



Proceedings 8th United States/Japan Conference on Sewage Treatment Technology 1981

October 12-13
Cincinnati, Ohio

October 19-20
Washington, D.C.



PROCEEDINGS

EIGHTH UNITED STATES/JAPAN CONFERENCE ON
SEWAGE TREATMENT TECHNOLOGY

CINCINNATI, OHIO: OCTOBER 12-13, 1981
WASHINGTON, D.C.: OCTOBER 19-20, 1981

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FOREWORD

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

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

A handwritten signature in black ink, appearing to read "Anne M. Gorsuch", with a stylized, flowing script.

Anne M. Gorsuch
Administrator

Washington, D.C.

CONTENTS

FOREWORD.....	iii
JAPANESE DELEGATION.....	vi
U.S.-CINCINNATI DELEGATION.....	vii
U.S.-WASHINGTON, D.C.....	viii
JOINT COMMUNIQUE.....	1
JAPANESE PAPERS.....	3
UNITED STATES PAPERS.....	627

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UNITED STATES AND JAPAN DELEGATES TO THE EIGHTH CONFERENCE,
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CINCINNATI, OHIO



MR. FRANCIS T. MAYO, DIRECTOR, MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY AND MR. JOHN J. CONVERY, GENERAL CHAIRMAN OF THE EIGHTH CONFERENCE AT A WORKING SESSION, CINCINNATI, OHIO



MRS. ANNE M. GORSUCH, ADMINISTRATOR, USEPA GREETES DR. TAKESHI KUBO, JAPANESE TEAM LEADER AND JAPANESE DELEGATES TO THE EIGHTH CONFERENCE IN WASHINGTON, D.C.



DR. JOHN W. HERNANDEZ, JR., DEPUTY ADMINISTRATOR, USEPA AND DR. TAKESHI KUBO, JAPANESE TEAM LEADER EXCHANGE GREETINGS AT THE WASHINGTON, D.C. SEMINAR OF THE EIGHTH CONFERENCE



JAPANESE AND U.S. DELEGATES ON A FIELD VISIT IN THE WASHINGTON, D.C. AREA

JOINT COMMUNIQUE

EIGHTH UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

CINCINNATI, OHIO OCTOBER 12-14, 1981
WASHINGTON, D.C. OCTOBER 19-20, 1981

1. The Eighth United States/Japan Conference on Sewage Treatment Technology was held in Cincinnati, Ohio from October 12-14, 1981 and in Washington, D.C. from October 19-20, 1981.
2. The Japanese Delegation headed by Dr. Takeshi Kubo, President of the Japan Sewage Works Agency was composed of two representatives from the Ministry of Construction, two representatives of the Japan Sewage Works Agency and one each from the Cities of Tokyo, Kyoto and Yokohama.
3. The United States Delegation in Cincinnati consisted of Mr. John J. Convery, Director, Wastewater Research Division, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency as General Conference Chairman, eleven federal government representatives, one representative from industry and one representative from local government (Indianapolis). The Washington, D.C. Delegation consisted of Mr. William J. Lacy, Director, Water and Waste Management, Monitoring and Research Division, Office of Monitoring and Quality Assurance, U.S. Environmental Protection Agency as Washington Conference Chairman as well as eleven federal government representatives, one representative of the Water Pollution Control Federation, one representative from industry and one representative from academia.
4. The Chairmanship of the Conference was shared by Dr. Takeshi Kubo and Mr. John J. Convery in Cincinnati and Dr. Takashi Kubo and Mr. William J. Lacy in Washington, D.C.
5. During the Conference, papers relating to joint research projects on sludge treatment/disposal including utilization of sludge as a resource, co-incineration of refuse and sludge, and composting were presented by both sides. Data and findings on the joint research projects were useful to the develop-

ment of improved control technology practices for each country. A decision was made to expand the scope of the joint research projects conducted by the two countries to include wastewater reuse.

6. Principal topics of the Conference were innovations in wastewater treatment as well as sludge management disposal and toxics removal, pretreatment, combined industrial-municipal treatment and nutrient control. The discussions which followed the presentations were also useful to both countries.
7. Field visits in Cincinnati, Ohio; Tampa, Florida; Washington, D.C.; Denver, Colorado and Los Angeles, California are planned to inspect wastewater treatment facilities in these areas.
8. Recent engineers exchanges between the two countries included a three-week visit in 1981 to Japan by Dr. Irwin J. Kugelman of the Wastewater Research Division, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency. Mr. M. Hirabayashi of the Japan Sewage Works Agency is now studying at the County Sanitation Districts of Los Angeles County. Both parties agreed to continue the engineers exchange program.
9. Dr. Takeshi Kubo announced that after ten years he would transfer the future responsibilities of Japanese Chairman of the Project Agreement to Mr. T. Tamaki, Director, Department of Sewerage and Sewage Purification, Ministry of Construction. Dr. Kubo's leadership and significant contributions to the cooperative agreement was acknowledged in the form of an EPA commemorative plaque presented by the EPA Deputy Administration, Dr. John W. Hernandez, Jr.
10. Both sides agreed that the time interval between conferences should be extended from once every eighteen months to once every two years.
11. It was proposed by the Japanese side that the Ninth Conference shall be held in Tokyo, Japan about October 1983.
12. A Proceedings of the Conference will be printed in English and in Japanese.

JAPANESE PAPERS

PERFORMANCE OF SEWAGE SLUDGE DEWATERING BY SCREW PRESS.....	5
Kazuhiro Tanaka, Section Chief, Research and Technology Division, Japan Sewage Works Agency	
UTILIZATION OF SEWAGE SLUDGE AS RESOURCES.....	41
Tetsuichi Nonaka, Senior Technical Advisor, Sewage Works Bureau, Tokyo Metropolitan Government	
EXPERIMENT ON INCINERATION OF MUNICIPAL REFUSE AND SEWAGE SLUDGE IN KYOTO CITY.....	67
Takashi Yoneda, Director, Sewage Works Bureau, City of Kyoto	
NEW ASPECTS OF SLUDGE INCINERATION IN YOKOHAMA.....	89
Shigeki Miyakoshi, Senior Technical Advisor, Sewage Works Bureau, City of Yokohama	
CURRENT STATUS OF AUTOMATIC MONITORING OF WATER QUALITY IN SEWAGE TREATMENT.....	143
Ken Murakami, Water Quality Section, Water Quality Control Division, Public Works Research Institute, Ministry of Construction	
PILOT PLANT STUDY FOR TREATMENT OF COMBINED FISH-PROCESSING AND DOMESTIC WASTE IN MAKURAZAKI CITY.....	163
Kazuhiro Tanaka, Section Chief, Research and Technology Development Division, Japan Sewage Works Agency	
TECHNICAL EVALUATION OF DEEP WELL BIOLOGICAL PROCESS.....	199
M. Kuribayashi and K. Murakami, Water Quality Control Division, Public Works Research Institute, Ministry of Construction	
DEPHOSPHORIZATION OF SEWAGE BY CONTACT CRYSTALIZATION OF CALCIUM APATITE.....	283
Tetsuichi Nonaka, Senior Technical Advisor, Sewage Works Bureau, Tokyo Metropolitan Government	
ADVANCED TREATMENT PROJECT FOR EUTROPHICATION CONTROL IN LAKE BIWA.....	313
Kazuhiro Tanaka, Section Chief, Research and Technology Development Division, Japan Sewage Works Agency	

(continued)

JAPANESE PAPERS (continued)

EFFECT OF DETERGENTS AND SOAP ON MUNICIPAL WASTEWATER TREATMENT...	385
K. Kobori, H. Watanabe and K. Murakami, Water Quality Control Division, Public Works Research Institute, Ministry of Construction	
FIFTH FIVE-YEAR SEWERAGE SYSTEM DEVELOPMENT PROGRAM AND LONG- RANGE PROSPECTS FOR DEVELOPMENT OF SEWERAGE SYSTEMS.....	441
Tsutomu Tamaki, Director, Department of Sewerage and Sewage Purification, Ministry of Construction	
CURRENT ISSUES IN WATER POLLUTION CONTROL ADMINISTRATION IN JAPAN.....	477
Toshiki Oshio, Councilor for Engineering Affairs, Environment Agency, Government of Japan	
REGIONAL SLUDGE MANAGEMENT PROGRAM IN THE TOKYO BAY BASIN.....	489
Dr. Takeshi Kubo, President, Japan Sewage Works Agency	
STATUS AND OUTLOOK OF SLUDGE TREATMENT AND DISPOSAL IN KYOTO CITY.....	531
Takashi Yoneda, Director, Sewage Works Bureau, City of Kyoto	
COLLECTIVE PRETREATMENT OF INDUSTRIAL WASTEWATER FROM MINOR ENTERPRISES IN YOKOHAMA.....	549
Shigeki Miyakoshi, Senior Technical Advisor, Sewage Works Bureau, City of Yokohama	
AGRICULTURAL USE OF SEWAGE SLUDGE.....	585
Kazuhiro Tanaka, Section Chief, Research and Technology Division, Japan Sewage Works Agency	

Eighth US/JAPAN Conference
on
Sewage Treatment Technology

PERFORMANCE OF SEWAGE SLUDGE DEWATERING BY SCREW PRESS

October 13-14, 1981

Cincinnati, Ohio USA

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

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1. INTRODUCTION

Dewatering devices used in sewage treatment plants in Japan have largely depended on vacuum filters or filter presses using slaked lime and ferric chloride.

It has been found on the application of these dewatering devices that the dewatered cake dosed of large amounts of slaked lime produces hexavalent chromium when incinerated in a furnace, and is too bulky to handle.

In recent years, the use of polymers (cationic and anionic polyelectrolytes) as dewatering coagulants has solved these problems.

In order to dewater polymer-coagulated sludges, centrifuges were put to practical use, but were found defective in that moisture content of the cake remains 80% or more.

Very recently, belt filter presses have come into use as they save electrical energy, and reduce the moisture content of the cake, more than do centrifuges.

In many sewage treatment plants belt filter presses are considered for a new or additional installation of dewatering devices because of their excellent performance.

In addition to belt presses, screw presses discussed here are another alternatives for sewage treatment plants since they are claimed to be able to reduce the cake moisture content to 60% or lower.

The screw press uses polymers as dewatering coagulants as does the belt filter press, and also employs steam heating in compression processes.

Originally, the screw press was developed for dewatering sludge generated from wastewater treatment facilities of fish meal processing. Its modification for application to dewatering sewage sludge has been studied for years.

The screw press as a dewatering device is a tapered Archimedes' screw consisting of a screw shaft and a perforated metal cylinder (cylindrical screen). The space between the screw shaft and the perforated metal cylinder is made smaller the nearer it goes toward the outlet.

As the screw blades turn, the sludge moves toward the outlet while being compressed gradually for filtration and dewatering.

As a result, the dewatering efficiency increases with the progress of the sludge along the screw press dewatering device.

The screw press has been claimed to achieve very low moisture content of sludge cake, to run without noise and vibration in operation, to be easy to maintain, and to further improve the dewatering efficiency by making use of steam.

A total of six screw presses has now been operated at three sewage treatment plants in Japan.

The results of a survey on the dewatering performance of the screw presses at these three sewage treatment plants are reported here.

2. DESCRIPTION OF SCREW PRESS

A cross section of the screw press is shown in Fig. 1.

A detailed view of the screen is shown in Fig. 2.

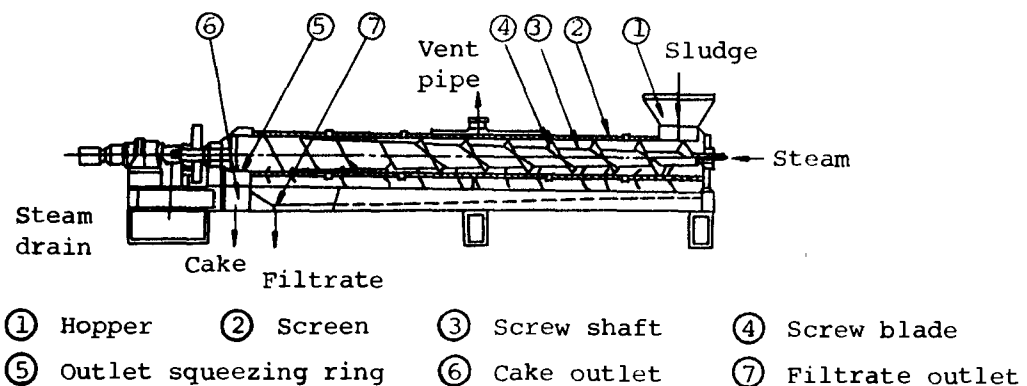


Fig. 1 Cross- sectional view of screw press dewatering device

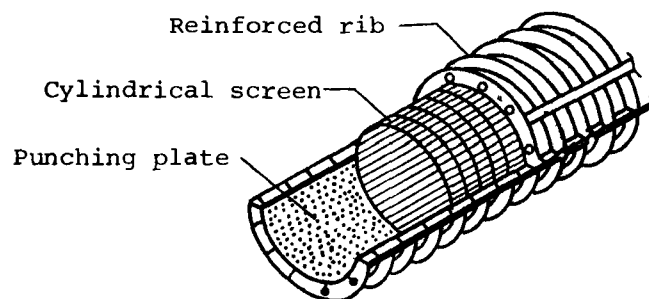


Fig. 2 Detailed view of the screen

Polymer-conditioned sludge is charged into the hopper of the screw press. Free porewater contained in the sludge floc is gravitationally discharged from the coarse-mesh punching plate set near the hopper. This zone including a coarse-mesh punching plate is called the draining zone.

The polymer-conditioned sludge is then gradually transferred by revolving screw-blades. As the space between the cylindrical punching plate and the main body of the screw gets smaller the farther it goes away from the inlet, the remaining porewater and adhering water contained in the sludge are gradually removed. This zone is called the pressing zone. Finally, the sludge is compressed at high pressure by a tapered cone. Namely, the sludge is worked under a shearing force for further compression and filtration. This zone is called the squeezing zone.

In a screw press dewatering device, the draining, pressing and squeezing processes are integrated for continuous production of dewatered cake.

The cylindrical screen is an assembly of stainless steel punching plates having a number of small holes (2 to 3 mm ϕ). The filtrate is discharged from these small holes. The sludge flocs form bridges at the inside of small holes of the punching plates, and function to trap sludge flocs smaller than the punching plate holes. The filtrate which includes much suspended solids is guided into a solid-liquid separation tank where suspended solids are settled for separation from liquid. The supernatant of the solid-liquid separation tank is then returned to the wastewater treatment system, while the settled suspended solids are returned to the sludge storage tank where they are mixed with thickened sludge and charged again into the hopper of the screw press.

The screw press is equipped with a hollow screw shaft into which steam ($1 \text{ kg/cm}^2 \cdot \text{G}$) is fed for heating of sludge through thermal conduction.

The dewatering efficiency can be increased by steam heating. It is inferred that the steam heating will reduce the viscosity of the sludge and hence reduce the drag resistance to increase the dewatering efficiency.

The cylindrical screen is covered with a shell to prevent foul odors generated from the heated sludge and also to stop the filtrate from spattering

The specifications of the screw presses surveyed are listed in Table 1.

Table 1 Specifications of screw presses

Item \ Model	ø100	ø200	ø500	ø800	ø900	ø1000	Remarks
Diameter of cylindrical screen D (mm)	100	200	500	800	900	1000	
Screen length, L (mm)	1000	2000	6000	8000	8000	10000	
Screw pitch, P (mm)	100	200	450	700	800	800	
Overall length (mm)	1800	3000	7500	10200	10200	12500	
Width (mm)	900	1000	1400	2300	2500	2700	
Overall height (mm)	1100	1200	1700	2000	2200	2300	
Weight (ton)	0.7	1	5	16	20	23	
Power requirement (kW)	0.4	1.5	2.2	5.5	5.5	5.5	
Revolution speed (rpm)	0.1~ 0.53	0.08~ 1.28	0.08~ 0.32	0.026~ 0.316	0.02~ 0.24	0.046~ 0.18	
Steam supply rate (kg/hr)	20	30	90	-	200	200	1kg/cm ²
Surveyed model at Nishinomiya	0	0	0		0		
Surveyed Model at Kurume						0	
Surveyed Model at Sakai		0		0			

The models of the screw presses are expressed in terms of diameter of cylindrical screen (mm). Namely, the cylindrical screen diameter of the 900 model is 900 mm.

3. SLUDGE CHARACTERISTICS AND SELECTION OF COAGULANTS

3.1 Sludge Characteristics

Of the sludge treatment facilities of the three sewage treatment plants surveyed, two adopt anaerobic digestion and mechanical dewatering, and the other dewateres raw mixed sludge mechanically. The sewage treatment plants are outlined in Table 2. The characteristics of the sludge to be dewatered at each treatment plant are shown in Table 3.

In the sludge characteristics analyzed, the ignition loss of coarse suspended solids is measured as follows.

Ignition loss of coarse suspended solids

Take a 100 ml sample of sludge, pour it over a 100-mesh sieve, and wash with tap water.

Heat the residue at 105°C, and weigh the evaporation residue.

Then, heat the evaporation residue at 600°C, and weigh the ignition residue.

Calculate the ignition loss of coarse suspended solids according to the following formula.

Ignition loss of coarse suspended solids (%)

$$= [100\text{-mesh evaporation residue (g/l)} - 100\text{-mesh ignition residue (g/l)}] / [\text{SS of sludge sample (g/l)}] \times 100$$

Table 2 Outline of surveyed sewage treatment plants

Name of municipality	Sewer system	Maximum daily sewage flow (m ³ /day)	Sewage treatment system	Sludge treatment system
Nishinomiya	Combined system	126,000	Conventional activated sludge process	Gravitational thickening - Anaerobic digestion - Elutriation - Mechanical dewatering - Incineration (by multiple hearth furnace)
Kurume	Separate system	21,300	Conventional activated sludge process	Gravitational thickening - Anaerobic digestion - Elutriation - Mechanical dewatering
Sakai	Separate system	3,500	High rate trickling filter and conventional activated sludge process	Mechanical dewatering

Table 3 Characteristics of tested sludges

Name of municipality	Type of sludge	Season	TS (%)	SS (%)	DS (%)	VTS/TS (%)	PH (-)	M-alkalinity (mg/l)	Ignition loss of coarse suspended solids (%)
Nishinomiya	Anaerobically digested and elutriated sludge	Summer	3.92	3.54	0.38	54.3	7.6	3,150	4.1
		Winter	3.36	3.00	0.37	53.9	7.6	2,890	4.8
Kurume	Anaerobically digested and elutriated sludge	Summer	2.02	1.96	0.06	65.2	2.0	1,110	5.2
		Winter	2.26	2.17	0.09	66.1	7.6	2,310	6.1
Sakai	Raw mixed sludge	Summer	1.78	1.68	0.10	58.4	6.9	1,210	8.2
		Winter	1.14	0.91	0.23	72.0	6.8	815	21.7

3.2 SELECTION OF COAGULANTS

It was found that the use of cationic polyelectrolyte in combination with anionic polyelectrolyte is very effective for sludge dewatering.

It was also found that the strength of floc coagulated by the polyelectrolytes alone is weak, and therefore, that the use of quick lime or slaked lime as a coagulation aid improves this strength.

Tests were conducted at the sewage treatment plants to select the appropriate coagulants. The selected polymers and coagulation aids are as shown in Table 4.

4. DEWATERING PERFORMANCE

4.1 Test Results

The factors affecting dewatering performance of the screw press are sludge characteristics, chemical dosing conditions (particularly lime dosing), steam supply rate, screw shaft revolution speed, and blade height at the cake outlet, punching plate hole size, steam pressure and temperature, etc. The dewatering tests were conducted with the screw shaft revolution speed, slaked lime dosing rate, and steam supply rate as operational parameters. The results summarized in Figs. 3 through 5 are discussed below.

Table 4 Kinds of coagulants and aids, and their dosing rates

Name of municipality	Type of sludge	Season	Chemicals					
			Anionic polymer		Cationic polymer		Inorganic aids	
			Brand	Dosing rate (%/DS)	Brand	Dosing rate (%/DS)	Aids	Dosing rate (%/DS)
Nishinomiya	Anaerobically digested sludge	—	Diafloc AP-520	0.3	Diafloc KP-201G	0.2	Slaked lime	10
Kurume	Anaerobically digested sludge	—	Kurifloc PA-331	0.4	Kurifix CP-604	0.8	Quick lime	30
Sakai	Raw mixed sludge	Summer	Kurifloc PA-331	0.2	Kurifix CP-634	0.6	Slaked lime	20
		Winter				0.8		30

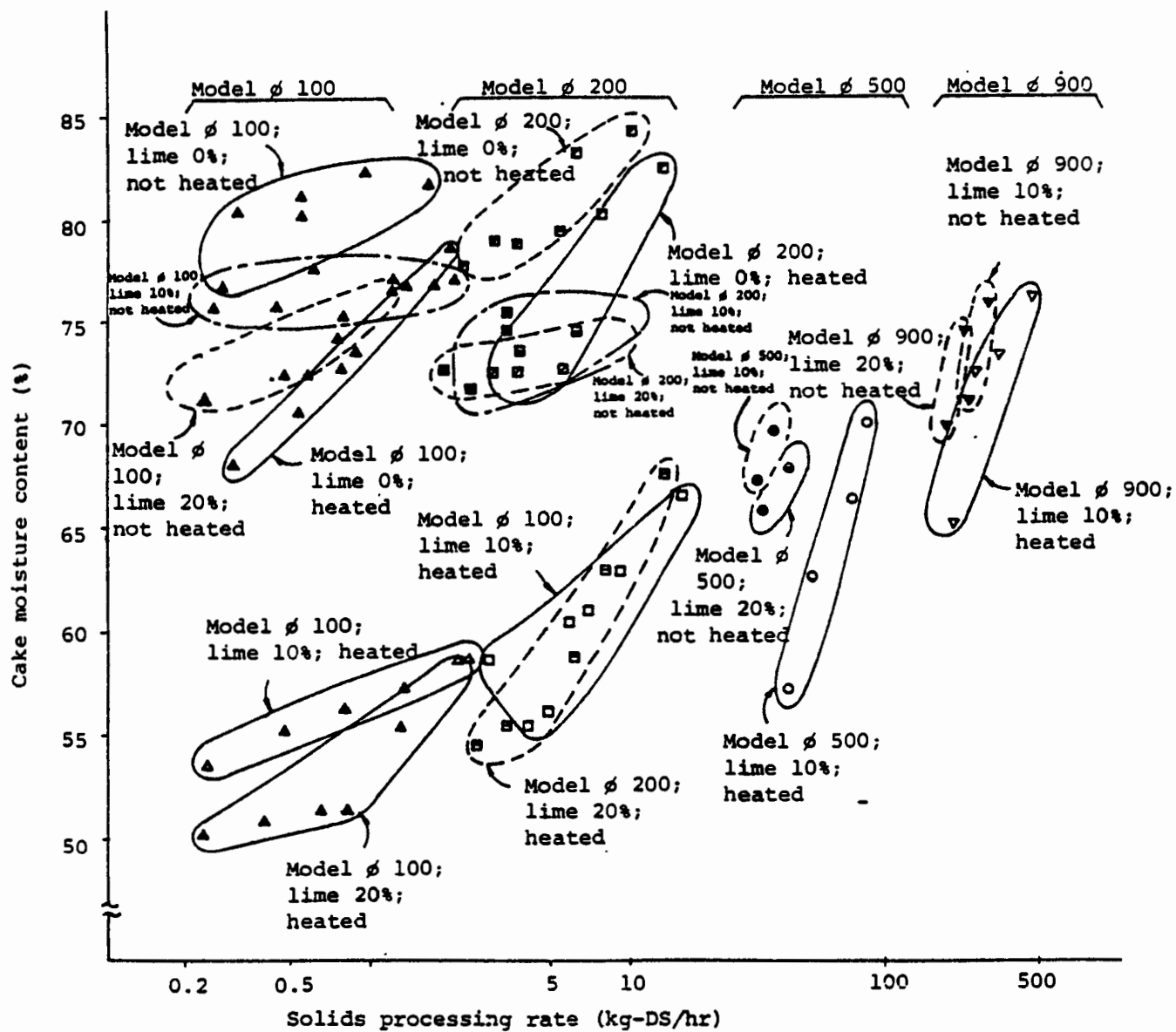


Fig. 3 Relationship between cake moisture content and solids processing rate under various operational conditions (1)
(Nishinomiya)

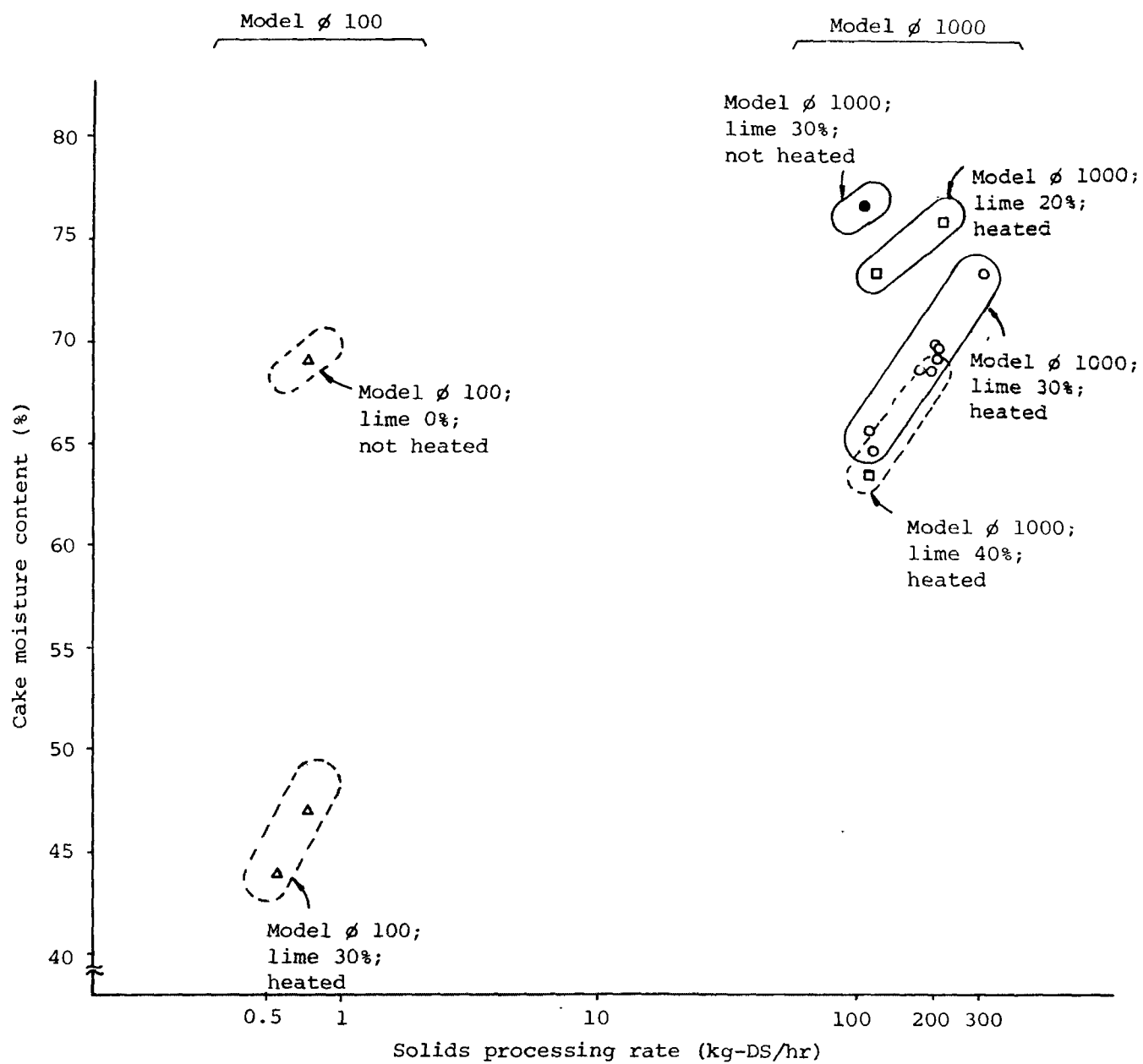


Fig. 4 Relationship between cake moisture content and solids processing rate under various operational conditions (2) (Kurume)

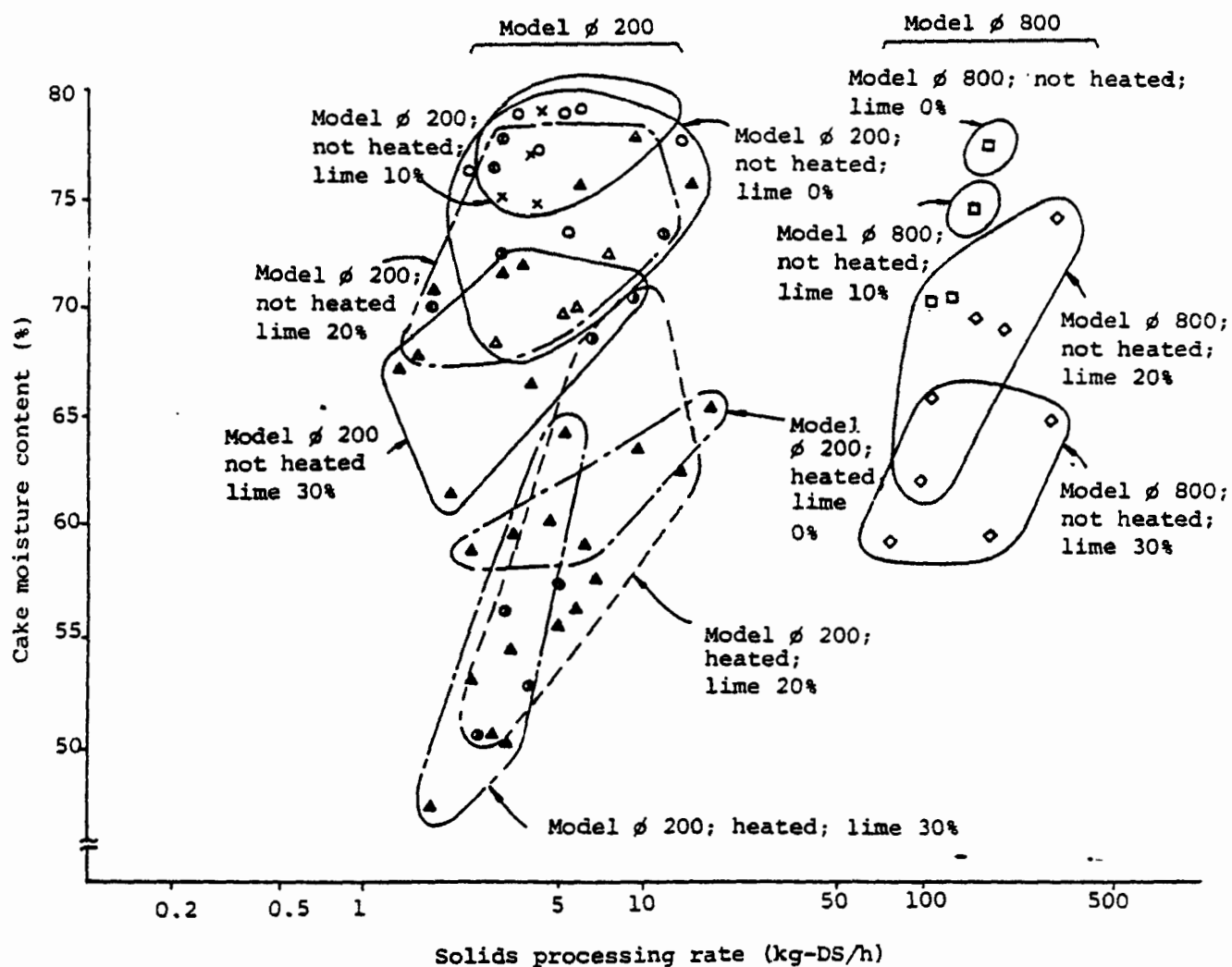


Fig. 5 Relationship between cake moisture content and solids processing rate under various operational (3) (Sakai)

4.2 Effect of Screw Shaft Revolution Speed

In a Sewage Treatment Plant at Kurume City, tests were conducted on a Model $\phi 1000$ type screw press to determine the dewatering performance with the revolution speeds of 0.046, 0.069, 0.092 and 0.123 rpm.

The dewatering performance is shown in Fig. 6

With increase in the revolution speed, the solids processing rate was increased, but the moisture content in the cake also increased. With increase in the speed of the dewatering device, the screw feed rate increases, increasing the solids processing rate.

Therefore, the compressing and dewatering time in the dewatering device becomes shorter, which increases the moisture content in the sludge cake.

In the other two sewage treatment plants, in Nishinomiya and Sakai, the same tendency as in Kurume was noticed.

4.3 Effect of Lime Dosing Rate

In a Sewage Treatment Plant, in Nishinomiya, Hyogo, anaerobically digested and elutriated sludge was tested with the lime dosing rate changed to 5%, 10%, and 20%.

In this test, the cake moisture content changed as shown in Fig. 7.

It was found that lime dosing is highly effective in reducing the moisture content in the cake. In Kurume, the same survey was conducted, changing the lime dosing rate to 20%, 30% and 40%. Up to 30% dosing of lime, the same tendency as in Nishinomiya was noticed. However, more than 40% dosing of lime, the moisture content in the cake was not significantly improved.

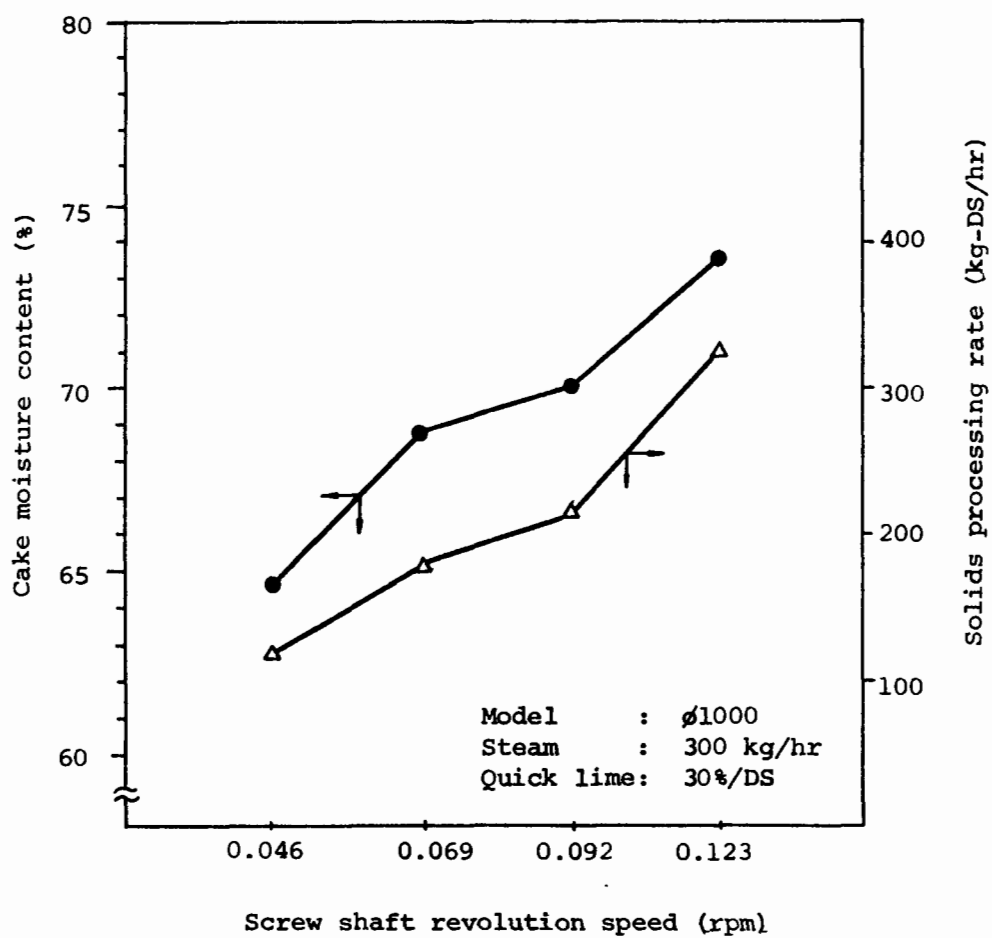


Fig. 6 Relationship between cake moisture content, solids processing rate and screw shaft revolution speed (Kurume)

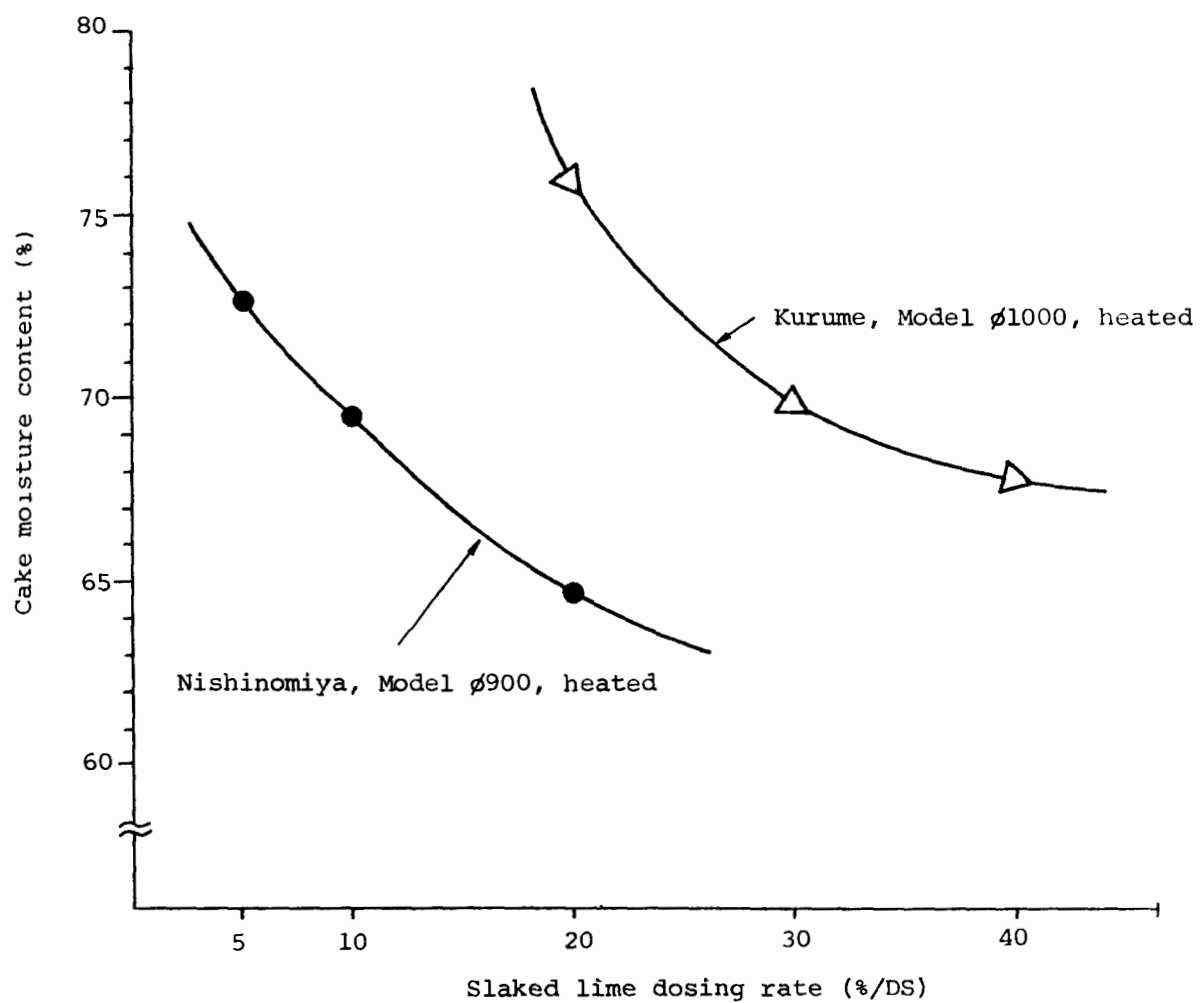


Fig. 7 Slaked lime dosing rate vs. cake moisture content

4.4 Effect of Steam Supply

In Nishinomiya, anaerobically digested and elutriated sludge was tested with the steam supply rate changed by using the model $\phi 900$ screw press. The steam supply rate vs. cake moisture content relationship is shown in Fig. 8. It can be seen from Fig. 8 that the steam supply contributes toward improving the cake moisture content.

The temperature distribution within the steam-heated screw press is shown in Fig. 9.

To assess the effect of steam heating on viscosity and filtration rate of sludge, laboratory experiments were conducted using Nishinomiya sludge. The results are shown in Fig. 10 and Fig. 11. It is found that the viscosity of polymer-coagulated sludge decreases with increase in the steam supply rate; the viscosity is minimized at a sludge temperature of 80°C to 90°C, and the filtration time of sludge is minimized at a temperature of about 60°C. The reduction in the liquid-phase drag resistance due to change in the liquid-phase viscosity of polymer-coagulated sludge is considered beneficial to the improvement of the sludge dewatering efficiency.

The characteristics of the filtrate and its supernatant obtained in Nishinomiya are as shown in Table 5.

According to Table 5, COD_{Mn} and BOD_5 of the filtrate and its supernatant are higher when heated with steam than when not supplied with steam. Whether this phenomenon is ascribable to temperature rise due to steam supply or other factors is as yet unknown. Anyway, the increase in the organic loadings to be sent back to the wastewater treatment system is negligibly small.

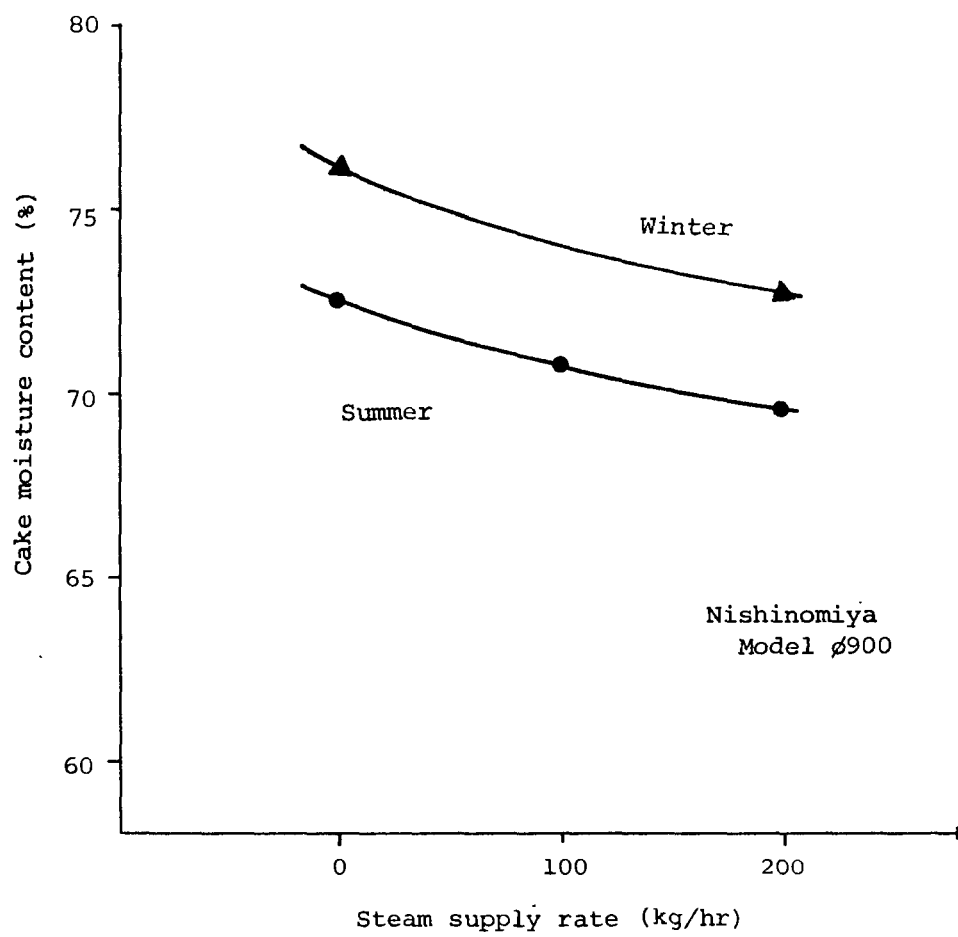


Fig. 8 Cake moisture content vs. steam supply rate

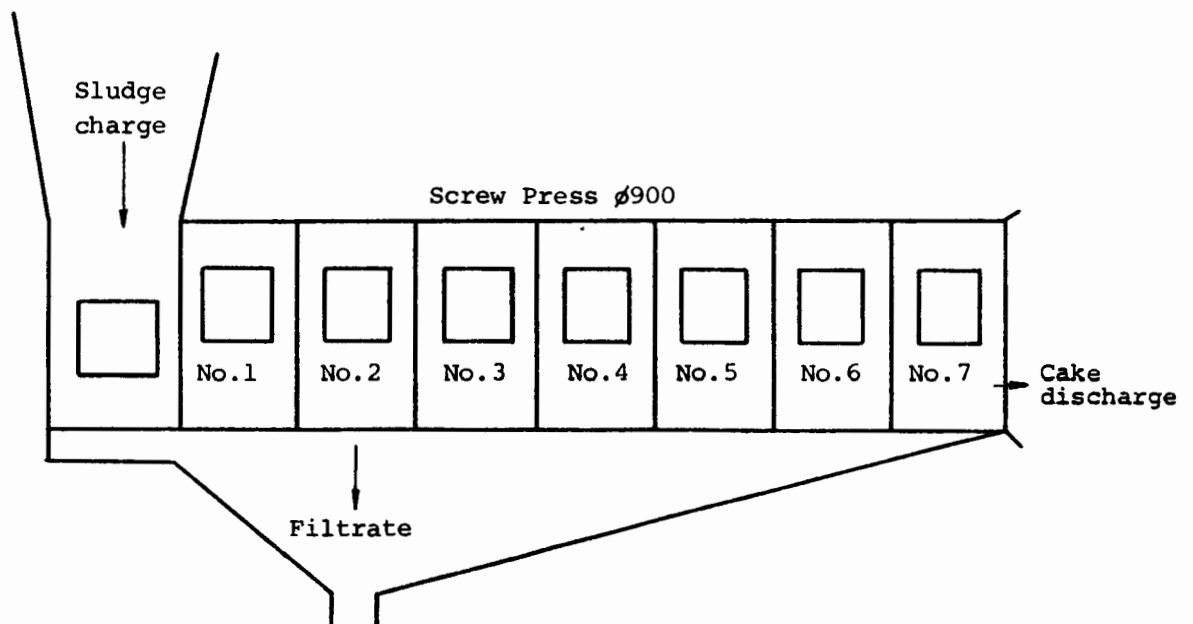
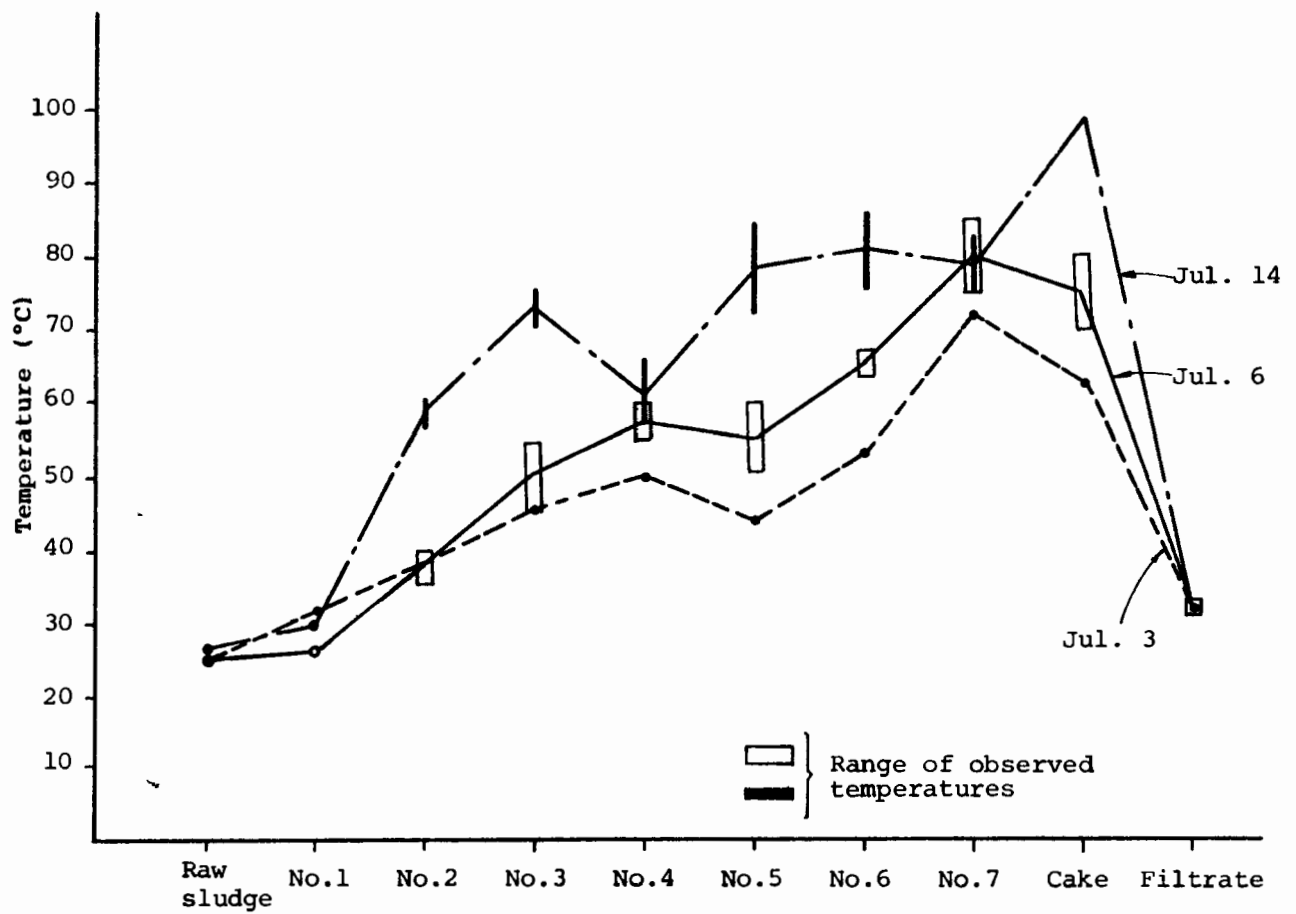


Fig. 9 Temperature distribution in Screw Press Model ø900

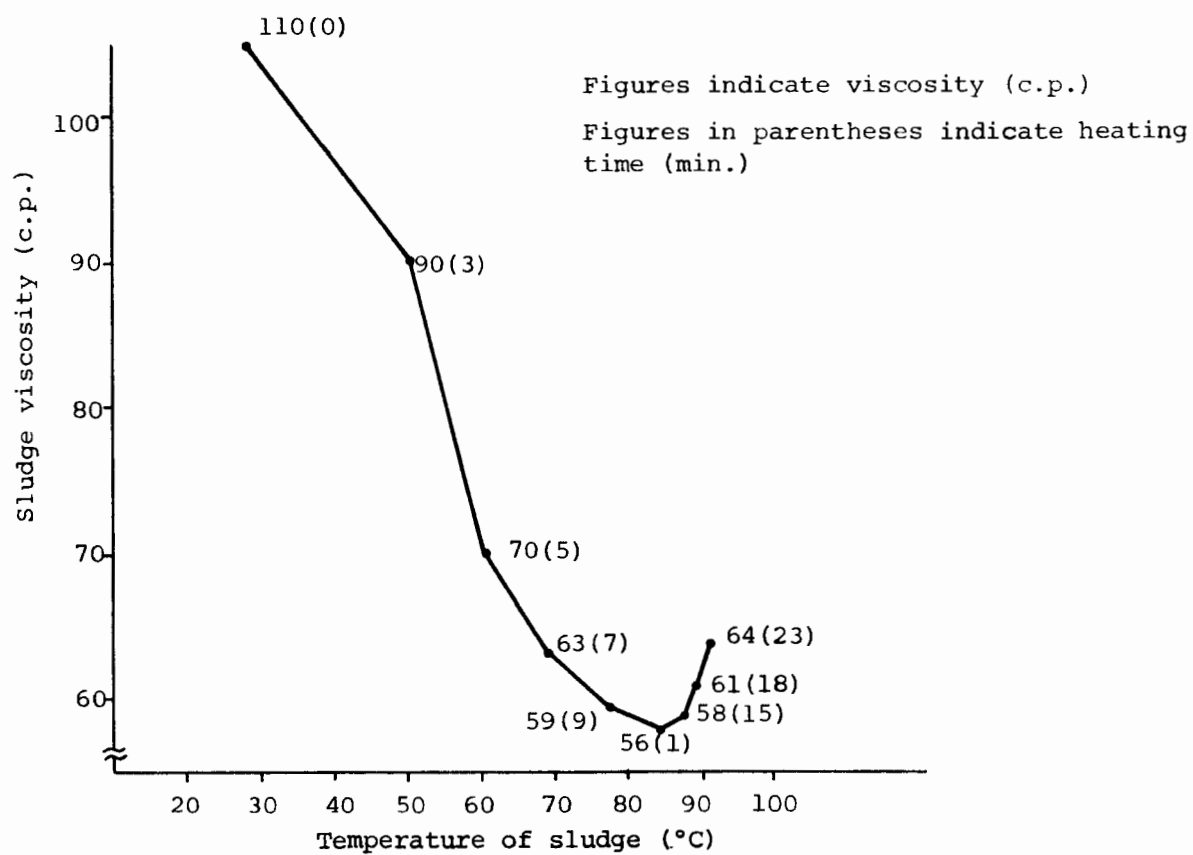


Fig. 10 Temperature vs. viscosity

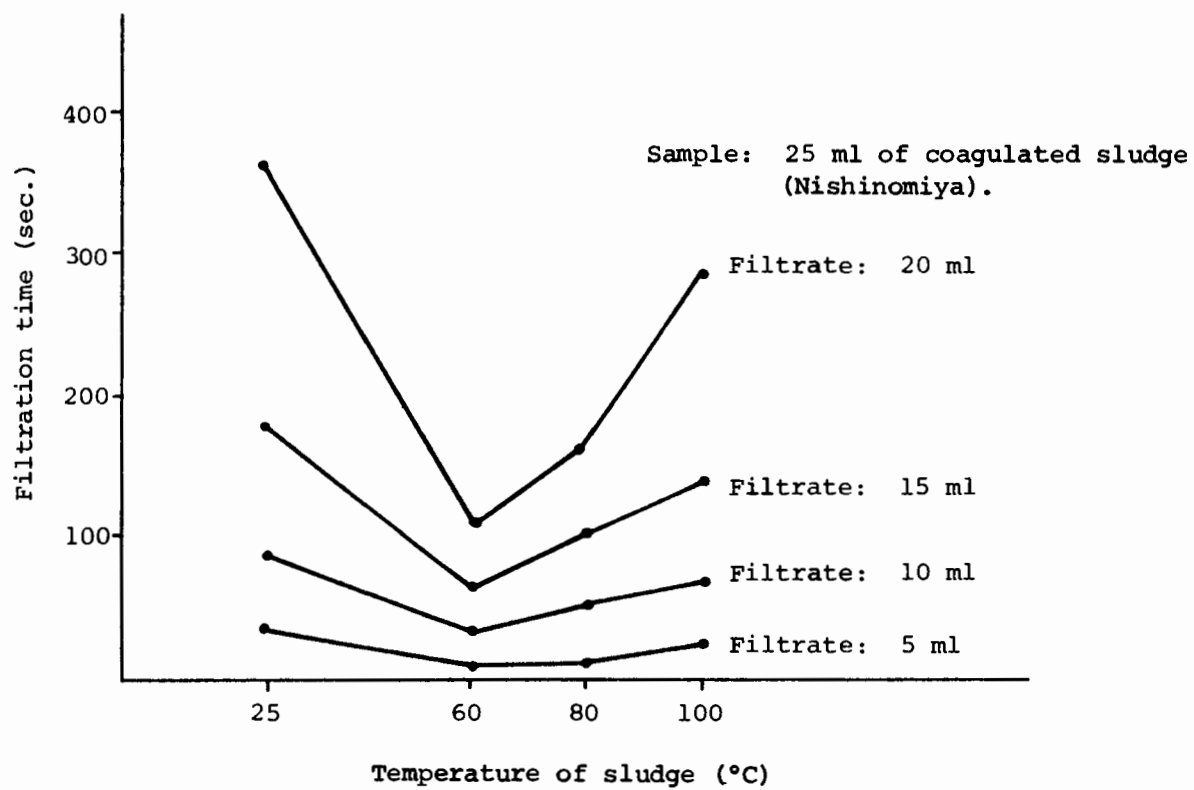


Fig. 11 Effect of steam supply on filtration time

Table 5 Characteristics of filtrate and its supernatant obtained by sedimentation tank
(Screw press Model ø900, Nishinomiya)

	Steam	TS	SS	DS	PH	D-COD	D-BOD	Remarks
		(mg/l)	(mg/l)	(mg/l)	-	(mg/l)	(mg/l)	
Filtrate	Supplied at a rate of 200 kg/hr	5,400 (6840~4410)	1,740 (3370~660)	3,660 (3850~3470)	12.0 (12.1~11.9)	174 (195~146)	273 (444~172)	Steam 1 kg/cm ² G
	Not supplied	5,570 (7970~4250)	1,785 (4530~540)	3,790 (4190~3450)	12.4 (12.5~12.2)	147 (173~134)	173 (371~80)	
Supernatant obtained by sedimentation tank	Supplied at a rate of 200 kg/hr	4,030 (4360~3770)	370 (890~156)	3,660 (3850~3470)	11.8 (11.9~11.6)	213 (237~190)	299 (428~185)	Steam 1 kg/cm ² G
	Not supplied	4,010 (4340~3750)	220 (514~39)	3,790 (4190~3450)	12.3 (12.5~12.2)	158 (195~106)	180 (304~64)	

4.5 Effect of Diameter of Cylindrical Screen

In Nishinomiya comparisons of the dewatering efficiencies and scale effects were studied using models $\phi 100$, $\phi 200$, $\phi 500$, and $\phi 900$ type screw presses.

The solids processing rate increased with increase in screw shaft revolution speed and also with increase in diameter of the cylindrical screen (Fig. 3 and Fig. 6).

The solids processing rate was 0.3 to 2.5 kg-DS/hr. for the Model $\phi 100$, 3 to 15 kg-DS/hr. for the Model $\phi 200$, 30 to 80 kg-DS/hr. for the Model $\phi 500$ and 170 to 370 kg-DS/hr. for the Model $\phi 900$.

Generally, the solids processing rate of the screw press is given by the following formula.

$$Q = (\pi/4) \times (D_1^2 - D_2^2) \times P \times R \times (1 - M/100) \times 60 \times r \times \eta \dots (1)$$

Where, Q : Solids processing rate (kg-DS/hr.)

D_1 : Diameter of cylindrical screen (m)

D_2 : Final outside diameter of screen shaft (m)

P : Screw pitch (m)

R : Screw shaft revolution speed (rpm)

M : Cake moisture content (%)

r : Cake density (kg/m^3)

η : Cake transfer efficiency (-)

The final outside diameter of screw shaft (D_2) of the test screw presses is portional to the diameter of cylindrical screen (D_1), and the screw pitch (p) is also nearly proportional to the same. If the cake transfer efficiency and cake density are constant, and the variations in cake moisture content are neglected, Eq. (1) can be rewritten as follows.

$$Q \propto D^3 \cdot R \dots \dots \dots (2)$$

The theoretical sludge retention time in the screw press is given by the formula below.

$$T = L/S = L/(P \times R) \dots\dots\dots (3)$$

Where, T : Theoretical sludge retention time in device (min)

L : Length of cylindrical screen (m)

S : Cake transfer rate (m/min.)

P : Screw pitch (m)

R : Screw revolution speed (rpm)

The sludge dewatering time can be considered to be nearly equal to the theoretical retention time.

As explained in the foregoing, the surveyed screw presses have screw pitches (p) nearly proportional to the diameter of cylindrical screen (D₁). The length of cylindrical screen (L) is also designed to be nearly proportional to D₁.

Thus, Eq. (3) is rewritten as follows.

$$T \propto 1/R \dots\dots\dots (4)$$

By combining Eqs. (2) and (4), the following formula is obtained.

$$Q \propto D_1^3/T \dots\dots\dots (5)$$

Of the test data obtained in Nishinomiya, those obtained for a case where slaked lime was dosed 10%, anionic polymer 0.3% and cationic polymer 0.2% with steam supplied to the screw press were selected to determine an empirical formula for the solids processing rate as follows.

$$\log Q = 2.9 \log D_1 - 0.72 \log T - 4.7 \quad \text{..... (6)}$$

Where, Q : Solids processing rate (kg-DS/hr.)

D₁: Diameter of cylindrical screen (mm)

T : Theoretical retention time (min.)

Eq. (6) is graphically represented in Fig. 12.

This difference between Eq. (5) and Eq. (6) is probably due to the assumption of a fixed cake moisture content in Eq. (5).

An empirical formula of cake moisture content was also derived by using the same test data as follows.

$$\log M = 0.15 \log D_1 - 0.10 \log T + 1.61 \quad \text{..... (7)}$$

Where, M : Cake moisture content (%)

T : Theoretical retention time (min.)

D₁: Diameter of cylindrical screen (mm)

Eq. (7) is graphically represented in Fig. 13, in which the data obtained in Sakai and Kurume are also plotted.

When the sewage sludge was dewatered by the screw press with slaked lime dosing rate of 10% and steam supply, the cake moisture content varied depending upon D₁ as follows.

- o Model ø100: 53 to 58%
- o Model ø200: 55 to 68%
- o Model ø500: 57 to 70%
- o Model ø900: 65 to 75%

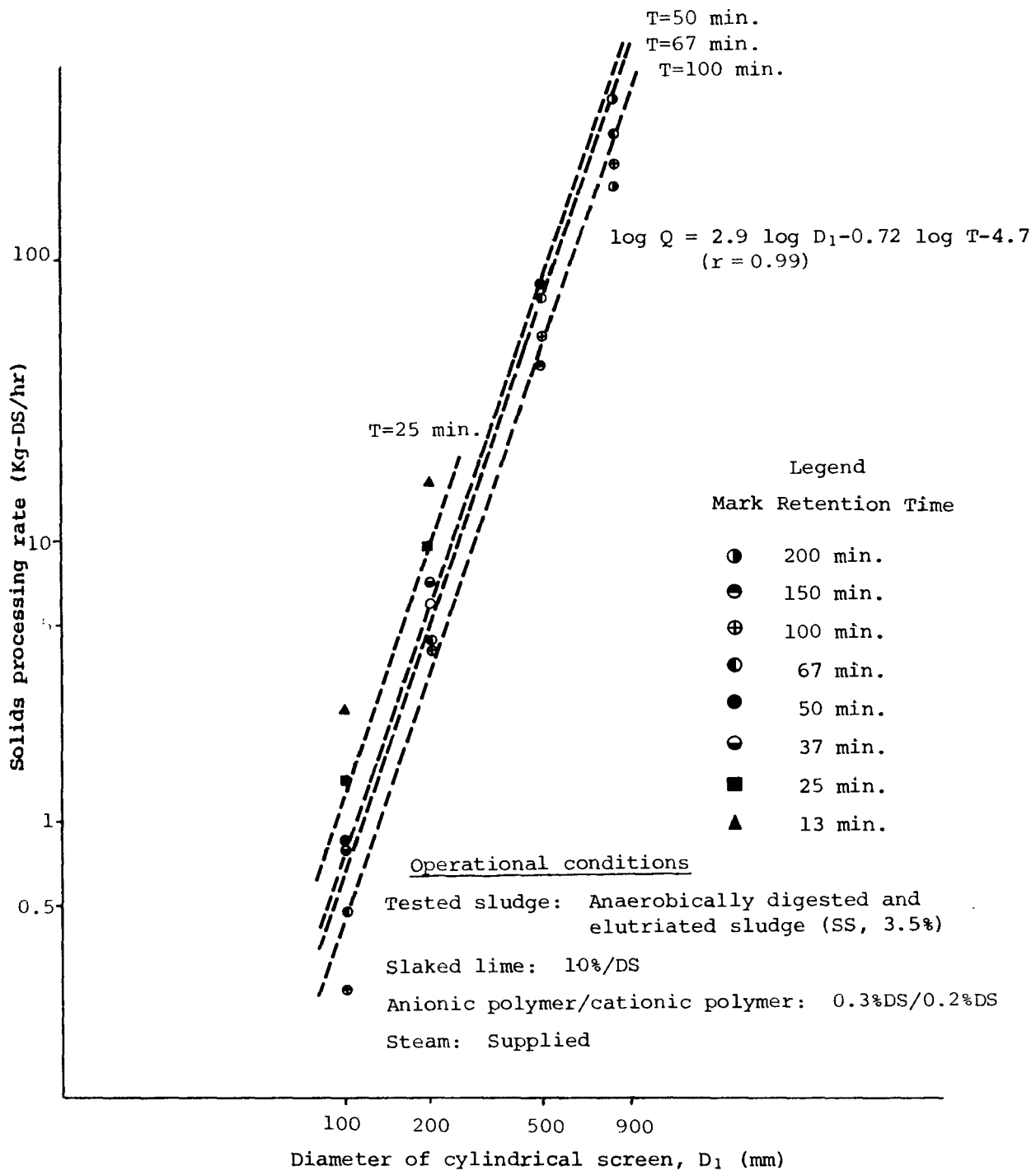


Fig. 12 Relationship between solids processing rate and diameter of cylindrical screen

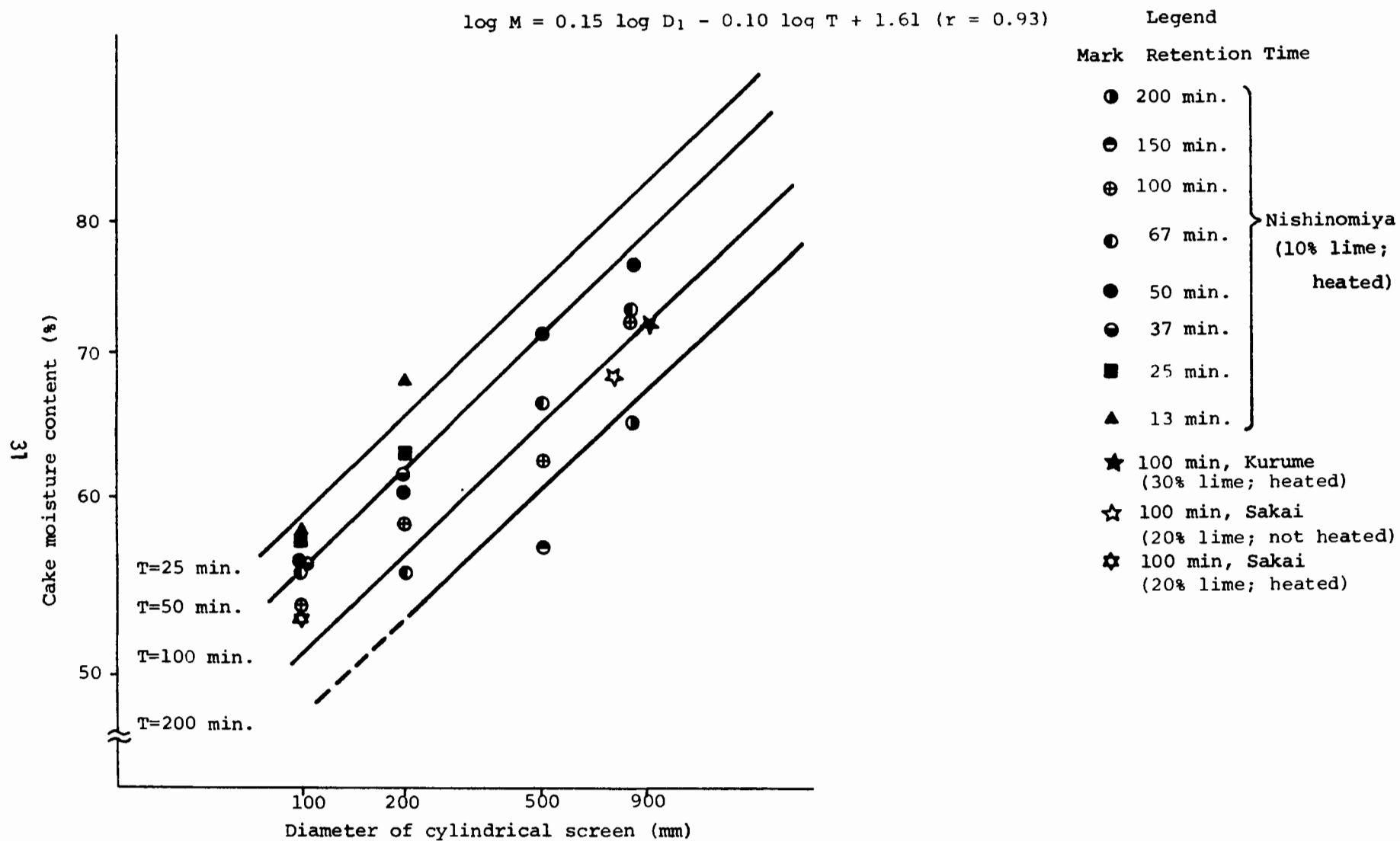


Fig. 13 Cake moisture content vs. diameter of cylindrical screen

When steam was not supplied to the screw press, the cake water content of the same sludge was as follows.

- o Model $\phi 100$: 75 to 77%
- o Model $\phi 200$: 72 to 76%
- o Model $\phi 500$: 67 to 70%
- o Model $\phi 900$: 71 to 76%

As demonstrated above, in case of steam supply dewatering the cake moisture content increased with increase in the diameter of cylindrical screen. On the other hand, when steam was not supplied, the effect of the diameter on the cake moisture content was not observed. As compared with the case without steam supply, the steam supply improved the cake moisture content by about 20% for Model $\phi 100$, about 15% for Model $\phi 200$, about 10% for Model $\phi 500$, and about 5% for Model $\phi 900$. It is therefore deduced that under the constant steam pressure, the effect of steam supply on the dewatering efficiency becomes greater the smaller the diameter of cylindrical screen. This is probably due to the fact that the steam supply per unit weight of solids increases with decrease in the diameter of cylindrical screen.

Fig. 14 shows the relationship between the diameter of the cylindrical screen and the steam supply rate per unit weight of solids based upon the data in Nishinomiya. From Fig. 14, the following can be deduced.

- (1) The steam supply rate is proportional to the diameter of the cylindrical screen (D_1).
- (2) The solids processing rate is proportional to D_1 to the power of 2.3 to 2.9.

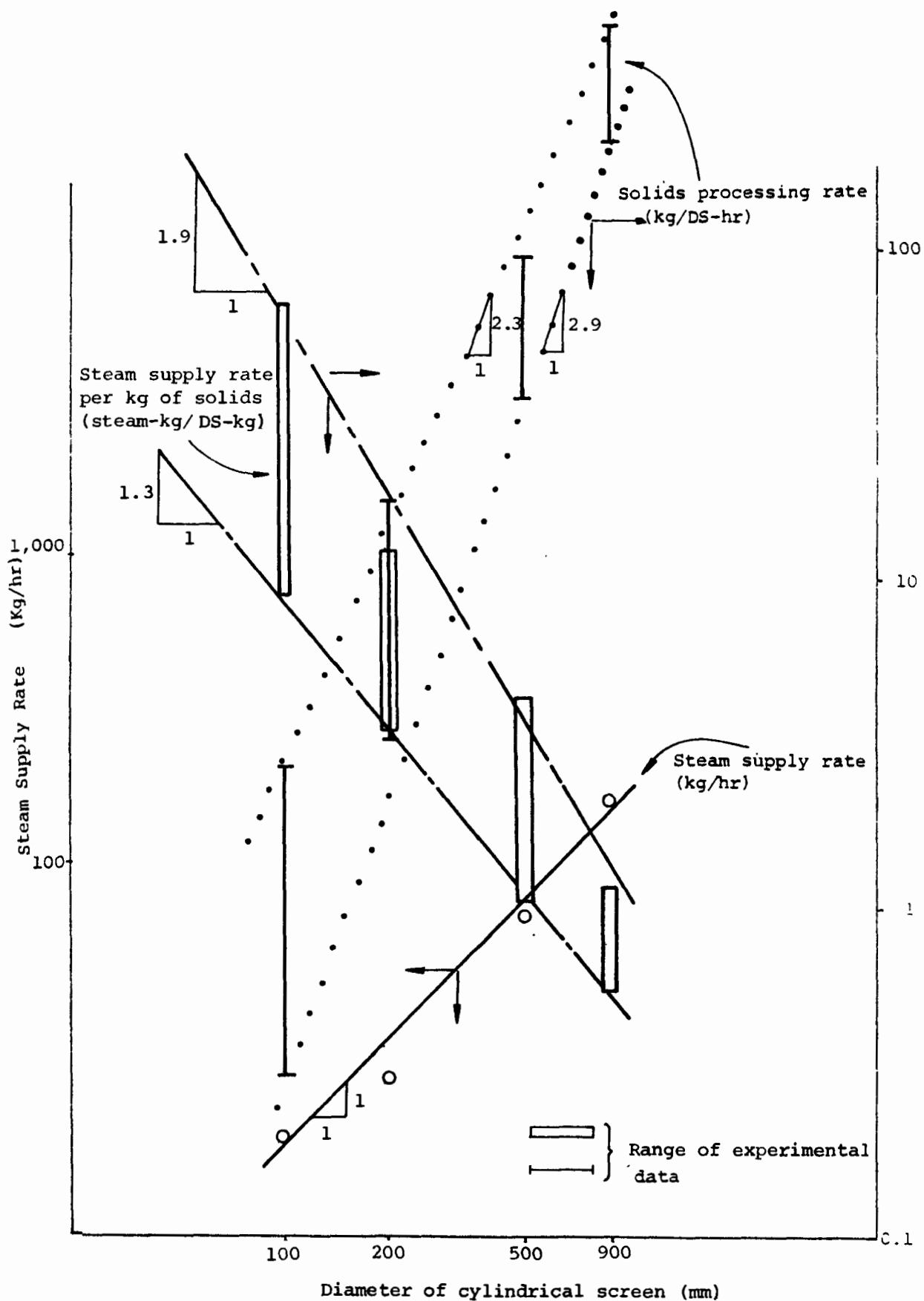


Fig. 14 Diameter of cylindrical screen vs. steam supply rate per unit weight of solids

- (3) Namely, the steam supply rate per unit weight of solids is in inverse proportion to D_1 to the power of 1.3 to 1.9.

Accordingly, when steam is supplied, the cake moisture content rises with increase in the diameter of the cylindrical screen. On the other hand, in case of no steam supply, the cake moisture content is unrelated to the diameter of the cylindrical screen.

The steam supply rate is limited by the steam heat to be transferred to sludge. The currently available screw presses are designed to transfer steam heat to the sludge through their screw shafts.

The surface area of the screw shaft is proportional to the square of the diameter of the cylindrical screen.

When the steam temperature is increased by increasing the steam pressure, sludge sticks on the heating surface of the screw shaft, degrading the heating efficiency. Therefore there is a upper limit to which the steam pressure can be increased. The theoretical steam supply rate at the optimum steam pressure is governed by the square of the diameter of the cylindrical screen.

In the actual screw press, however, filling up efficiency of the sludge is reduced with increase in the diameter of the cylindrical screen, and the ratio of the heating surface available to the total screw shaft surface area is accordingly reduced. For this reason, the actual steam supply rate is proportional to the diameter of the cylindrical screen (D_1) as shown in Fig. 14.

Thus, for large diameters of cylindrical screen, the following measures will be necessary:

- (1) The screw shaft should be designed so that steam can be supplied not only to the shaft, but also to the screw blade, in order to increase the heat transfer area.
- (2) The sludge should be filled up well in.

- (3) Solids processing rate should be limited to meet the steam supply rate.

5. ECONOMICS OF SCREW PRESS AS COMPARED WITH OTHER DEWATERING DEVICES

The Edagawa Sewage Treatment Plant, in Nishinomiya, has, in addition to screw presses, three belt type rotary vacuum filters and three stand type filter presses to dewater anaerobically digested and elutriated sewage sludge.

Accordingly, data regarding the operation of these three types of dewatering devices on the same sludge was collected in Table 6. Based on this data, an economic evaluation of the three types of dewatering devices was made. The dewatered sludge cake is incinerated in multiple hearth furnaces, and ash is disposed of for landfill outside the plant.

The utility costs of 15.8 tons-DS/day dewatering and incineration facilities were estimated. The transportation costs required for ash disposal were also considered.

The results of cost estimation are given in Table 7.

Table 6 Operational conditions of dewatering devices

Item	Type of dewatering device	Screw press	Vacuum filter	Filter press
	Unit			
Raw sludge concentration (SS)	%	Anaerobically digested and elutriated sludge, 3.5	Anaerobically digested and elutriated, sludge, 3.5	Anaerobically digested and elutriated sludge, 3.5
Sludge processing rate	m ³ /hr/unit	7.5	7.0	5.5
Ferric chloride dosing rate	%/DS	-	3	15
Slaked lime dosing rate	%/DS	10	25	35
Anionic polymer dosing rate	%/DS	0.3	-	-
Cationic polymer dosing rate	%/DS	0.2	-	-
Steam supply rate	kg/hr	200	-	-
Cake moisture content	%	70	79	58
Solids processing rate	Kg/m ² /hr	*8.4	11	3.9
Solid removal rate (SS)	%	73	95	98
Running time		6 hrs./day (300 days/yr.)	6 hrs./day (300 days/yr.)	6 hrs./day (300 days/yr.)
* Per unit screen area				

Table 7 Comparison of utility cost for dewatering and incineration

(based on 1979 price)

			Screw press		Vacuum filter		Filter press	
			Specific unit	Cost (¥/t-DS)	Specific unit	Cost (¥/t-DS)	Specific unit	Cost (¥/t-DS)
Utility cost	Dewatering cost	Slaked lime	137 kg/t-DS	2,700	263 kg/t-DS	5,300	357 kg/t-DS	7,100
		Anionic polymer	4.11 kg/t-DS	4,100	-	-	-	-
		Cationic polymer	2.74 kg/t-DS	4,100	-	-	-	-
		Ferric chloride	-	-	83.7 kg/t-DS	2,100	403 kg/t-DS	10,000
		Total of chemicals		10,900		7,400		17,100
		Water	12 m ³ /t-DS	400	28 m ³ /t-DS	800	6 m ³ /t-DS	200
		Electricity	80 KWH/t-DS	1,500	145 KWH/t-DS	2,600	140 KWH/t-DS	2,500
		Steam	1050 kg/t-DS	5,300	-	-	-	-
		Sub-total		18,100		10,800		19,800
	Incineration cost			21,000		37,900		19,000
	Sub-total			39,100		48,700		38,800
Ash disposal cost				3,200		4,300		5,500
Grand Total				42,300		53,000		44,300

In terms of the total amount of costs for utilities and ash disposal, the screw press system costs about 5% less than the filter press system and about 25% less than the vacuum filter system.

In terms of utility costs of dewatering, the screw press system costs about 60% more than the vacuum filter system, but about 5% less than the filter press system. The reason why the vacuum filter system is most inexpensive is that its chemical consumption is the lowest. The utility costs for the filter press system are high because it uses large amounts of chemicals and electricity.

In the screw press system, the utility costs are not so low because steam generation is costly though little electric power is consumed.

In terms of the total amount of utility costs including incineration costs, the screw press system is almost the same as the filter press system, and costs about 25% less than the vacuum filter system.

The difference in utility costs between the vacuum filter system and the screw press system is due mainly to the incineration costs; the sludge cake from the vacuum filter system contains much moisture and requires much auxiliary fuel, while the sludge cake from the screw press system can burn itself with little auxiliary fuel.

The filter press system and screw press system are nearly on a par with each other in terms of both total utility cost and incineration cost.

6. SUMMARY

The performance of the screw press was investigated using anaerobically digested and elutriated sludge, and raw mixed sludge obtained from the three sewage treatment plants. The results are summarized below.

- (1) The factors affecting the dewatering performance of the screw press include sludge characteristics, chemical dosing conditions (particularly slaked lime dosing rate), steam supply rate, screw shaft revolution speed, height of screw blade at the cake outlet, and punching metal pore diameter. Of these factors, the screw shaft revolution speed, slaked lime dosing rate, and steam supply rate were varied to investigate the dewatering performance.
- (2) It was found that increase in the slaked lime dosing rate leads to reduction of cake moisture content, that increase in the screw shaft revolution speed increases the cake moisture content and solids processing rate, and that increase in the steam supply rate reduces the cake moisture content.
- (3) The theoretical solid processing rate is proportional to the cube of the diameter of the cylindrical screen, and in inverse proportion to the sludge retention time.
- (4) The moisture content in cake increases with increase in the diameter of the cylindrical screen when the sludge is steam-heated, but remains almost unchanged if steam is not supplied. The steam supply rate is proportional to the diameter of the cylindrical screen. On the other hand, the solids processing rate proportional to the cube of the diameter of the cylindrical screen. Accordingly, the steam supply rate per unit weight of solids is in inverse proportion to the square of the diameter of the cylindrical screen.

Accordingly, the moisture content in the cake increases with increase in the diameter of the cylindrical screen.

- (5) From the test data available in Nishinomiya, empirical formulas for solids processing rate and cake moisture content were obtained for screw press with steam supply and 10% dosing of slaked lime.

$$\log Q = 2.9 \log D_1 - 0.72 \log T - 4.7$$

$$\log M = 0.15 \log D_1 - 0.10 \log T + 1.61$$

Where, Q : Solids processing rate (kg-DS/hr.)

M : Moisture content in cake (%)

D_1 : Diameter of cylindrical screen (mm)

T : Theoretical retention time (min.)

- (6) Operational costs were compared for the screw press, vacuum filter, and filter press.

In terms of total utility costs including incineration and ash disposal cost, the screw press system costs about 5% less than the filter press system, and about 25% less than the vacuum filter system.

In terms of the total utility costs inclusive of incineration costs, the screw press system is nearly on a par with the filter press system, and costs about 25% less than the vacuum filter system.

In terms of utility costs for dewatering alone, the screw press costs about 60% more than the vacuum filter, but about 5% less than the filter press.

**UTILIZATION
OF
SEWAGE SLUDGE AS RESOURCES**

October 1981

Cincinnati, U. S. A.

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1. INTRODUCTION

Recently, turning sewage sludge into fuel or other useful resources has been seriously talked about.

In the past, however, sewage sludge was dumped as it was regarded as useless.

Historically speaking, there was once a time when sewage sludge was used as fertilizer to a very limited extent. Until recently, most sewage sludge has been disposed of chiefly in the form of landfill.

Although landfill is productive in the sense that it fills hollows, lowlands and shores, etc. to turn them into useful lands, the sewage sludge, which is not a good filling material, has so far been disposed of as useless rather than as useful.

If we continue to dispose of sewage sludge as useless, the dump sites, that is, the landfill sites, will soon be filled up. It is therefore mandatory to establish some improved form of disposal by recycling sludge into marketable products.

If the sewage sludge produced in a city is harnessed for urban development, it will go a long way toward improving the distribution economy as there will be no need to haul wastes out of the city and at the same time bring superfluous materials into the city.

In order to establish an improved form of sludge disposal, it will be necessary to promote research and development for sludge recycling technology, application technology and distribution system for recovered products.

The feasibility of sewage sludge harnessing is dependent on the wise use of organic substances and trace inorganic salts in the sludge. The requirements for harnessing the sludge may be summarized as follows.

1) Agricultural use

- a. To be effective as fertilizer or for soil conditioning.
- b. Not to contaminate the surface and ground waters through accumulation, leaching, and infiltration of pollutants.
- c. Not to be detrimental to plants.
- d. Not to communicate contaminants through the food chain.

2) Use for grassy land

Almost the same as agricultural use.

3) Civil engineering use

- a. To meet engineering requirements such as strength.
- b. Not to leach out contaminants.

Namely, the ideas of harnessing sewage sludge must be implemented while fulfilling these requirements. For example, when it is planned to make products out of sludge cake or its ashes, it is necessary to carefully study the sludge as a material physically and chemically, particularly from the viewpoint of safety relating to the heavy metals contained, how to use and apply the products, and how to make the products on an industrial scale for the purpose of establishing a stabilized sludge recycling system.

2. DEVELOPING COMPOST FROM SEWAGE SLUDGE

Since 1976, the Tokyo Metropolitan Government has promoted experiments on composing of sewage sludge, along with tests on the agricultural use of compost, phytotoxicity surveys, and marketing system, etc.

According to the interim results of these surveys, the Tokyo Metropolitan Government installed a 10 m³/day composting plant at the Minami-Tama Wastewater Treatment Plant in March 1980. At present, this composting plant is performing well, producing 2 to 3 tons of compost a day.

This paper deals with the harnessing of sewage sludge with emphasis on the Minami-Tama composting plant and the results of tests on the agricultural use of sludge compost.

2.1 COMPOSTING PLANT OUTLINE

2.1.1 Minami-Tama Wastewater Treatment Plant

The Minami-Tama Wastewater Treatment Plant is a separate sewer type plant treating the domestic wastewater of Tama New Town, and is located in the west of Tokyo.

The sludge produced there contains low concentrations of heavy metals, and is suitable for agricultural use. At the Minami-Tama Wastewater Treatment Plant, raw sludge is added with slaked lime and ferric chloride and then is dewatered through a filter press.

The following are the 1980 daily average statistics recorded by the Minami-Tama Wastewater Treatment Plant.

- . Domestic wastewater treated : 17,970 m³
- . Sludge produced : 209 m³
- . Sludge cake produced : 15 tons
- . Sludge cake incinerated : 12 tons

2.1.2 Composting Plant

a. Principal features of the plant

- . Area of premises : 3,164.9 m²
- . Building floor space
 - Plant : 1,243.8 m²
 - Warehouse : 360.0 m²
- . Processing capacity : 10 m³/day (approx. 7 tons/day)
in terms of sludge cake
- . Output : 2 to 3 tons/day
- . Construction cost : ¥400,000,000

b. Flow sheet and operating method

Figure 1 shows a flow sheet of the plant. (Refer to Fig. 1)

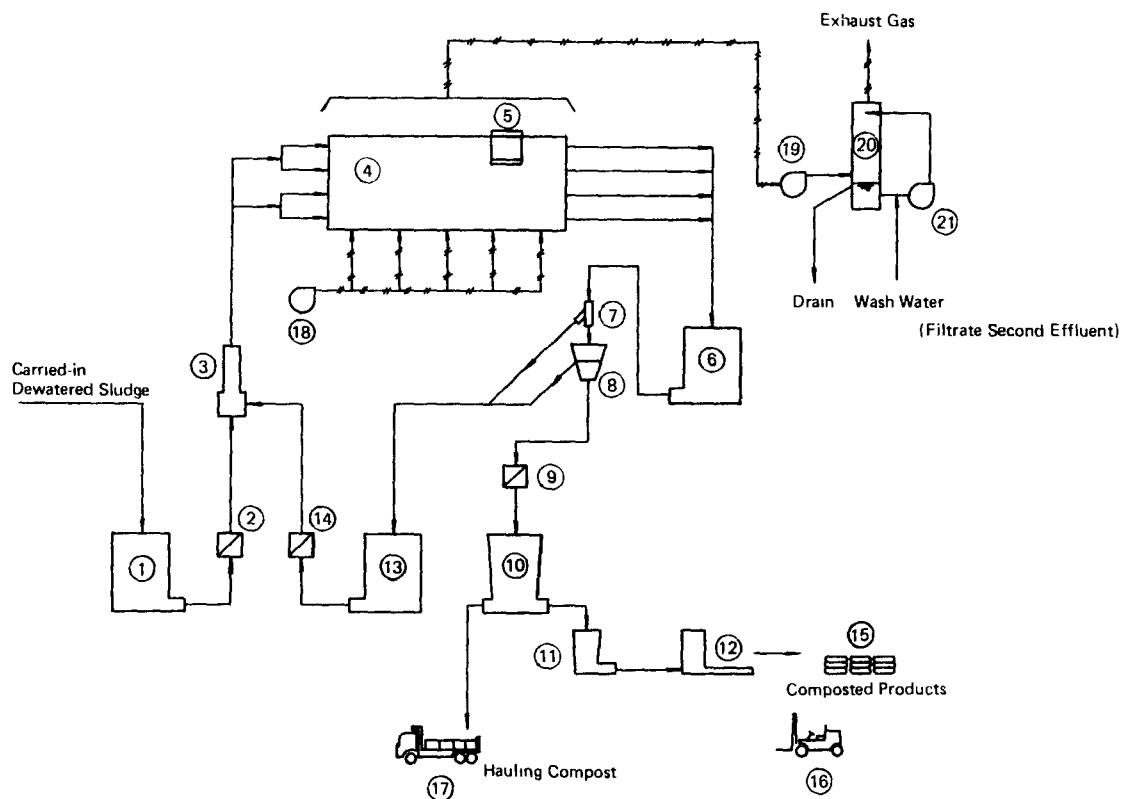


Fig. 1 Flow diagram of full-scale composting plant

Fig. 1 (continued)

No	Description	No of Unit	Particulars
1	Dewatered sludge hopper	1	Capacity : 10 m ³
2	Belt-scale of sludge cake	1	Load cell, 2 Tons/H, 0.75 kW
3	Crushing, mixing, granulating unit	1	3.8 Tons/H, 30 kW
4	Composting tank	4	Capacity : 52.5 m ³
5	Transfer) Mechanical turner	1	Self-travel bucket, 45 m ³ /H
6	Hauling hopper	1	Capacity : 10 m ³
7	Damper	1	
8	Vibrating screen	1	Circular vibrating screen, 0.4 kW
9	Belt scale of products	1	Load cell : 1.5 tons/H, 0.75 kW
10	Products' hopper	1	Capacity : 21 m ³
11	Measuring hopper	1	Capacity : 0.5 m ³
12	Bagging machine	1	Belt conveyor type, 2 Tons/H
13	Return compost hopper	1	Capacity : 28 m ³
14	Belt-scale of return compost	1	Load cell, 3 tons/H, 0.75 kW
15	Composted products		
16	Forklift	1	1 Ton
17	Truck for hauling		
18	Blower for composting tank	3	Turbofan, 35 m ³ /min, 7.5 kW
19	Exhaust gas fan	1	Turbofan, 250 m ³ /min, 15 kW
20	Scrubber	1	Vertical 2-stage wash, 250 m ³ /min.
21	Circulation pump	2	Centrifugal pump, 0.5 m ³ /min, 5.5 kW

There are four composting tanks, each measuring 18.8 m in length, 2.1 m in width and 2.8 m in depth. A single mechanical turner is installed to handle these four tanks.

The plant is fully automated, and is operated as follows.

Sludge cake with a moisture content of about 70% is mixed with return compost of about 110% in weight ratio to produce a mixture with a moisture content of about 55%. The mixture is then charged into the composting tanks.

The mixture is turned over once every other day, reaches the outlet of the composting tank after being subjected to the fifth turnover, and is finally discharged from the composting tank. The sludge cake is retained in the composting tank for nearly 10 days. The fermented sludge discharged from the composting tank is classified through 3-mesh vibrating screen, and the

smaller is delivered as product compost, while the larger is used as return compost.

The plant is operated to adjust the moisture content of product compost to 30 to 35%. Namely, it is required to properly adjust the ratio of return compost to meet the changes in sludge cake moisture content and fermentation conditions.

The aeration of the composting tanks is carried out by air injected through five blow pipes provided longitudinally in the bottom of the composting tank. The standard aeration rate is set at 200 lit/m³-sludge/min. The aeration rate is adjusted while monitoring the temperature rise in the composting tank.

The product compost is packaged and delivered in 20-kg bags.

c. Plant performance

The composting plant was put into commission in May 1980. The achievements from that time till March 1981 were as follows.

. Sludge cake processed	: 1,271.4 tons
. Compost production	: 391.6 tons
. Compost shipments	: 367.7 tons
. Running cost (excl. depreciation cost)	: ¥20,672/ton-sludge cake ¥67,116/ton-compost

The marketing route of product compost is as shown below.

A ton of product compost fetches Sewage Works Bureau ¥5,000.

Tokyo Metropolitan Government's Sewage Works Bureau →
Tokyo Metropolitan Economic Agricultural Cooperatives → Local
agricultural cooperatives in Tokyo → Farmers

2.2 SURVEYS ON THE AGRICULTURAL USE OF COMPOST

2.2.1 Topics Discussed for Agricultural Use of Compost

The Tokyo Metropolitan Government organized a compost research and development project team in 1976. Since then, the team has undertaken a variety of researches and studies.

As regards the matters concerning the agricultural effects of compost, safety, demand and physical distribution, the surveys have been promoted under the leadership of the Bureau of Labor and Economic Affairs in order to study the following subjects.

- 1) Soil-conditioning effects and fertilizing effect of compost.
- 2) Compost application technology, and limitations on applications.
- 3) Analysis of effective and toxic substances in compost.
- 4) Formulation of compost merchandizing requirements and their indices.
- 5) Compost demand, and marketing problems.
- 6) Compost pricing.
- 7) Other related matters.

Each of these surveys is briefly described below.

2.2.2 Chemical Analysis for Preservation of Compost Quality and Assurance of Health Safety

In Japan, sewage sludge compost for agricultural use is classified as a special fertilizer, and is subject to control according to the Fertilizer Control Law with respect to its heavy metal concentrations, etc.

The limitations stipulated in the Fertilizer Control Law are 50 mg/kg for arsenic (As), 5 mg/kg for cadmium (Cd) and 2 mg/kg for mercury (Hg). In addition, the sludge compost is required to meet the standards as specified in Table 1. (Refer to Table 1)

Table 1 Limitations on hazardous industrial wastes

	Item	Criteria
1	Mercury alkylides	Not detectable
	Mercury or its compounds	Less than 0.005 mg per liter of sample liquid
2	Cadium or its compounds	Less than 0.3 mg-Cd per liter of sample liquid
3	Lead or its compounds	Less than 3 mg-Pb per liter of sample liquid
4	Organic phosphorous compounds	Less than 1 mg-phosphorous compound per liter of sample liquid
5	Sexivalent chromium compounds	Less than 1.5 mg-sexivalent chromium compounds per liter of sample liquid
6	Arsenic and its compounds	Less than 1.5 mg-As per liter of sample liquid
7	Cyanides	Less than 1 mg-cyanides per liter of sample liquid
8	PCB	Less than 0.003 mg-PCB per liter of sample liquid

The compost manufactured by the composting plant was examined according to these requirements. In addition, an analysis to determine whether the denaturalization of compost is sustained during the distribution process was conducted.

a. Analysis of fertilizer components

The sludge cake, primary compost (hot from the composting plant), heaped compost (primary compost cured for 45 days in the open air) were subjected to an analysis of fertilizer components, heavy metals, etc.

The results are as shown in Table 2. (Refer to Table 2)

Table 2 General characteristics and heavy metal content of compost

Product Characteristic	Dewatered sludge cake	Primary compost	Heaped compost	Product Characteristic	Dewatered sludge cake	Primary compost	Heaped compost
Moisture content(%)	70.7	21.4	23.4	T-K ₂ O (%)	0.04	0.04	0.04
pH (H ₂ O)	12.2	7.8	7.7	T-CaO (%)	28.11	24.61	
EC (mΩ/cm)	6.62	4.80	5.63	T-MgO (%)	0.75	0.63	0.61
COD (ppm)	2,290	2,340	1,000		29.15	25.49	25.08
TOC (%)	22.6	14.9	13.0	T-As (ppm)	2.3	3.9	4.5
T-N (%)	3.14	2.28	2.24	T-Cd (ppm)	1.3	1.5	2.1
C/N	7.2	6.5	5.8	T-Hg (ppm)	1.0	1.2	1.6
Ignition loss (%)	42.4	36.5	33.6	T-Cu (ppm)	117	124	153
NH ₄ -N (mg/100g)	29	458	498	T-Zn (ppm)	610	730	984
NO ₃ -N (mg/100g)	3	14	11	T-Ni (ppm)	34	40	44
T-P ₂ O ₅ (%)	2.43	3.18	3.54	T-Pb (ppm)	34	44	59

Notes: 1. pH, EC and COD when one part of dry product is mixed with 10 parts of water.
2. TOC and the characteristics listed below it refer to dry products.

It is found that there is no significant difference in quality between the primary compost and heaped compost, and that the product compost necessitates no further working prior to application. The concentrations of As, Cd, and Hg are found to be below the limitations specified in the Fertilizer Control Law.

b. Storage test

Three 20-kg vinyl bags of product compost were put in a thick-walled cardboard case, covered with a heat-insulating material, and loaded with a weight, were left to stand for about eight and a half months from July 1980 till March 1981 in order to investigate qualitative change.

During the storage period, analysis was performed three times. The results are as shown in Table 3. (Refer to Table 3)

Table 3 Results of storage test

Characteristic Date	Moisture content (%)	Ignition loss (%)	T-N (%)	T-C (%)	*pH (H ₂ O)	**pH (KCl)	*EC (mΩ/cm)	*COD (ppm)	**NH ₄ -N (mg/100g)	**NO ₃ -N (mg/100g)
Jul. 21, 1980	34.35	37.56	2.25	19.39	7.71	7.93	4.89	2,120	497	6
Nov. 10, 1980	34.22	36.90	2.10	18.86	7.93	8.00	5.34	1,940	648	N.D
Mar. 31, 1981	34.08	35.16	2.10	18.70	7.82	7.99	5.38	1,910	645	trace

Notes: 1. Ignition loss, T-N, and T-C refer to dry compost.

2. The characteristics marked with an asterisk refer to mixture of 1 part of dry compost and 10 parts of water.

Those marked with two asterisks refer to a mixture of 1 part of compost and 10 parts of KCl (10% solution).

During the eight and a half month storage period, little change occurred in moisture content and pH value. The ignition loss, T-N, T-C, and COD decreased a little, while the electric conductivity (EC) and NH₄-N increased slightly. While fermentation was still in progress, this was not so violent as to cause denaturalization.

c. Mineralization test

A sample of compost was sieved through a 1-mm screen, and a 10 mg N equivalent was mixed with 50 g of dry soil. The mixture was then cultured for 100 days at a temperature of 29 to 30°C with its moisture content controlled at 50 to 60% of the maximum water holding capacity, for the purpose of examining the changes in inorganic nitrogen content.

It was found that the primary compost originally contains inorganic nitrogen accounting for 20% of its total nitrogen, is easy to nitrify and decompose, and has an immediate effect as a fertilizer.

2.2.3 Tests on Application Effects and Safety

a. Application to vegetables

Since 1978, the growing of Japanese radish for spring cropping and cabbage for autumn cropping has been tested at the Kuroboku (humous soil containing volcanic ash) soil farms of the Agricultural Experiment Station.

For two years after the start of the tests, efforts were made to study the fertilizer effects of compost. Now a study of the effects of continued application of compost on the crops, particularly regarding accumulation of lime in the soil, has been under way.

With respect to the fifth spring crop of Japanese radishes and the sixth autumn crop of cabbages, an outline of test farms, yield from each farm, and an analysis of edible parts of the crops for heavy metals, etc. are shown in Tables 4, 5 and 6. The transition of yields from the first and subsequent crops is shown in Table 7. (Refer to Table 4, Table 5, Table 6 and Table 7)

Table 4 Test sections

Crop and fertilizer application Section	Japanese radish for spring crop (5th crop)							Cabbage for autumn crop (6th crop)						
	Chemical fertilizer as starter			Chemical fertilizer as top dressing		Calcium carbonate	Compost	Chemical fertilizer as starter			Chemical fertilizer as top dressing		Calcium carbonate	Compost
	N	P ₂ O ₅	K ₂ O	N	K ₂ O			N	P ₂ O ₅	K ₂ O	N	K ₂ O		
Control section (chemical fertilizer section)	15	25	15	10	10	283	—	15	30	10	10	10	386	—
1-ton compost plus chemical fertilizer section	15	25	15	10	10	—	1,000	15	30	10	10	10	—	1,000
1-ton compost without chemical fertilizer section	—	25	15	—	10	—	1,000	—	30	10	—	10	—	1,000
1-ton heaped compost section	15	25	15	10	10	—	1,000	15	30	10	10	10	—	—

Note: 1 ton of dry compost contains 23 kg of N and 32 kg of P₂O₅, and 1 ton of dry heaped compost contains 22 kg of N and 35 kg of P₂O₅.

Table 5 Yields from the test sections

Crop and survey item Section	Japanese radish spring crop (5th crop)				Cabbage for autumn crop (6th crop)			
	Total weight per plant	Per plant total weight index	Root weight per plant	Per plant root weight index	Total weight per plant	Per plant total weight index	Head weight per plant	Per plant head weight index
Chemical fertilizer section	910 g	100	343 g	100	2.61 kg	100	1.51 kg	100
1-ton compost plus chemical fertilizer section	845	93	332	97	2.59	99	1.47	97
1-ton compost without nitrogen chemical fertilizer section	788	89	372	108	2.13	82	1.17	77
1-ton heaped compost section	942	104	365	106	2.53	97	1.38	91

Table 6 Content of heavy metals, etc. in edible parts of crops (ppm)

Chemical composition		(1980)							
Crop		NO ₃	As	Pb	Cd	Zn	Cu	Cr	Hg
Japanese radish for spring crop (5th crop)	Chemical	1,890	less than 0.1	less than 0.1	less than 0.1	1.63	0.37	less than 0.1	less than 0.01
	1-ton compost plus chemical fertilizer section	1,970	"	"	"	1.73	0.27	"	"
	1-ton compost without nitrogen chemical fertilizer section	1,890	"	"	"	1.30	0.33	"	"
	1-ton heaped compost section	2,080	"	"	"	1.77	0.37	"	"
Cabbage for autumn crop (6th crop)	Chemical fertilizer section (control section)	1,440	less than 0.1	less than 0.1	less than 0.1	1.77	0.57	less than 0.1	less than 0.01
	1-ton compost plus chemical fertilizer section	1,490	"	"	"	1.67	0.53	"	"
	1-ton compost without nitrogen chemical fertilizer section	830	"	"	"	1.57	0.40	"	"
	1-ton heaped compost section	1,260	"	"	"	1.80	0.27	"	"

Table 7 Transition of yield index during farm tests

Section \ Crop and year of survey	1978		1979		1980	
	Japanese radish spring crop	Cabbage autumn crop	Japanese radish spring crop	Cabbage autumn crop	Japanese radish spring crop	Cabbage autumn crop
Chemical fertilizer section	100	100	100	100	100	100
1-ton compost plus chemical fertilizer section	109	102	90	104	97	97
1-ton compost without nitrogen chemical fertilizer section	103	107	86	108	108	77
1-ton heaped compost section	96	96	96	93	106	91

The results of the first two years of operation show that all the crops but the third spring crop of Japanese radishes yielded more than those grown on chemical fertilizer alone, and that the use of compost to supplant nitrogen fertilizer also produced a high yield.

The test results for the year 1980 show that the compost farms were a little inferior in yield to standard chemical fertilizer farms.

It is inferred that this decline in yield may have been invited by the lime contained in compost, and that the yield will be increased if the application of compost is reduced. When the heaped compost was used, all the crops but the spring crop of Japanese radishes in 1980 were lower than the chemical fertilizer-grown counterparts, and a further study will be needed in this respect.

The intake of heavy metals by the plants was studied. In the first year of compost application, the concentrations of copper and zinc in the crops rose a little. In the second year, however, there was no noticeable rise in heavy metals such as those shown in Table 6. The concentration of nitrate rose a little, but fell off when nitrogen fertilizer was eliminated. (Refer to Table 6)

An analysis of soil from fallow land showed no accumulation of copper, zinc and nickel. As regards the nutrient balance of soil, the application of compost gradually increased exchangeable calcium and potassium, but decreased magnesium; namely, the magnesium/potassium ratio and the magnesium/calcium ratio became low, making the soil susceptible to magnesium

deficiency. In future, measures will have to be provided to supplement magnesium.

According to experiments on spinach, it was found that N and P in compost showed an effectiveness of 50 to 60% and about 30% respectively as compared with the corresponding chemical fertilizers.

Namely, a ton of dry compost per 10 a is expected have an effect equivalent to about 10 kg of nitrogen fertilizer and about 10 kg of phosphate fertilizer. As the compost is destitute of potassium, potassium must be made up in the form of chemical fertilizer.

b. Application of compost to orchards

A compost application test was conducted on Japanese pears, Japanese apricots, and Japanese chestnuts in the Agricultural Experiment Station and also on Japanese pears grown by commissioned farmers.

In the Agricultural Experiment Station, three farms were prepared: a control farm using chemical fertilizer only; a pH-conditioned farm using chemical fertilizer and calcium carbonate; and a compost farm using 2 tons of compost per 10 a and chemical fertilizer to supplement nitrogen and potassium.

The commissioned farmers used a control farm using chemical fertilizer, and a compost farm using 2 tons of compost per 10 a. The fertilizing effect of compost on each tree species was correlated with the yield index as shown in Figure 2.

(Refer to Fig. 2)

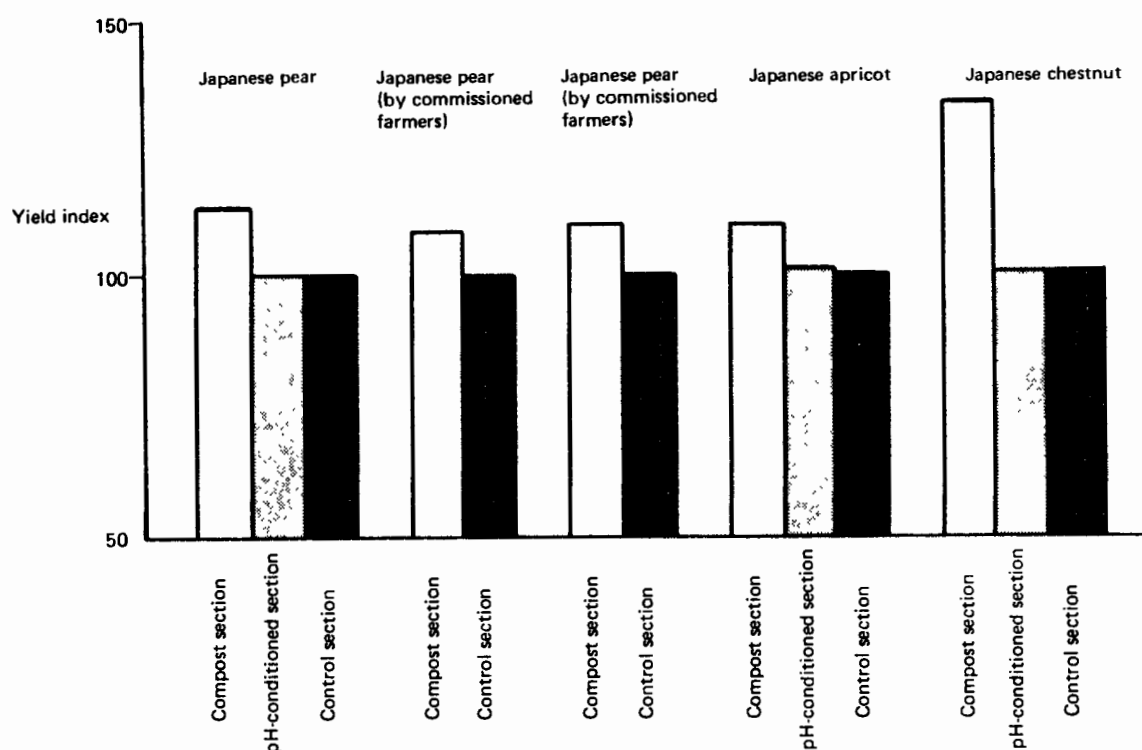


Fig. 2 Fertilizing effect of compost

The crop of Japanese chestnuts increased by about 30%, followed by Japanese pears and Japanese apricots with an increase of about 10% each. The application of compost brought about no change in the nitrogen concentration in the leaves of Japanese chestnuts, but a slight increase in nitrogen concentration was noticed in the leaves of Japanese pears and Japanese apricots, suggesting that too much nitrogen was supplied.

If the compost application is moderated, the yield will be increased more.

As regards the quality of fruits, the application of compost slightly increased N, P, Ca, and Zn in Japanese apricots, but the concentration levels of other inorganic substances, heavy metals, nitrate, etc. remained the same as they were before application of compost. The total amount of sugar, glucose, sorbitol, and sucrose, etc. tended to rise, improving the quality of the fruits.

The same tendency was seen in the Japanese chestnuts; the total amount of sugar, fructose, and glucose increased

while sucrose and starch declined, improving the commercial value of chestnuts as a whole. As regards Japanese pears, however, no substantial improvement in quality was noticed. The quality of Japanese pears will be improved by reducing the supply of nitrogen, however. An analysis of toxic substances, such as heavy metals, contained in the pears is shown in Table 8. (Refer to Table 8)

Table 8 Chemical analysis of Japanese pear

Section	(in %)					(in ppm)							
	N	P	K	Ca	Mg	Pb	Cu	Zn	Cr	Cd	Hg	As	Nitrate
Compost section	0.397	0.042	0.900	0.034	0.042	0.1	0.4	0.7	less than 0.1	less than 0.1	less than 0.1	less than 0.1	6
pH-conditioned section	0.393	0.037	0.766	0.033	0.034	-	-	-	-	-	-	-	-
Control section	0.439	0.039	0.888	0.031	0.041	0.1	0.3	0.5	less than 0.1	less than 0.1	less than 0.1	less than 0.1	8

2.2.4 Use of Compost by Farmers

The farmers in Tokyo were entrusted with the task of growing vegetables such as cabbage, spinach, and cauliflower on compost. In the main, the compost worked well, and the opinions of the commissioned farmers were favorable.

The results of the growth of cabbage and spinach on compost are given below.

a. Cabbage

Compost was applied to a farm overworked by twice annual cropping for more than 20 years, and its effects on cabbage growth were studied.

The farm was divided into three sections: A control section using chemical fertilizer alone; a 1-ton compost section using a ton of compost per 10 a, together with chemical fertilizer; and a 2-ton compost section using 2 tons of compost. The yields and the incidence of plant diseases are shown in Table 9. (Refer to Table 9)

Table 9 Survey of yields and incidence of diseases

Section	(a) Yield			(b) Incidence of black rot					
	Per head average weight	Total weight per 10 a	Index	Extremely high	High	Medium	Low	None	Damage index
Control section	1.33 kg	5,990 kg	100	16	24	8	2	0	77
1-ton compost section	1.44	6,460	108	0	3	10	36	1	33
2-ton compost section	1.35	6,050	101	1	4	8	35	2	34

Notes: (1) In the yield survey, 20 heads were sampled from the middle row of each section and weighed.
 (2) The morbidity was investigated by sampling 50 heads from the middle row of each section according to the following criteria.
 Extremely high: : Head affected
 High : Leaves affected
 Medium : All the external leaves affected
 Low : Part of external leaves affected
 None : No part affected

As is clear from Table 9, the 1-ton compost section showed the highest yield, and the compost sections were generally lower in the incidence of black rot than the control section, warranting the application of compost.

b. Spinach

A farm debilitated by overcropping and oversupply of chemical fertilizer was selected in order to examine the effects of compost on spinach. Just as with cabbage, the farm was divided into:

A control section; a 1-ton compost section using a ton of compost per 10 a, together with chemical fertilizer; and a 2-ton compost section using 2 tons of compost per 10 a.

The growth conditions and yields were investigated. The results are shown in Table 10. (Refer to Table 10)

Table 10 An example of investigation of agricultural use for spinach

	Investigation of growth and harvest (1st harvest)					
	Study of growth			Study of harvest		
	Length of blade	Width of blade	Number of blade	Total weight	Adjusted weight	ratio
Control farm conditioned with chemical fertilizer	21.5 (cm)	4.9 (cm)	6.3 (blades)	0.85 (kg)	0.74 (kg)	100
Farm using compost (1 ton per 1000 m ²)	24.0	5.3	7.3	1.45	1.29	174
Farm using compost (2 tons per 1000 m ²)	22.3	5.4	7.8	1.35	1.22	165

	Investigation of growth and harvest (2nd harvest)					
	Study of growth			Study of harvest		
	Length of blade	Width of blade	Number of blade	Total weight	Adjusted weight	Ratio
Control farm conditioned with chemical fertilizer	22.1 (cm)	7.2 (cm)	10.4 (blades)	0.80 (kg)	0.71 (kg)	100
Farm using compost (1 ton per 1000 m ²)	24.2	7.5	10.2	1.00	0.91	128
Farm using compost (2 tons per 1000 m ²)	23.2	7.3	9.9	0.95	0.85	120

As demonstrated in Table 10, the compost sections outperformed the control section in both growth and yield.

In the 1-ton compost section, the first crop showed a yield increase of 74% in terms of adjusted weight, and the second crop 28%. The quality of the crops raised in the compost sections was also better.

An analysis of fallow soil indicated that the soil which was pH 5.12 before application of compost was improved to pH 5.61 in the 1-ton compost section and to pH 6.28 in the 2-ton compost section, proving that compost is highly effective in improving acid soils.

The rise in electric conductivity was also low. It is therefore concluded that this degree of compost application will not aggravate the concentration hazards.

2.2.5 Quality and Application Standards for Compost

According to the results of the compost application tests detailed in the foregoing, a quality standard and an application standard were formulated. 20 kg bags of compost are labeled with these standards as follows.

a. Quality and safety

The compost is quite safe from arsenic, cadmium, mercury and other heavy metals, (The quality of the compost is as shown in Table 11) (Refer to Table 11)

Table 11 Compost quality labeling

Nitrogen	Phosphate	Lime	pH	Organic matter	Moisture content
1 ~ 2 %	2 ~ 3%	16 ~ 18%	7 ~ 8	20 ~ 30 %	30 ~ 35 %

b. Features and usage

This fertilizer is made of dewatered domestic wastewater sludge of the Tama New Town, fermented for about 10 days. It contains lime and nitrogen, and is effective as a fertilizer and a pH improver.

- i. Highly effective in curing acidosis, a ton of this fertilizer is equivalent to 300 kg of calcium carbonate.
- ii. The nitrogen component contained in a ton of this fertilizer is equivalent to about 50 kg of ammonium sulfate. This

fertilizer retains fertilizing effectiveness for an extended period.

iii. This fertilizer contains no potassium.

Supplement potassium as required.

iv. This fertilizer contains organic substances by 20 to 30%, and should preferably be used in combination with manure or other organic fertilizer for enhanced fertilizing efficiency.

v. Apply this fertilizer at a rate of within 1 ton per 10 a. Mix well with soil, and wait several days before planting.

vi. If you have any questions, do not hesitate to contact your local Agricultural Improvement and Diffusion Center.

3. PRODUCTION OF ARTIFICIAL LIGHTWEIGHT AGGREGATE, USING SEWAGE SLUDGE ASH

So far, the Tokyo Metropolitan Government has disposed of sewage sludge by dumping. But available dump sites have been decreasing rapidly. In addition, the dump sites are problematic from the viewpoint of environmental safety. Concerned over these problems, the Tokyo Metropolitan Government has promoted a project for disposal of sewage sludge in the form of ash.

In the future, the installed capacity of incinerators will be increased, and the volume of ash to be generated will accordingly rise. A survey was thus conducted to determine ways in which the ash could be utilized.

After evaluating the various conceivable products from various angles, including the energy necessary for production, added values of products, potential demand and environmental safety, the production of artificial lightweight aggregate was considered one of the quickest and simplest approaches to recycling of sewage sludge ash. Outlined below are the perspective of the industrial manufacturing through the development of this manufacturing technique.

3.1 ARTIFICIAL LIGHTWEIGHT AGGREGATE

The artificial lightweight aggregate currently available on the market is made of sedimentary rock or shale either crushed to appropriate sizes or pulverized and pelletized, and then sintered at 1,000°C to about 1,200°C.

Shale is also called mudstone, and is an incomplete agglomeration of clay. It is mainly composed of silicon and aluminum, and contains

volatile matter that is dissipated at high temperatures. It is light and fragile.

When shale is sintered under proper conditions, silicon and other elements become semi-fused and softened, and at the same time, volatile matter is gasified to make the stone intumescence.

When cooled, the pieces of shale become a hard and tough aggregate, with a finely-textured glazed shell on the outside and a finely pored nest inside. (Refer to Photograph)

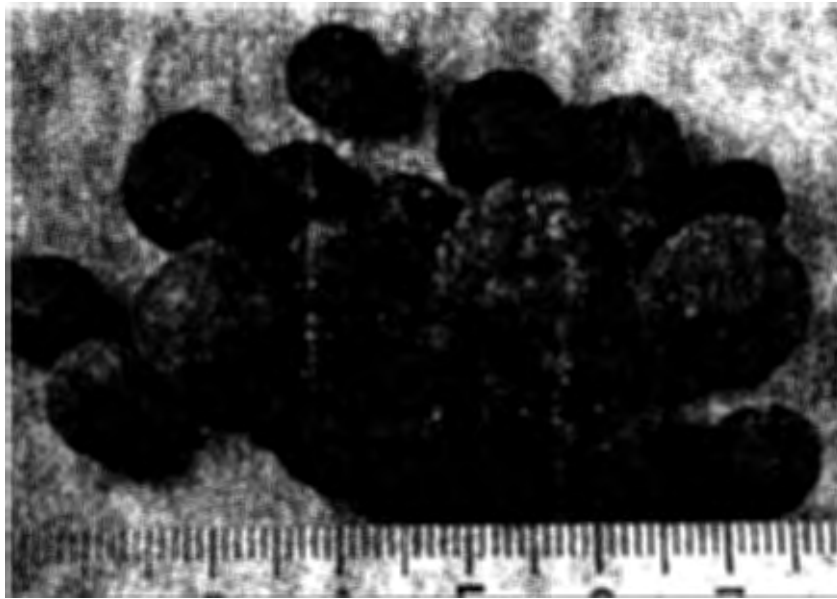


Photo. Artificial lightweight aggregate made of sludge ash
(independent pores are seen inside.)

The specific gravity of this artificial lightweight aggregate is about 1.4, or about half that of natural aggregate.

3.2 USE OF SEWAGE SLUDGE ASH AS ARTIFICIAL LIGHTWEIGHT AGGREGATE

Compared with shale, the chemical composition of the sludge ash varies widely depending on the types and quantity of coagulants injected for the purpose of dewatering the sludge.

This is particularly so in the case of the ash of sludge cake dewatered with inorganic coagulants, which shows a high variation in the lime content. Compared with shale, the ash of sludge cake dewatered with polymer or by heat treatment or by freezing and defrosting, contains more lime and ferric oxide, but less silicon. The amount of volatile matter that serves as a foaming agent is less in the ash than in shale. (Refer to Table 12)

Table 12 Chemical composition of samples (%)

Chemical composition	Ash A (Coagulant: Lime and iron chloride)	Ash B (Coagulant: Polymer)	Expansive shale (ore available from company A)	Expansive shale (ore available from company B)	Bentonite
SiO ₂	29.86	55.6	64.30	62.14	71.07
P ₂ O ₅	5.41	2.51	—	—	—
Al ₂ O ₃	11.18	18.48	16.34	15.42	14.21
Fe ₂ O ₃	11.09	10.48	1.36	3.07	1.60
FeO	1.22	1.84	3.23	2.30	0.04
CaO	25.19	4.28	1.20	2.46	1.52
MgO	1.86	2.67	1.23	2.04	2.30
K ₂ O	0.89	1.64	2.55	1.74	0.44
Na ₂ O	0.81	1.54	1.94	1.59	2.16
SO ₃	2.92	0.69	0.65	0.10	0.01
Cl	0.59	—	—	—	—
C	1.08	0.77	0.66	0.27	—
CO ₂	3.51	0.13	1.75	0.61	—
Ign-loss	5.60	1.67	7.34	8.74	5.94

Note: "—" denotes less than 0.00.

In general, it was surmised that it would be feasible to convert sludge ash obtained without lime injection into artificial lightweight aggregate.

3.3 SINTERING OF SLUDGE ASH INTO ARTIFICIAL LIGHTWEIGHT AGGREGATE

Sludge ash containing large amounts of lime shows a high melting point, and its fusion shows a low viscosity, whereas sludge ash with less lime has almost the same melting temperature and viscosity as shale.

So far as the practical sintering temperature (approx. 1,150°C) is concerned, the lower the lime content in the sludge ash, the better. What makes the sludge ash quite different from shale is the fact that the viscosity of a dough made up of sludge ash powder and water is far below that of shale, and the strength of dry pellets is far below that of shale.

Namely, the pellets are turned into powder as they cannot hold their shapes when sintered in the kiln. It is therefore necessary to use a proper binder for the purpose of increasing the strength of pellets. The sludge ash is deficient in volatile matter either in the form of organic or inorganic substance as it has been sintered at a temperature of 800°C to 900°C, and is not able to intumesce enough at heat.

To counteract these problems, it is necessary to select an additive which serves not only as a binder but also as a foaming agent. Shale and bentonite combine the binding power with the foaming power, and can solve these problems.

When bentonite or shale is used as a foaming binder, the relationship between the process flow (Figure 3) and production requirements is as follows. (Refer to Fig. 3)

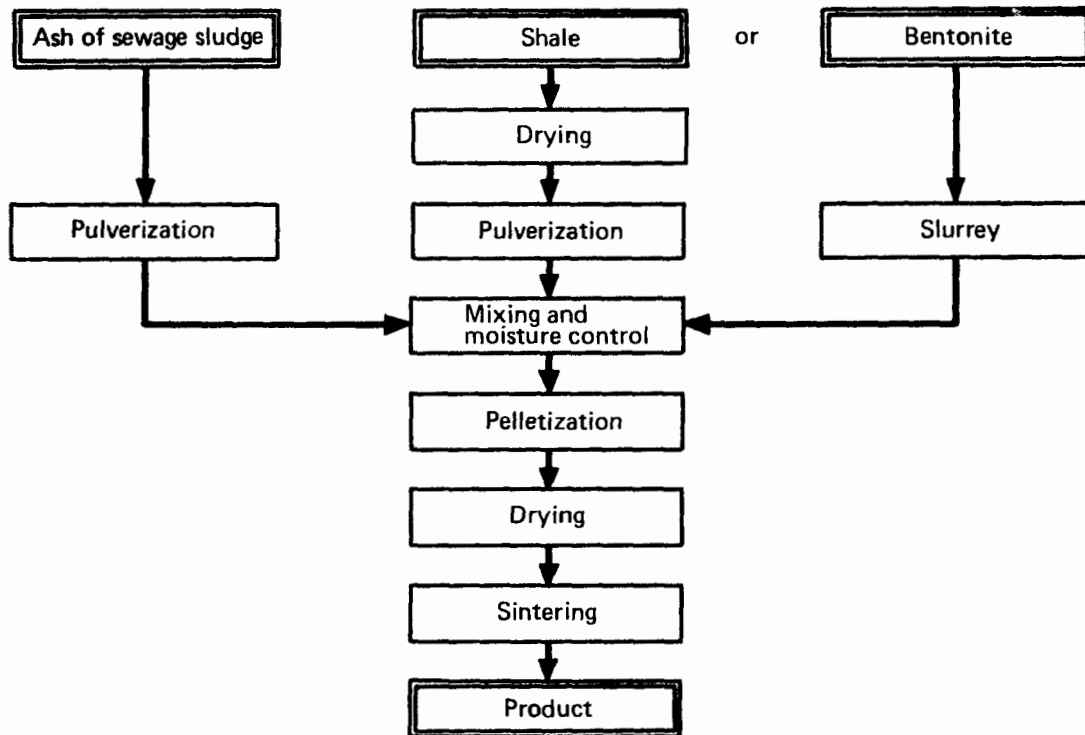


Fig. 3 Process flow of artificial lightweight aggregate

a. When bentonite is used:

When added with water, bentonite becomes highly adhesive, making the pelletizing of sludge ash easy. When sintered, the pellets puff up. Both intumescibility and distensibility increase with increase in the addition of bentonite as shown in Figure 4.

(Refer to Fig. 4)

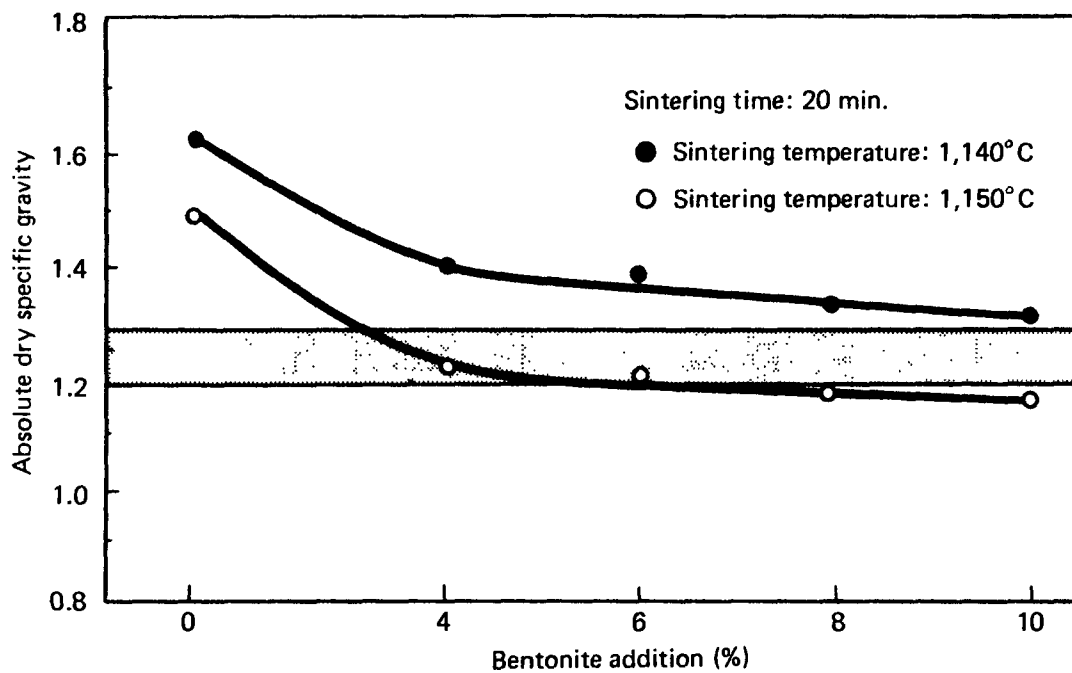


Fig. 4 Relationship between bentonite addition and absolute dry specific gravity

The best results are obtained under the following conditions.

- . Sintering temperature : 1,140°C to 1,150°C
- . Sintering time : 15 to 20 min.
- . Addition of bentonite : 4 to 6%

b. When shale is used:

Bentonite is costly, and can be substituted satisfactorily by shale (expansible shale) which has long been the material for artificial lightweight aggregate. When the addition of shale is increased, the specific gravity of sintered pellets becomes small as shown in Figure 5. (Refer to Fig. 5)

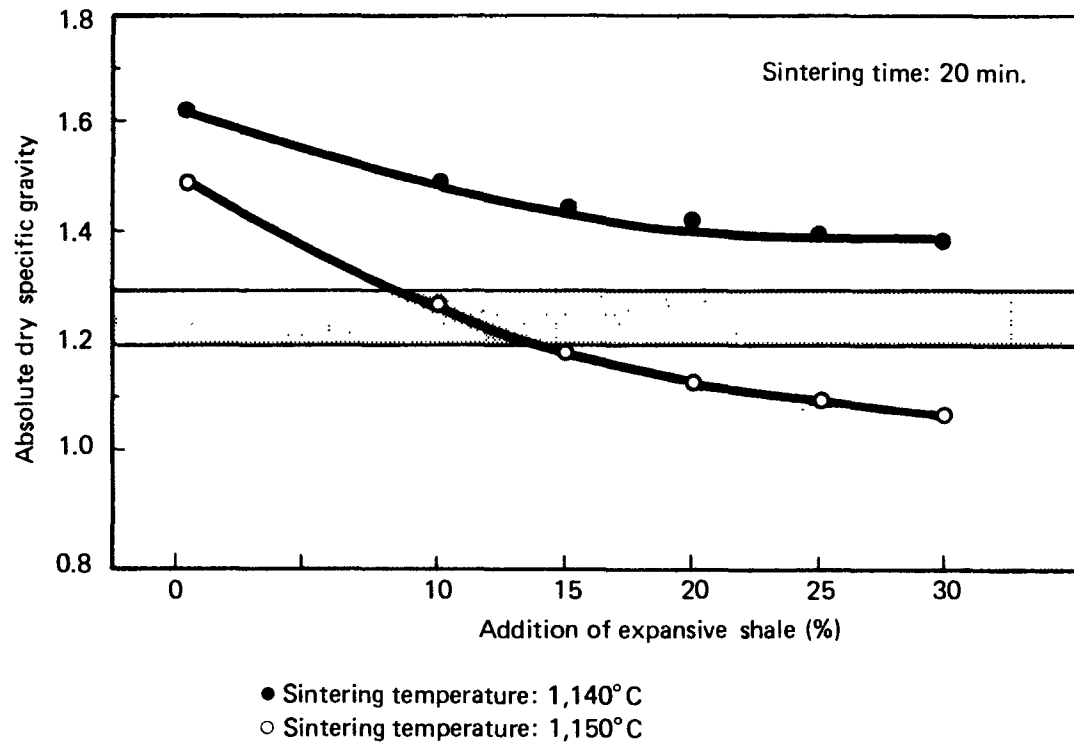


Fig. 5 Relationship between addition of expansive shale and absolute dry specific gravity

With the addition of shale fixed at 15% and 20% and the sintering temperature fixed at 1,150°C, the specific gravity decreasing with sintering time as shown in Figure 6. (Refer to Fig. 6)

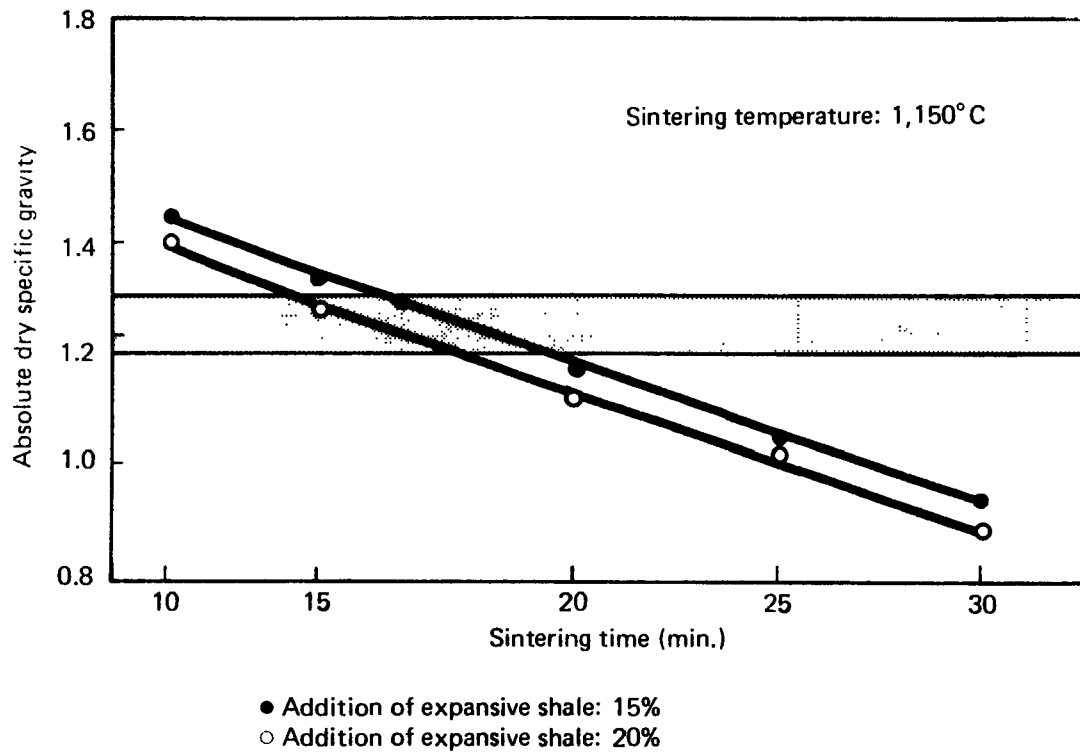


Fig. 6 Relationship between sintering time and absolute dry specific gravity of expansive shale-added pellets

A similar trend is also seen in bentonite as shown in Fig. 7.
(Refer to Fig. 7)

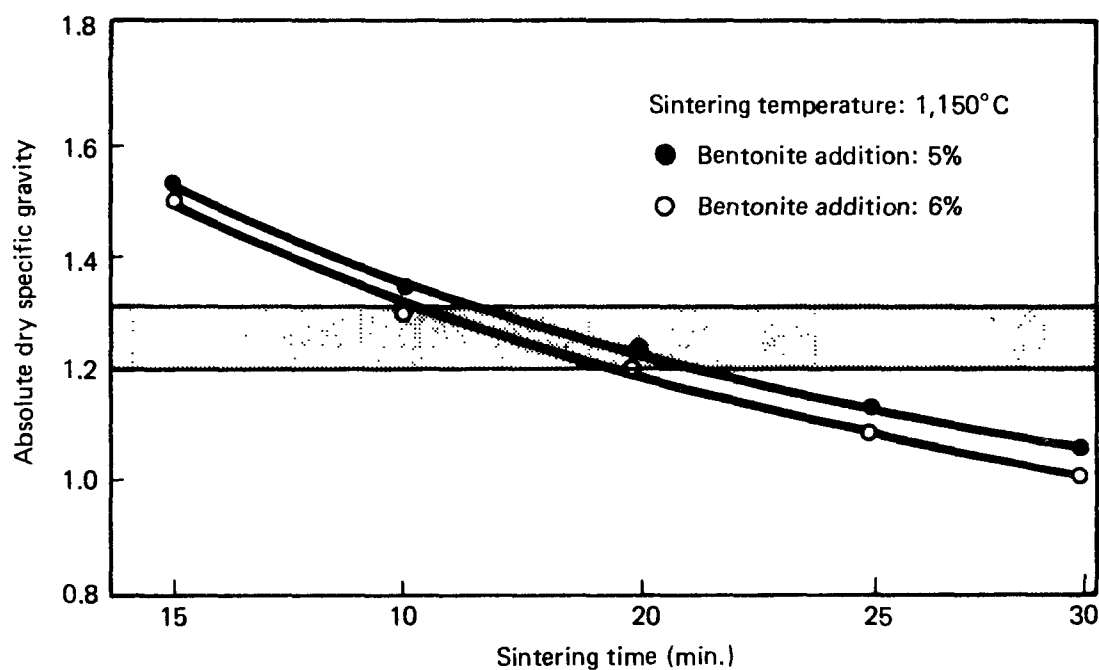


Fig. 7 Relationship between bentonite addition, sintering time and absolute dry specific gravity

The quality of sinter and the physical test results of concrete are as shown in Table 13, 14, and 15.

(Refer to Table 13, Table 14 and Table 15)

Table 13 Physical properties of artificial lightweight aggregate

Type Characteristics	Fine aggregate	Coarse aggregate		
	Commercially available fine aggregate	Ash aggregate (L _A)	Commercially available pelletized aggregate (L _B)	Commercially available non-pelletized aggregate
Nominal dry specific gravity	1.85	1.26	1.33	1.47
Absolute dry specific gravity	1.69	1.24	1.23	1.36
Hygroscopicity (%)	9.9	1.3	8.1	8.4
Unit weight (kg/m ³)	1,079	789	806	881
Solid volume percentage (%)	—	63.6	65.5	64.8
Ratio of floating particles in coarse aggregate (%)	—	1.7	0.3	1.8
10% crushing value* (tons)	—	9.1	13.1	16.2
Aggregate shock value (%)	—	35.4	33.8	25.4
Wear loss (%)	—	28.4	25.5	18.2

*: Load required to crush 10% of aggregate.

Table 14 Concrete mix proportions

Type	Maximum size (mm)	Slump (cm)	Air volume (%)	Water to cement ratio (W/C %)	Unit water weight (kg)	Unit cement weight (kg)	Sand percentage (%)	Fine aggregate weight (kg)	Coarse aggregate weight (kg)	Admixture (kg)	Unit weight (kg/m ³)
L _A	15	7.5	6.5	40	149	373	46.0	585	462	0.933	1632
	15	5.5	7.2	50	149	298	47.0	618	468	0.746	1578
	15	5.5	8.3	60	150	250	50.5	678	446	0.625	1552
L _B	15	14.0	8.2	40	150	375	44.0	558	549	0.938	1647
	15	8.5	7.1	50	150	300	46.5	611	543	0.750	1619
	15	7.5	10.1	60	150	250	48.5	652	535	0.625	1581

Table 15 Strength, elastic modulus and strength ratio

Item	W/C %	Slump cm	Air volume %	L _A aggregate concrete			W/C %	Slump cm	Air volume %	L _B aggregate concrete			Strength ratio, L _A /L _B		
				7 days	28 days	91 days				7 days	28 days	91 days	7 days	28 days	91 days
Compression strength kgf/cm ²	40	7.5	6.5	278	375	424	40	14.0	6.2	274	395	448	1.01	0.95	0.95
	50	5.5	7.2	150	240	294	50	6.5	7.1	215	318	400	0.70	0.78	0.74
	60	5.5	8.3	98	169	233	60	7.5	10.1	110	169	210	0.89	1.00	1.11
Bending strength kgf/cm ²	40	7.5	6.5	—	46.2	52.7	40	14.0	6.2	—	47.3	52.6	—	0.98	1.00
	50	5.5	7.2	—	42.4	51.7	50	6.5	7.1	—	43.2	49.0	—	0.98	1.06
	60	5.5	8.3	—	30.8	39.1	60	7.5	10.1	—	29.4	34.8	—	1.04	1.12
Tensile strength kgf/cm ²	40	7.5	6.5	—	30.7	31.1	40	14.0	6.2	—	28.6	31.8	—	1.07	0.98
	50	5.5	7.2	—	24.5	29.7	50	6.5	7.1	—	27.2	28.8	—	0.97	1.03
	60	5.5	8.3	—	18.4	23.6	60	7.5	10.1	—	21.2	21.7	—	0.86	1.09
Static modulus of elasticity x10 ⁴ kgf/cm ²	40	7.5	6.5	—	15.3	—	40	14.0	6.2	—	13.8	—	—	1.13	—
	50	5.5	7.2	—	13.5	—	50	6.5	7.1	—	13.4	—	—	1.01	—
	60	5.5	8.3	—	12.5	—	60	7.5	10.1	—	11.9	—	—	1.05	—
Dynamic modulus of elasticity x10 ⁴ kgf/cm ²	40	7.5	6.5	—	17.8	—	40	14.0	6.2	—	17.0	—	—	1.05	—
	50	5.5	7.2	—	15.8	—	50	6.5	7.1	—	14.4	—	—	1.10	—
	60	5.5	8.3	—	13.4	—	60	7.5	10.1	—	13.2	—	—	1.02	—

c. Others

As previously mentioned, the ash of sewage sludge contains more lime and ferric oxide than shale. These compounds reduce the viscosity of the vitreous phase, narrowing the sintering temperature range. But when shale and sludge ash are pulverized to finer than 200 meshes, the mix proportion of shale can be reduced to as small as 30% without sacrificing the sintering temperature stability. In addition, the variation in specific gravity and strength of the sinter decreases.

If unpulverized sludge ash is used, it is difficult to industrially produce merchandizable sinter unless the mix proportion of shale is increased to as much as about 70%.

These results have been obtained through sintering tests using a pilot kiln and a production kiln.

3.4 PROPOSED RESEARCH

Thanks to the current state of the art in sintering, the industrial production of artificial lightweight aggregate from sludge ash has now become possible. Many technical improvements for the purpose of saving energy are still required, however. The waste heat developed in the system must be recovered to a maximum extent; it will be used effectively to dry pellets containing a great amount of water. In addition, the sintering method must be improved and the amount of additives must be reduced.

Although the efforts have so far been expended to study the conversion of sludge ash into artificial lightweight aggregate, the high calorific value of sludge cake may make it possible to develop a system in which the incineration of sludge and the production of artificial lightweight aggregate can be carried out concurrently.

In view of this, we are planning to study processes that will directly turn sludge cake to good account.

The recycling of sewage sludge within a city will eliminate the problem of transporting sludge out of the city and at the same time will reduce the amount of resources to be brought into the city from outside. Sewage sludge may become a valuable resource if developed. It may be used as a material for slag cement and slag brick, etc. We would like to gradually implement sludge recovery measures which will be feasible and viable not only at present but in the future.

Eighth US/JAPAN Conference
on
Sewage Treatment Technology

EXPERIMENT ON INCINERATION OF MUNICIPAL REFUSE AND SEWAGE SLUDGE IN KYOTO CITY

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1. PREFACE

1.1 CURRENT STATUS IN KYOTO CITY

Kyoto City is located on the middle of the Yodo River and is an inland city surrounded by hills on three sides with a population of 1,460,000. The Yodo River is water resources for about 11 million people downstream in the Osaka-Kobe district. Consequently, the construction of sewer system in this city is important for the preservation of water quality in the Yodo River. The sewer network of Kyoto City covers 60.2% of its total residents as of April 1981. Influent to the wastewater treatment plants is discharged into the Yodo River through the secondary treatment. All of sludge produced is thickened, partly digested, and dewatered. For the volume reduction and stabilization, dewatered cake is all incinerated and landfilled on the municipal disposal site.

The current ash disposal site has a limited area of about 4 ha for landfill. It is difficult to secure the new site. With the progress of the sewerage construction, the quantity of sludge produced is expected to increase more and more in the future. So the treatment and disposal of sludge have become an important problem.

1.2 IDEA OF COMBINATION OF WASTEWATER TREATMENT PLANT WITH MUNICIPAL REFUSE INCINERATION PLANT

Wastewater and refuse are representatives of municipal wastes, whose treatment and disposal consumes various types of energy enormously. The energy flow in the treatment system for wastewater and municipal refuse is shown in Fig. 1.

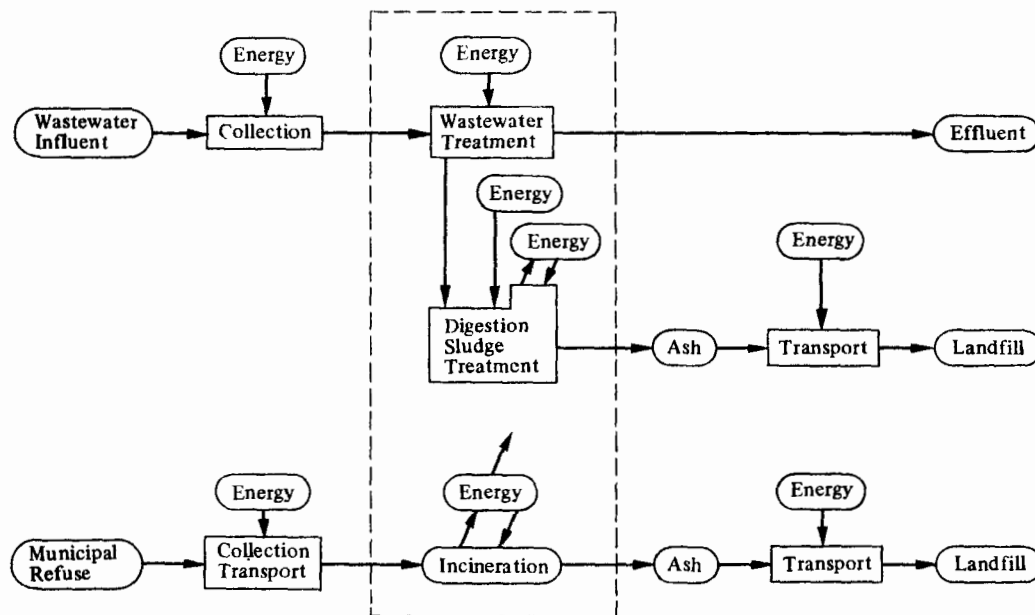


Fig. 1 Energy Flow in Treatment of Municipal Wastewater and Refuse

Note the wastewater treatment plant and the municipal refuse incineration plant in the dot line in Fig. 1. The former plant consumes electric power for blower, pump and other machines in the wastewater treatment process and for various machines in the sludge thickening, dewatering and incineration processes. Heavy oil is also utilized mainly as auxiliary fuel for incinerator. In case that digestion process is well operated, it is possible to recover energy as digestion gas, which is used both for heating the digestion tank and for auxiliary fuel of incinerator.

The total energy consumption at the wastewater treatment plants in Kyoto City (converted into electric power) was about 0.66 kWh/m³ influent in fiscal 1980. Digestion gas covered about 18% of the entire energy consumption. As mentioned above, a huge quantity of energy is required for pumping, wastewater treatment and sludge treatment at the plant. It may be called an energy-consuming plant.

On the other hand, the low heat value of municipal refuse is about 1,500 Kcal/kg at the municipal refuse incineration plant. About 180 kWh/ton refuse is generated in general with thermal energy produced by incineration although the efficiency depends on the generation system. As electric power required at the incineration plant is about 60 kWh/ton refuse, about 120 kWh/ton refuse is surplus electric power. That is, the municipal refuse incineration plant may be called an energy producing plant.

On the assumption that the wastewater treatment plant and the municipal refuse incinerator are combined in one system as shown in Fig. 1, the latter is capable of supplying surplus energy to the former as electric power or steam. The idea of combination is illustrated in Fig. 2 with the mutual utilization of treatment facilities as well as energy.

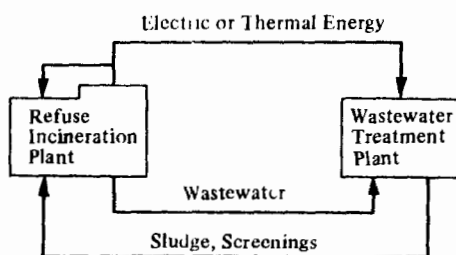


Fig. 2 Combination of Wastewater Treatment Plant and Waste Incineration Plant

About 60~70% of dry solids of sewage sludge is combustibles (organic matter). By reducing fully the water of sludge and lowering the latent heat of moisture evaporation, its low heat value will become similar to that of municipal refuse, and it will be combustible without auxiliary fuel or be incinerated with municipal refuse. On the other hand, wastewater produced in the municipal refuse incineration plant will be treated at the wastewater plant when the toxic substances like heavy metals to the biological treatment is removed in the pretreatment.

The combination of the two plants will save the construction cost and the maintenance and operation cost including energy consumption. The feasibility study of the abovementioned combination has been conducted, as a construction plan of the Ishida plant, the fourth in Kyoto and that of a municipal refuse incinerator adjacent to the former were ongoing simultaneously.

This report presents the results of investigation conducted in 1977–1979 to evaluate the feasibility of sewage sludge incineration by the municipal refuse incinerator, especially with attention to the pretreatment of sewage sludge and its characteristics.

2. CONSTRUCTION PLAN OF WASTEWATER TREATMENT PLANT AND MUNICIPAL REFUSE INCINERATION PLANT

A plan of the Ishida plant of the Sewage Works Bureau and the Higashi Refuse Incineration Plant of the Cleaning Bureau is shown in Table 1. Fig. 3 shows the layout of two plants. As the vicinity of the plants is an exclusively residential district, several pollution preventives are taken into consideration. As there has been no plant operated for coincincineration of sewage sludge and municipal refuse in Japan, the feasibility study was conducted.

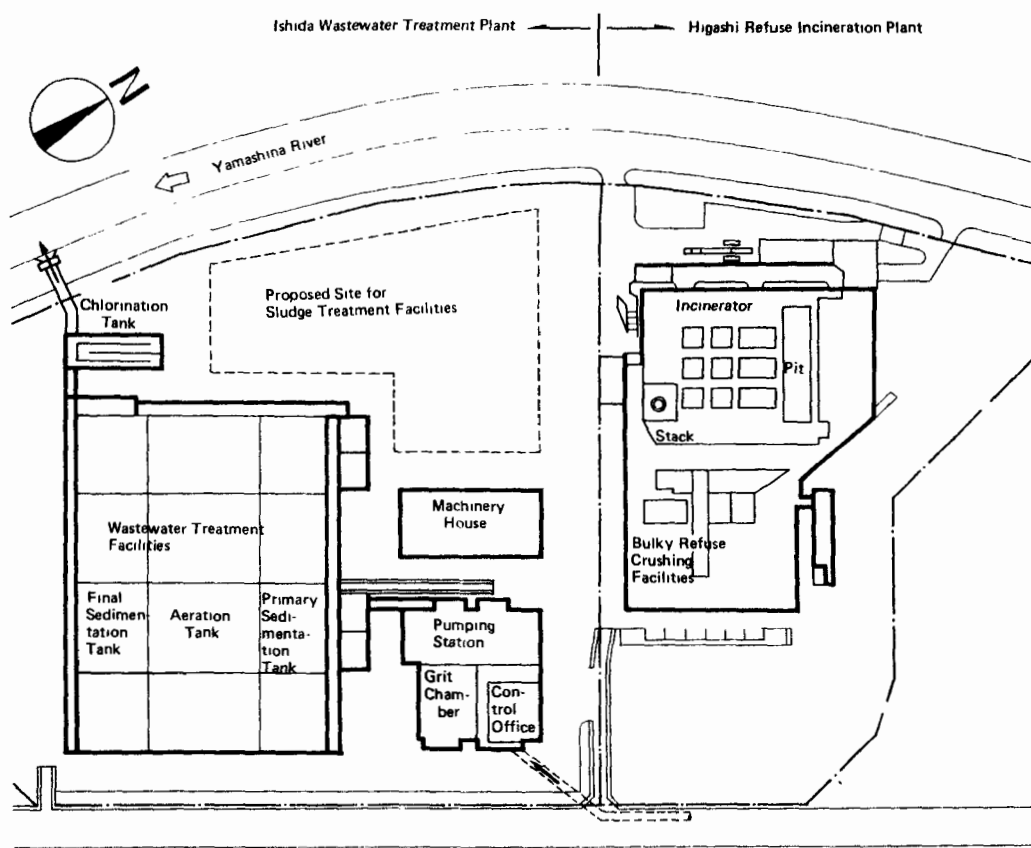


Fig. 3 Ishida Wastewater Treatment Plant and Higashi Refuse Incineration Plant

Table 1 Plans for Ishida Sewage Treatment Plant and Higashi Refuse Incineration Plant

Ishida Sewage Treatment Plant		Higashi Refuse Incineration Plant	
Plant Site Area	87,600 m ²	Plant Site Area	43,000 m ²
Design Sewer Service Area	1,984 ha	Type of Incinerator	Rotary Stoker (7 stages)
Design Population Served	230,900	Capacity	600 t/day (200 t/day × 3 units)
Design Flow of Influent (Day Max.)	163,000 m ³ /day	Burning Temperature	800~900°C (Outlet Gas)
Collection System	Separate	Low Heat Value of Refuse	1,100~2,500 kcal/kg
Wastewater Treatment Process	Conventional Activated Sludge Process	Unburned Content in Ash	Below 3% (800°C, 3 hours)
Sludge Produced	28t DS/day (VS: 70%)	Waste Heat Boiler	3 Units
Sludge Treatment Process	Incineration with Municipal Refuse	Independent Generator	4,000 kW × 2 units

3. BASIC RESEARCH AND EXPERIMENT ON COINCINERATION

3.1 CONDITIONS OF COINCINERATION EXPERIMENT

The type of a refuse incinerator was already decided as shown in Table 1 and Fig. 4 before the start of this research. Then, the following experimental conditions were given.

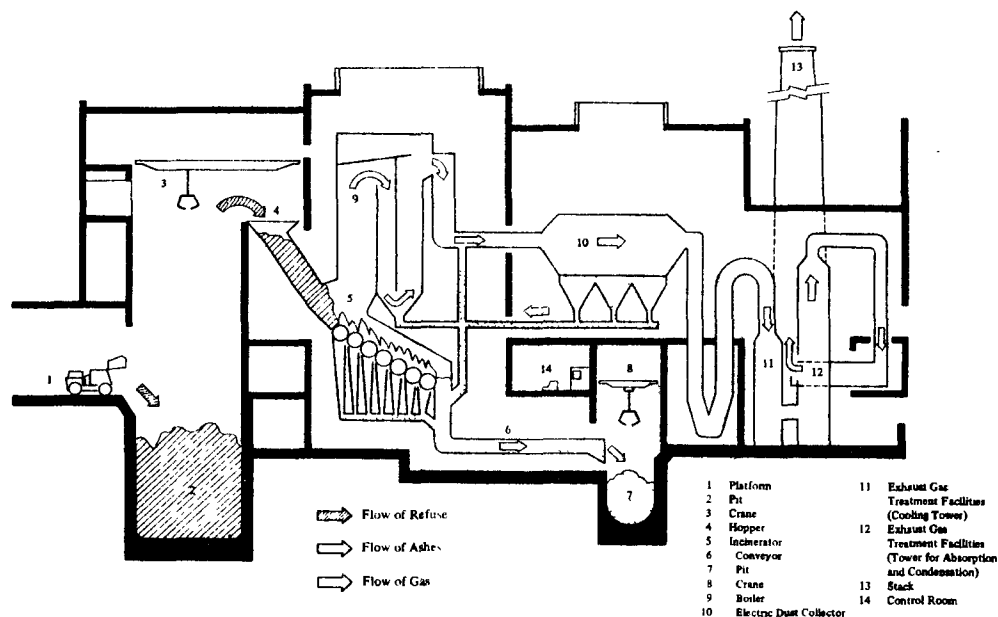


Fig. 4 Schematic of Incineration Plant

- a. The incinerator burns exclusively municipal refuse. No consideration is taken for the incineration of sewage sludge.
- b. With the incineration of sludge, no adverse effects are given to the incinerator. (On combustibility, composition of exhaust gas, amount of unburned substances and like that.)
- c. To reduce the quantity of ash and bad effects on the exhaust gas treatment facilities, lime and ferric salt are not used as sludge conditioner.
- d. The digestion process is not adopted because it reduces the heat value of sludge.
- e. Sludge is completely burned within one hour in the incinerator, particularly with care to prevent sludge from falling through the slit between rotary stokers (about 8 mm wide).
- f. For overhaul of the incinerator, the operation is shut down for a week once a year.
- g. Sludge is dried to the extent of similar low heat value to refuse (1,100~2,500 Kcal/kg sludge).

If the heat value of organic matter is assumed to be 5,000 Kcal/kg, the low heat value of sludge is expressed in the following equation:

$$H_L = \frac{a}{100} \left(1 - \frac{w}{100}\right) \times 5,000 - \frac{w}{100} \times 600$$

where, H_L : Low heat value of sludge (Kcal/kg)

a : Organic content in sludge (%)

w : Moisture content in sludge (%)

The relation between the low heat value computed by the above equation and the moisture content is shown in Fig. 5. The Figure shows clearly that the low heat value of sludge is in inverse proportion to its moisture content, and that the moisture content should be reduced by 60~25% to make sludge's low heat value similar to municipal refuse in case that organic content of sludge is 70%.

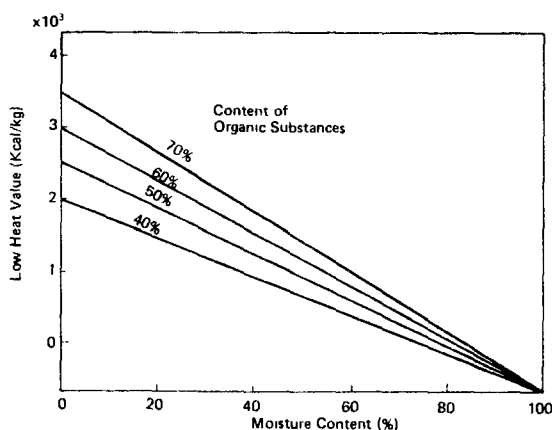


Fig. 5 Relation Between Low Heat Value and Moisture Content of Sludge

The low heat value of about 1,200 Kcal/kg is necessary for the stable incineration of municipal refuse. So the moisture content of sludge is set below 50% in this research.

3.2 ALTERNATIVES OF SLUDGE TREATMENT

Considering the conditions mentioned in Section 3.1, two processes (A) and (B) have been thought out as shown in Figs. 6 and 7 respectively, where the combination of unit processes possible for practical use at present is designed.

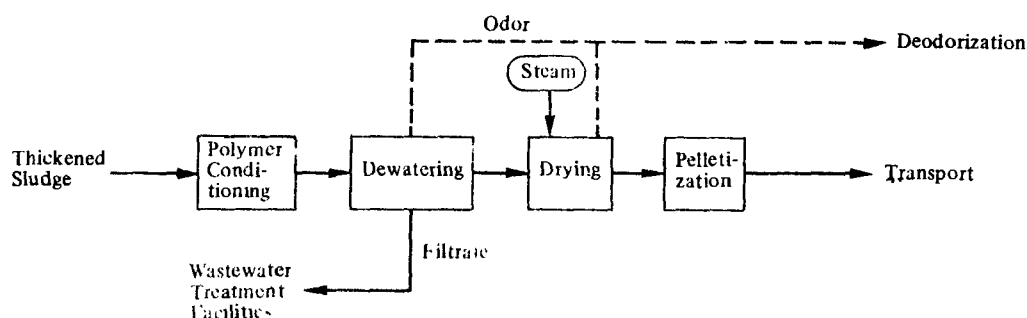


Fig. 6 Polymer Conditioning Process (A)

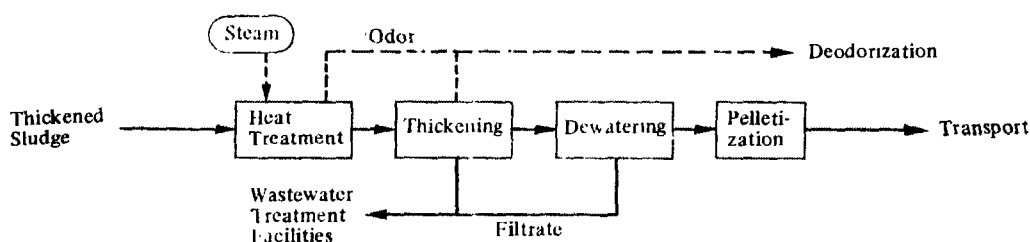


Fig. 7 Heat Treatment Process (B)

Under the condition that inorganic coagulants such as lime and ferric salts are not used, two dewatering processes seem practical: one is the dewatering of polymer-conditioned sludge and the other is that of heat-treated sludge without dosage. In the former process, the centrifuge, belt press filter or screw press are available. As the centrifuge is said to produce sludge cake of higher moisture content than two other machines, more amount of thermal energy is required in the subsequent drying process to gain the designed moisture content. That's why it has been excluded from this plan.

In the latter process, the moisture content of well heat-treated sludge cake will be less than 50% by the filter press without dosage of conditioner. Accordingly, the drying process by steam seems unnecessary.

There are three types of the drying system, indirect steam heating with paddle, steam pipe heating and direct air current heating. In this plan, the paddle type heating is adopted because the heat source is steam and the second type has a problem that sewage sludge containing much organic substance adheres to heat transmitting surface.

Although their effectiveness to sewage sludge is not confirmed, there are two types of pelletizing machines. One is by briquetting, the other is by pushing out with screw or piston. Pelletized cake is required strength not to break due to vibration and moisture not to adhere to each other on the way to the incinerator.

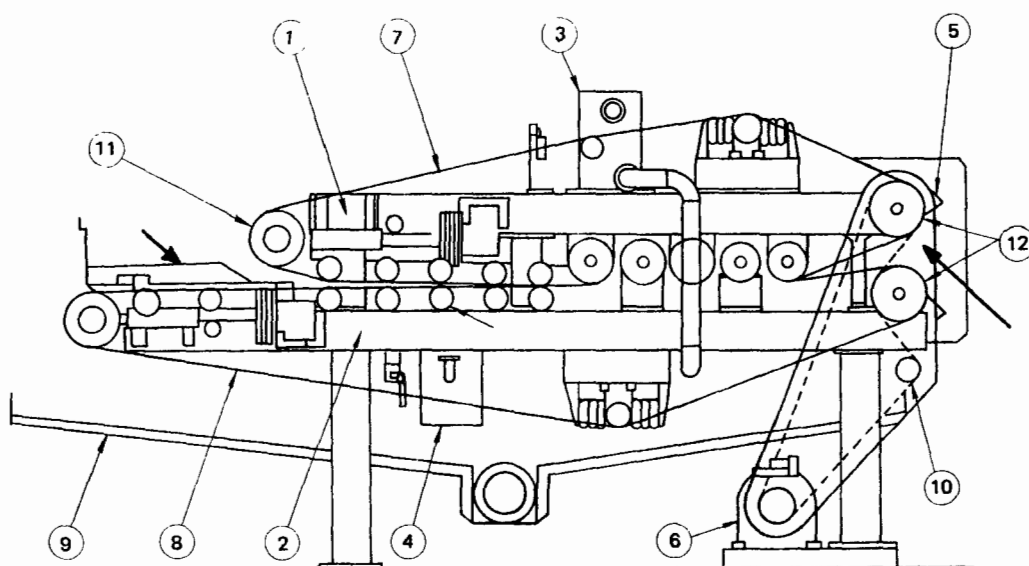
3.3 EVALUATION OF EACH UNIT PROCESS BY EXPERIMENT

3.3.1 DEWATERING

Belt press filter and screw press have been evaluated for the sludge conditioned with polymer and filter press for the heat-treated sludge.

a. Belt Press Filter

The structure of the belt press filter used in the test is shown in Fig. 8 and its belt is 50 cm wide. The typical result is indicated in Table 2. As belt press filter has been put into practical use, the result of this test is estimated to be reliable. Consequently, dewatered cake of moisture content of 70 to 75% will be produced at filter yield of 100 to 200 kg DS/m-hr with polymer dosage of 0.7 to 1.0% for the sludge of solids content 3.5% although the result depends on the kinds of sludge.



- | | |
|--|---------------------------|
| 1 Upper Frame | 7 Upper Belt |
| 2 Lower Frame | 8 Lower Belt |
| 3 Upper Washwater Unit | 9 Filtrate Drain |
| 4 Lower Washwater Unit | 10 Belt Drive Unit |
| 5 Scraper | 11 Belt Tensioning Roller |
| 6 Belt Driving Motor and Speed Reducer | 12 Belt Driving Roller |

Fig. 8 Belt Press Filter

Table 2 Dewatering Characteristics of Belt Press Filter

Experiment Number	Polymer Dosage (%/DS)	Solids Content in Sludge (%)	Sludge Feed Rate (m ³ /hr)	Belt Tension (kg/cm ²)	Moisture Content in Dewatered Cake (%)	Filter Yield (kgDS/m ² ·hr)	Solids Recovery (%)
1	0.4	3.3	2.0	3.0	68.1	137	98.6
2	0.7	3.3	2.8	3.0	71.1	192	99.3
3	1.0	3.3	2.8	3.0	70.7	192	99.1

(Note) Sludge used was mixed sludge of the Toba plant.
Polymer used was KAYAFLOC C-599-IH.

b. Screw Press

There have been still few cases of full-scale press used for dewatering sewage sludge. Polymer-conditioned sludge is put into the cylindrical screen made of the punched steel, slowly pressurized and dewatered by screw. The screw press used in the test is shown in Fig. 9. This is a pilot machine with the inside diameter of 200 mm and the length of 2,000 mm. It is able to feed steam into screw shaft and heat sludge at 40~70°C for the improvement of dewatering efficiency.

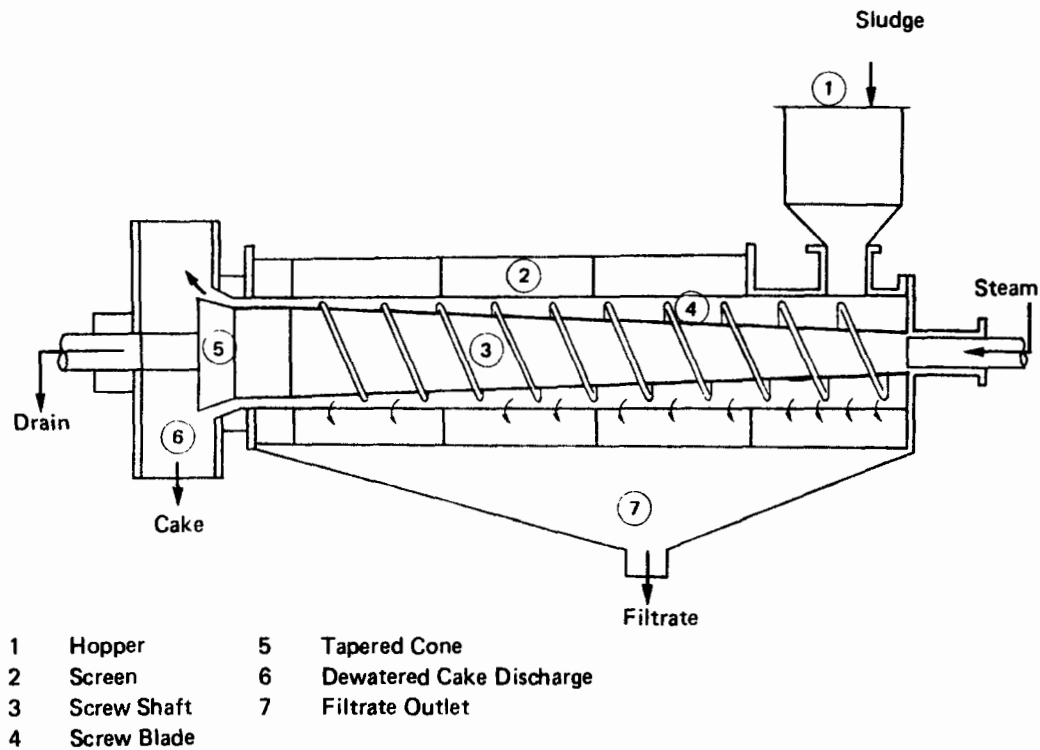


Fig. 9 Screw Press

The typical test result is indicated in Table 3. The result shows that the properties of dewatered cake was stable on any condition of heating or not and change of rotating speed. But with heating, the moisture content of dewatered cake was reduced by 6~10% and the sludge feed rate increased by 10~20% as compared with the operation without heating. When the screw press is scaled up, the moisture content of dewatered cake tends to increase. So the reasearch is required in the practical scale.

Table 3 Dewatering Characteristics of Screw Press

Experiment Number	Polymer Dosage (%/DS)	Solids Content in Sludge (%)	Sludge Feed Rate (kgDS/hr)	Rotation Speed (r.p.m.)	Heating	Steam Pressure (kg/cm ² G)	SS of Filtrate (ppm)	Solids Recovery (%)	Moisture Content in Cake (%)
1	0.3	3.9	2.8	0.08	Yes	1	1,400	95.5	56.3
2	0.3	3.9	5.7	0.16	Yes	1	590	98.3	58.1
3	0.3	3.9	10.3	0.32	Yes	1	810	97.6	65.5
4	0.3	3.9	12.3	0.48	Yes	1	1,400	95.5	69.4
5	0.3	3.9	2.1	0.08	No	0	815	96.7	65.4
6	0.3	3.9	5.1	0.16	No	0	1,010	98.1	68.1
7	0.3	3.9	9.2	0.32	No	0	1,500	95.5	72.3
8	0.3	3.9	11.0	0.48	No	0	1,400	95.2	78.0

(Note) Sludge used was mixed sludge of the Toba Plant.
Polymer used was DIAFLOC KP-201A.

c. Filter Press

In the heat treatment process studied in this test, the low temperature system (145°C~175°C) is adopted. As compared with the high temperature system (200°C), it is effective for the characteristics of filtrate and deodorization. The pilot machine with the filtration area of 0.07 m² was used to evaluate its performance of dewatering heat-treated sludge and to get the condition required for the effective cake moisture content. To evaluate the subsequent process, the dewatered cake was produced by the test machine with the filtration area of 4.0 m². The result of the dewatering test for the two machines is shown in Table 4. There are many full-scale filter presses operated for heat-treated sludge, so it may be possible to get dewatered cake of moisture content 35~50% at filter yield of 5~10 kg/m²-hr.

Table 4 Dewatering Characteristics of Filter Press

Item Filtration Area	Solids Content in Sludge (%)	Operational Conditions						Results								
		Injecting			Squeezing			Cake Removal			Moisture Content in De-watered Cake			Filter Yield (kgDS/m ² hr)		
		Time			Pressure											
		(min.)			(kg/cm ²)											
0.07 m ²		A	B	C	4	10	13	A	B	C	A	B	C	A	B	C
	6.3							Good	Good	Good	38.6	30.1	40.3	6.43	5.47	5.03
	8.1	5	10	15				Good	Good	Good	38.7	43.8	45.0	5.98	5.24	5.89
	9.6							Good	Good	Good	40.4	42.6	48.3	7.10	6.42	5.86
4.0 m ³	9.4	5			5	13	13	Good			48.7			5.82		

(Note) The sludge used was mixed sludge of the Toba Plant.

3.3.2 DRYING

The moisture content of polymer-conditioned sludge cake is about 65~75%. Such a cake has not enough to low heat value and has adverse effects on combustibility because it is soft enough to aggregate in the transport and refuse pit. To settle the problem, a test was conducted with the paddle-type indirect steam drier for drying the dewatered cake with steam produced from the waste heat boiler of the refuse incinerator. The test was conducted using sludge cake produced from the screw press at the Toba plant. The paddle-type indirect steam drier is shown in Fig. 10. The characteristics of dewatered cake used for the test are shown in Table 5 and the test result is shown in Table 6.

It has been confirmed that the continuous drying of polymer conditioned sludge was conducted well by the paddle-type indirect steam drier without the sludge adhering to the heat-transmitting surface. The particle size distribution of dried cake is changed with the change of the moisture content of dried cake as shown in Fig. 11. The appropriate pelletization is required because the result indicates that the dried cake produced by the test drier is expected to fall from between grates of incinerator.

Table 5 Characteristics of Dewatered Cake

Moisture Content (%)	Ignition Loss (%)	Heat Value (kcal/kgDS)	C (%)	H (%)	N (%)
63.6	62.9	3,660	37.9	5.7	3.7

Table 6 Drying Test of Dewatered Cake

Steam Pressure (kg/cm ²)	2.5
Steam Temperature (°C)	138
Moisture Content in Dewatered Cake (kg/hr)	63.6
Dewatered Cake Feed Rate (kg/hr)	140
Moisture Content in Dried Cake (%)	33.2
Dried Cake Discharge Rate (kg/hr)	76.3
Cake Temperature in Drier (°C)	95

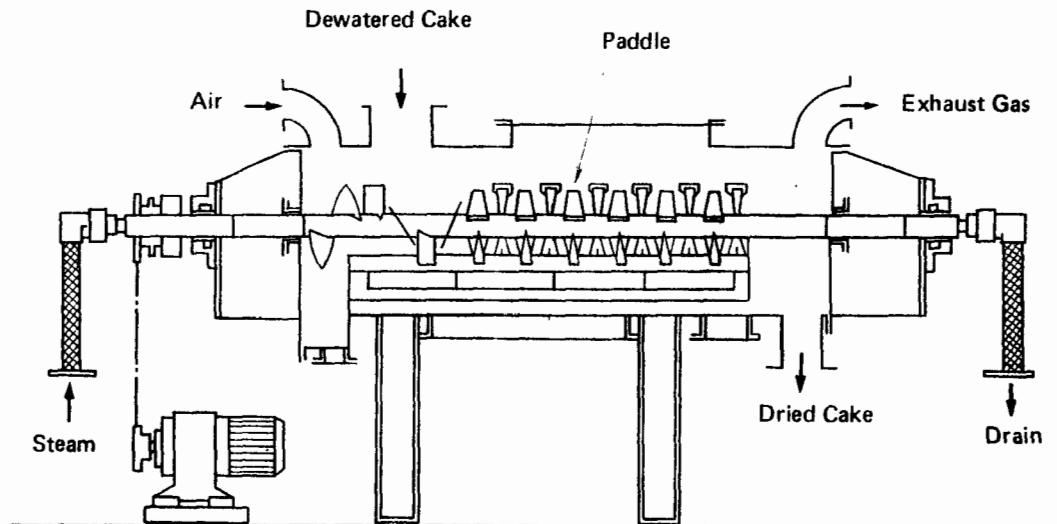


Fig. 10 Paddle-Type Indirect Steam Drier

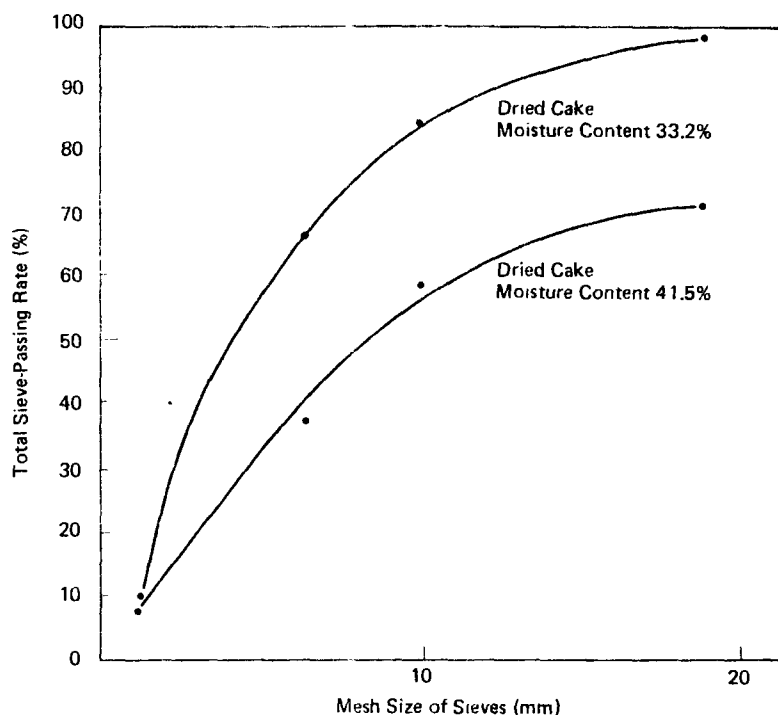


Fig. 11 Particle Size Distribution of Dried Cake

3.3.3 PELLETIZATION

The moisture content of dewatered cake by the belt press filter or screw press with polymer conditioning is higher and should be reduced to proper moisture content by drying. Although it depends on the moisture content, dried cake is not uniform in the particle size and may cause the pulverulent condition. Dewatered cake by filter press after the heat treatment has the moisture content required without the drying process. But, as dewatered cake is in the shape of large plate, it should be pulverized mechanically. The particle size of the pulverized cake is not uniform and too large that the bad effects are given to the combustibility and too small that the fall from between grates of incinerator is caused.

In this plan, the test was conducted for the screw-type pelletization machine under the condition that dewatered cake should be pelletized to the proper size even if any type of the dewatering machine is used. The pelletized cake should be so strong in resisting the shock and consolidation in process and be well combustible. Accordingly, the test was also conducted for the strength of pelletized cake.

a. Test of Polymer-Conditioned Sludge Cake

Dewatered cake by screw press was dried to the moisture content of 23.6%, 33.3%, 41.5% respectively for the pelletization test using the screw-type pelletizing machine as shown in Fig. 12. The excellent cylindrical pelletized cake of $\phi 10$, 12, 14 mm is obtained as shown in Fig. 13.

b. Test of Heat-Treated Sludge Cake

It is unable to pelletize the heat-treated sludge cake if the moisture content

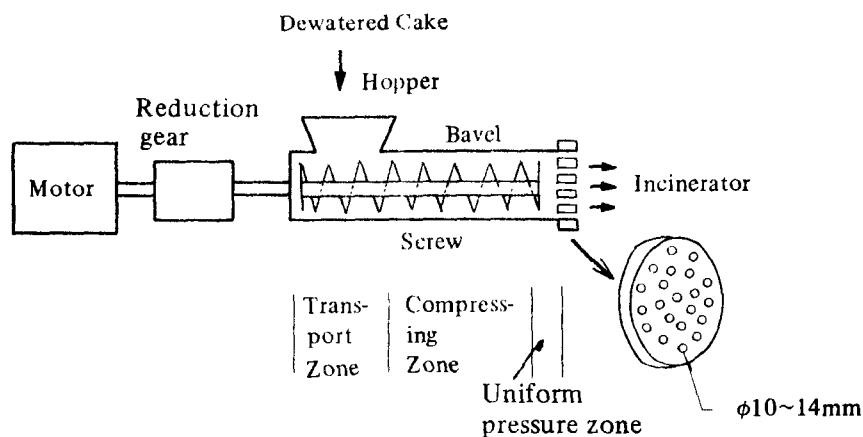


Fig. 12 Screw-Type Pushing Pelletizer

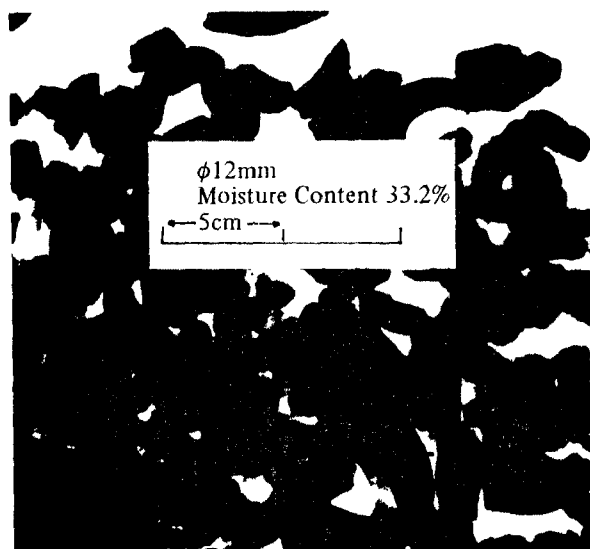


Fig. 13 Pelletized Cake

becomes less than 30%, and the pelletized cakes are adhered each other to produce a large cake if the moisture content becomes more than 50%. At the moisture content of 30~50%, the excellent pelletized cake with the diameter of 12 mm and the length of 30~50 mm is obtained.

To test the strength of pelletized cake, 500 grams of cake in the above (a) and (b) cases are put in an 18 liter can and then dropped 3 meters down on the concrete floor 12 times. As a result of the test, the strength has been adjusted as good because the rate of passing through 10 mm sieve was less than 5% for both cases.

3.3.4 COMBUSTIBILITY

To investigate the combustibility of pelletized cake, the incineration test was conducted in the electric furnace (200V, 7.5kW, W200×H150×L300). The typical test result for polymer-conditioned sludge cake and heat-treated sludge cake is shown in Fig. 14.

Table 7 Incineration of Dried Cake

	Ignition Loss Before Incineration (%)	Size of Pelletization (mm)	Moisture Content (%)	Ignition Loss of Ash after Incineration at 800°C	
				10 min. Burning	60 min. Burning
Polymer Conditioned Sludge Cake	67.3	φ12 Cylindrical	23.6	6.3	0.05
		φ12 Cylindrical	33.2	8.3	0.09
		φ12 Cylindrical	41.5	13.2	0.11
Heat-Treated Sludge Cake	52.7	φ12 Cylindrical	52.7	9.9	0.37

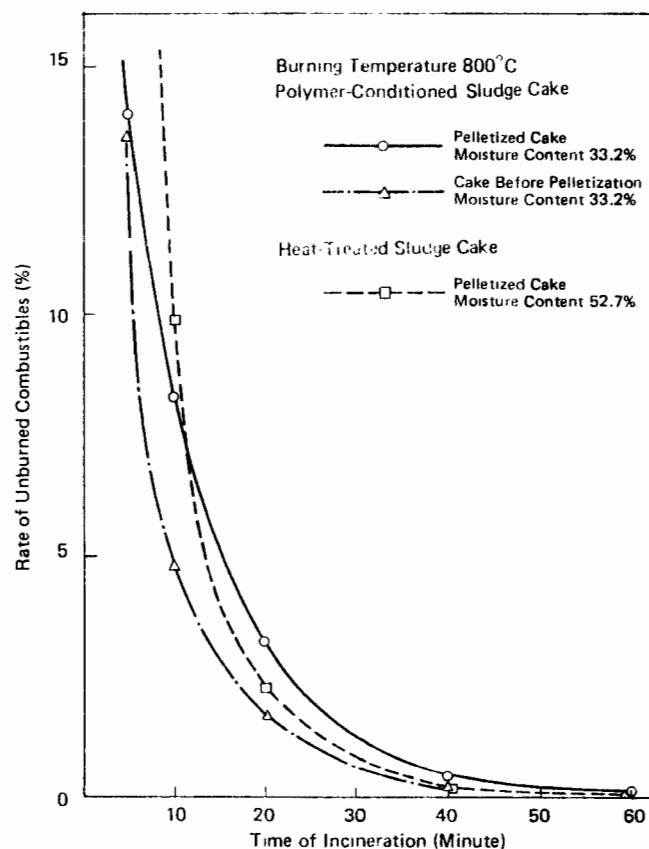


Fig. 14 Decreasing Curve of Combustibles Content

In the incineration test with the electric furnace, considerable non-combustibles remained in ash at the temperature of 800°C and the incineration period of 10 minutes. In case of the one-hour incineration period, it was almost completely reduced to ash and the non-combustible rate was less than 1%. Fig. 14 shows that the incineration speed of pelletized cake is slower than that of cake before pelletization, but it was almost the same speed 40 minutes later. Although the incineration with the electric furnace differs from the incineration with the full-scale incinerator, it is expected to be possible to incinerate pelletized cake with refuse unless the ratio of pelletized cake to refuse is so high.

3.3.5 CONCLUSION ON LABORATORY TEST

The laboratory test was conducted for unit process of two treatment processes ---(A) polymer conditioning and (B) heat treatment---under the condition to incinerate sewage sludge and municipal refuse at the full-scale plant. The result was obtained that both processes could produce the pelletized cake for coincineration. It also showed that there is no particular problem on its combustibility.

4. COINCINERTION TEST BY FULL-SCALE PLANT

4.1 OBJECT

This test was conducted to investigate the incineration of the pelletized cake by full-scale refuse incinerator based on the result obtained by the laboratory test described in the previous section. The investigation was carried out for the following problems that may be caused by the difference of physical and chemical characteristics between sewage sludge and municipal refuse.

- a. Effect on NO_x concentration in exhaust gas and effect of dust load on electric dust collector.
- b. Effect on the incineration state in an incinerator and the ignition loss of ash.

4.2 METHOD

The incinerator used for the test is a rotary stoker-type refuse incinerator (the same type as that at the Higashi Refuse Incineration Plant) in M city. The capacity is 150 ton/day and it passed four years after the operation of incinerator. As the exhaust gas treatment facilities, the electric dust collector (dry type) and the alkali washing facilities are installed. Fig. 15 shows the schematic of the incinerator and the monitoring points and the sampling points for investigation.

At the incineration plant, the collected refuse is stored in the refuse pit and then weighed by the refuse supply crane. After the refuse is fed into the incinerator from the feed hopper, it is dried and incinerated in moving from the upper side to the lower side on the rotary grates. Both the incinerated ash and the riddlings from the rotary grates are put in the ash cooling tank and then are transported by truck from the ash bunker. The exhaust gas passes through the duct and is washed by water jet at the gas cooling chamber. After smoke dust is collected by the electric dust collector, toxic gases such as HCl are removed with the alkali purification and then the gas is discharged from stack.

The monitoring and sampling were conducted at each point shown in Fig. 15 in order to confirm on whether any effect is caused on by-products (exhaust gas, ash, riddlings, wastewater, etc.) of the coincineration with sludge in the incineration process abovementioned.

Two kinds of sludge were used for the test---(A) pelletized cake of polymer-conditioned sludge and (B) pellerized cake of heat-treated sludge. Pelletized cake was fed into the hopper with man power from a 20 kg bag so that the ratio of the weight of fed pelletized cake to that of the refuse fed by crane might be fixed. The coincineration test was carried out for three days October 24-26, 1978. The ratio of fed sludge quantity to refuse and the feeding pattern in the test are shown in Table 8.

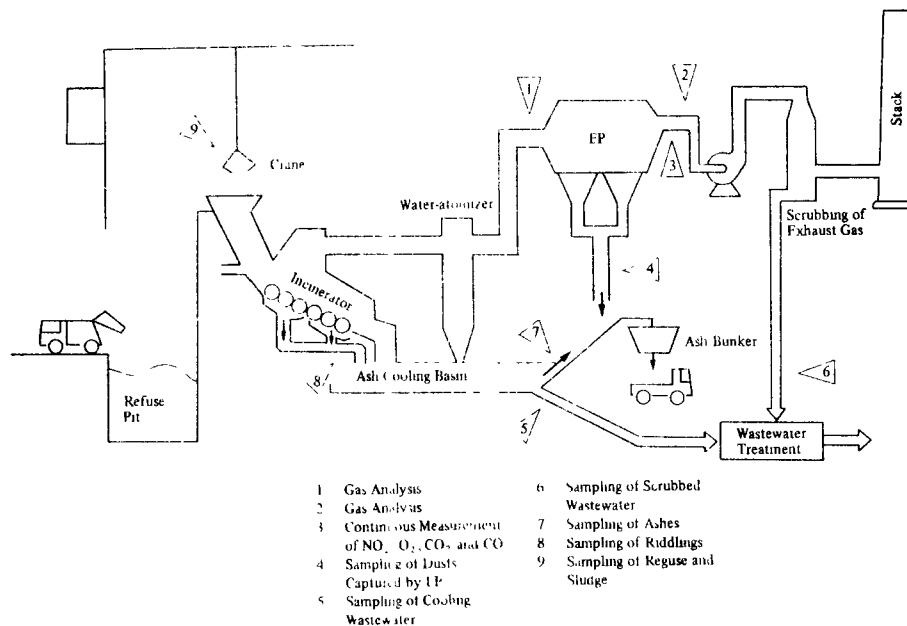


Fig. 15 Schematic of Incineration Plant and Sampling Points

Test was conducted at the average sludge mixing rate of 10% (which is estimated from the construction plan of incinerator shown in Table 1) and 20%. On the fourth day of the test (October 27), the sludge was not fed, and the test was conducted for the incineration of only refuse.

Table 8 Ratio of Sludge Cake to Refuse

Item	Date		Ratio of Sludge Cake (%)
Polymer Conditioned Sludge Cake	Oct. 24	10:00~12:00	9.8 (8.9)
		13:00~15:00	19.1 (16.0)
Heat-Treated Sludge Cake	Oct. 25	10:00~12:00	9.9 (9.0)
		13:00~15:00	18.8 (15.8)
	Oct. 26	10:00~12:00	19.1 (16.1)
		13:00~15:00	17.7 (15.1)

(Note) Figures in parentheses show the ratio of sludge cake to total.

4.3 ANALYSIS OF REFUSE AND SLUDGE

The composition of refuse of M city in the test is shown in Table 9. The analysis was conducted for about 3~4 kg of refuse taken out by crane from the refuse pit once a day. The composition of refuse of Kyoto City is also shown for comparison. The composition of refuse in M city is featured by the high rate of paper and the small rate of kitchen wastes. During the test period, the low heat value of refuse was 2,160~2,530 Kcal/kg.

Table 10 shows the characteristics of sludge used for the test. Although there was no large difference of the characteristics between two kinds of sludge prepared for the test, the ignition loss was much lower than that of normal sludge, and the low heat value is estimated at about 1,000 Kcal/kg. The fact that the moisture content of pelletized cake was lower than the objective value is ascribed to the natural drying because the pelletized cake in paper bag was stored inside the building for 7 to 10 days until the incineration test was conducted.

Table 9 Composition of Refuse (On Dry Basis) (%)

	Paper	Plastics	Garbage	Other Combustibles	Metal	Non-Combustibles	Total		Moisture Content (%)
M City	56.0	13.0	12.5	10.2	4.6	3.7	100	4-Day Average	39.0
Kyoto City	32.6	10.0	34.7	12.6	3.6	6.5	100	Data in 1975	45.5

Table 10 Characteristics of Dried Cake

	Polymer Conditioned Sludge Cake	Heat-Treated Sludge Cake
Sludge Treatment Process	Polymer Conditioning → Dewatering → Drying → Pelletization	Heat Treatment → Dewatering Pelletization
Shape	Cylindrical Diameter 12 mm Length 20 mm	Cylindrical Diameter 12 mm Length 20 mm
Moisture Content	14.6%	30.5%
Apparent Specific Gravity	0.69	0.63
Ignition Loss	36.6%	39.4%
Elemental Composition	H	2.6%
	C	14.8%
	N	3.2%
		2.6%

4.4 RESULT

4.4.1 MEASUREMENT OF EXHAUST GAS

Concentrations of nitrogen oxide (NO_x), oxygen (O₂), carbon dioxide (CO₂) and carbon monoxide (CO) of exhaust gas are shown in Table 11.

The Table shows that there were no big differences in concentrations of four substances of exhaust gas at the incineration and non-incineration of sludge and no effect was found by the incineration of sludge. There was also almost no effect of the sludge incineration on quantity of dust, composition of elements (C, H, N) in unburned materials, exhaust gas flow rate and characteristics of wastewater from exhaust gas washing.

4.4.2 ANALYSIS OF ASH AND RIDDINGS

As described in Section 4.2, the fallen ash and riddlings from grates are mixed in the cooling water basin to produce ash. The ash was then sampled on the ash conveyor and was analyzed. The riddlings was sampled just before reaching the ash cooling basin from the conveyor for riddlings.

**Table 11 Analysis of Exhaust Gas
from Incinerator**

Date		Item	NOx (ppm)	O ₂ (%)	CO ₂ (%)	CO (ppm)
Incineration with Sludge Cake	Oct. 24		107	14.9	5.9	193
	25		104	14.6	6.3	215
	26		104	14.3	6.0	215
	Average		105	14.6	6.1	208
Incineration Without Sludge Cake	Oct. 24		104	15.3	6.1	174
	25		92	14.9	5.6	250
	26		106	14.4	5.7	221
	27		105	14.4	6.6	171
	Average		102	14.7	6.1	201

Ash is classified in substances of incinerate residual ash content, metals, glass ware, non-combustibles and sludge (all those adjusted as sludge). The rate of sludge in the ash increases with time after it passes about one hour from the start of sludge feeding. And the quantity of riddlings also increases, and the rate of sludge in riddlings became more than 50% in some cases.

As shown in Table 12, the ignition loss of incinerate residual ash is found to be slightly higher in the incineration with sludge than that in the incineration of only refuse.

In the incineration with sludge, the non-combustibles of incinerate residual ash content in ash increases. It indicates that the incineration state in the incinerator is expected to have been worsened to some extent due to the feed of sludge. But it seems to have been caused by the fact that non-combustible sludge in riddlings was considerably remained in ash. For both the temperature and incinerating state in the incinerator, there was no large difference between the incineration and non-incineration of sludge.

There was much sludge in riddlings. It was caused by the factors—it passed four years after the operation of incinerator used for the test; the gap of grates became larger than that assumed at the outset due to wear and damage; and the difference of the physical characteristics between pelletized cake and municipal refuse. As the particle size of pelletized cake is small and its apparent specific gravity is large, sludge is stirred and concentrated on the lower part of refuse layer (on grates) while it moves on grates. A huge quantity of the cake is believed to fall from the slit of grates. There is another factor that the refuse layer thickness is about 0.3 m, very thinner than the normal layer thickness of about 1.0 m at the entrance of incinerator.

Judging from these factors, it is necessary to set the sludge mixing rate properly and to feed the sludge uniformly to the incinerator for the coincineration of refuse and sludge. The completely-incinerated sludge in ash remains unchanged in shape of the pelletized cake and this shows that the pelletized cake will not be destroyed in the incinerator.

**Table 12 Ignition Losses of Residual Ash
and Sludge in Ash**

Date	Item	Ignition Loss of Residual Ash (%)	Ignition Loss of Sludge (%)
Oct. 24	9:30	7.5	—
	10:30	7.1	—
	11:30	5.6	10.9
	14:30	9.1	12.3
	15:30	7.6	7.6
Oct. 25	9:30	5.3	—
	10:30	10.2	—
	11:30	7.9	12.5
	14:30	10.1	18.4
	15:30	11.4	22.2
Oct. 26	9:30	7.2	—
	10:30	5.9	—
	11:30	6.7	18.5
	14:30	9.7	17.3
	15:30	12.3	29.5
Oct. 27	9:30	8.7	—
	10:30	Not Measured	—
	11:30	9.4	—
	14:30	9.8	—
	15:30	8.6	—
Average in Incineration Without Sludge Cake		8.0	
Average in Incineration With Sludge Cake		8.9	
Average		8.4	16.6

4.5 CONCLUSION OF FULL-SCALE PLANT TEST

The following items have been made clear for the coincineration of sewage sludge and municipal refuse in the test.

- a. It may be possible to incinerate sewage sludge with municipal refuse in the rotary stoker type incinerator if pretreatment of sludge is conducted properly.
- b. No effect of coincineration was found on exhaust gas.
- c. No big change was also found on smoke dust, exhaust gas flow, characteristics of exhaust gas and wastewater.
- d. To prevent the fall of sludge from grates, it is necessary to pelletize the dewatered cake as big as possible under the condition that the combustibility may not be worsened.
- e. In the case that sludge and refuse are stored and mixed in a pit in the incinerator, it is necessary to mix them as uniformly as possible for feeding into hopper.

4.6 ADDITIONAL TEST (ENLARGEMENT OF PELLETIZED CAKE)

As described in the previous section, the pelletized cake is expected to fall from grates if the size of pelletized cake is around 12 mm on incineration by full-scale incinerator. Then, it was attempted to produce the pelletized cake of larger particle size. The characteristics was examined in the laboratory test as in the similar test conducted in Section 3. The result is briefly described.

Sludge to be pelletized is the combined sludge from domestic wastewater, and its organic content is about 80%. The pelletized cake was produced by both (A) and (B) processes described in Section 3.2.2. Its moisture content is 33~40% and its shape is about $\phi 20$ mm \times 30 mm long, $\phi 30$ mm \times 30 mm long. It was possible to produce the excellent cake. Results of the test for the strength and incineration by the electric furnace was also satisfactory.

In the coincineration test by the electric furnace, various shape of pelletized cake was produced and tested to investigate the effects of the shape and size of pelletized cake on combustibility. It was found that the pelletized cake of larger surface area and smaller initial weight is more combustible if it is the same volume. It means that the cake of thin flat board shape and light weight is more combustible. The $\phi 20 \sim 30$ mm cylindrical cake produced for the test won't cause any adverse effect on the combustibility especially in the incinerator. Rather, it is better to reduce the quantity of riddlings by making larger pelletized cake than $\phi 12$ mm cylindrical cake produced for the full-scale plant test.

5. PROBLEMS IN FUTURE

The test for coincineration of sewage sludge in the municipal refuse incinerator was carried out to investigate mainly the effective utilization of energy and the pre-treatment technology of sludge for coincineration. As a result of investigation, it was found that the incineration of sludge with municipal refuse is possible for cake produced in the treatment processes—(A) polymer conditioning process and (B) heat treatment process.

But for the enforcement of this plan, there still remains the problems to be studied. These problems are described below:

a. For both the (A) and (B) processes, the huge quantity of thermal energy is required to produce the objective pelletized cake. It is covered by the surplus heat produced in the refuse incinerator but the quantity is limited. Consequently, the maximum efficiency and the smaller amount of heat consumption are regarded better for the heat to be used in the sludge treatment. For this purpose, the solids content of thickened sludge should be increased in both processes. And, for (A) process, it is necessary to reduce the load for the drier by producing lower moisture content of sludge cake with increasing dewatering efficiency. For (B) process, it is necessary to make the volume of heat treatment reactor smaller and to reduce the heat consumption for heat treatment.

b. It is necessary to do comparative evaluation of the construction cost and the operation and maintenance costs of the total system after learning about the necessary energy and other factors for both processes.

c. The evaluation on sludge treatment process by tests was carried out for

each unit process. At the full-scale plant, these processes are organically combined for operation. Accordingly, it is necessary to study the operational efficiency of the whole sludge treatment process which was not confirmed in the laboratory test, and the countermeasures during the period of shut-down caused by the overhaul of incinerator.

6. CONCLUSION

The role of the wastewater treatment plant and the refuse incineration plant as the municipal environment facilities will become more important in the future. The construction of effective and economic facilities, and the maintenance and operation are necessitated in consideration of the effective use of surplus energy because the huge quantity of energy is consumed at these facilities. In this situation, the coincineration of sewage sludge and municipal refuse is regarded as a subject to be studied for the construction of these facilities. The construction of full-scale plant is expected to be started as early as possible after the further investigation is made for the enforcement of the plan.

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NEW ASPECTS OF SLUDGE INCINERATION IN YOKOHAMA

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Foreward

With the spreading of sewerage system, the amount of sludge produced in sewage treatment plants has increased and serious problems have been brought about regarding its treatment and disposal.

In Yokohama city, part of the sludge has been returned to green zones and farm lands as "dried sludge fertilizer" since 1977 and the bulk has been disposed of in garbage dumping plots.

However, with the advance of urbanization everywhere in the city, which has been taking place with the high economic growth since the beginning of the 1960s, it has become more and more difficult for the city authority to secure a new plot for sludge disposal.

Although it has been the basic policy for sludge to be disposed of by being returned to the recycling system of natural materials so that the ecological environment can be improved rather than harmed, it was decided as a tentative measure to incinerate all the sludge in order to extend the useful life of existing disposal grounds which are precious in the circumstances described above and to improve the quality of sludge.

Eleven treatment plants in all are involved in the sewerage works plan of the city (eight plants are in operation as of the end of fiscal 1980). The sludge from these treatment plant is to be processed by two base treating plants, viz. Kanazawa sewage treatment plant and the Hokubu second sewage treatment plant located in coastal areas. Sewage treatment plants are planned to be connected to the base plants with pipe lines which transport sludge under pressure. Until the pipe line is made complete, sludge cake must be carried by trucks for incineration at the base plants.

On the basis of the plan described above, a multiple-hearth furnace incinerator was constructed at Kanazawa sewage treatment plant and put into operation in 1978.

Aiming at energy conservation, a fluidized bed furnace, which utilizes pulverized coal as the supplementary fuel instead of oil, was completed in Hokubu second sewage treatment plant in March 1981.

In order to further conserve energy, experiments have been carried out on a pilot plant with the object of establishing a combination system of "drying" and "incineration".

Detailed descriptions of the treatment facilities are given in the following paragraphs.

1. Sludge Incineration by Multiple-hearth Furnace Located Apart from Sewage Treatment Plant Site

1.1 Outline of facility

Since it was the first experience for the city to construct a sludge incinerator, the sewerage works bureau formed a committee consisting of the bureaus officials which made preliminary investigations for two years starting with a survey of incineration performance in other cities. On the basis of the general plan and of the estimated amount of sludge cake that would be produced in a year when it is put into operation, the capacity of the facility was set at 100 ton per day. Whereas there was only one fluidized bed furnace in operation with a capacity of less than 40 ton a day

at that time, several multiple-hearth furnace with a capacity of 150 ton per day were already in practical use and it was decided to adopt a multiple-hearth furnace.

Design specifications of the sludge incinerator were set forth as follows:

- a. Multiple-hearth furnace
- b. Incinerating capacity:
100 ton per day, capable of operating for 30 minutes at the maximum capacity (120 percent of the rated capacity)
- c. Properties of the sludge to be incinerated:

Kind:

Dewatered cakes of raw sludge and anaerobic digested sludge.

Method of dewatering:

Vacuum or pressure filtration using inorganic coagulant, and centrifuge using organic coagulant

Water content:

65 to 80 percent

Calorific value:

1400 to 3000 kcal/kg-DS

Ash content:

About 40 percent

Combustibles:

About 60 percent

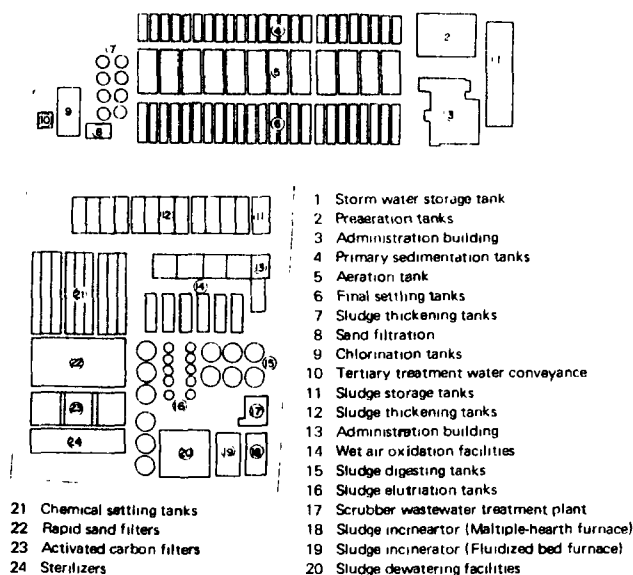


Fig. 1-1 General plan of Kanazawa sewage treatment plant

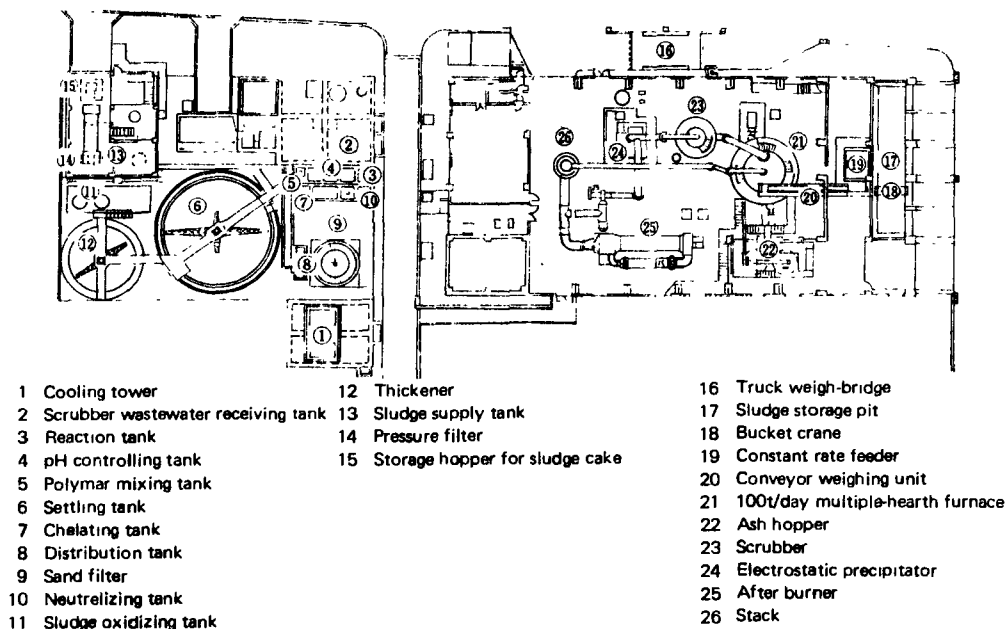


Fig. 1-2 Layout of sludge incineration facility

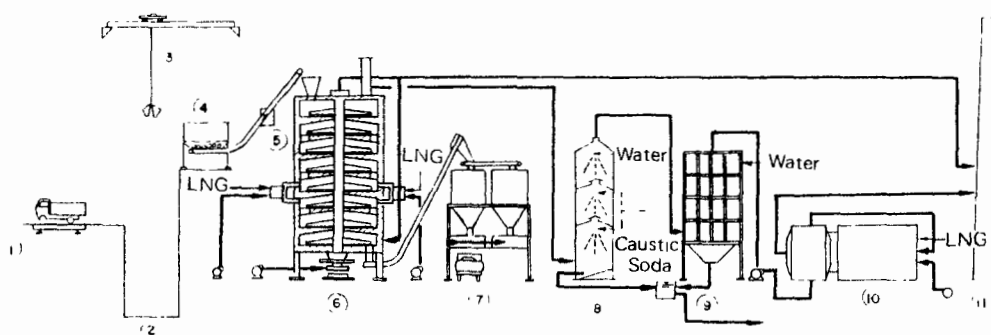


Fig. 1-3 Flow diagram of sludge incineration facility

- | | |
|---------------------------|-------------------------------|
| 1 Truck weigh-bridge | 7 Ash hopper |
| 2 Sludge storage | 8 Scrubber |
| 3 Bucket crane | 9 Electro-static precipitator |
| 4 Constant rate feeder | 10 After burner |
| 5 Conveyor weighing unit | 11 Stack |
| 6 Multiple-hearth furnace | |

Table 1-1 Summary of sludge incineration facility

Name	Description	
Building	Total floor space 1437m ² Steel-frame with ALC plates partially reinforced concrete structure	
Truck weighbridge	Weighing method	Static weighing 1 set
	Weighing capacity	30 ton (load cell type) minimum 20 kg
Crane	Overhead travelling with grab bucket	2 sets
	Rated load	1 ton (by grab bucket)
	Span	8.4m
	Range of lift	20.2m
	Length of travel	25m
	Bucket	Double wired clamshell type
Constant rate feeder	Steel, square, screw feeder, downward conveying type 1 set	
	Capacity	25m ³
	Rate of conveyance	0.55 – 6.6 ton/hr
Incinerator	Multiple-hearth furnace 1 set	
	Capacity	100 ton/day
	Number of hearths	8
	Diameter	5710 mm
Auxiliary burner	Hot air generation by gas burning type 2 sets	
	Hot air temperature	900–1100°C
	Combustion capacity	LNG 1,500,000 kcal/hr x 2
Ash hopper	Steel, truncated cone shape 2 sets	
	Capacity	35m ³
	Ash humidifier	Batch type (paddle mixer) Kneading capacity Max. 1m ³
Scrubber	3-stage spray tower 1 set	
	Dimensions	2000 mm (D) x 13m (H)
	Sulfur oxide	Less than 30 ppm (scrubbing with caustic soda)
	Percentage of removal	Higher than 90% (dust concentration at the exit 0.2g/Nm ³)
Electrostatic precipitator	Vertical wet-type 1 set	
	Dimensions	3016 mm(sq) x 10900 mm (L)
	Volume of gas treated	20,000 m ³ /hr (at 40°C)
	Percentage of removal	Higher than 90 % for the containing 0.2g/Nm ³ -DG at the entry
Afterburner	Direct heating type 1 set	
	Volume of gas treated	17,000 Nm ³ /hr
	Temperature of treatment	For deodorizing 750°C
		Maximum 800°C
	Combustion capacity	Large burner 3,000,000 kcal/hr
Stack	Small burner 300,000 kcal/hr	
	Height	45m 1 set
	Material	SUS304

Figure 1-1 shows the general plan of Kanazawa sewage treatment plant; Figure 1-2 the layout of the sludge incinerating facility; Figure 1-3 their flow chart, and Table 1-1 their summary.

1.2 Characteristics of facility

a. Receiving storage and supply facility of sludge cake

Since sludge cakes (SC) from several sewage treatment plants are to be transported to and treated by the incinerator, it was decided to install a facility for temporarily storing different kinds of cakes from different treatment process and dehydrators and to supply them at a constant input rate. A tentative route of transportation of SC is shown in Figure 1-4.

b. Scrubber wastewater treatment plant

The operation of wastewater treatment at Kanazawa sewage treatment plant was scheduled to be started after 3 years, and it was decided to install a treatment plant for the exhaust gas scrubber effluent water.

c. Automatic control system

It was expected that the sludge incineration facility under consideration would require a far larger number of control works (such as checking the amount of SC received and stored, controlling the combustion conditions in accordance with the change in properties of SC, and controlling the operating conditions in the scrubber wastewater treatment plant) than with ordinary type sludge incinerators. Hence, it was decided to introduce micro- and sequence-controllers and to adopt supervisory and control to enable each piece of equipment to fully perform its function and to reduce the amount of operating and control works to the minimum through the incorporation of automation and labor saving devices.

d. Energy saving

It was planned that at the initial stage of operation the facility would entirely relay on city gas (LNG) and to convert to digested gas when the sludge anaerobic digesters at Kanazawa sewage treatment plant are completed.

Since the facility was required to burn and deodorize its exhaust gas for maintaining environmental conditions in its peripheral areas, it was decided to install a heat-exchanger, which preheats the exhaust gas to be treated by use of the deodorized gas, to economize in the fuel used for the afterburner. Also, it was

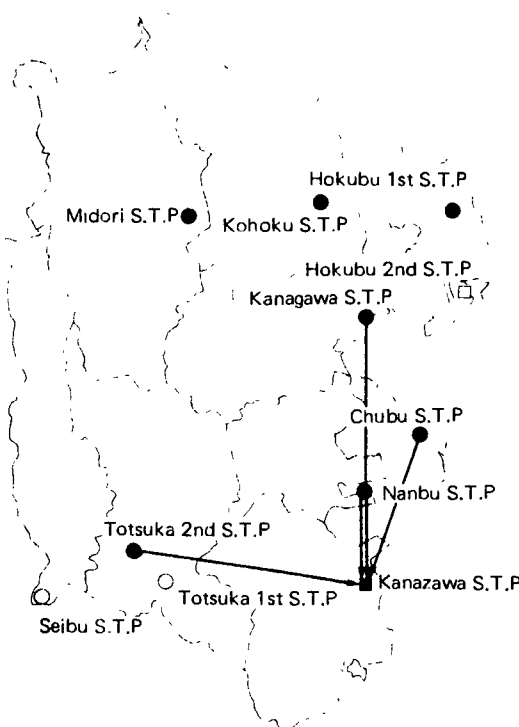


Fig. 1-4 Transportation route of sludge cakes

planned to secure stable combustion conditions and to save the supplementary fuel by automatically controlling the volume of secondary air on the basis of the temperature at the exit of the scrubber.

1.3 Outline of each facility

1.3.1 Facility for receiving, storage and supply of sludge cake

a. Receiving equipment

A truck weigh-bridge was installed to accurately account for the weight of sludge cake received from each sewage treatment plant, ash produced at the sludge incinerator and sludge produced at the scrubber wastewater treatment plant.

Figure 1-5 shows the flow of operation at the weigh-bridge. At the weigh-bridge, the car number of each incoming truck is read by a card reader, and receiving and shipping slips are printed out. A micro-controller reads the truck number, the kind, weight and source of sludge cake and transmits the data to the central control room, where data is sorted, totaled and printed by a logging typewriter.

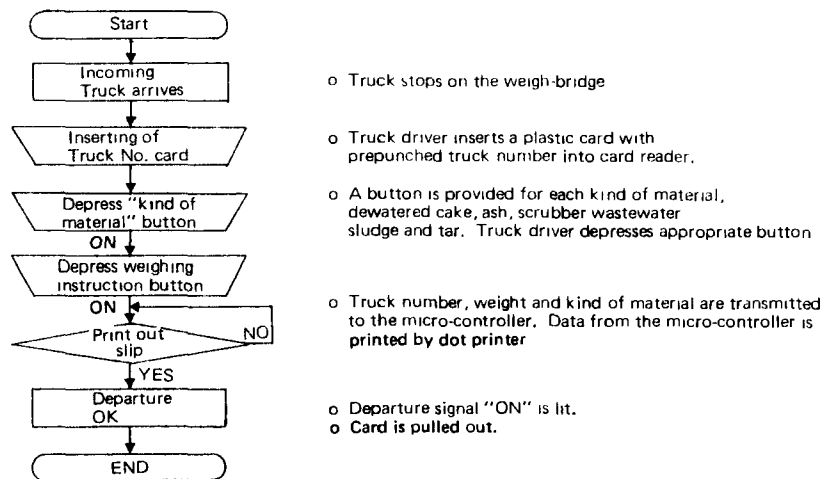


Fig. 1-5 Flow diagram of truck weigh-bridge operation

There are two modes to control the weigh-bridge:

- ① The weight of transported sludge cake is calculated from the difference between the weight of the loaded truck and that of the empty truck which is registered beforehand. This mode requires the truck to be weighed only once and is more convenient.
- ② The weight of transported sludge cake is calculated from the difference between the weights of the truck before and after unloading. This mode requires each incoming truck to be weighed twice.

Under normal conditions the single weighing mode is used.

b. Storage equipment

① Receiving at the pit

Sludge cake dehydrators at sewage treatment plants are operated only during daytime and are stopped throughout the silent hours, Sundays and holidays,

while the multiple-hearth furnace should be operated continuously (24 hours a day) even on Sundays and holidays. It was decided to install a storage pit capable of storing dewatered cake for 2 days operation, i.e., 250 m³ (effective).

Five gates to the pit are provided for access by incoming trucks. To receive weighed sludge cake in the pit, the destination is selected from 5 gates and is indicated by a flashing signal at the gate and the gate is made "OPEN". The destination is automatically determined on the basis of the amount of

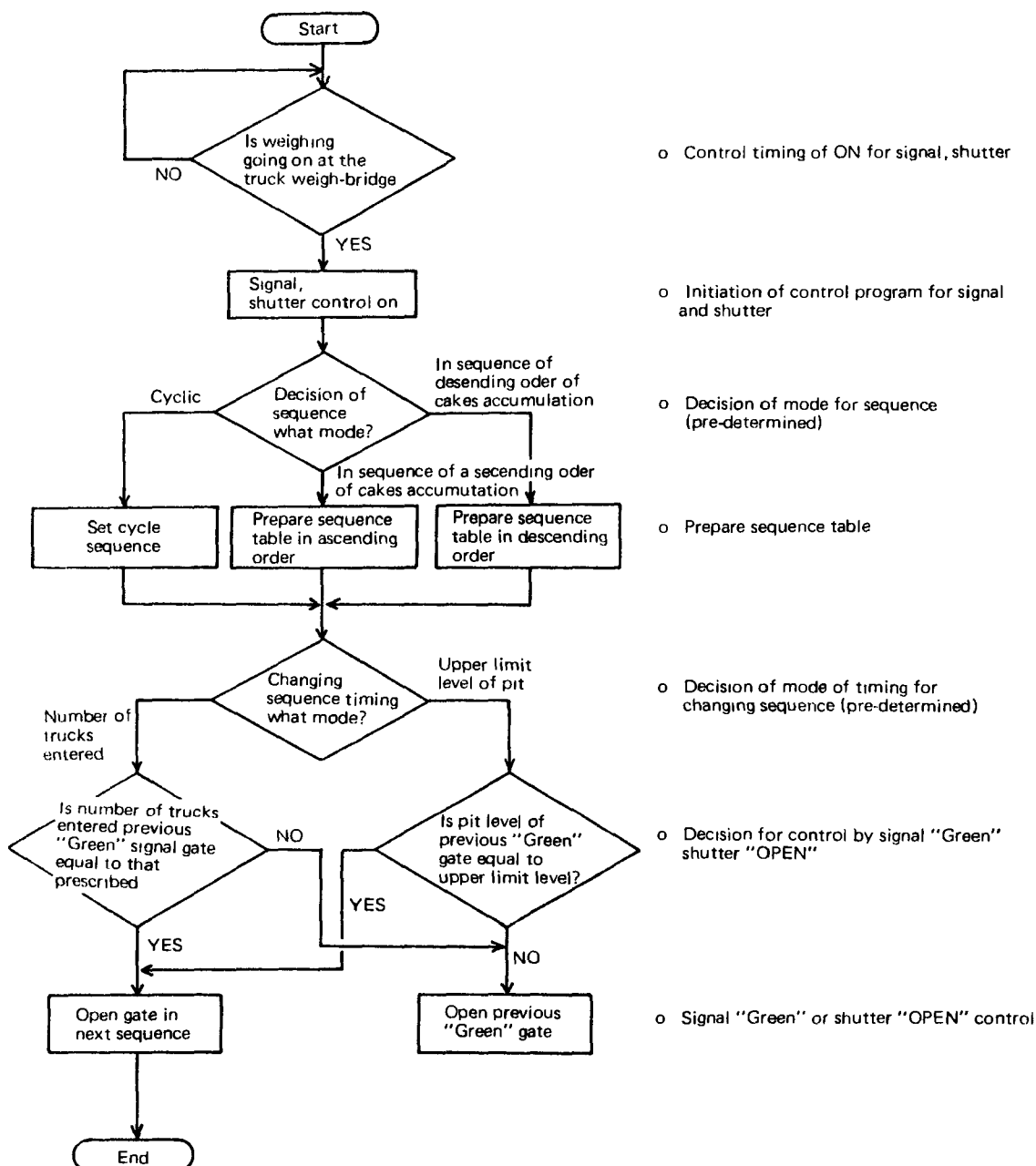


Fig. 1-6 Flow diagram of signal light and shutter control

sludge cake accumulated in each section of the pit, number of the trucks waiting for unloading and the operating conditions of the crane.

The control flow diagram at the pit station is shown in Figure 1-6, and the plan of receiving and storing equipment in Figure 1-7.

② Transfer to the constant rate feeder

The sludge cake stored in the pit is transported to the constant rate feeder operated automatically based on the amount of cake stored at the feeder, and that stocked in the pit, and on the presence of incoming trucks.

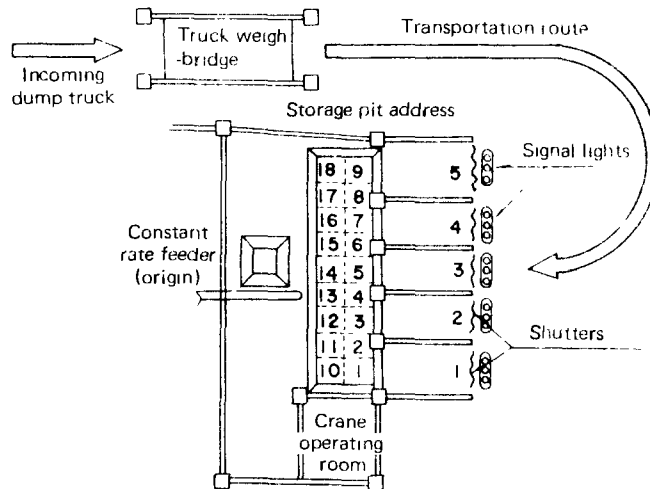


Fig. 1-7 Plan of receiving and storage facility

Four control modes

are provided to disperse and stratify the control functions as follows.

1) Manual mode

In this mode the equipment is manually operated by an operator in the crane operation room.

2) Automatic control mode at the site

In this mode, the operation of the crane is automatically controlled by the sequence control and the setting of the pinboard in the crane operation room.

The pit is divided into 18 addressed sections determined from the size of the grab of the crane. The crane moves to the addresses in the sequence determined by the pinboard setting, skipping the address in which cake is being unloaded from a truck.

3) Set point control mode [I]

Address setting of the automatic control is performed by a high order micro-controller instead of the pinboard at the site. In this mode, the sequence of crane destination is automatically controlled on the basis of the data of signal control (indication of truck destination) and on the stocked amount of sludge cake at each address so that conflict between the destinations of the crane and trucks may be avoided.

4) Set point control mode [II]

In this mode, corrections to the braking instruction to the crane are added to the set point control [I] and this is the highest level of control. Without frequent manual adjustment it is difficult for the brake of the crane to stop accurately at the designated position. Thus the corrections to the braking position are made automatically in units of 5 cm. Normally, operation is conducted by this mode.

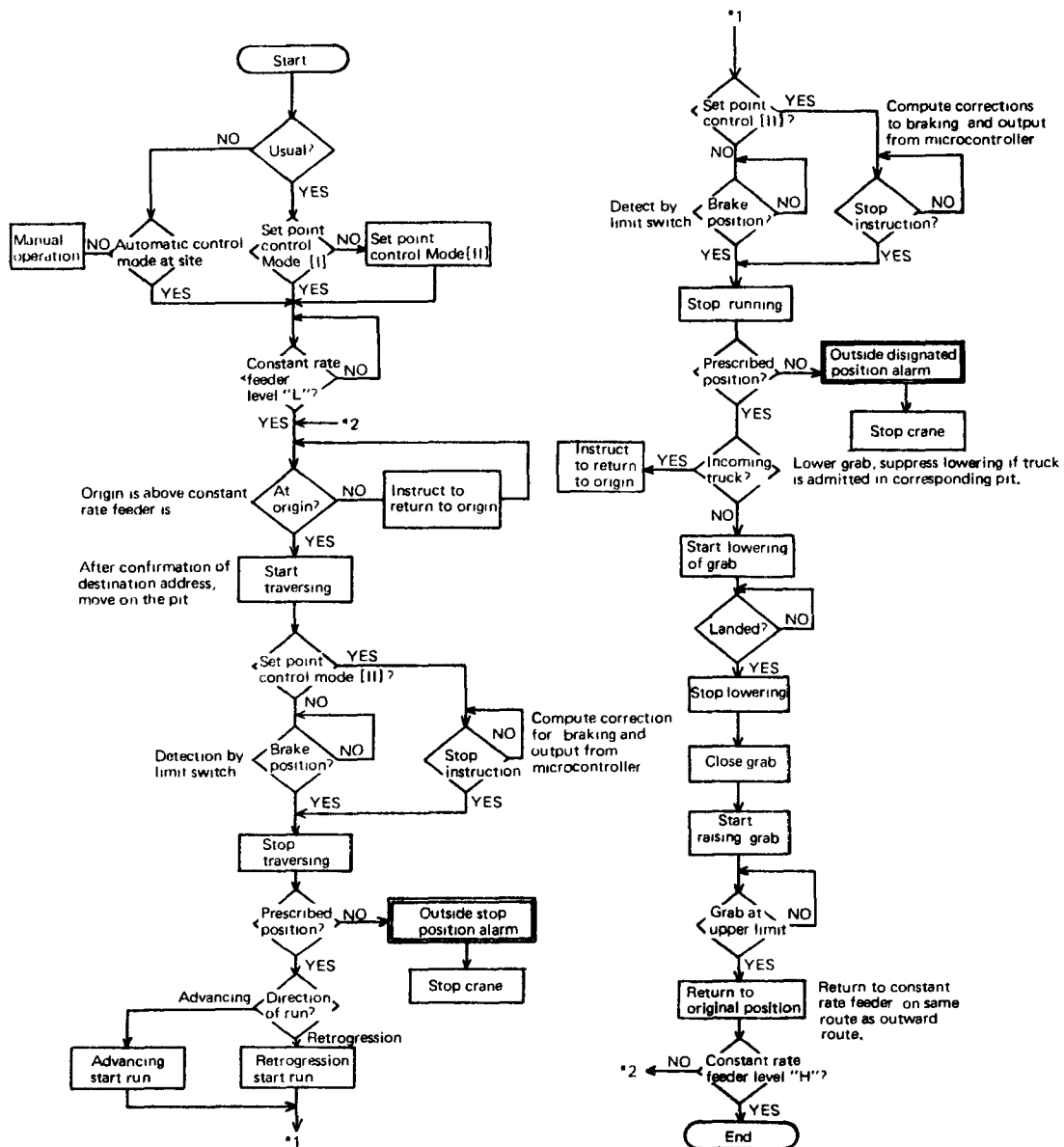


Fig. 1-8 Flow diagram of bucket crane control

Table 1-2 Control functions

Mode of control		Manual	Automatic control at local station	Set point control [I]	Set point control [II]
Equip- ment	Wired relay sequence	○	○	○	○
	Sequence-controller	—	○	○	○
	Micro-controller	—	—	○	○
Automatic Dec's on Element	State of constant rate feeder	x	○	○	○
	State of storage pit	x	x	○	○
	State of signal	x	x	○	○
	Presence of incoming truck	x	○	○	○
	Stop position	x	Fixed limit switch	Fixed limit switch	Compute by micro-controller Provide correction to brake

Control flow of the bucket crane and control functions are shown in Table 1-8 and Table 1-2 respectively.

c. Supplying equipment

Sludge cake is supplied at a fixed rate from a constant rate feeder to the incinerator by a conveyor.

A conveyor weighing unit is installed on the feed conveyor to keep the amount of sludge cake at a fixed rate by controlling the speed of the constant rate feeder.

1.3.2 Sludge incinerator

Sludge cake is fed through the opening at the top of the incinerator, dried and preheated as it passes from hearths 1 to 4, incinerates at hearths 5 and 6, cooled as hearths 7 and 8, and taken out as ash from the bottom.

The temperatures inside the incinerator are maintained constant by a micro-controller, which automatically sets the maximum temperatures in hearths 4 through 6.

In addition, the following modes of control can be used for controlling the temperatures inside the incinerator.

a. Control for raising temperatures

This mode is used when the temperatures is raised to the incinerating temperature, 800°C, after starting from the relighting stage of an idle incinerator.

b. Control for preserving temperatures

This mode is used when incinerating operation is switched to that of preserving the temperature at 500°C.

c. Control for lowering temperature

This mode is used when the incinerator is shut down.

d. Control for maintaining temperature at a fixed point

This mode is used for controlling the temperature of hot air generated by auxiliary burners at 1000°C.

Control valves are installed on the gas line to the burners and on the air line. The control valve on the gas line is controlled by the temperature inside the incinerator and the air control valve is controlled to keep the temperatures of hot air from the

auxiliary burners constant. As a result the incinerator body is protected, and clinker is prevented. At the initial stage of operation, the amount of secondary air was manually controlled to suit the combustion conditions. This method of control, however, tended to increase the proportion of air. At present it is automatically controlled.

1.3.3 Exhaust gas treatment plant

a. Scrubber

This is a three-stage spray tower for cooling, dehumidifying, removing dust from, and desulfurizing the exhaust gas discharged from the incinerator.

The amount of caustic soda supplied to the recycling tank is controlled by the pH of the overflowing water.

The concentration of sulfur oxides should be less than 30 ppm at the exit of the tower when the concentration at the entry is 300 ppm. The removal rate should be higher than 90 percent. The amount of water supplied to the scrubber is controlled so that the temperature of exhaust gas at the exit of the tower may be maintained at 40°C.

b. Electrostatic precipitator

A wet vertical type electrostatic precipitator is adopted, since it requires less floor space to install and less volume than a dry type. The concentration of dust at the exit of the collector should be less than 0.02 mg/Nm³.

c. Afterburner

This is required to deodorize the exhaust gas and to prevent white smoke. The reaction temperature should be 800°C at the maximum and the set temperature can be readily maintained by controlling the combustion of burners. A heat exchanger is installed to preheat the gas to be deodorized with heat from the exhaust gas.

1.3.4 Scrubber wastewater treatment plant

a. Pilot plant experiments

The scrubber wastewater contains dust from the incinerator dissolved substances from dust, salts of sulfur oxides in addition to various metals in a low concentration. The following experiments were conducted to select the best process for treating the scrubber wastewater from the two prospective processes, i.e., ferritizing and chemical sedimentation.

① Quality of water to be treated

The sample used in the experiment was the wastewater obtained by scrubbing the gas from the pilot plant incinerator. The cakes incinerated were produced by vacuum filtration and centrifugation at Nanbu sewage treatment plant and by pressure filtration at Totsuka second plant. Since the concentrations of metals in some samples were very low, the experiments were conducted on the assumption that the wastewater contains metals in the concentrations shown in Table 1-3.

The values in the table were determined by referring to the examples of measurement, published in Japan.

Table 1-3 Metals contained in scrubber wastewater

Item	SS	COD	Cd	T-Cr	Cu	Ni	Pb	Zn	Mn	Fe	T-Hg
Conc. in wastewater mg/l	414	40	0.27	2.26	3.29	0.56	1.00	16.55	5.44	197.6	0.07
Target mg/l	70 ^(ave) ₅₀	25	0.1	2.0	1.0	1.0	1.0	1.0	1.0	3.0	0.005

Note 1. Target values of treatment are based on the standards of wastewater specified in the pollution control ordinance of Kanagawa Prefecture.

② Experiments on treating by ferritization

After basic data was obtained by bench scale tests, experiment were conducted in accordance with the flow diagram shown in Figure 1-9. Ferritizing was found to be ineffective with copper, although it could remove metals other than copper.

Formation of ferrites depended on the ratio of the amount of iron added to that of suspended solid (SS) present in wastewater. A good yield of ferrite could be obtained only under conditions satisfying the following ratio; $SS/FE \leq 1/4$

The amount of iron added increases with an increase of the concentration of SS. As an example, where the concentration of SS was 350 mg/l and the ratio of iron to SS is 4 to 1, the amount of ferrous sulfate required was about 7000 mg/l. The amount of caustic soda required in the example was about 1200 mg/l and that of ferrite sludge produced was about 3000 mg/l.

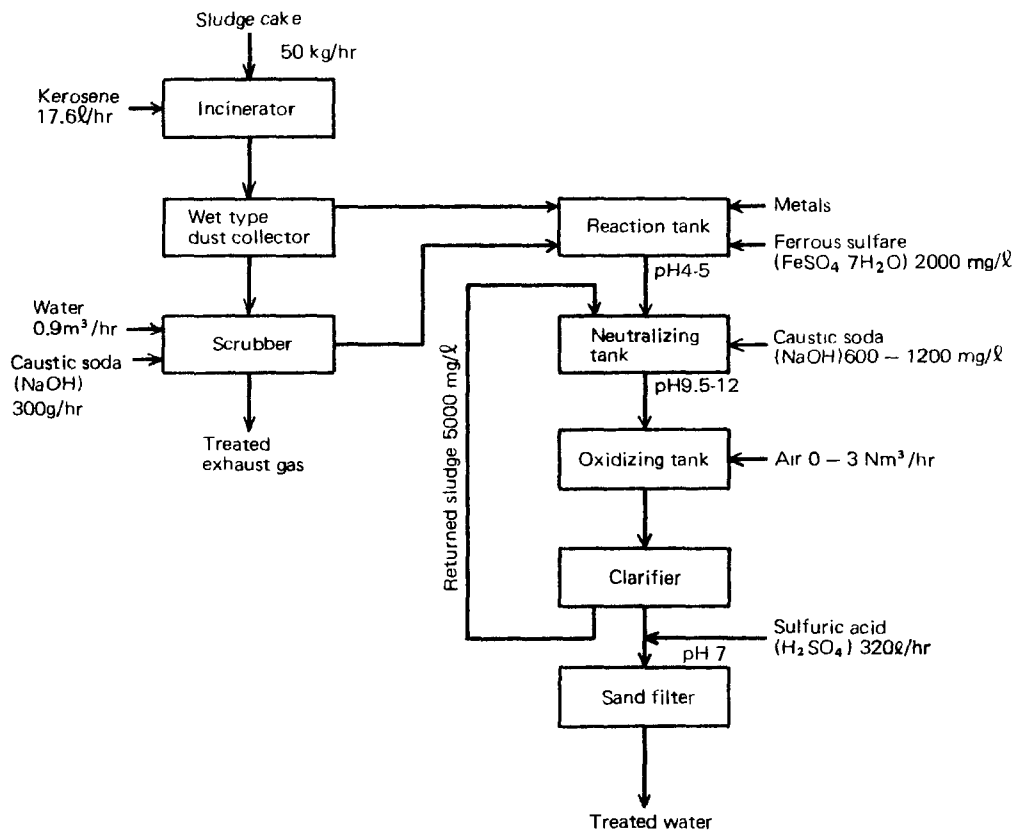


Fig. 1-9 Flow diagram of ferritizing pilot plant

(3) Experiments on treatment by chemical sedimentation

After basic data on the kind of coagulant, the amount of addition, and so on were obtained by jar tests, experiment were made according to the flow diagram shown in Figure 1-10. Using ferrous sulfate as the coagulant at a concentration of 300 to 500 mg/l at a pH value of 9.5 to 11, the concentration of metals other than copper was reduced to less than the desired values. The addition of 300 to 500 mg/l of ferrous sulfate to the wastewater containing 350 mg/l of SS, gave about 500 mg/l of sludge.

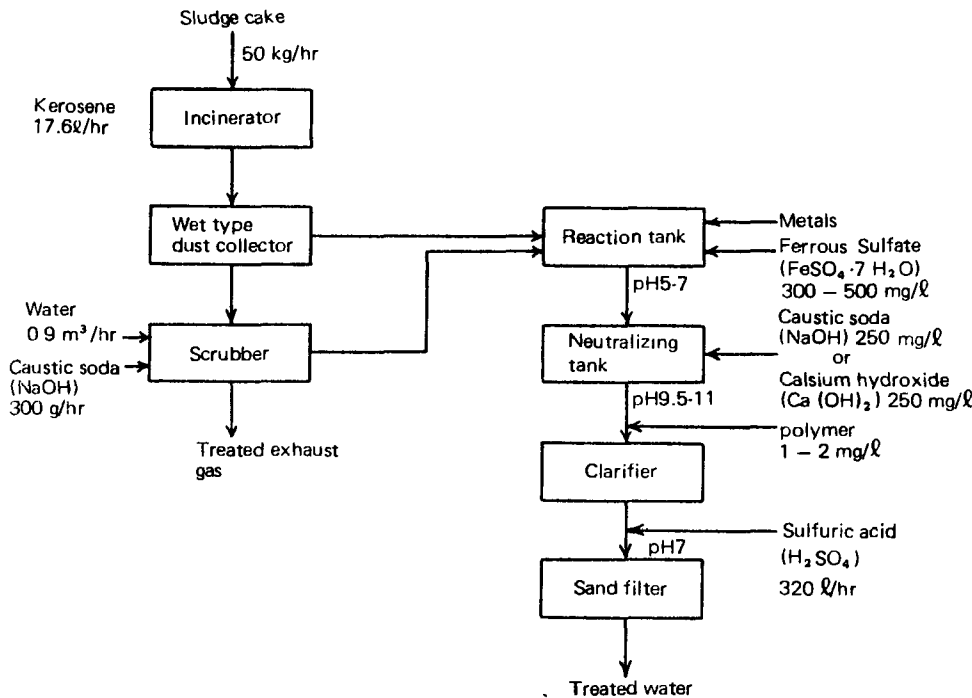


Fig. 1-10 Flow diagram of chemical sedimentation pilot plant

(4) Experiments using chelating agents

Jar test experiments for removing copper the unwanted substances by use of a chelating agent were conducted under the following conditions:

Conc. of ferrous sulfate solution
500 mg/l

Conc. of caustic soda solution
250 mg/l

pH value at the time of precipitation
7.1

Formula of chelating agent

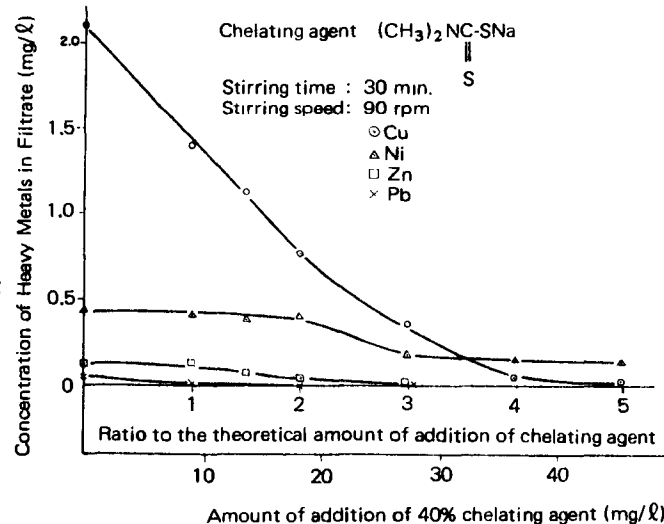
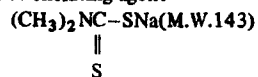


Fig. 1-11 Removal of metals by chelating agent

Most of the copper could be removed by using 30–40mg/l of chelating agent (about 4 times the theoretical value). It was decided, however, duct filtration should be provided by a sand layer at the final stage before discharging the treated water, since the floc produced was very fine. The results of the experiments are shown in Figure 1-11.

⑤ Determination of treating process

The ferritizing process was not considered suitable for use against the wastewater used in the experiment, because it could not remove copper and it produced about 9 times as much solid sludge as the chemical sedimentation. It was decided to adopt chemical sedimentation as the principal process, taking into consideration such factors as the effect of treatment, economy, and the stability of operation, and also to install a “chelating and sand filtration process” for the sake of safety.

Further, it was decided to process the sludge by pressure filtration without the addition of any chemicals, since the sludge consisted almost entirely of inorganic precipitates.

b. Actual plant

The design specifications of the scrubber wastewater treatment plant are as follows:

- | | |
|---------------------------------|--|
| ① Process | Chemical sedimentation and sand filtration |
| ② Capacity sulfate | 120 m ³ /hr |
| ③ Chemicals requirement | |
| Ferrous sulfate (15% conc.) | 400 ppm |
| Caustic soda (48% conc.) | 250 ppm |
| Polymeric coagulant (1% conc.) | 1 ppm |
| Sulfuric acid (75% conc.) | 60 ppm |
| Sodium hypochlorite (12% conc.) | 80 ppm |
| ④ Dehydrator | |
| Capacity | 3.5 cake-ton/day (water content 70%) |

The flow chart of the scrubber wastewater treatment plant and the summary of equipment are shown in Figure 1-12 and Table 1-4 respectively.

Since the scrubber wastewater is discharged from the scrubber at a temperature of between 50° to 60°C, it must be cooled by the cooling tower before it is fed into the receiving tank. Then, it is pumped into the reaction tank, where ferrous sulfate solution is added in proportion to the rate of flow, and the mixture is agitated. Then it is fed into a conditioning tank to adjust pH to 9.5 by adding caustic soda. The resultant floc is further flocculated by adding polymeric

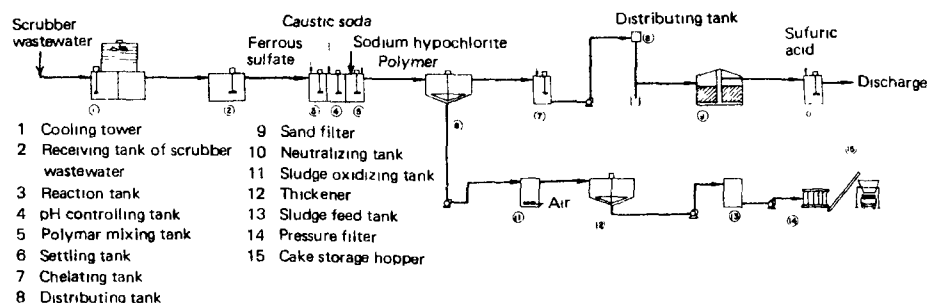


Fig. 1-12 Flow diagram of scrubber wastewater treatment plant

Table 1-4 Outline of scrubber wastewater treatment plant

Name	Description
Reaction tank and pH controlling tank	Holding time: 2.5 min, 2 groups x 2 tanks Effective capacity: 2.5m (sq) x 2.5m (H)=6.25m ³ /tank
Polymer mixing tank	Mixing time: 1 min, 1 tank Effective capacity: 2.5m ³
Settling tank	Settling time: 3 hours, 1 tank Overflow rate: 1.5m ³ /hr Capacity: 530m ³ Circular center-driven type
Chelating reaction tank	Holding time: 2.5 min, 1 tank Effective capacity: 6.25m ³
Sand filter	Filtering speed: 7m ³ /m ² -hr. Effective filter surface: 21.4m ² Gravity type with automatic reverse washing.
Neutralizing tank	Holding time: 2.5 min. Capacity: 7.2m ³ x 2 tanks
Cooling tower	Capacity: 150m ³ /hr Liquid temp.: Entrance 60°C, exit 40°C Atmospheric air temp.: Wet bulb temperature 27°C Blower capacity: 2900m ³ /min x 7 mmAq
Sludge oxidation tank	Holding time: 15 hours Capacity: 14m ³ x 2 tanks
Sludge thickening tank	Detention time: 7 days Capacity: 210m ³ x 1 tank Solid loading: 50 kgDS/m ² day Picket fence type
Dehydrator	Filter area: 148m ² Filtering pressure: 9.9 kg/cm ² Number of filter chambers: 60 chambers Pressure filter

coagulant into the mixing tank.

The effluent from the mixing tank is fed into a settling tank and the SS is removed by precipitation. The resultant solution is fed into a chelating reaction tank. No chelating agent, however, is added in normal operation. If the concentration of copper is high, a chelating agent will be added and the resultant SS will be removed by a sand filter down to a concentration of about 10 ppm.

After the measurement is taken on the volume, the filtrate is fed into a neutralizing tank to be neutralized to pH7 by adding sulfuric acid, and then it is discharged.

The sludge separated by precipitation in the settling tank to the concentration of about 5% is fed to a sludge oxidation tank, where ferrous oxide contained in the sludge is oxidized to ferric oxide by blowing compressed air into it.

The sludge is further fed into a sludge dehydrator after it is concentrated to a concentration of 7% in a sludge thickening tank.

1.3.5 Automatic control system

One micro-controller set and two sets of sequence-controllers are used to ensure intensive and flexible automatic control. To support control equipment, wired-relay circuits for direct control by operators and lower-level automatic control using regulators for S.C.C. (supervisory computer control) are provided. The configuration of the automatic control equipment is shown in Figure 1-13. In addition to the fully automatic control mode, modes of interlocking operation by blocks are provided for the period of running-in operation and anticipated partial breakdown of the facility. These conceptions of automatic control are shown in Figure 1-14. The instrumentation and items of control throughout the whole system are listed in Table 1-5.

Table 1-5 Overall instrumentation and control items

System	Control item	Outline of contents of control
1. Receiving, storage, supply of sludge cake	(1) Control of operation of truck weigh-bridge.	(1) Computation, sorted and printing or receiving data, classified by treatment plant. (2) Issuing of receiving and shipping slips. o Truck No. day and time, total weight, tare, net weight. (3) Detection of incoming trucks o Instruction for shutter signal control.
	(2) Shutter signal control	(1) Decision on the corresponding pit level from among gates 1 thru 5. o Decision on priority sequence. o Instruction for opening of corresponding gate to incoming truck. (2) Detection of admission of truck o Instruction to bucket crane on suppression of descent to the pit corresponding to the gate.
	(3) Control of crane	(1) Decision on the weight of constant feeder hopper. o Instruction to the crane on starting operation (2) Decision of destination address o Memorize level of each address of pit. o Decision of admission of a truck to gate. (3) Instruction to the crane on traversing, traveling and grabbing.
	(4) Control of feeding of cake.	(1) o Constant feeding of cake o Consideration to idle time.
2. Incinerator	(1) Pressure control (2) Temperature control	(1) Constant control of pressure inside incinerator. (1) Control of raising and lowering of temperature. (2) Control of hot air temperature. (3) Control of constant temperature inside incinerator.
	(3) Combustion air control	(1) Control of secondary air volume
3. Exhaust gas treatment	(1) Control of scrubber	(1) Control of constant pH (2) Control of exhaust gas temperature at the exit.
	(2) Control of electro-static precipitator	(1) Control of voltage and current
	(3) Control of after-burner	(1) Control of constant deodorizing temperature. (2) Control of temperature for white smoke prevention. o Decision of temperature to prevent white smoke referring to humidity chart.
4. Scrubber wastewater treatment	(1) Control of scrubber wastewater treatment plant	(1) Control of flow of untreated water. (2) Control of rate of addition of ferrous sulfate. (3) Control of constant pH in pH control tank (4) Control of rate of addition of polymer solution. (5) Control of rate of addition of chelating agent. (6) Control of constant pH at neutralizing tank.

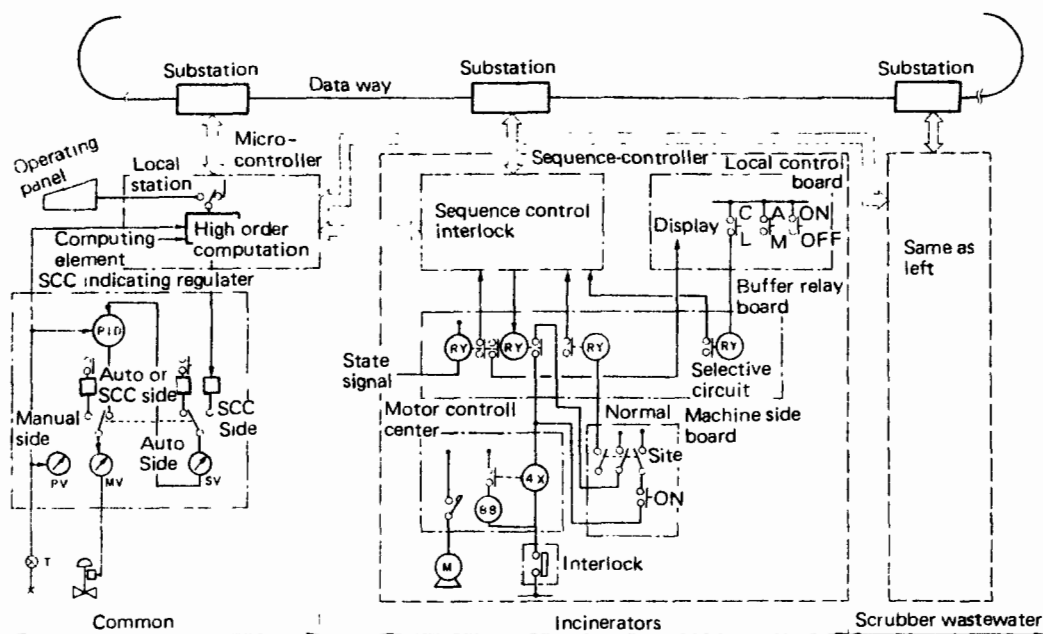
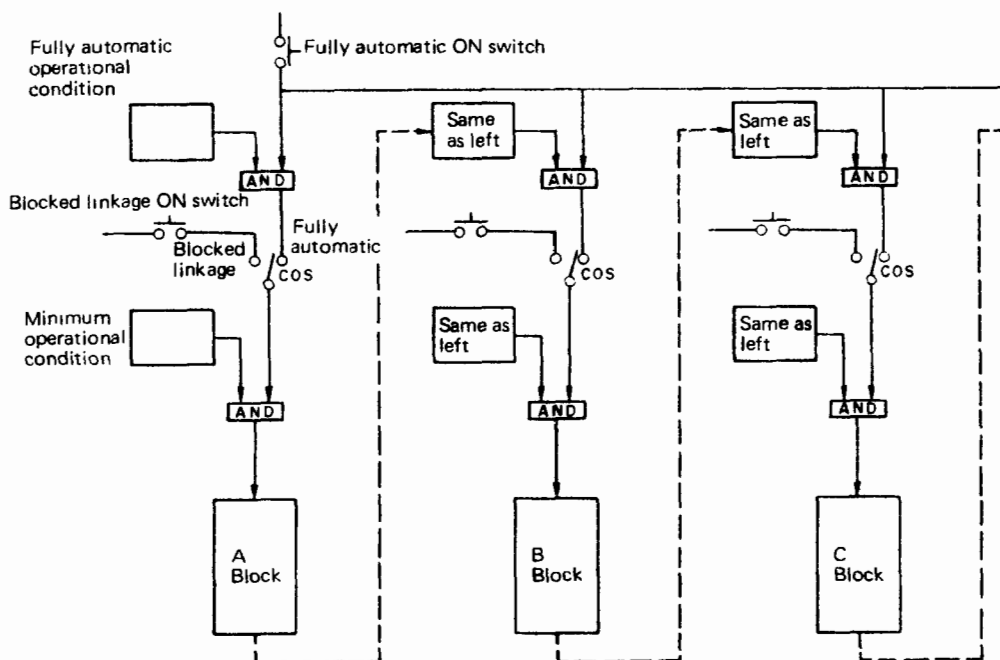


Fig. 1-13 Configuration of automatic control equipment



Block diagram of incinerating facility — water supply pump, cake feeding, operation of incinerator, auxiliary burners, scrubber, electrostatic precipitator, induced draft fan, afterburner, ash storage.

Block diagram of scrubber wastewater treatment plant — raw wastewater pump, ferrous sulfate polymer, chelating agent, reaction tank agitator, settling tank scraper, sludge thicker, dehydrator sulfuric acid, cooling tower.

Fig. 1-14 Conceptual diagram of "Fully automatic" and "Block linkage"

1.4 Present state of treatment operations

Although several new conceptions, which have few records of performance, such as, integrated treatment of sludge cakes from various sources and independent treatment of scrubber wastewater, are incorporated in the system, no serious difficulties have been encountered so far.

1.4.1 Sludge incineration

The results of sludge incineration in fiscal 1980 are shown in Table 1-6.

Table 1-6 Performance of sludge incineration

			Fiscal 1980
Amount incinerated			t
32,171			
Ash	Amount	t	3,386
	Rate of production	%	10.5
	Water content of ash shipped	%	24.9
Fuel	Incinerator	Amount used	m ³
		701,688	
	After burner	Per ton of cake	m ³ /t
		21.8	
	Total	Amount used	m ³
		714,451	
Water	Incinerator	Per ton of cake	m ³ /t
		22.2	
	After burner	Amount used	m ³
		1,416,139	
Caustic soda	Total	Per ton of cake	m ³ /t
		44.0	
Electric power	Amount used	m ³	438,154
		Per ton of cake	m ³ /t
Time of operation of feeding conveyor	Amount used	t	185
		Per ton of cake	kg/t
	Amount used	kWH	1,613,425
		Per ton of cake	kWH/t
	Amount used	hr	50.2
		7,192	

Type of Dehydrator	Amount produced and incinerated	Percentage of production	Percentage of amount incinerated to amount produced
Total	32,000t 79,600t Incineration	100%	40.2%
CF	24,740t 51,600t Incineration	64.9%	47.9%
BF	5,120t Inc. 6,520t	8.2%	78.5%
PF	2,140t 21,380t Inc.	26.9%	10.0%

Fig. 1-15 Amount of dewatered cake produced and incinerated in fiscal 1980

a. Properties of sludge cakes

The amount of sludge cake produced and incinerated in fiscal 1980, classified by the process of dewatering are shown in Figure 1-15.

Three kinds of dehydrator, vacuum filter (BF), pressure filter (PF), and centrifuge (CF) are used in this city's sewage treatment plants.

Of about 79,600 ton of sludge cake produced in 1980, 32,000 ton were incinerated and the rest were disposed of by reclamation. Incinerated cakes were the sludge cakes from CF which were of the kind ill-suited for handling, because they contained a high percentage of water and had a strong odor. The yearly average water content and calorific value of the mixed sludge cakes were about 76% and 2500 kcal/KG-DS, respectively.

b. Properties of ash

Since ash is comprised of fine particles and does not contain water, it tends to scatter into the air when it is discharged from the incinerator. It is humidified (to a water content of about 25%) before transportation. The ratio of the weight of humidified ash to that of sludge cake is about 1 to 7.

Table 1-7 Results of analysis of exhaust gas

Item	Analysed value	Regulated value
Volume of dry gas Nm ³ /hr	8750 - 16650	None
O ₂ %	10.6 - 14.3	None
Dust g/Nm ³	0.0014 - 0.0185	0.20
SO _x (at O ₂ =12%) ppm	Less than 0.5 - 73	300
NO _x (at O ₂ =12%) ppm	73 - 160	None
Ammonia ppm	0.9 - 2.55	247
Trimethylamine ppm	0.001 - 0.023	1.21
Hydrogen sulfide ppm	0.0016 - 0.710	4.85
Methyl mercaptan ppm	Less than 0.0005 - 0.270	*0.002
Methyl sulfide ppm	0.0017 - 0.202	*0.01
Dimethyl sulfide ppm	0.0004 - 0.040	2.18
Acetaldehyde ppm	Less than 0.0005 - 0.015	12.0
Styrene ppm	Less than 0.0001	97.0

Note 1. Analysed values were obtained from gas sampled at the stack exit.

Note 2. Regulated values with a * mark indicate values at the land surface at the boundary of the plant area.

c. Exhaust gas

The results of analysis of exhaust gas are shown in Table 1-7. The percentages of removed harmful substances were much higher than the values specified by the regulation: 98 to 99.9% for dust and 92 to 95% for sulfur oxides.

1.4.2 Scrubber wastewater treatment

The quality of the scrubber wastewater, the results of treatment and the amount of chemicals used in fiscal 1980 are shown in Tables 1-8, 1-9, and 1-10 respectively. Because the concentration of copper in the scrubber wastewater has been as low as 0.05 mg/l since the beginning of operation, the addition of chelating agent has never been required. Variations in the concentrations of heavy metals through the year showed that the addition of ferrous sulfate has not been required either. The commencement of operation as Kanazawa Sewage Treatment Plant in December, 1980 allows for the pumping of scrubber wastewater to the plant and a ban on the addition of polymeric coagulant. Because the concentration of cyanide was recognized as being as high as 0.85

Table 1-10 Actual rates of use of chemicals

Name of Chemicals	Rate of use
Ferrous sulfate (15% conc.) ppm	317
Caustic soda (45% conc.) ppm	104
Polymer coagulant (0.1% conc.) ppm	0.86
Sulfuric acid (75% conc.) ppm	128
Sodium hypochlorite (12% conc.) ppm	58

Table 1-8 The quality of raw wastewater

Item	Tests	Quality
pH	-	5.5 - 7.5
BOD mg/l	-	7.4 - 49
COD mg/l	40	15 - 40.4
SS mg/l	414	44 - 110
T-CN mg/l	-	0.85 - 5.32
Cd mg/l	0.27	0.02 - 0.12
Pb mg/l	1.00	0.22 - 0.95
Cu mg/l	3.29	0.05 - 0.09
Zn mg/l	16.55	0.43 - 1.36
T-Hg mg/l	0.07	0.0015 - 0.006
F mg/l	-	0.15 - 0.93
Phenols mg/l	-	0.12 - 0.20

Table 1-9 Results of treatment of scrubber wastewater

	Fiscal 1980	Regulated value
Volume of treated water m ³	699,365	-
Volume of thickened sludge m ³	303	-
Sludge Weight produced t	46	-
Cake Water content %	67	-
pH	8.3	5.8 8.6
BOD mg/l	12.6	(20) 25
COD mg/l	13.0	25
SS mg/l	1.0	(50) 70
T - CN mg/l	0.08	1.0
Cd mg/l	N.D	0.1
Pb mg/l	N.D	1.0
Cu mg/l	0.02	1.0
Zn mg/l	0.046	1.0
T - Hg mg/l	0.0011	0.005
Fe mg/l	0.29	15
Phenols mg/l	0.18	0.5

Note 1. Analysed values are the average of 2 to 3 values in the year.

Note 2. Regulated values are cited from the environmental pollution control ordinance of Kanagawa Prefecture.

Note 3. Values in brackets are the average values.

to 5.32 ppm, it was decided to add sodium hypochlorite in the reaction tank.

Accordingly, the scrubber wastewater is at present pumped back to the sewage treatment plant after being conditioned to pH 9.4, having the cyanide removed and being neutralized.

1.5 Results of operation

1.5.1 Operating process

Receiving gate indication for trucks and automatic control of bucket crane operation have been going on quite satisfactorily.

Although suspension of operation of the constant rate feeder due to over-load caused by blocking of the spiral-blades of the conveyor by foreign matter mixed in sludge cake have occurred a few times each year, improvements, which will be described later, enabled expedite elimination of obstacles and maintaining of smooth operation.

Sludge incinerator was operated under constant temperature control (800°C) by use of auxiliary burners. However, the spontaneous combustion phenomenon took place 5 to 6 times a month, 20 to 30 hours in total. This phenomenon is commonly seen in plants which treat mixed cakes from various sources. It was possible to maintain the temperature inside the incinerator constant (set at 830°C) even when temperature tended to rise by spontaneous combustion, by automatically controlling the cooling damper device. Figure 1-16 shows the temperature at various parts in the

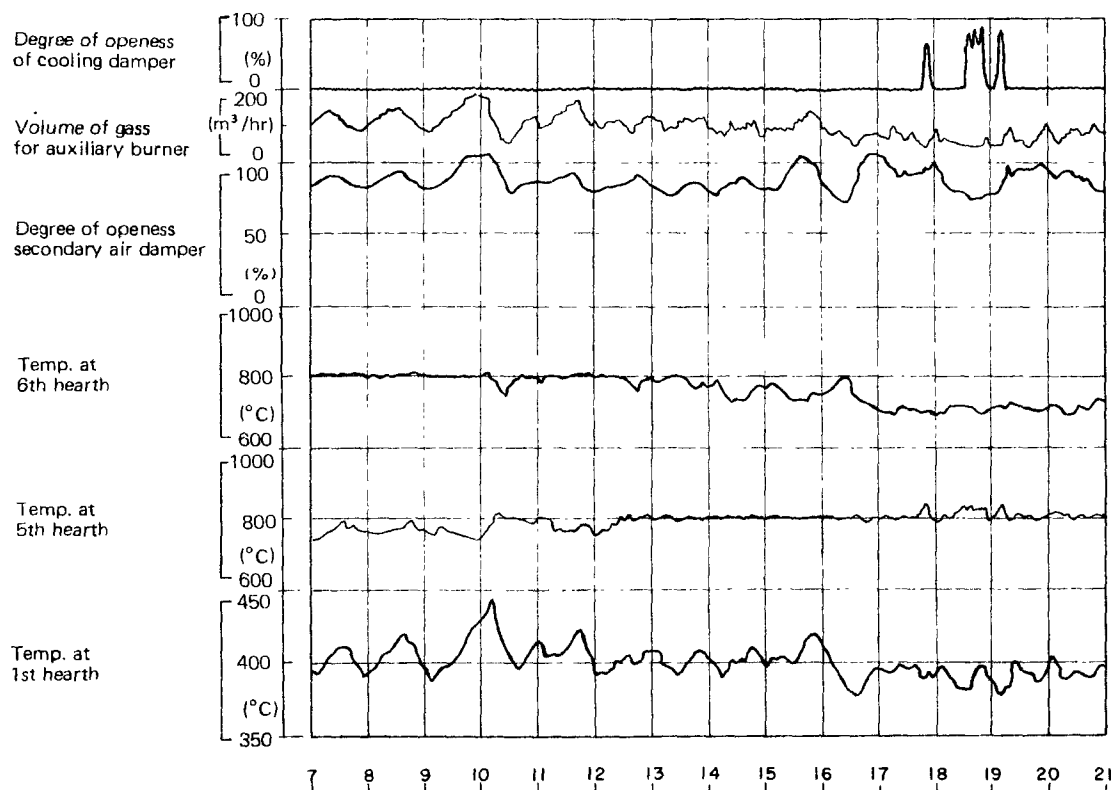


Fig. 1-16 Time-chart for controlling incinerator

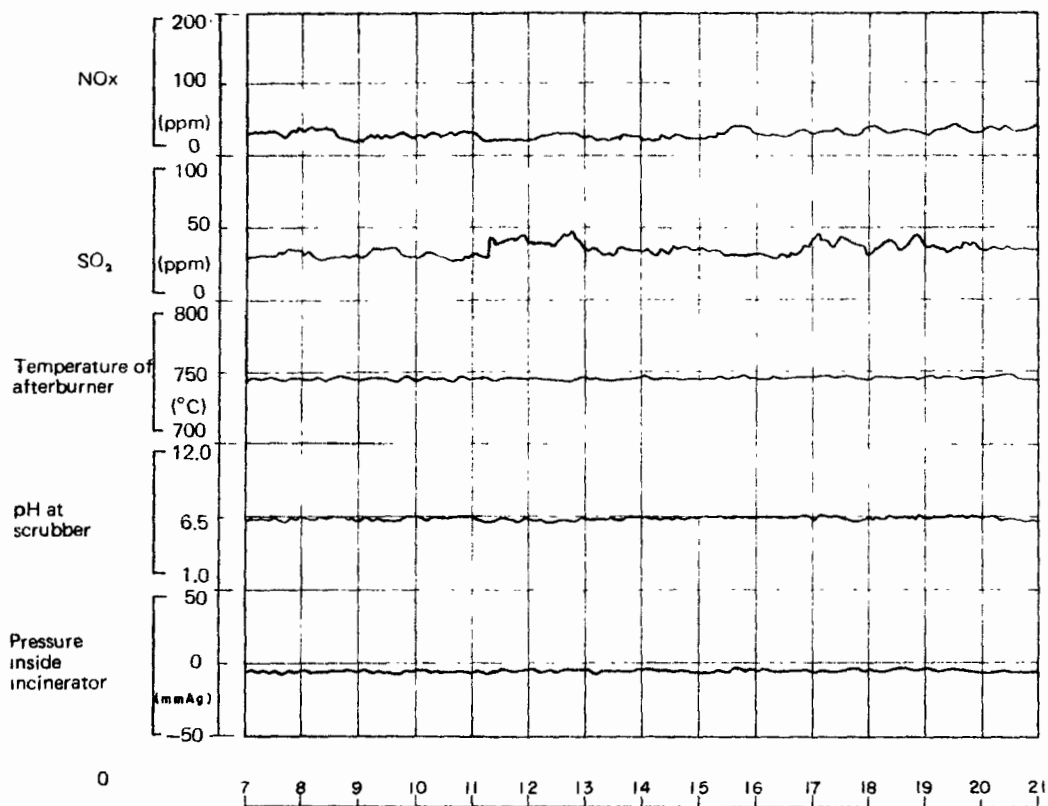


Fig. 1-17 Time-chart for treatment of exhaust gas

incinerator under normal conditions, flow of auxiliary burner gas, and a time chart indicating the operation of dampers.

In the exhaust gas treatment plant, stable operation could be maintained by the automatic control equipment for maintaining constant pressure inside the incinerator (-6 mmAq) by the furnace pressure control damper, constant pH at the absorption cooling tower (pH 6.4), constant voltage at the electrostatic precipitator (50 kV), and constant temperature inside the deodorizing furnace (800°C). Figure 1-17 shows the time chart of exhaust gas treatment operation.

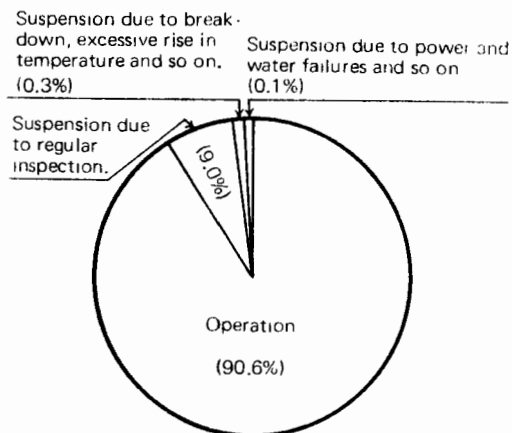
Operation of the ash conveyor and the whole incinerator had to be suspended several times because foreign matters mixed in the cake blocked the conveyor. Increasing the water content of ash to 25 percent facilitated the transportation of ash.

Functions of various kinds of control equipment incorporated in the scrubber wastewater treatment plant were quite satisfactory.

1.5.2 Rate of operation and load factor

The state of operation of the facility is shown in Figure 1-18. The rate of operation of the facility throughout fiscal 1980 was 90.6 percent and the rate of suspension due to power and water failures was 0.1 percent, and that due to breakdown of equipment and excessive rise in temperature was 0.3 percent. The load factor throughout fiscal 1980 was 108.7 percent. The fact that the facility could operate

above the rated load factor is credited to the stable operation brought about by automatic control notwithstanding incineration of various kinds of sludge cakes and to the control of the secondary air which yielded surplus capacity of the exhaust gas treatment facilities including the induced draft fan.



1.5.3 Improvements

After commencement of operation of the facility, the following improvements were incorporated to solve the difficulties brought about by the mixing of foreign matter in the sludge cake, to economize in energy usage, and to maintain stable operations of the whole facility.

- a. Improvement of the constant rate feeder for saving difficulties caused by the mixing of foreign matter

Figure 1-19 shows the improvement in the constant rate feeder. In the running in operation prior to full-scale operation, foreign matter, such as, concrete lumps, pieces of steel pipe, and empty cans, caused the overload protection device to actuate several times and also resulted in deformed screw shaft and broken bearing cases and finally the suspension of operation of the whole incinerator.

Equipment was improved to make the conveyor less liable to damage and more readily repairable so that operation could be resumed without serious difficulties. In order to prevent foreign matter from blocking the screw conveyor, the intervals between the spiral blades were made longer and the clearance between the blades and the bottom plate was increased to about 200 mm to allow foreign matter to fall to the bottom.

To eliminate difficulties before they result in suspension of operation the driving mechanism of the conveyor was divided into 3 sections so that each section may be broken down and checked separately. By virtue of the improvement, no suspension of the feeding operation to the incinerator occurred, although the over-load protection device was actuated three times.

Also, improved equipment allowed faster repairing work. At the annual cleaning of the constant rate feeder, foreign matter with a total volume of 0.1 m³ was taken out of the bottom of the conveyor.

- b. Improvements in energy saving and securing of stable operation

Improvement in the control of secondary air is shown in Figure 1-20. Originally, secondary air was controlled manually by an operator who monitored the temperature inside the incinerator and adjusted the volume of secondary air as a matter of experience. Although stable control of temperature could be attained by this method, manual operation tended to make a large allowance for the volume of air, because the facility treated various kinds of cakes which were

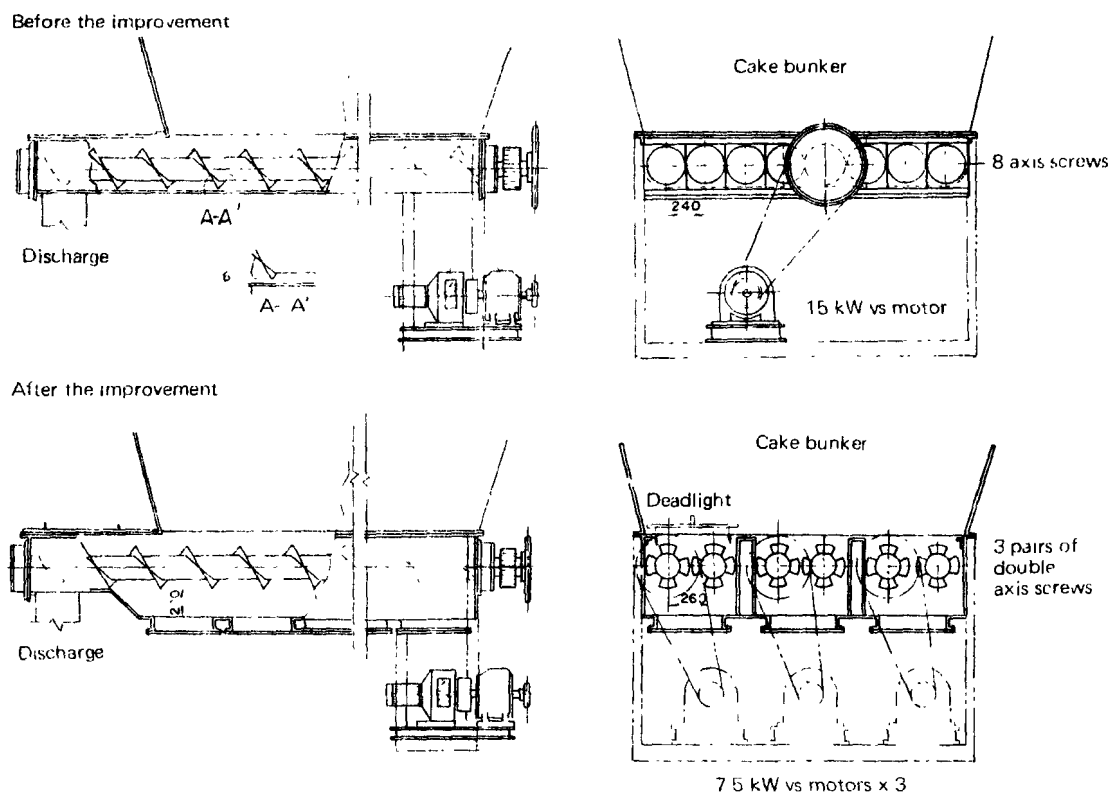


Fig. 1-19 Improvements to constant rate feeder

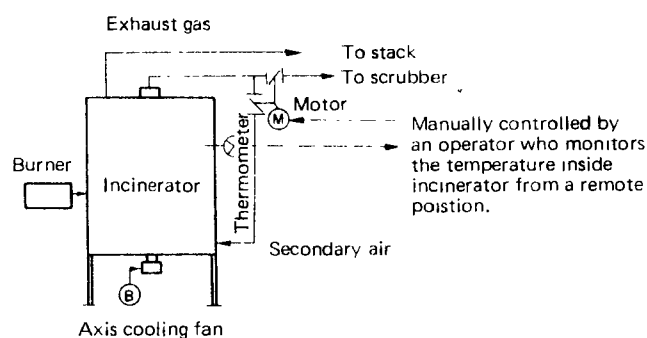
produced at different dewatering processes, consequently their quality varied greatly. Improvement was made to limit the volume of secondary air to the minimum to save both fuel and scrubbing water and to stabilize the operation. In order to maintain the concentration of O_2 in the exhaust gas at the prescribed value a cascaded type control system was adopted. The high order control system was made to transmit the set value of the temperature of exhaust gas at the exit of the scrubber to the low order control system so that the temperature at the 2nd hearth of the incinerator may be kept constant, and the low order control system, which controls the secondary air damper, was employed so that the exhaust gas temperature at the exit of the scrubber may be maintained at the set value. As a result, stable operation, which required the resetting of the 2nd hearth temperature only once or twice at the most, could be attained.

As for the saving in operation cost, the improvement brought about the saving of fuel by 34 percent (¥95 million a year) and of scrubbing water by 25 percent

Table 1-11 Comparison of results of operation before and after the improvements in secondary air control

Items	O_2	Air ratio	Volume of Gas			Volume of scrubbing water	Weight of cake incinerated
			Auxiliary burners	After burners	Total		
Before improvements	About 12%	2.6	29.8 m ³ /t	50.2 m ³ /t	80.0 m ³ /t	28.9 m ³ /t	96.3 t/day
After improvement	About 9%	1.7	17.6 m ³ /t	35.2 m ³ /t	52.8 m ³ /t	21.6 m ³ /t	107.5 t/day
Rate of increase or decrease	—	—	-41.0%	-30.0%	-34.0%	-25.0%	+ 10.4%

Before Improvements



After Improvements

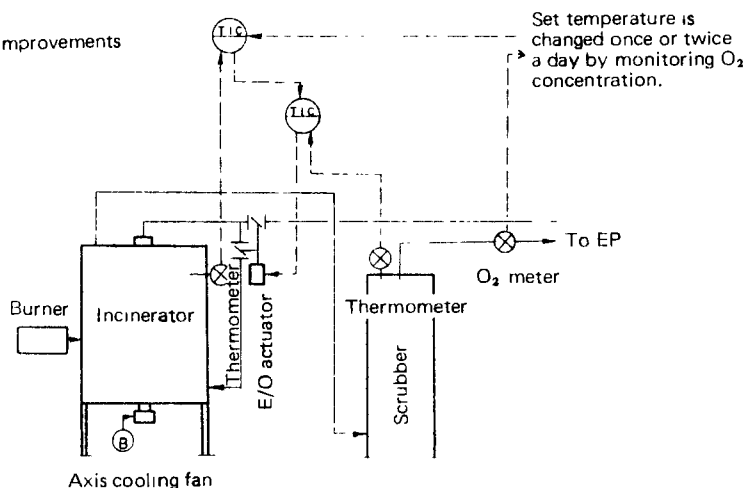


Fig. 1-20 Improvements in control of secondary air

(¥51 million a year). Further, in order to save the cost of handling and disposal of ash (consignment fee for disposal), an improvement was made on the spray nozzle to the ash humidifier resulting in an increase in spraying effect and reduction of usage by about 29 percent (more than ¥4 million a year).

1.6 Costs

1.6.1 Construction cost

Construction work on these facilities was started in July 1975 and it took about 3 years before the facilities started operation. Construction cost of the facilities including the sludge incineration and the scrubber wastewater treatment plant was about ¥2,240 million.

Details of the expenses are listed in Table 1-12.

Table 1-12 Construction cost

Unit 1,000 yen			
	Sludge incineration facility	Scrubber waste-water treatment plant	Total
Mechanical equipment	773,000	304,900	1,077,900
Electrical equipment	320,500	213,300	533,800
Construction works	482,200	149,754	631,954
Total	1,575,700	667,954	2,243,654

1.6.2 Maintenance and operations costs

The cost of incinerating 1 ton of sludge cake in fiscal

1980 was ¥13,492 (utility cost ¥8,193), of which about 50 percent was fuel cost and 16 percent was labour cost.

If kerosene had been used as the fuel, the cost of incineration would have been ¥8,911 per ton (utility cost ¥3,611 per ton). The cost of treating the scrubber wastewater was ¥1,513 per ton of sludge cake, of which about 45 percent was the cost of chemical and 42 percent was the cost set against depreciation.

Details of operating costs are shown in Table 1-13.

Table 1-13 Operation cost

Unit: yen	
Item	Cost during April, '80 through March '81
Fuel	214,386,000
Lighting and water	12,663,000
Electric power	31,811,000 (6,249,000)
Chemical	3,307,000 (21,586,000)
Total cost of utilities	262,167,000 (27,835,000)
Repairs	11,317,000
Fixtures & expendables	9,148,000 (39,000)
Ash disposal	30,150,000 (131,000)
Labour costs	71,015,000
Depreciation	47,975,000 (20,440,000)
Grand total	431,772,000 (48,445,000)
Cost for treatment of dewatered cake (yen per ton)	13,492 (1,513)

Note 1 Fuel: City gas ¥154 per Nm³ (11,000 kcal/Nm³)
Note 2 Electric charges: ¥15 per kWh
Note 3 Basis of depreciation: Residual price 10%. Period of depreciation 20 years
Note 4 Treated water is used for scrubbing
Note 5 Figures in brackets indicate the cost of scrubber wastewater treatment process

2. Incineration of Sludge Mixed with Pulverized-coal in a Fluidized Bed System

The construction of incineration facilities for pulverized-coal mixed with sludge was started in December 1978 in the Hokubu second sewage treatment plant. It was designed on the basis of results obtained from experiments conducted at the pilot plant from January to October in 1977 (presented at the 6th U.S./Japan Conference on Sewage Treatment Technology). The plant was completed in March 1981 and went on stream after adjustment during a trial operation period.

2.1 Outline of incineration processes

2.1.1 Circumstances

Conventional sludge incineration systems require a large quantity of oil energy as the supplementary fuel. The prevention of secondary pollution from these systems should also be considered concerning the exhaust gases and burned ashes generated in the incineration process. The recent worsening of the oil situation makes it essential to save energy by heat recovery etc., and to conserve oil by using alternative forms of energy, and also to take effective countermeasures for preventing secondary pollution.

To eliminate various problems associated with the incineration of sludge, pilot plant experiments with a fluidized bed furnace were carried out by using pulverized coal as a supplementary fuel and cement clinker as a fluidizing medium. Results were obtained on the spontaneous-burning of sludge cakes, furnace bed load, SO_x control, O₂ and NO_x concentrations, CO concentration and inhibition of oxidation to hexavalent chromium, etc. These observations confirmed that the incineration system with pulverized-coal was effective and the process was adopted for practice.

2.1.2 Treatment process

The treatment process of this incineration system is composed of feeding, incineration, exhaust gas treatment and ash disposal. The overall plan of the plant is given in Fig. 2-1, the flow sheet in Fig. 2-2, the layout in Fig. 2-3, and the main equipment specifications in Table 2-1.

a. Feeding and incineration

Sludge cakes and pulverized-coal are simultaneously metered into the kneader, continuously mixed, and fed into the fluidized bed incinerator. The kneaded cakes are fed into the furnace and

Table 2-1 Specifications of main equipment

Item of equipment	Specification	Q'ty	Remarks
Kneader	Double-axle continuous type Drive Driven = 2 1 rotation ratio 6t/h \times 15 kw	2	1 for spare
Pulverized-coal silo	Effective volume 100m ³ 330-3060 kg/h \times 5.5 kw Discharger with pipe	2	10 days' stock
Pulverized-coal feed hopper	Effective volume 1.5m ³ 250-1000kg/h \times 1.5 kw with constant rate discharger	2	1 for spare
Incinerator	Fluidized bed furnace Sludge cake 100t/day ϕ 3400mm \times H 8000 mm	1	
Medium silo	Effective volume 20m ³ 3t/h \times 1.5 kw with discharger	1	10 days' stock
Medium feed hopper	Effective volume 0.2m ³ 300 kg/h \times 1.5 kw with feeder	1	
Double cyclone	Heat-resistant type 16,290 Nm ³ /h \times 900°C	1	
Air preheater	Vertical cylinder type heat exchanger 14,500 Nm ³ /h Heating surface 146m ²	1	Inlet 8, 0°C Outlet 730°C
Waste-heat boiler	Twin bodied natural circulation boiler 14,500 Nm ³ /h Heating surface area 420m ²	1	Inlet 730°C Outlet 300°C
Multi-cyclone	Multi-cyclone type 14,500 Nm ³ /h \times 300°C	1	6 cyclones
Electrostatic precipitator	Dry horizontal flow type 14,500 Nm ³ /h \times 300°C Outlet dust content 0.02g/Nm ³ max	1	
Stack	Stainless steel stack supported by iron tower ϕ 1,500 mm \times H 30,000 mm	1	
Forced draft blower	Turbo-blower 180m ³ /min (at 20°C) \times 2,000mm Aq \times 3,000 rpm \times 100 kw	1	Driven by steam turbine & motor
Positive draft fan	Turbo-fan 150m ³ /min (at 20°C) \times 300mm Aq \times 1,500 rpm \times 15 kw	1	
Induced draft fan	Turbo-fan 750m ³ /min (at 300°C) \times 450mm Aq \times 1500 rpm \times 100 kw	1	Driven by steam turbine & motor
Gas recirculation blower	Turbo-blower 70m ³ /min (at 300°C) \times 1,800mm Aq \times 3,000 rpm \times 75 kw	1	
Burner air compressor	Recipro-automatic unloader type 5.0m ³ /min \times 7 kg/cm ² G \times 37 kw	2	
Incombustibles separator	Amplitude type 200 - 300kg/h (at 800 - 200°C) \times 1.5 kw	1	
Ash cooler	Screw type conveyor with water-cooled jacket 200 - 1060 kg/h (at 600°C) \times 1.5 kw	1	
Ash bunker	Effective volume 45m ³ with ash humidifier	2	days' stock

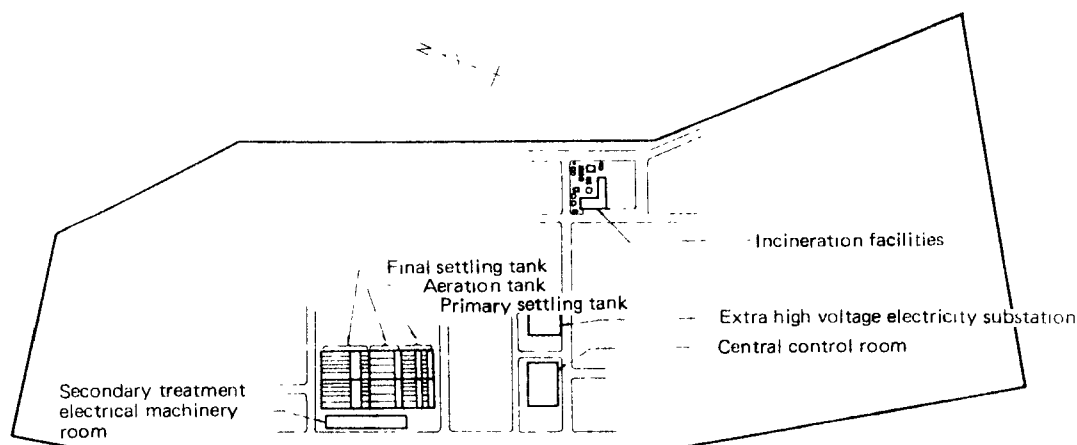


Fig. 2-1 Comprehensive layout of the Hokubu Second sewage treatment plant

Fig. 2-2 Flow sheet of incineration facilities

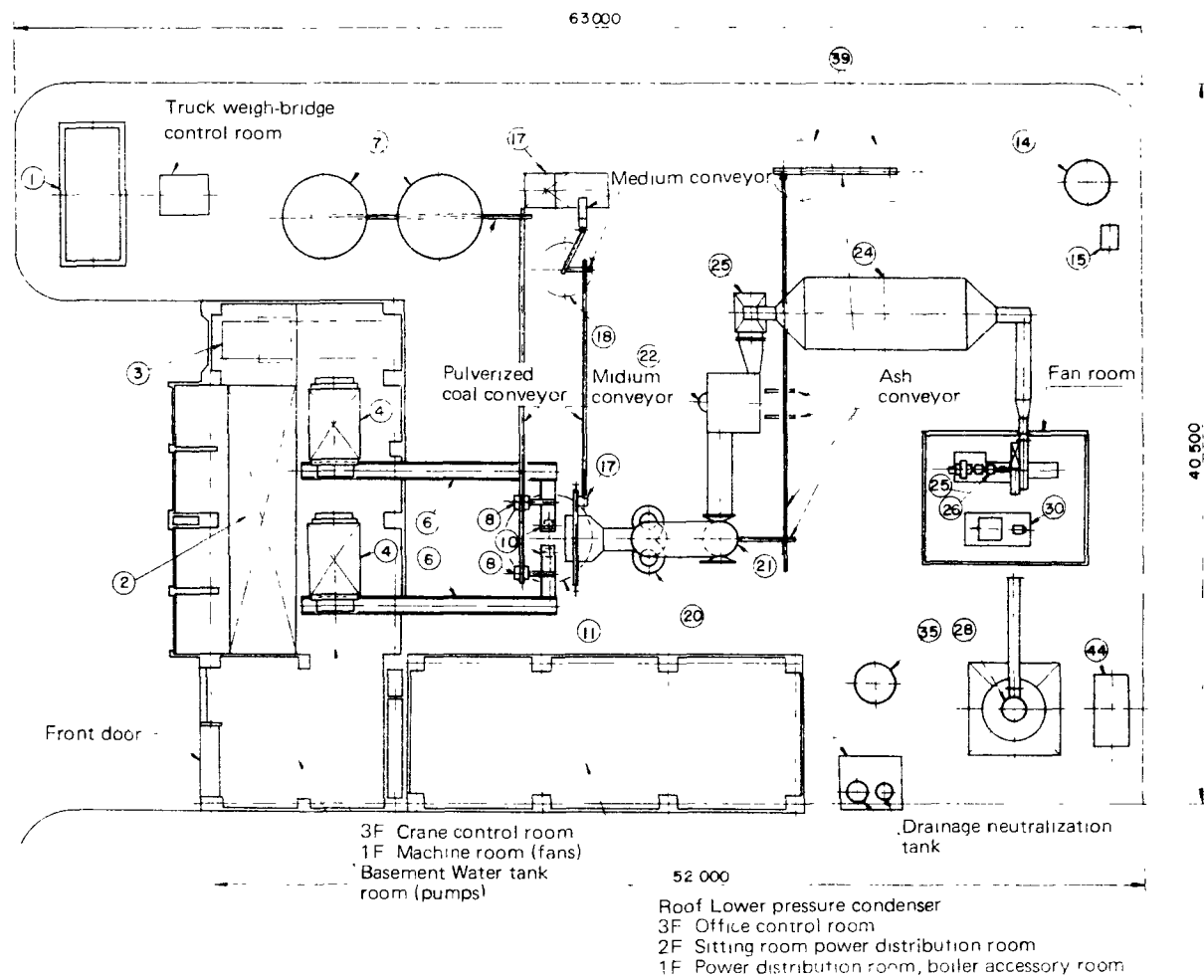


Fig. 2-3 Layout

incinerated in a fluid medium at about 800°C.

b. Exhaust gas treatment

Most of the high temperature ash is removed from the exhaust gas by means of double cyclones and heat is recovered from the gas with a reduced dust concentration in the air preheater and waste-heat boiler. Fine dust is removed in the multi-cyclone and dry-type electrostatic precipitator, and the residual gas is discharged from the stack into the air.

c. Ash treatment

Ash discharged from each piece of equipment in the exhaust gas treatment process is transferred to an ash bunker by the conveyor, moistened, and disposed of.

2.1.3 Design elements

Design elements of the incineration system are illustrated in Table 2-2.

Table 2-2 Design elements

Item				Specification					
Incineration capacity				Sludge cake 100 ton/day (max. 120%/30 min)					
Properties of sludge cake	Kind of sludge cake			Pressure dewatered cake with inorganic coagulant (Sludge VTS 40 – 80%)		Sludge cake centrifuged with high polymer (Sludge VTS 40 – 80%)			
	Water content			55 – 75%		70 – 80%			
	Gross calorific value			1,300 – 2,500 kcal/kgDS		2,500 – 4,500 kcal/kgDS			
	Solid composition			13.5 – 22.6%	2.9 – 4.6%	1.8 – 2.9%	20.6 – 41.3%	3.3 – 6.6%	2.7 – 5.3%
	C	H	N						
Pulverized-coal consumption				About 570 – 690 kg/h		About 490 – 530 kg/h			
Cement clinker consumption				About 100 kg/h					
Hazardous components	Dust quantity			0.02 g/Nm ³ max.					
	Sulfur oxides			50 ppm max.					
	Nitrogen oxides			80 ppm max. (O ₂ 5%) [45 ppm (O ₂ 12%)]					
	Dissolved hexavalent chromium from ash			0.3 mg/l max.					
Gas temperature at furnace outlet				800°C min. 900°C max.					
Heat loss of incinerated ash				3% max.					

2.1.4 Construction cost

The incineration facilities were built during Fiscal 1978 to 1980 and the total sum of the construction costs amounted to ¥2,626 million. The cost in detail is shown in Table 2-3.

Table 2-3 Items of construction cost

Item	Construction cost	Ratio
Civil	185 Mil Yen	7.1%
Building	455 Mil Yen	17.3%
Machinery	1,292 Mil Yen	49.2%
Electricity	694 Mil Yen	26.4%
Total	2,626 Mil Yen	100%

Note: 1. Year of construction Fiscal 1978 to 1980.
2. Building includes the electricity costs for such as illumination, ventilation and sanitary equipment.

2.2 Characteristics of incineration facilities

Characteristics of the incineration facilities include the application of pulverized-coal as a supplementary fuel, the use of cement clinker as a fluid medium, adoption of the dry-type exhaust gas treatment, heat recovery by the waste-heat boiler etc.

2.2.1 Facilities for pulverized-coal

The facilities for pulverized-coal are composed of the equipment concerning storage and feeding. The flow sheet of pulverized-coal is given in Fig. 2-4, the constant rate feeder of pulverized-coal in Fig. 2-5, and the structure of the kneader in Fig. 2-6.

The ease with which pulverized-coal can be applied to constant rate feeding and kneading is a factor directly affecting the stability and uniformity of fluidized bed temperature.

The constant rate feeder of pulverized-coal provides a model which is less affected by the variation of bed thickness in the hopper.

The kneader is required to perform continuously, uniform mixing and discharge, and hence it provides reverse blades in the outlet side and variable speed biaxial paddles with a rotation ratio of 2:1.

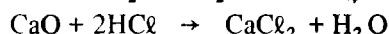
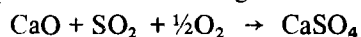
Besides which, the facilities for pulverized-coal incorporate safety measures for inhibiting temperature rise due to solar heating effect which includes heat shielding plates on the upper part of silos for pulverized-coal, cooling water spray nozzles around the perimeter, heat shielding plates which cover pulverized coal conveyors, and heat insulation layers on the outside surface of pulverized-coal feeding hoppers. In addition, all equipment has static electricity grounding as well as being equipped with carbon dioxide fire extinguishers for fire prevention.

2.2.2 Sludge incineration furnace

The structure of the sludge incineration furnace is given in Fig. 2-7 and its specifications in Table 2-4.

a. Inhibition of hazardous components

Hazardous components are SO_x, HCl and NO_x in the exhaust gas, hexavalent chromium in the incinerated ashes, etc. SO_x and HCl are fixed by being absorbed into the cement clinker (Principal component is CaO. Activity exceeds 50%) applied as the fluidizing medium according to the following equations.



The generation of NO_x and the oxidation of trivalent chromium in the incinerated ashes to hexavalent chromium are inhibited by a reducing environment produced in the furnace by recycling waste gas or decreasing the air content. Malodorous materials are completely decomposed by oxidation at exhaust gas temperatures of 800 – 900°C.

b. Feed and discharge equipment for fluidizing medium

The furnace provides feed, overflow and underflow equipment for automatically controlling the correct bed height and properties of cement clinker which con-

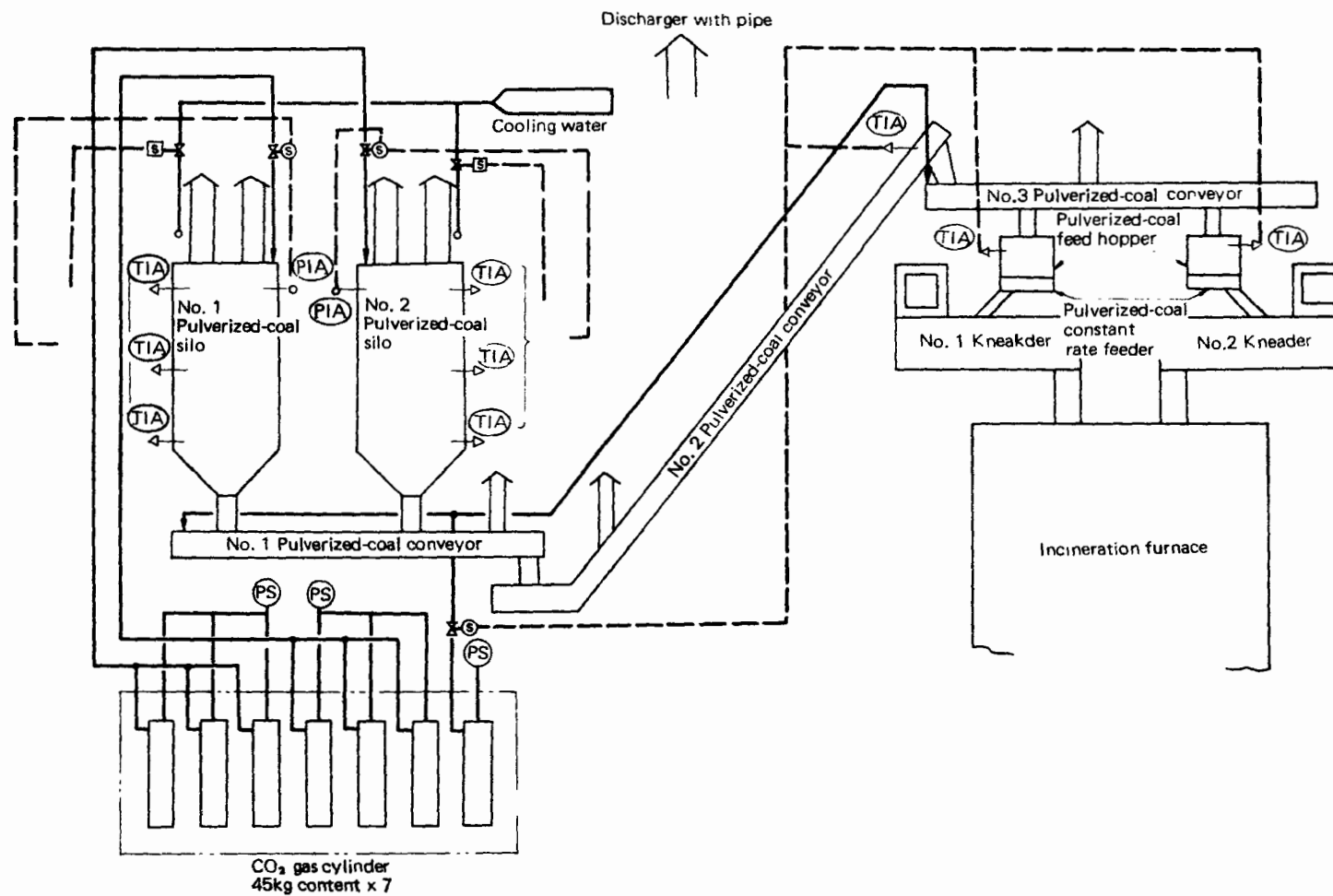


Fig. 2-4 Flow sheet for pulverized-coal system

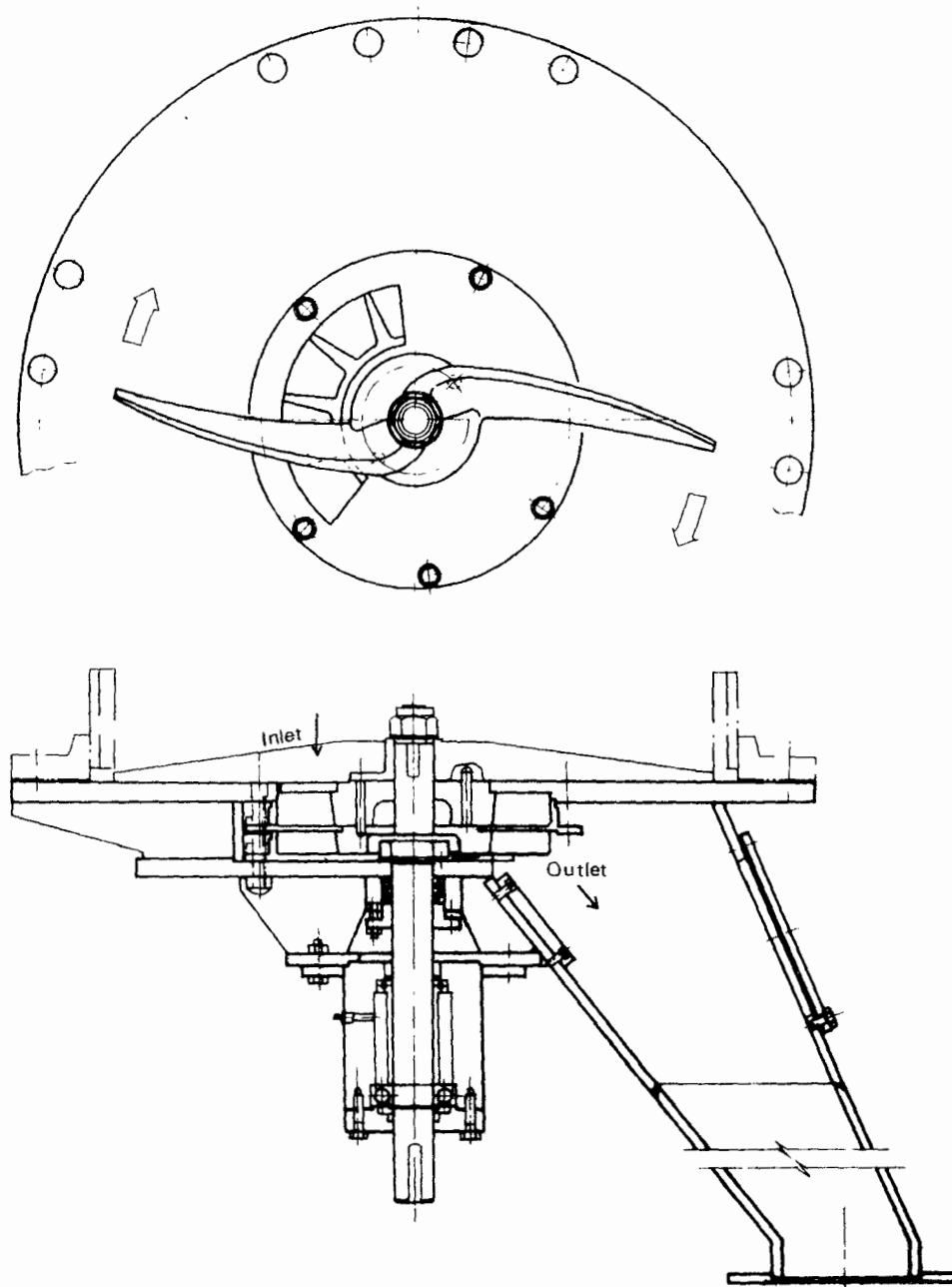


Fig. 2-5 Structure of pulverized-coal constant rate feeder

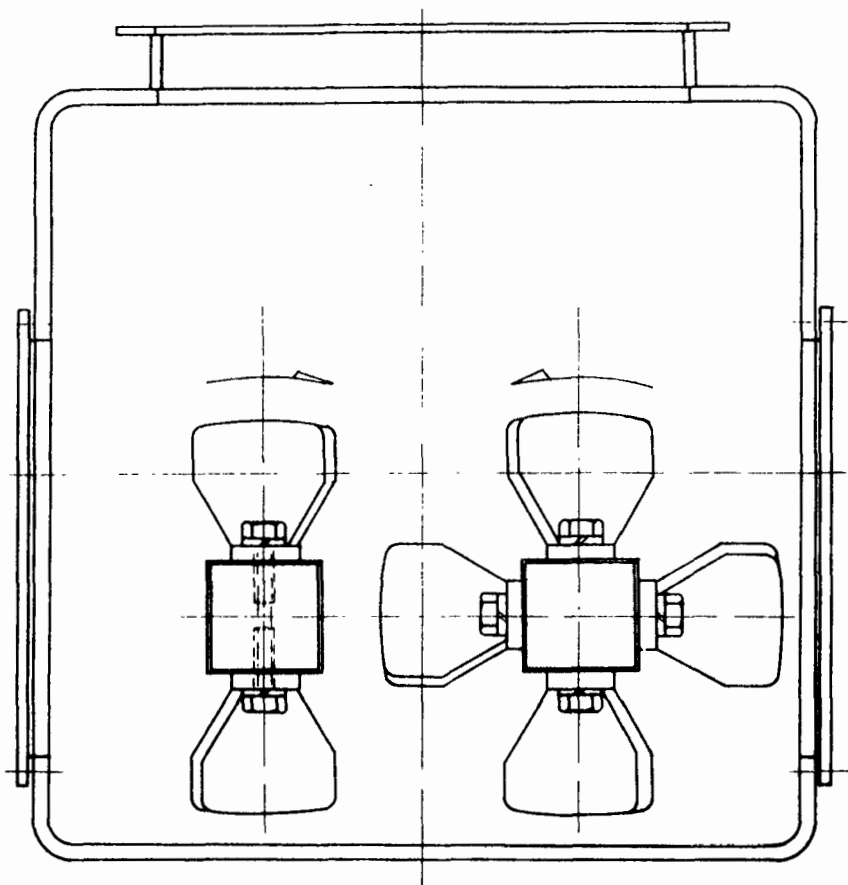


Fig. 2-6 Structure of kneader (Section)

Table 2-4 Specifications of incineration furnace

Items		Specification
Model		Fluidized bed furnace
Incineration quantity		4,720 kg/h (including pulverized coal 553 kg/h)
Furnace bed load		520 kg/m ² ·h
Dimensions	Inner diameter of layer	φ3,400 mm
	Inner diameter of empty drum	φ4,000 mm
	Incinerator height	H 5,400 mm
Materials of main parts	Fire brick	Thickness 230 mm at empty drum, clay brick SK34
	Insulating fire brick	Thickness 230 mm at empty drum, siliceous brick B1
	Fire castable	Crown, bed, layer side, burner chamber, wind box 1,350–1,550°C
	Insulating castable	Crown, bed, layer side, burner chamber, wind box 1,000°C
	Casing	Lower empty drum, layer, din-box, SS41, thickness 12 mm upper empty drum, crown, SS41, thickness 9 mm
	Perforated plate	SB42
	Perforated nozzle	SCH2 x 414
Accessories	Starting burner	300 ℓ/h x 2
	Auxiliary burner	30 ℓ/h (50ℓ/h max.)x10
	Overflow damper	0.75 kw x 1
	Underflow damper	0.2 kw x 1
	Other items	Pressure, temperature, & O ₂ measuring sensors etc.

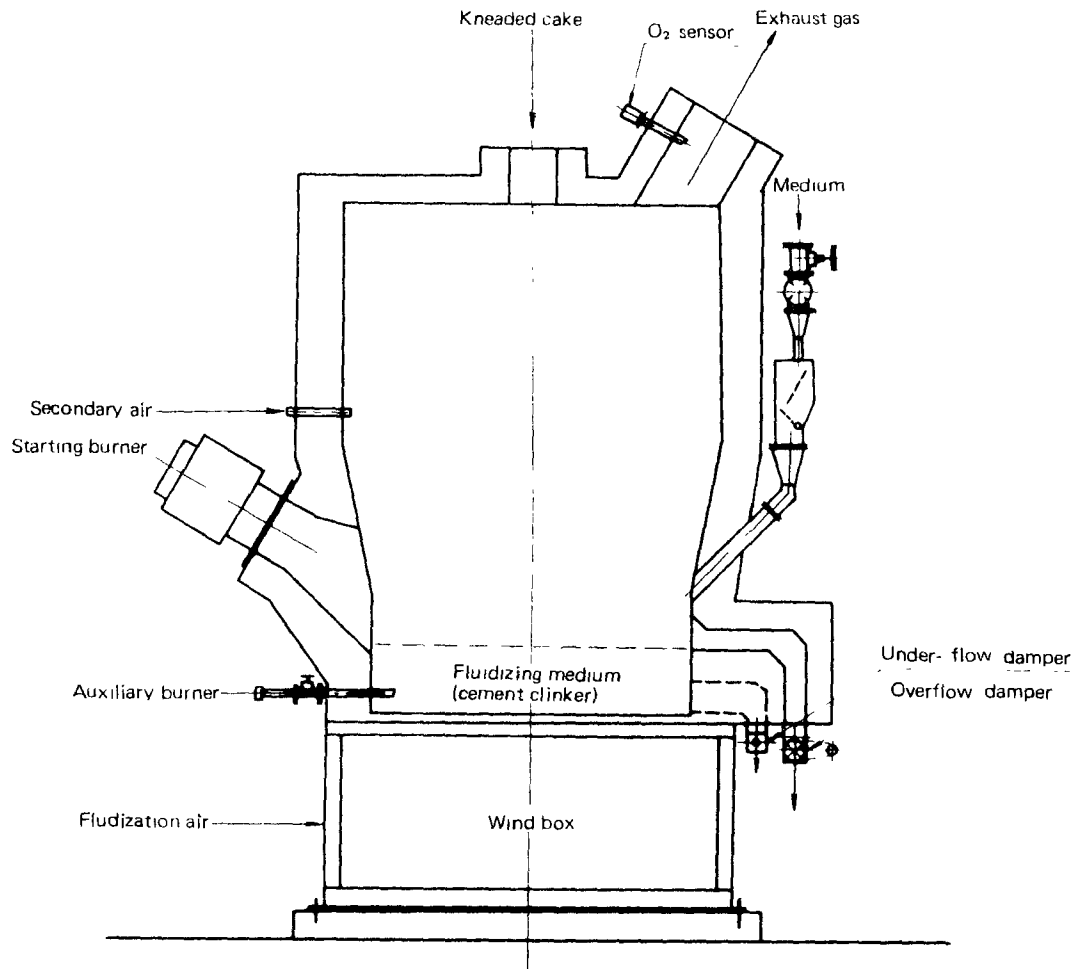


Fig. 2-7 Structure of incinerator

stitutes the fluidized bed (Refer to Fig. 2-7).

c. Fuel

Pulverized-coal is used during normal operation as the supplementary fuel, whereas kerosene fuel is applied during start up and if there is a rapid drop of bed temperature in normal operation.

An example of combustion flow chart is shown in Fig. 2-8.

2.2.3 Equipment for heat recovery

The high temperature exhaust gas discharged from the fluidized bed furnace is subject to heat recovery in the air preheater and the waste-heat boiler. The heat recovery flow is given in Fig. 2-9.

a. Air preheater

The function of the air preheater is to heat fluidizing air up to about 400°C by means of a vertical tubular heat exchanger which is low in dust adhesion and easy to inspect. To prevent blockage the tube is relatively large in diameter and

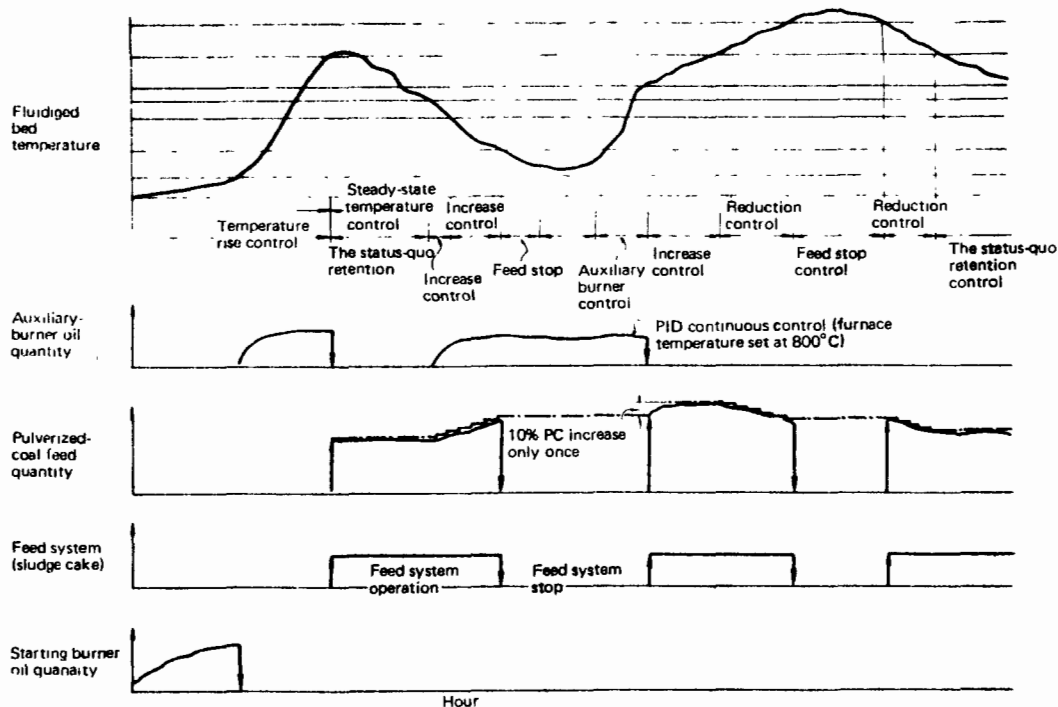


Fig. 2-8 An example of combustion flow chart

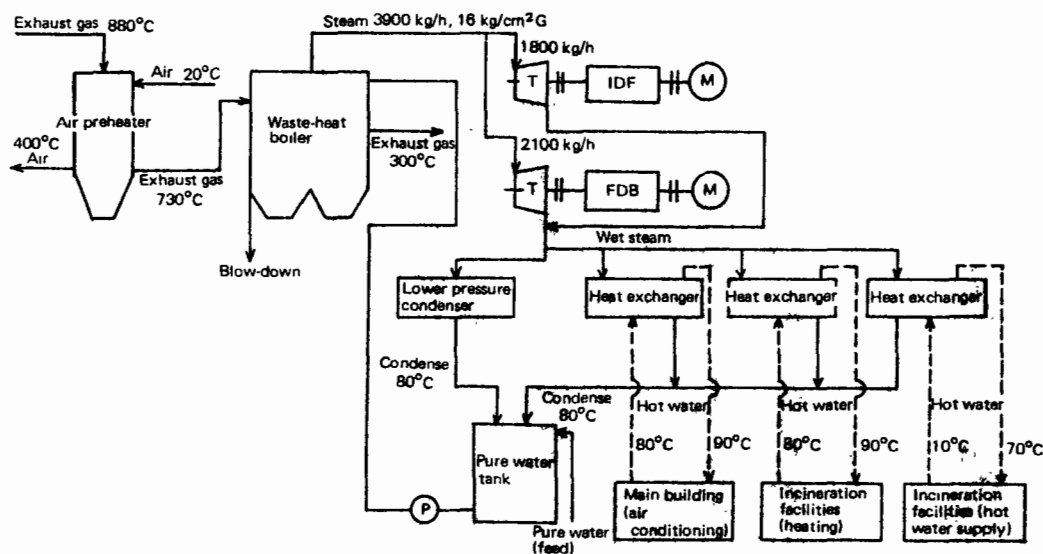


Fig. 2-9 Heat recovery flow diagram

is equipped with a guide sleeve at its upper end. The structure of the air pre-heater is shown in Fig. 2-10.

b. Waste-heat boiler

The waste-heat boiler is a twin bodied natural circulation boiler. The structure of the waste-heat boiler is given in Fig. 2-11. Generated steam is supplied to the

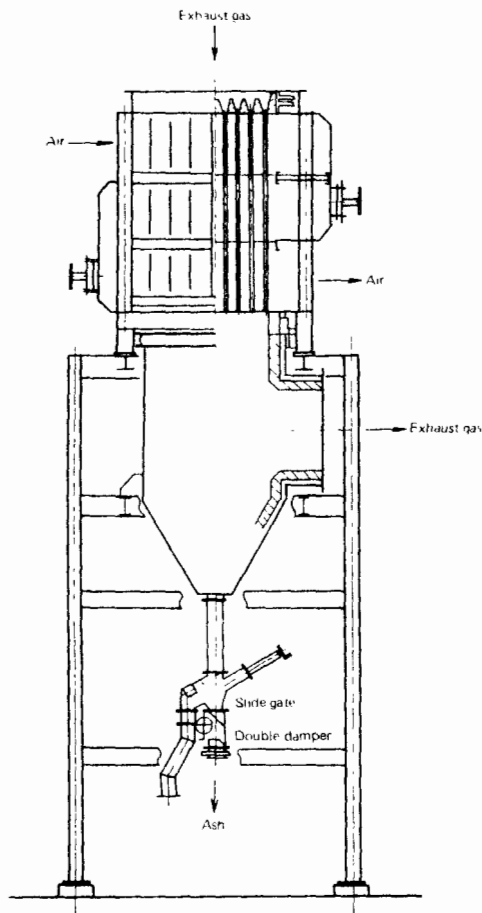


Fig. 2-10 Structure of air preheater

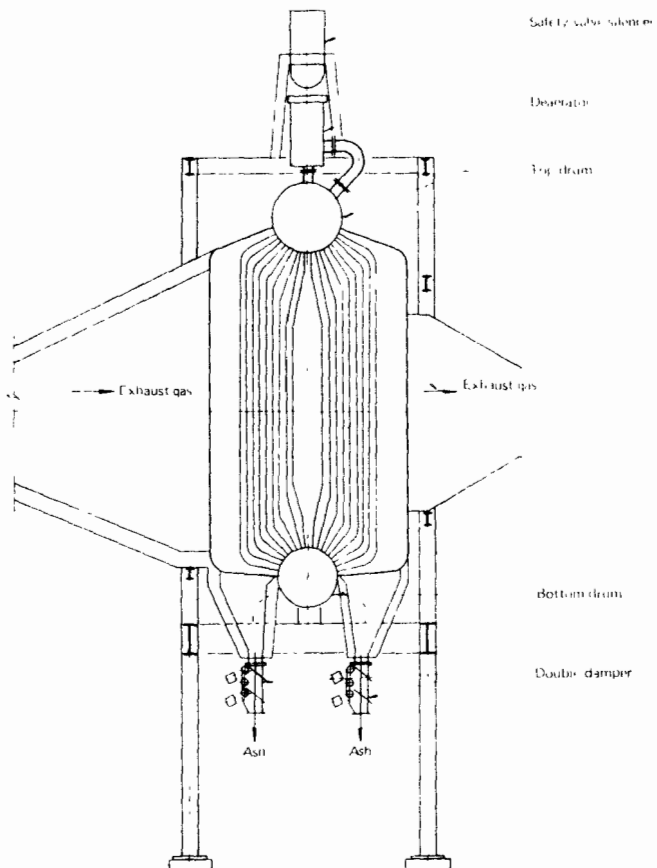


Fig. 2-11 Structure of waste-heat boiler

back pressure turbine which drives the induced draft fan and forced draft blower in the system and reduces by about 32% the electric power consumption in the system. Besides which, the waste steam from the back pressure turbine provides a heat source for the plant management buildings for heating, air conditioning and hot water supply. The condense is used for replenishing boiler water. In addition, the induced draft fan and forced draft blower are driven by two means, steam and motor, because of the requirement for stability at the start and during normal running.

2.2.4 Exhaust gas treatment

As the incineration system inhibits SO_x and NO_x in the incineration process it requires no incidental equipment for alkaliwashing, waste liquid treatment, white smoke prevention, etc. Therefore, dust removal is the sole requirement for waste gas treatment and hence the dry method of treatment is applied. The dust in the exhaust

gas is removed by about as much as 65% in the double cyclones, then further reduced in the air preheater and waste-heat boiler. About 70% of the residual dust is removed in the multi-cyclones, followed by a further elimination of about 99.75% or more of the residue in the dry type electrostatic precipitator, and the resultant gas is discharged from the stack into the air with a dust concentration of 0.02g/Nm^3 or less.

The double cyclones, in addition, are arranged just after the furnace outlet to reduce the dust load on the air preheater. The dust extracted by the double cyclones is at high temperature and great in quantity, and hence a cyclone ash cooler is provided as illustrated in Fig. 2-12.

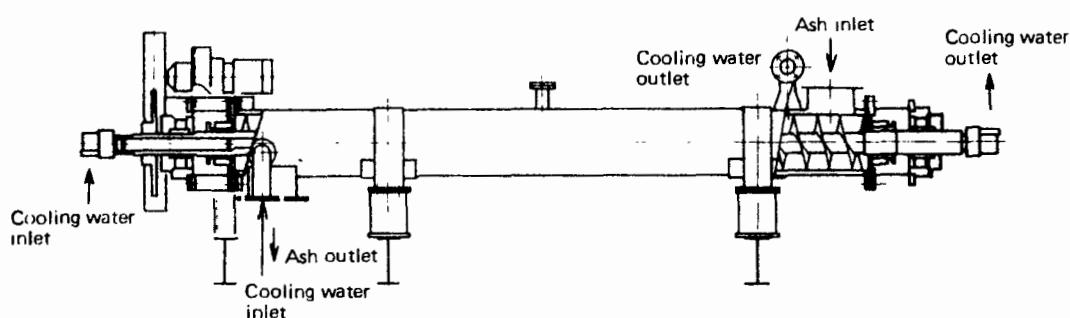


Fig. 2-12 Structure of cyclone ash cooler

2.2.5 Automatic control

The monitoring and control equipment to operate the incineration system is mainly composed of automatic control by micro-controllers etc. in consideration of safety and labor-saving for managing the operation.

Automatic and manual control of the process and monitoring of faults during operation can be carried out in the control room. The examination of each unit can be performed at the position of that instrument (at site). Also, a major difference between this control system and a conventional one lies in the reduction in quantity of control cables and supplementary relays by the use of signal transmitters.

Henceforth the incineration facilities will become more widely established. The monitoring and operation of the facilities are intended to be carried out centrally in the existing incineration-control room.

The outline of management by microcontrollers is given in Table 2-5, the constitution of the control system in Fig. 2-13, and managing procedure for start and stop in Figs. 2-14 and -15. As examples of instrumental control, acceptance managing flow for sludge cakes etc. is shown in Fig. 2-16 and the control flow of air quantity related matter is illustrated in Fig. 2-17.

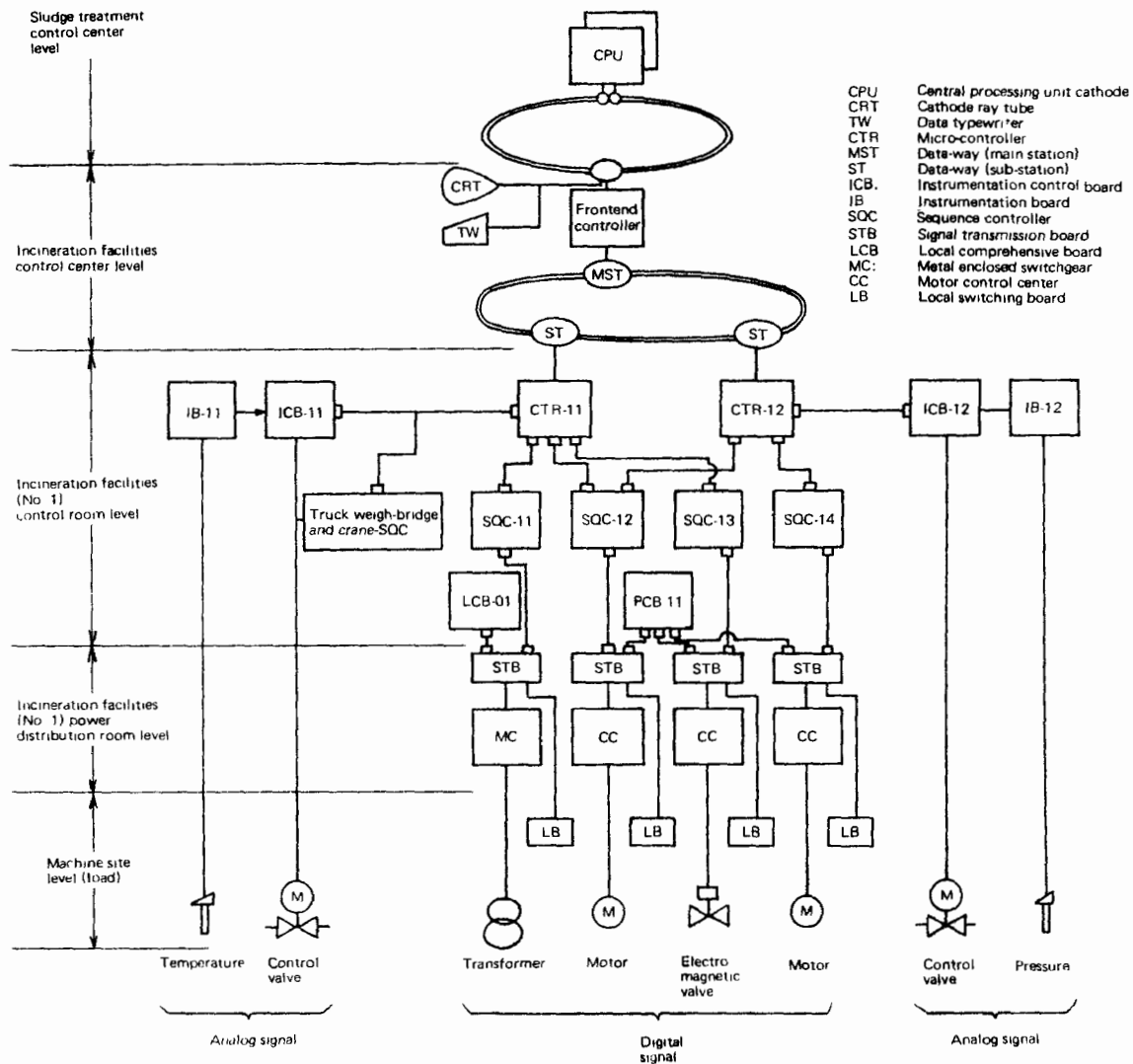


Fig. 2-13 Constitution of control system

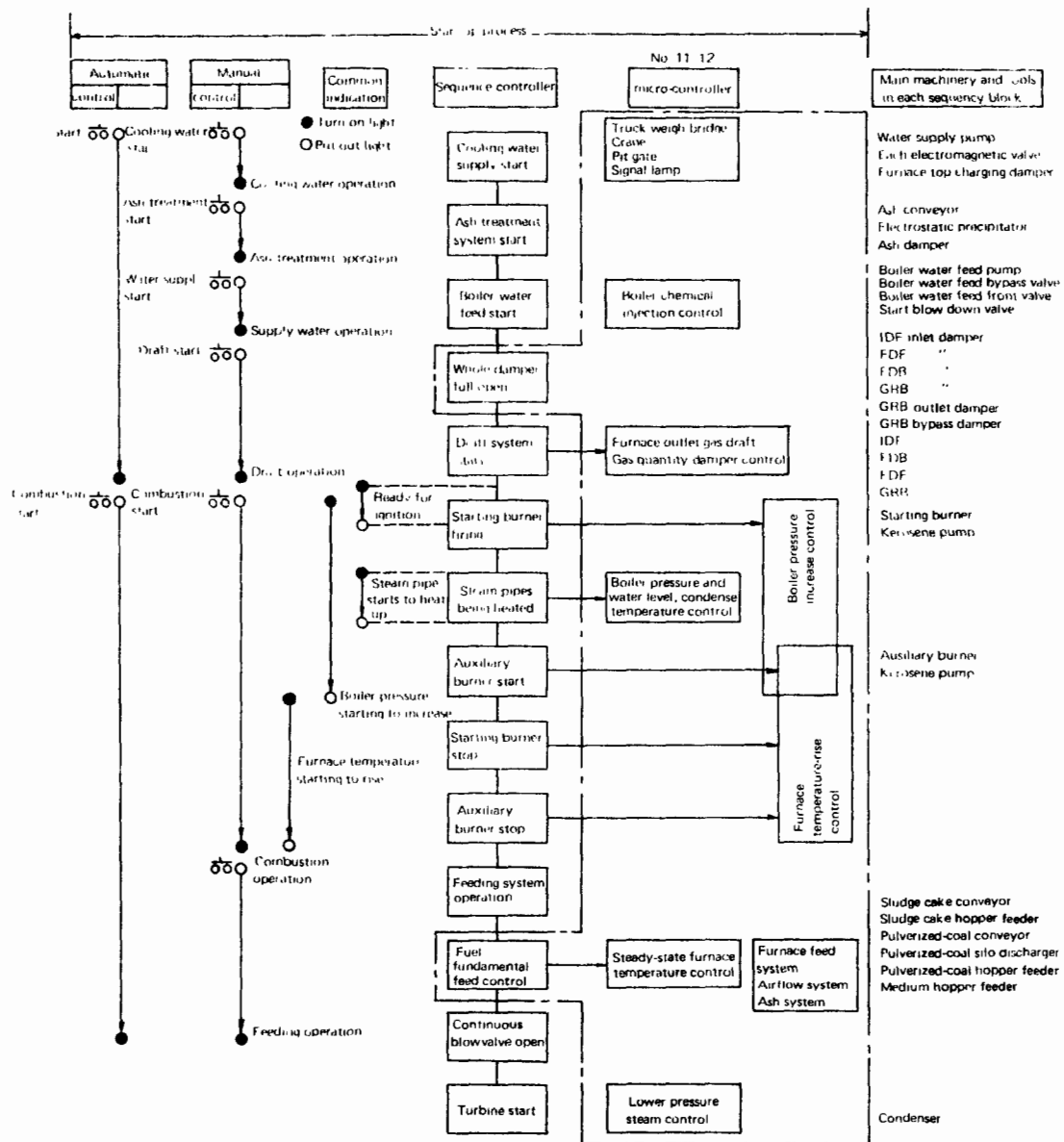


Fig. 2-14 Start-up process

Fig. 2-15 Shut-down process

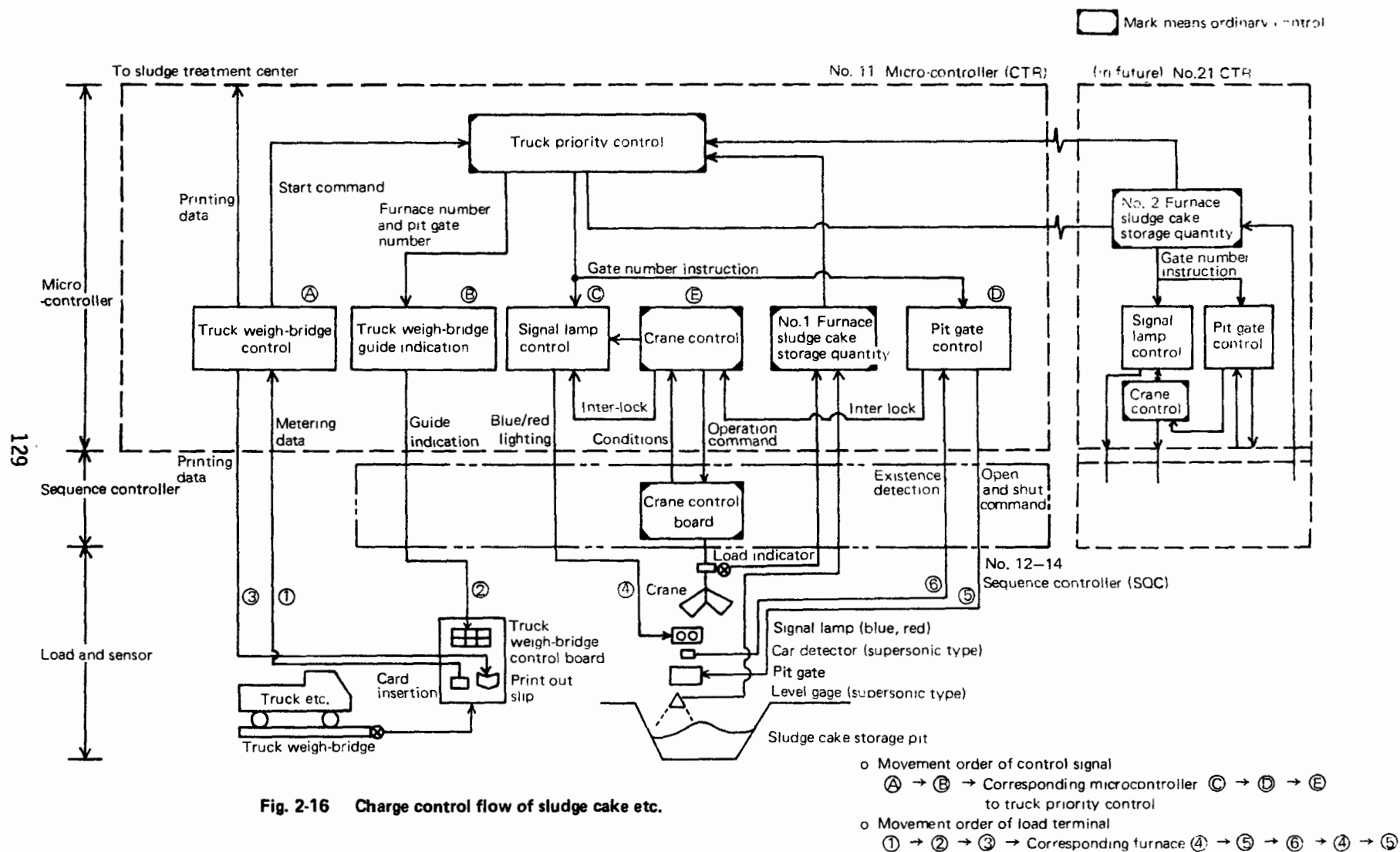


Fig. 2-16 Charge control flow of sludge cake etc.

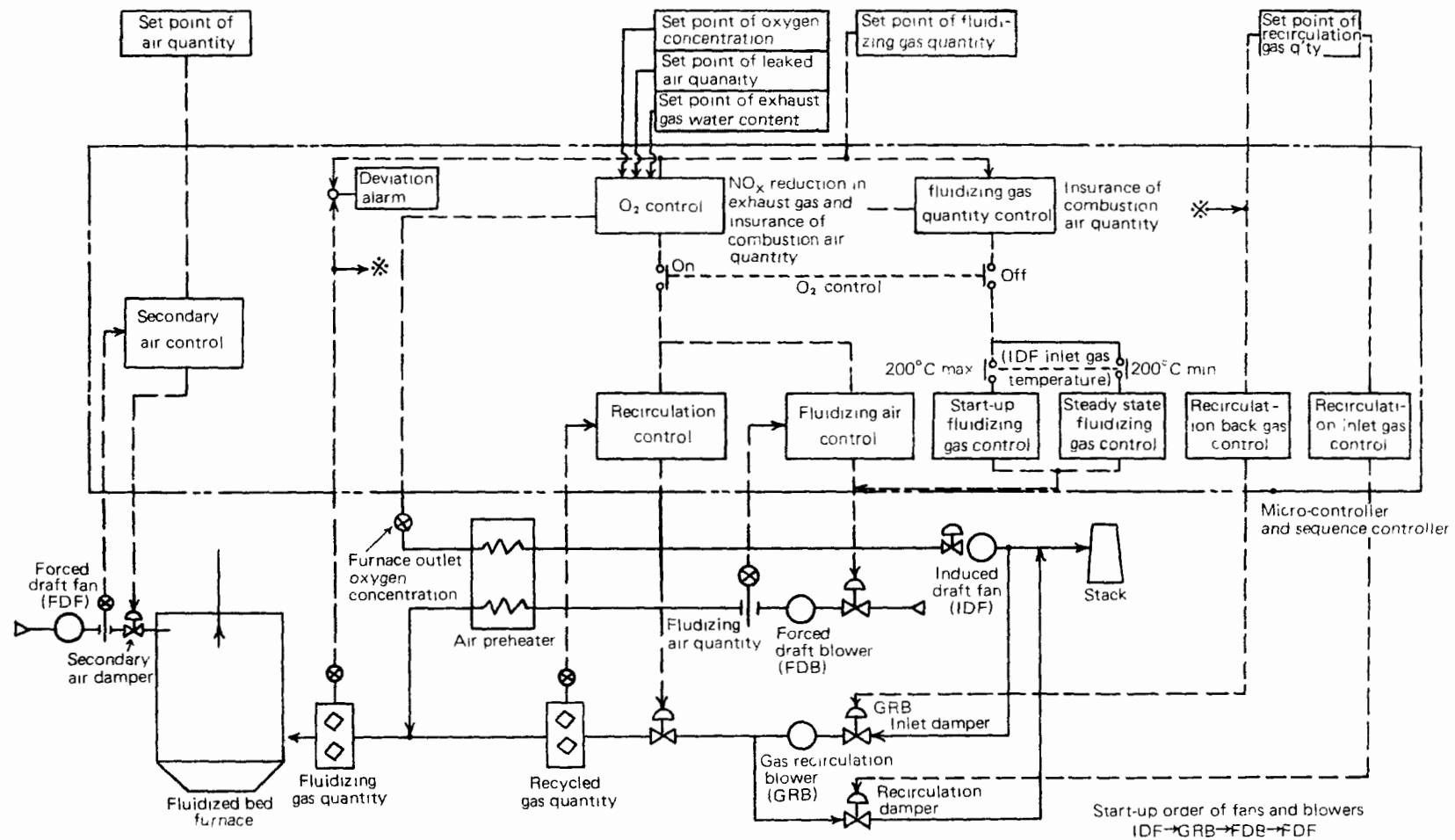


Fig. 2-17 Air quantity control flow

Table 2-5 Outline of micro-controller operation

System	name of control	Outline of control	Main equipment to be controlled
No. 11 Micro-controller			
Acceptance system	Truck weighing control	Weight calculation of cake acceptance etc. and issue of data prints. by cooperation with truck weighing equipment.	Truck weigh-bridge controller
	Crane control	Control of crane address indication etc.	Crane No. 11, No. 12
	Pit gate signal lamp control	Pit gate guidance for cake carrier, opening and shutting of pit gate, on-off control of signal lamp.	Pit gate, signal lamp
	Boiler water level & pressure control	Retention control of boiler drum water level within a definite range. Retention control of constant steam pressure in drum.	Water supply flow control valve Drum steam pressure control valve
Boiler system	Boiler pressure rise-up control	Oil control for pressure increase, by comparing with temperature gradient on starting fluidized bed	Starting burner control valve
	Lower steam pressure control	Retention control of constant steam pressure at lower pressure condenser inlet	Lower steam pressure control valve
	Boiler chemicals injection control	Chemicals control proportional to boiler water supply	Metering pump
	Condense temperature control	Control to maintain constant condense temperature.	Lower pressure condenser
Ash system	Cyclone ash temperature control	Retention of constant ash temperature from cyclone.	Cooling water control valve.
No. 12 Micro-controller			
Feed system	Cake feed control	Control to maintain constant cake feed	Sludge cake constant rate feeder
	Pulverized-coal	Ratio control against cake quantity to keep constant furnace temperature	Pulvarized coal constant rate feeder
Temperature system	Furnace temperature-rise control	Control to maintain fluidized bed temperature constant after reaching a temperature of 650°C or above	Auxiliary burner control valve
Airflow system	Exhaust gas O ₂ control	Ratio control of positive draft blower inlet valve and recycle outlet valve, to obtain target airflow (a constant fluidized gas quantity) calculated from O ₂ control	Forced draft blower inlet valve recycle outlet valve
	Fluidized gas quantity control	Control to keep required air quantity for combustion without exhaust gas O ₂ control	Forced draft blower inlet valve
Other system	Medium feed control	SO _x inhibition control by medium feeder and retention control of constant draft under perforated plate	Medium feeder
	Furnace outlet gas-draft control	Retention control of constant gas-draft at furnace outlet	Induced draft fan inlet dumper
	Furnace crown spray water control	Spray water control to inhibit abnormal temperature rise of furnace exhaust gas	Spray water control valve

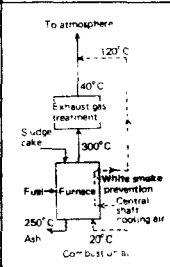
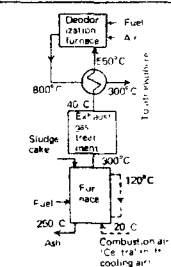
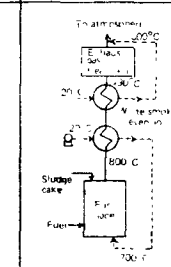
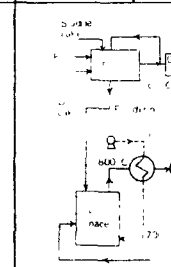
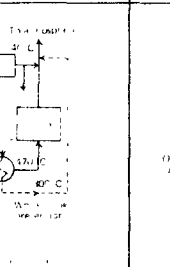
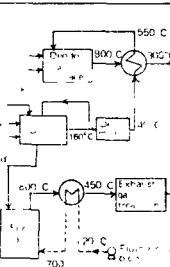
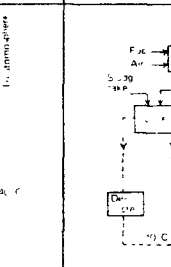
3. Experiments on Separately Dried Sludge Incineration

3.1 The purpose of experiments

Since sludge cakes are generally high in water content and of low calorific value, a large amount of supplementary fuel is required for their incineration. When deodorizing is needed, the quantity of fuel consumed will be increased. The relationship between fuel and exhaust gas quantity in each incineration method is given in Table 3-1.

According to Table 3-1, the multiple hearth furnace has an excess-air ratio of 2 and the exhaust gas quantity at the outlet of the furnace is the largest compared with other methods. The low outlet temperature of 300°C, in this method however, makes the incinerator efficient and requires less fuel consumption compared with the fluidized bed furnace. When deodorization is performed, the required gas quantity is so large

Table 3-1 Comparison of fuel consumption and exhaust gas quantity in each incineration process

Incineration process Items for comparison	Multiple hearth furnace		Fluidized bed furnace (C)	Separately dried sludge incineration			Indirect drying -fluidized bed furnace (G)
	Without deodorization (A)	With deodorization (B)		Direct drying - fluidized bed furnace			
				Without deodorization (D)	With deodorization		
				Deodorization in the fluidized bed furnace (E)	Deodorization in the deodorization furnace (F)		
Incineration conditions	Sludge calorific value kcal/kg-DS	2550 (Net calorific value)					
	Capacity t/day	100					
	Water content of sludge cake %	79 [VTS 55 Ash 45]					
	Excess air ratio	2.0		1.3			
Fuel consumption kg/h	117	182	162	110	149	126	77
Fuel consumption per ton of sludge cake kg/t	28	44	39	26	36	30	18
Incineration outlet exhaust gas Nm ³ /h	12310 [DG7510] [MV4800]	11840 [DG7080] [MV4760]	10270 [DG5450] [MV4820]	6150 [DG3340] [MV2810]	7410 [DG5130] [MV2280]	6150 [DG3340] [MV2810]	5220 [DG3340] [MV1880]
Gas to deodorization Nm ³ /h		18250 [DG7680] [MV 570]			2030 [DG1890] [MV 140]	1580 [DG1470] [MV 110]	
Counted in fuel for power consumption kg/h (Power for equipment kW)	50 (220)	55 (240)	76 (335)	78 (340)	85 (370)	79 (345)	85 (370)
Flow sheet							

DG Dry gas, MV: Moisture volume.

that an almost equal quantity of fuel to that used for incineration is needed, even though heat is recovered by the heat exchangers. The gas quantity to be deodorized, although sealing air of the electrostatic precipitator is added on the way, decreases by defumidifying to about 3/4 of the exhaust gas quantity at the outlet of the furnace.

Fluidized bed furnace has a high outlet temperature of 800°C, hence fuel consumption becomes greater. The exhaust gas quantity, although a large amount of fuel is used, is somewhat less than that of the multiple hearth furnace due to a lower excess-air ratio of 1.3. The fuel consumption, however, is less than that of the multiple hearth furnace with deodorization. Heat recovery is performed by heating the fluid air in the heat exchanger.

There are two types of incineration methods with separate drying processes, which are classified into directly and indirectly heated dryers. Both types perform additional fuel combustion effectively in the process of drying sludge cakes to their spontaneous-burning points. Therefore, the exhaust gas quantity at the furnace outlet reduces to about one half that of the conventional multiple hearth furnace or fluidized bed incinerator. This is attributed to low water content of the sludge and the elimination of exhaust gas resulting from fuel combustion. If water were removed from the sludge by evaporation in the fluidized bed furnace, water must have been emitted as steam having a temperature of 800°C and this would require a heat quantity of over 950 kcal per kg of water. Therefore, water removal prior to incineration has a calorific advantage, along with reduction of exhaust gas quantity.

The combination of directly heated dryer and fluidized bed furnace is divided into necessary and unnecessary cases of exhaust gas deodorization. Even if deodorizing is needed, the required gas quantity is less than 1/4 that of the multiple hearth furnace. There are two deodorizing methods and these are performed either in the fluidized bed furnace or in another deodorizing furnace.

The combustion furnace deodorizing method causes exhaust gas increase by a corresponding quantity, thus results in calorific disadvantage and larger sized furnace.

When an indirectly heated dryer is combined with a fluidized bed furnace, the smallest quantity both of exhaust gas and fuel consumption is obtained compared to other methods. The heat recovery of exhaust gas is carried out in the waste-heat boiler and generated steam passes to the dryer. This process is more favorable than that of the heat exchanger because of lower exhaust gas temperature after heat recovery. The exhaust gas from the dryer is defumidified and utilized as combustion air for the incinerator together with deodorization. When dry steam from the waste-heat boiler is insufficient, additional heating is provided by another boiler. The exhaust gas from the boiler is used for white smoke prevention regulations.

In this experiment, after selecting the model of sludge dryer to reduce the water content of sludge cakes to about their spontaneous-burning point, we intend to confirm the characteristics in energy saving and operation control of the separate drying incineration systems composed of sludge dryer and incinerator.

3.2 Experiment plan

Experimental facility has a treatment capacity of 10 tons/day. Concerning the

sort of sludge, five kinds of sludge cakes from four sewage treatment plants were used.

Also, the dryers are selected by preliminary experiments. The full procedure for the experiments is given in Table 3-2.

Table 3-2 Schedule for experiments

Year	1980					1981					1982				
Month	4	6	8	10	12	2	4	6	8	10	12	2	4	6	
Preliminary experiment	Design, manufacture, installation		Experiment												
Main experiment						Design, manufacture, installation		Experiment							

3.3 Preliminary experiments

3.3.1 Outline

The sludge cakes have a large variation in their properties such as water content, calorific value, composition etc., and hence the dryer which can perform stable operation for any sludge should be selected.

There are two drying processes, direct and indirect heating methods, as shown in Fig. 3-1. As the indirect heating method is advantageous, comparative experiments were carried out on the three indirectly heated models.

The types and characteristics of the dryers are given in Table 3-3 and experimental methods are shown in Fig. 3-2.

3.3.2 Results of preliminary experiments

a. Properties of sludge cakes

Analytical results of sludge cakes used in the experiments are given in Table 3-4. Net calorific value and water content at spontaneous-burning point are briefly divided into two groups on the basis of Table 3-4.

(a) Inorganic coagulant added sludge

Net calorific value 1650 kcal/kg-DS

Water content at spontaneous-burning point 41 – 45%

(b) Organic coagulant added sludge

Net calorific value 2800 – 3400 kcal/kg-DS

Water content at spontaneous-burning point 62 – 65%

b. Outline of experimental results

Experimental data obtained from the three equipment models and six methods are given in Table 3-5 to 3-7, and the summary is illustrated in Table 3-8.

c. Discussion of results

If an indirect heating method is used to sewage sludge which properties vary widely, a low speed disk mixing dryer seems to be suitable. The dryer could not control effectively the water content of dried cakes, and yet the content was

Table 3-3 Types of indirect dryer

	Structure and characteristics	Structural figure
Steam tube heating type	<p>The steam tubes are concentrically arranged along the whole inside length of a rotating cylindrical container installed at a slightly inclined angle and fabricated so as to rotate with the body. Steam is passed through the tubes and sludge cakes are fed in and dried by means of the combing and stirring action of the tubes. Viscous sludge tends to stick, whereas this type requires less power. The speed of rotation is 2 – 10 rpm.</p>	
High speed paddle mixing type	<p>A shaft fixed with many stirring paddles rotates at high speed in the center of a cylindrical container which is equipped with a steam heating jacket. Steam is passed into the jacket of the container. The sludge cakes charged are dispersed by the centrifugal force of the stirring paddles and dried by repeated contact with the heating surface. High efficiency is obtained but unfortunately this type has the drawback of requiring power. The speed of rotation is 100 – 400 rpm.</p>	
Low speed disk mixing type	<p>Disc shafts having many notched disc conduction blades are biaxially arranged in a slightly inclined casing jacket. The two shafts rotate and stir at low speed in the direction that gives mutual engagement. Steam is passed both into the jacket and the disc shafts. The sludge cakes charged are dried by contact with the jacket and disc shafts. The speed of rotation is 10 – 40 rpm.</p>	

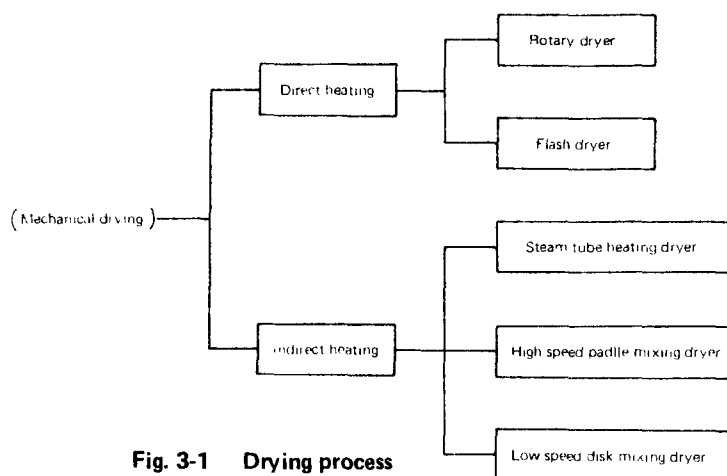


Fig. 3-1 Drying process

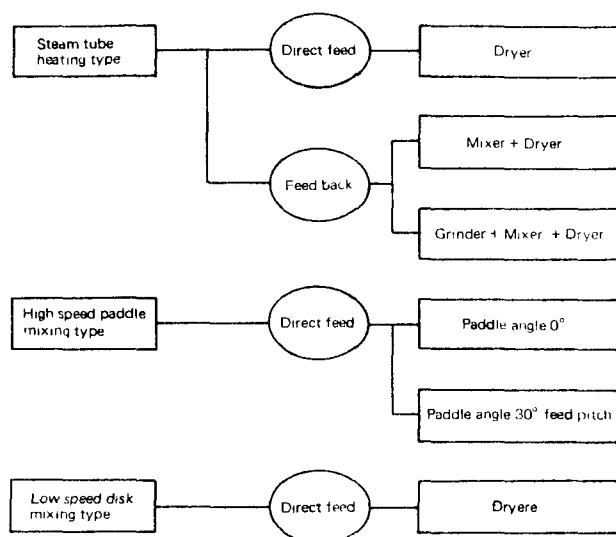


Fig. 3-2 Preliminary experimental process

affected by the variation of operating conditions such as steam pressure, rotating speed etc., though correlation could not be measured.

Therefore, concerning the selection of models, we adopted the low speed disk mixing dryer which is able to operate steadily and evaporate quickly from any sludge as shown in Table 3-8.

3.4 Outline of main experiments

The flow sheet and equipment specifications of experimental facilities are given in Fig. 3-3, and their layout is illustrated in Fig. 3-4.

The experiments are being performed on the site of Kanazawa Sewage Treatment Plant.

Sludge cakes in use for the experiments are the same as those used in preliminary experiments.

Currently the experiments are in continuation.

Table 3-4 Analytical value of sludge cakes

Sewage treatment plant		Chubu S.T.P.	Nanbu S.T.P.	Totsuka II S.T.P.		Kanagawa S.T.P.
Items						
Treatment		Anaerobic digestion	Anaerobic digestion	Direct dewatering	Direct dewatering	Direct dewatering
Dewatering machine		Belt filter	Centrifuge	Pressure filter	Centrifuge	Centrifuge
Coagulant	Sort	Ca(OH) ₂ Fe ₂ Cl ₃	Polymer	Ca(OH) ₂ Fe ₂ Cl ₃	Polymer	Polymer
	Addition %DS	55.2 6.9	1.2	32.8 6.3	0.8	0.9
Sludge cake	Water % WB	73.6 - 76.3	71.7 - 82.0	58.2 - 66.5	79.1 - 81.1	77.0 - 78.2
	Combustible matter %DS-B *1	35.9	51.4	31.1	64.4	48.5
	Component %DS-B	Raw protein	14.3	19.3	11.4	26.1
		Raw fat	2.0	6.8	1.4	7.8
		Raw fiber	6.9	7.5	16.2	13.3
		Calorific value	1805	3183	1805	3695
	Kcal/kgDS	Gross *2	1805	3183	1805	3695
		Net *3	1647	2904	1647	3371
Sludge cake	Water content at spontaneous-burning point %WB *4	40.7	62.1	45.2	65.2	61.5
		Soft clayey material with a low fibrous matter content	Viscous material with a high fibrous matter content.	Light yellow plates with a thickness of 5-7mm and a high fibrous matter content.	Viscous material with a high fibrous matter content.	Non-viscous material containing fibrous matter.

Note: *1 Analytical value at 600°C, 2hrs

*2 Analytical value by JIS M.8814

*3 Calculated value

*4 Calculated value (fluidized bed furnace with an air excess ratio of 1.3 and a furnace outlet gas temperature of 800°C)

Table 3-5 Experimental data on steam-tube dryer (Heating surface area as 4.8m²)

Conditions				Water content %WB		Evaporation rate	Power of dryer kW	Feeding process	Drying & sticking situations
Sludge	Feed kg/h	Steam pressure kg/cm ² G	Dryer's rotating speed rpm	Inlet	Outlet				
Vacuum filtered sludge cake (Digested sludge)	60	8	8	75.4	56.1	5.5	Unknown	Direct feed of sludge cake	Cakes are cloggin up the inside from the inlet to the mid-section, and tubes are buried after prolonged operation.
	70	8	8	41.5	6.4	5.4	0.46	Feed back process with grinder and mixer	Dried material is in grains of 1 - 10 mm diameter. No sticking is found.
Pressure filtered sludge cake (Raw sludge)	60	8	8	59.3	38.1	4.3	0.48	Direct feed of sludge cake	Dried material is a mixture of cotton dust fibrous matter and flat material having traces of the original form. No sticking is found.
	70	8	8	31.6	2.1	4.4	Unknown	Feed back process with grinder and mixer	Dried material is a mixture of grains with a diameter of 2 - 10 mm and fibrous matter. No sticking is found.
	120	10	8	35.5	5.0	8.0	Unknown	Feed back process with mixer	Repeated drying gradually increases grain diameter of dried material. No sticking is found.
Centrifuged sludge cake (Raw sludge)	60	8	8	73.7	53.6	5.4	0.41	Direct feed of sludge cake	Dried material is in grains of 10 - 40 mm diameter and it clogs between tubes and between tubes and shell.
	70	8	8	36.4	7.5	4.5	0.44	Feed back process with grinder and mixer	Dried material is in grains of 1 - 5 mm diameter. No sticking is found.

Table 3-6 Experimental data on high speed paddle mixing dryer (heating surface area 4.0 m²)

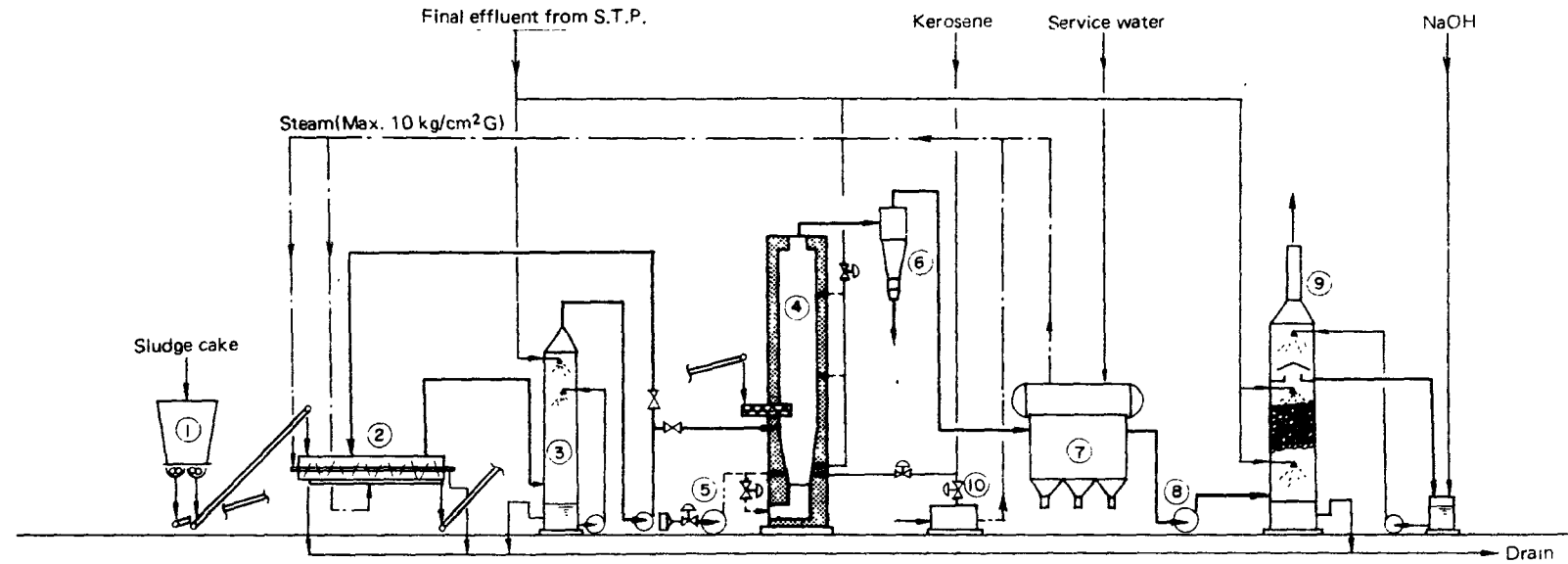
Sludge	Conditions			Water content %WB		Evaporation rate kg-H ₂ O/m ² ·h	Power of dryer kW	Drying and sticking situation
	Feed kg/h	Steam pressure kg/cm ² G	Dryer's rotating speed rpm	Inlet	Outlet			
Vacuum filtered sludge cake (Digested sludge)	150	8	150	75.0	58.9	14.7	2.22	Relatively uniform grains of 5 – 10 mm. Considerable sticking is found onto the rotation shaft.
Ditto	150	8	300	75.0	54.9	16.7	2.76–3.24	Relatively uniform grains of 5 – 10 mm. No sticking is found.
Ditto	150	8	400	75.0	53.6	17.3	4.20–4.80	The same as above. But re-start-up was impossible after sampling

Table 3-7 Experimental data on low speed disk mixing dryer (heating surface area 4.8 m²)

Sludge	Conditions			Water content %WB		Evaporation rate kg-H ₂ O/m ² ·h	Power of dryer kW	Weir height mm	Drying and sticking situations
	Feed kg/h	Steam pressure kg/cm ² G	Dryer's rotating speed rpm	Inlet	Outlet				
Vacuum filtered sludge cake (Digested sludge)	180	7	31	75.4	51.3	18.6	1.21	140	Dried material is in grains of 1 – 5 mm
	180	7	31	76.1	44.8	21.3	1.32–1.38	290	No sticking is found
Pressure filtered sludge cake (Raw sludge)	240	7	20	66.5	45.0	19.5	1.37–1.49	140	Dried material is in powder form of 0.1 – 1 mm.
	240	7	39	66.5	52.9	14.4	1.62–1.76	140	No sticking is found
Centrifuged sludge cake (Digested sludge)	240	7	39	79.6	53.2	28.2	1.22	140	Dried material is in grains on blocks of 30 – 50 mm.
	240	7	31	74.5	51.1	23.9	1.37	140	No sticking is found

Table 3-8 Summary of preliminary experiments

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



Equipment No.	1	2	3	4	5	6	7	8	9	10
Equipment name	Cake hopper	Dryer	Condenser	Incinerator	Fluidizing air blower	Cyclone	Waste-heat boiler	Induced draft fan	Scrubber	Boiler
Model	Cubical type	Low speed disk mixing type	3 stage spraying direct cooling type	Fluidized bed furnace	Multiple stage turbo-blower	Refractory castable lining	Water tube type	Plate type	Upper part. Spray type Lower part. Packed tower	Single pass type
Main dimensions	2353W/3000W x 2150L	670W x 2850L	φ762x6000H	φ1100/φ1600 x 9200H	φ 125	φ 980 x 4300H	Heating surface area 29m²	φ 300	φ 750 x 15000H	Heating surface area 5.77m²
Volume	12m³	500 kg/h	12m³/min at 100°C	Sludge cake 10t/day	10m³/min x 3500 mmAq	40m³/min at 800°C	380 kg/h x 10 kg/cm² G	30m³/min x 450 mmAq		350 kg/h x 10 kg/cm² G
Number	1	1	1	1	1	1	1	1	1	1

Fig. 3-3 Flow sheet and equipment specifications on separately dried sludge incineration experiments

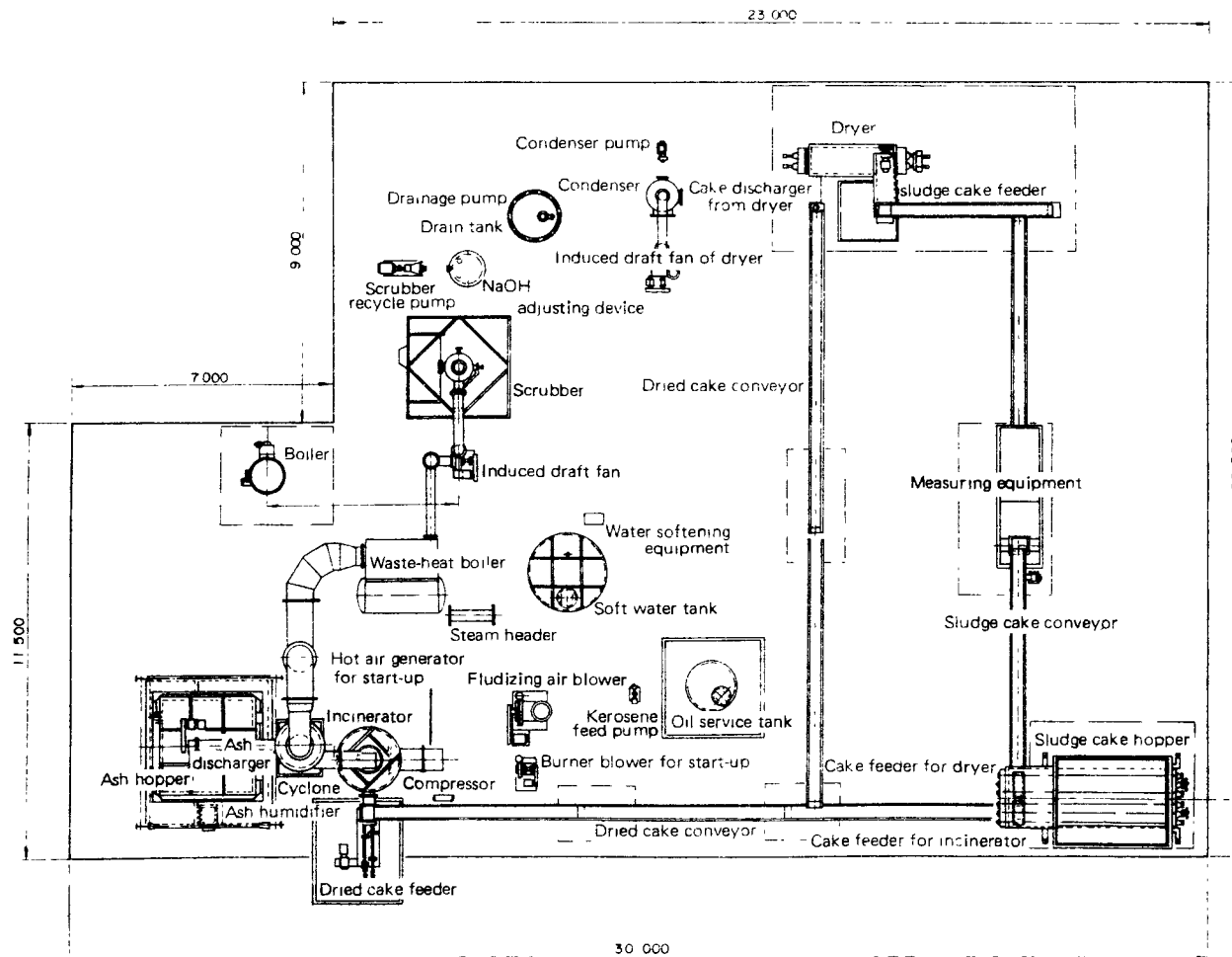


Fig. 3-4 Lay-out of equipment on separately dried sludge incineration experiments

Closing Remarks

As mentioned above, a couple of new aspects on the incineration of sludge cakes were introduced.

The need or situation demanding the incineration in the sludge treatment process depends upon the method for final disposal.

Accordingly, there is no need for incineration when the sludge cakes may be subsequently utilised as a fertilizer etc..Whereas it will be a valuable technology when land disposal can be continued for a long period in the area limited.

Yokohama City is obliged to take the latter case.

Henceforth we want to make every effort also for the development of stable and economical incineration technology.

Finally, the author would like to acknowledge the considerable assistance given in the preparation of this report by M. Noguchi, equipment manager, S. Yamaguchi, facilities sub-senior chief, and other member of facilities and maintenance sections.

Eighth US/JAPAN Conference
on
Sewage Treatment Technology

CURRENT STATUS OF AUTOMATIC MONITORING OF WATER QUALITY IN SEWAGE TREATMENT

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1. INTRODUCTION

The necessity for automatic measurement of water quality in sewerage system has been strongly recognized, particularly treatment process control and monitoring of hazardous substances in sewage. Both the users and manufactures are engaged actively in technological development in this field. This paper summarizes the recent progress in automatic water quality measurement in sewage treatment systems. As to instruments for monitoring hazardous substances in sewage, an automatic lead monitor and an automatic total-cynide monitor have been newly developed. Also developed is automatic samplers to take sewage samples in case abnormal pH-changes are detected. A project is in progress to evaluate the the reliability and performance of automatic water quality measuring instruments when they are operated under actual conditions. As a first step, an investigation to evaluate the performance of pH sensors under field conditions has started, including establishment of methodology to evaluate these instruments.

Along with the progress in construction of sewerage systems, the percentage of the loading discharged into public water bodies through sewage treatment plants is gradually increasing.

For this reason, the control of load discharged from sewage treatment plants in order to preserve the water quality of public water bodies is gaining greater and greater importance. In certain regions, today, automatic measurement and recording of the organic loading discharged from sewage treatment plants is being made compulsory, as it is for other sources of pollution and pollutors.

To implement similar restrictions on phosphorus also in the future, a total phosphorus automatic analyzer is now being developed.

2. AUTOMATIC MONITORING INSTRUMENTS FOR HAZARDOUS SUBSTANCES IN SEWAGE

Control of hazardous substances, especially heavy metals, contained in sewage is one the most important task in the operation and maintenance of sewage treatment plants. Sewerage authorities have assigned a large number of personnel to monitor water quality of industrial waltewater discharged into the sewerage system to enforce pre-treatment standards. At present, water quality monitoring of this kind is being done manually, but when automatic monitoring equipment becomes available in the future, still better monitoring will be accomplished. The Ministry of Construction, in the past several years, has been funding a contract research program to develop automatic water quality monitoring equipments.

In this project, automatic monitoring equipment for total cynide, chromium, cadmium, and copper have been developed. Summaries of work were presented at previous meetings.

In this section, recent activities for developing a lead monitor, improving the total cynide monitor and developemnt of automatic samplers operated by signals from a pH sensor are outlined.

2.1 Development of Lead Automatic Monitor

The lead automatic monitor being developed utilizes an ion-selective electrode. Major problems encountered in its developemnt are: (1) ordinary sewage contains copper, iron and organic substances in concentrations which interfere with the measurement of lead by an ion-selective electrode, (2) lead contained in sewage is mostly in the particulate form, and it has to be dissolved before measurement. Therefore, various experiments were conducted to determine the pre-treatment method, and finally a measuring instrument was devised based on the following principle.

Fig. 1 is block diagram of the automatic lead monitor.

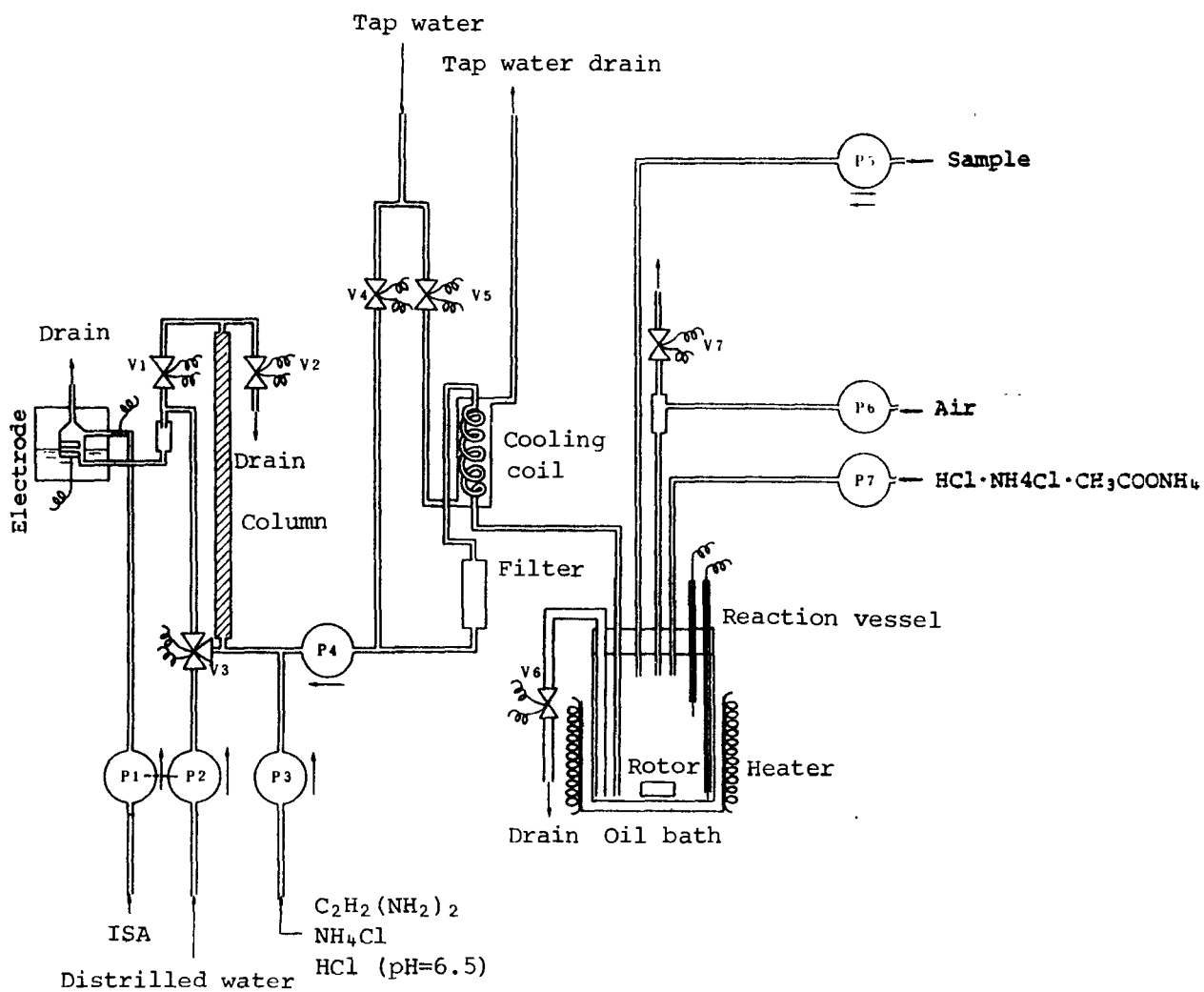


Fig. 1 Block diagram of the automatic Pb monitor

A 10 ml mixed solution of 2N hydrochloric acid, 10% ammonium chloride and 0.1 M ammonium acetate was added to a 200 ml sample, which was then heated to dissolve and convert lead into chloro-lead complex ions. The ammonium acetate is added to dissolve lead sulfate. The sample is then filtered to eliminate suspended solids and then introduced into a column with anion exchange resin. The column is of 6 mm ID, 65 cm long and made of PVC, while Dowex 1-X8 (type CL) of 20 to 50 mesh is used as the anion exchange resin. By this operation, the lead is adsorbed as chloro-lead complex ions onto the exchange resin. The anion exchange resin is then cleansed with elute to separate, as much as possible, the interfering substances that have been adsorbed onto the anion exchange resin. The elute is a 10 time diluted solution of 1M ethylenediamine, 10% ammonium chloride, and 2N hydrochloric acid mixed solution. The ethylenediamine is added to form chelate complex with copper and desorb them into the elute.

After this cleaning operation, distilled water is run into the column for 40 minutes at a flow rate (empty bed) of about 3.5 cm/min to desorb lead from the exchange resin. The desorption solution, after addition of a buffer solution, is transferred to a cell for measuring lead with an ion selective electrode. The buffer solution is a mixed solution of 1M ammonium acetate, 10% ammonium chloride, 0.2M sodium citrate, 1M sodium thiosulfate, and 1M potassium nitrate, whose pH value is adjusted to pH6 using acetic acid. It is added to the sample at the ratio of 1 : 10.

The purpose of adding this solution is to adjust pH value, to adjust ionic strength, and eliminate part of the interferences. After taking measurements, the anion exchange resin column is cleansed with distilled water for about 80 minutes. The measurement cycle is once every 2 hours.

To examine the mechanical reliability and precision of measurement under the actual operating condition, the device was transported to a sewage treatment plant and was continuously operated for a period of about 1 month. The test showed that, even with the complicated pre-treatment mentioned above, interference probably due to certain types of organic substances still remained.

Although certain correction can be made by data processing, improvement of electrodes, etc. are necessary. Fig. 2 shows an example of the measurement by the instrument. Here, sample A, B, and C are those prepared by adding lead to a raw sewage sample at concentrations of 1.0, 1.5, and 2.0 mg/l , respectively.

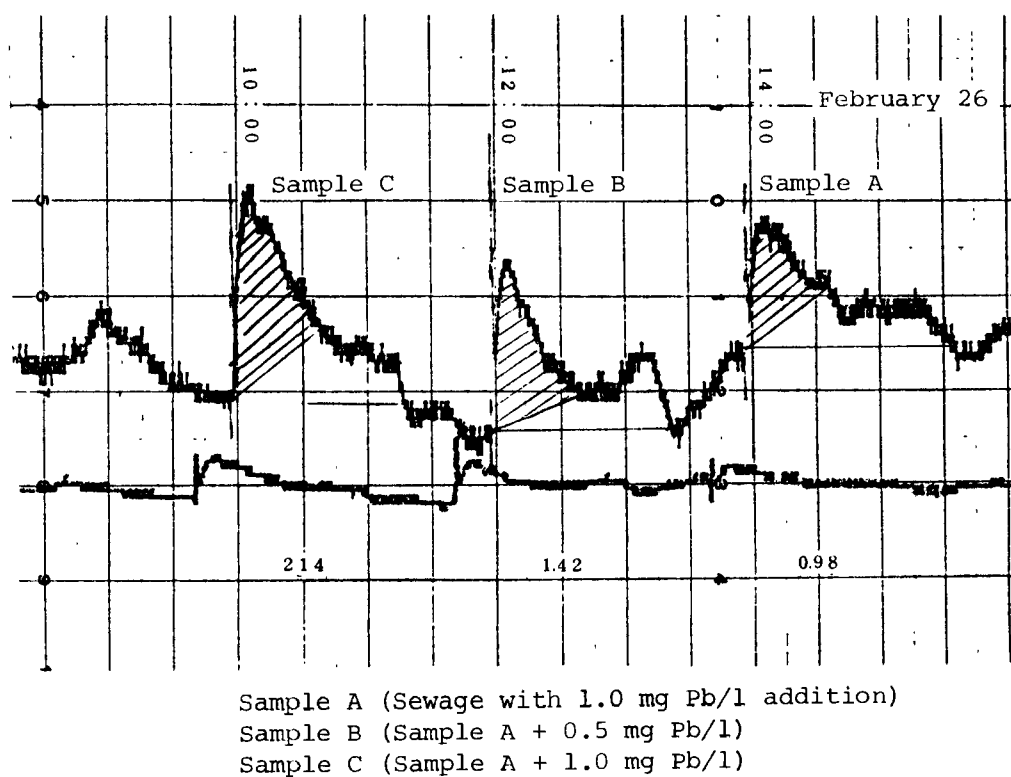


Fig. 2 Example of recording of the automatic Pb monitor
 - the known increment method -

As seen in the figure, the base line fluctuates considerably and tailing of the record after reaching its peak is significant. This tailing is considered to be due to interferences.

By integrating the hatched areas in the diagram, the lead concentrations in samples A, B, and C are found to be 0.98, 1.42, and 2.14 mg/l, respectively, that is fairly close to the added concentrations.

In this way, by means of data processing, the effect of interferences can be substantially corrected. However, improvement of electrode and/or sample pre-treatment is considered essential. Work on development of a reference electrode, which may compensate the effect of organic interferences, is being advanced.

Continuous operation revealed almost no mechanical troubles, but confirmed that atmospheric temperature variations have significant effect on the measurements, that is, temperature control or temperature compensation is necessary.

2.2 Improvement of Automatic Total Cyanide Monitor

As presented at the 5th and 6th conferences, automatic total cyanide monitors, in which the measurements are taken with an ion selective electrode after distillation, were developed and have already been put into service in a few plants.

Since the distillation step makes the monitor mechanically complicated, however, other methods especially that using ultraviolet rays (UV) for decomposing complex cyanide is being studied. In this method, UV light is irradiated on the sample under acidic condition, in order to analyze the complex cyanide. Free cyanide is transferred into an alkaline solution through a gas permeable membrane, and finally measured by a cyanide electrode.

Fig. 3 is a block diagram of a prototype model of the monitor.

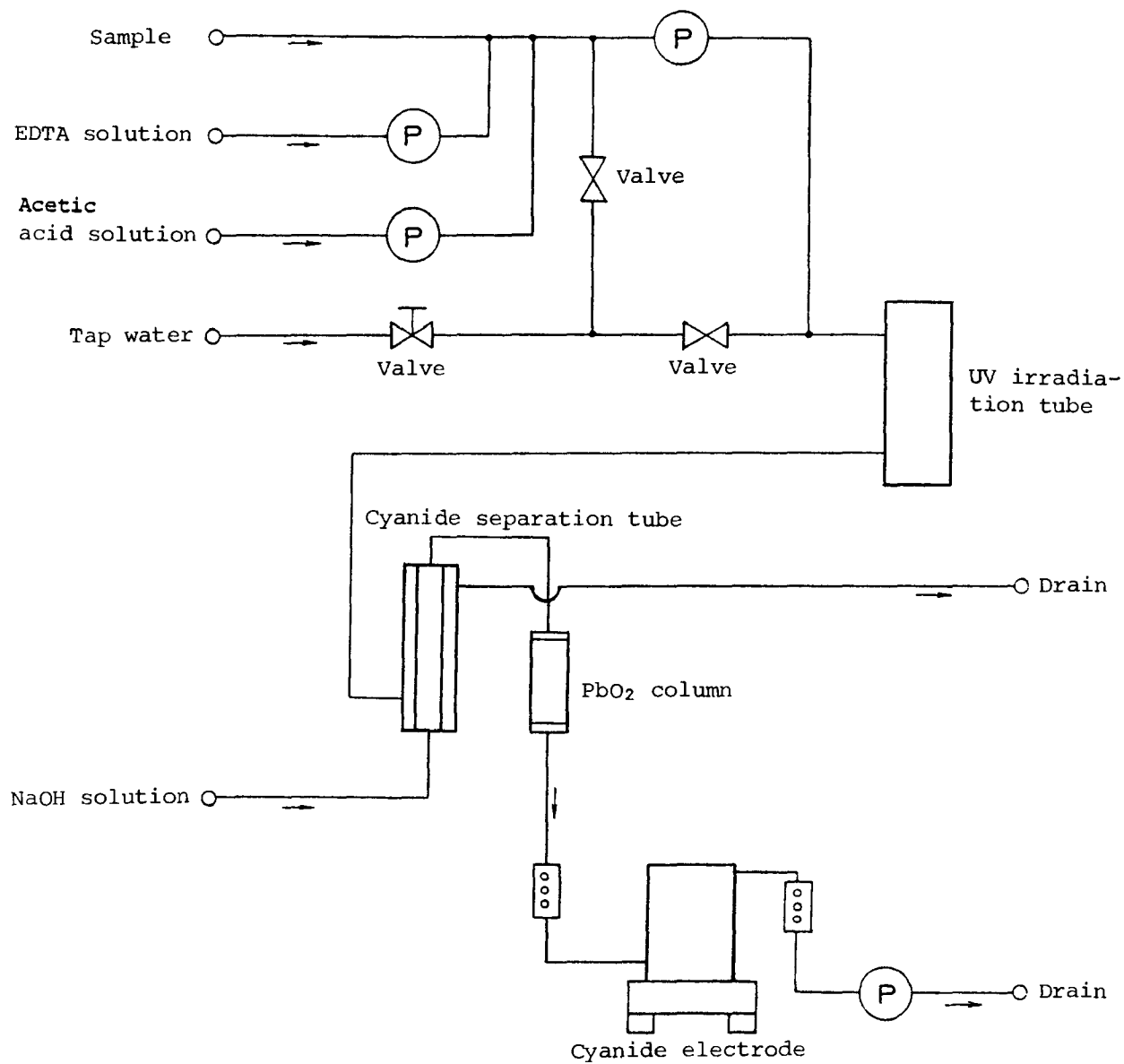


Fig.3 Block diagram of the automatic T-CN monitor

This monitor is of the continuous operating type, in which 5M acetic acid (1 ml/min) and 0.05M/EDTA solution (1 m /sec) is added to the sewage sample (10 m /min), then UV light from a 50W low voltage mercury lamp is irradiated onto it to convert complex cynides into free cynide.

EDTA solution is added to prevent free cynides from reacting again with metals. Acetic acid is used instead of mineral acid because mineral acids may cause precipitation of EDTA.

Next, the sample is transferred to a dual tube as shown in Fig. 4.

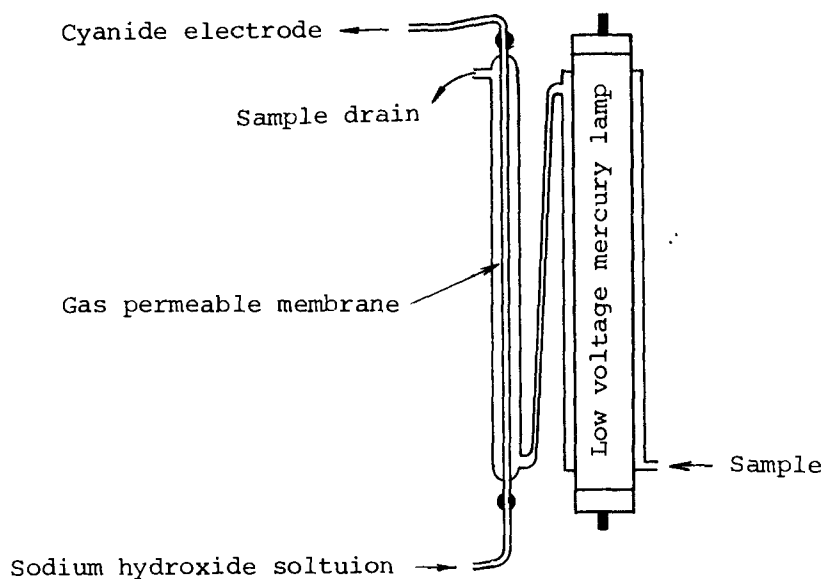


Fig. Complex cyanide decomposition and free cyanide separation portion of the T-CN monitor

The inner tube of the dual tube is made of a material which is permeable to gas (like, teflon). Free cyanide is transferred into the 0.2N sodium hydroxide solution which is flowing through the inner tube. When the sample contains sulfides, hydrogen sulfide is formed and also transferred into the sodium hydroxide solution. To eliminate this, the sample is passed through a column filled with particulate lead peroxide before being transferred to the ion electrode for cyanide measurement.

By controlling the flow rates of sample and sodium hydroxide, it is possible to concentrate cyanides to a certain degree.

In June 1981, a prototype monitor was installed and placed in operation at a pumping station. Many plating industries are located in the drainage area of the pumping station.

One of the problems revealed by the field test is the interferences probably caused by organic substances. When UV light is irradiated on the sample under acidic condition, the organic substances with high molecular weight are decomposed into those with low molecular weight, some of which pass through the membrane and interfere with the cyanide measurement. The degree of interference when teflon is used as the membrane, is about 0.05 to 0.1 mg/l as cyanide. This interference can be reduced to about 1/3 by passing the sample through an activated carbon column before taking the measurement with a cyanide electrode. However, since it is desirable to set the detection limit at about 0.01 mg/l, the material of gas permeable membrane, capacity of mercury lamp, and other factors are being re-investigated.

This type of total cyanide monitor requires much less maintenance than the previous ones, and is considered to be put into practical use more widely.

2.3 Development of Automatic Sampler Actuated by pH-meter

An automatic sampler, possessing the function to detect abnormal variations in water quality and begin sampling, will be a very effective tool in improving the industrial wastewater monitoring system. At present, the most suitable detector for this purpose would be a pH sensor, considering that it can be used inside the sewer. Therefore, automatic samplers operated and controlled by the signals from pH sensors have been developed and undergone field tests.

Three prototype models of the samples were manufactured: 2 portable and 1 stationary type.

a. Portable type automatic samplers

The portable type automatic samplers are designed to be placed in manholes. Their major specifications are as follows.

- (1) Shape : Shaped to be fitted in standard manholes
- (2) Power source : Battery (operates for a week without re-charging)
- (3) Sample volume : 500 ml, 12 times
- (4) Mode of sampling: Sampling with signal from pH sensor or periodic sampling, selectable
- (5) Suction head : 5 m or more
- (6) pH sensor : Accuracy of less than ± 0.1 pH with clean water, and equipped with electrode cleaning attachment
- (7) pH recording : Analog or digital.
If digital, the recording shall be done once every hour during normal condition, and once every 10 minutes if the pH deviates outside the set range. Sampled time also shall be recorded.

The success of this type of automatic sampler depends largely on the availability of reliable pH sensors. The pH sensors used for these models are shown in Fig. 5 and Fig. 6, respectively.

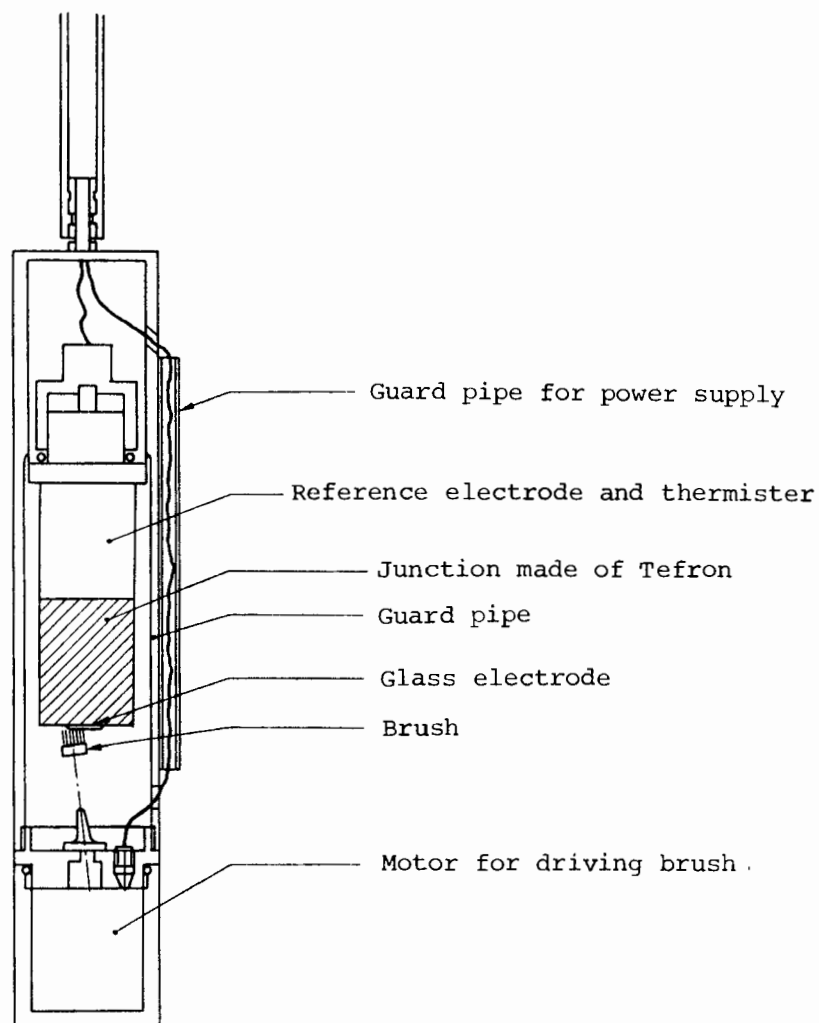


Fig. 5 pH sensor (Type I) for the automatic sampler

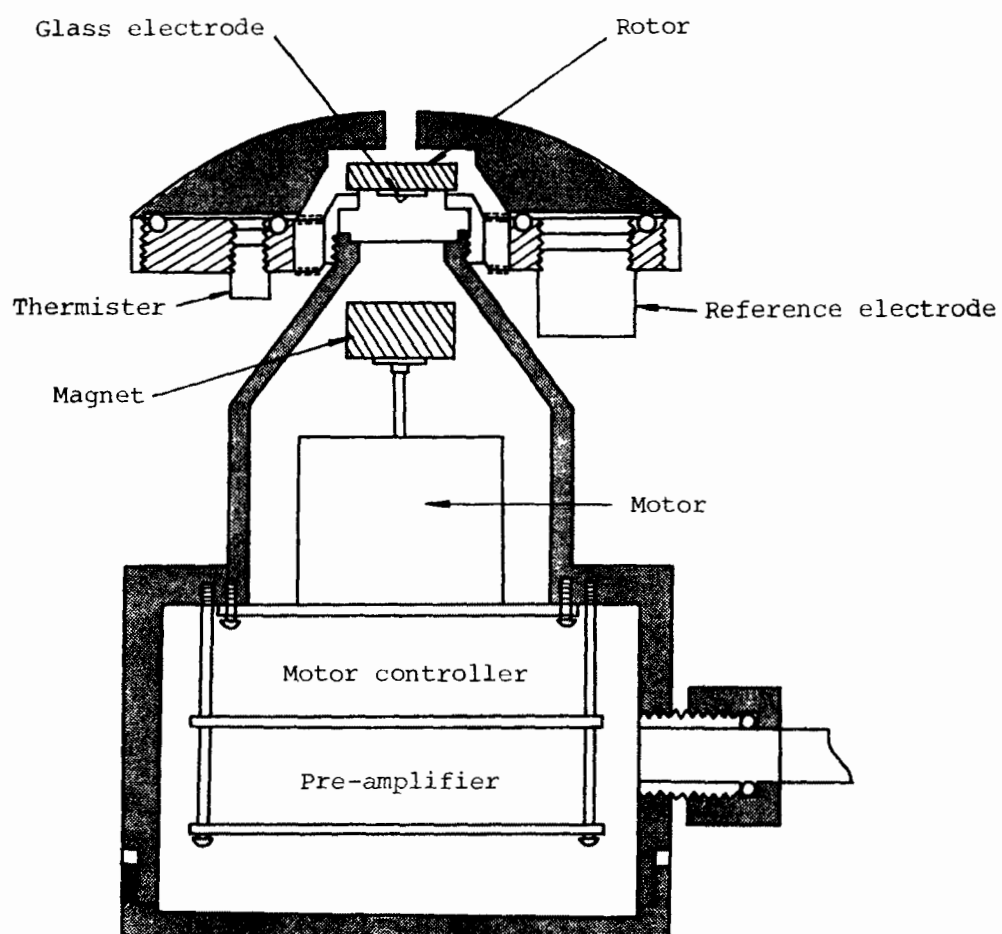


Fig. 6 pH sensor (Type II) for the automatic sampler

The Type I sensor shown in Fig. 5 has a built-in pre-amplifier and power source, which reduce noise caused by outside disturbances. The glass electrode is mechanically cleaned by a brush which is actuated by a timer. The Type II sensor shown in Fig. 6 has an all solid-state glass electrode whose surface is cleaned by a rotor driven by magnetic stirrer. One model uses a vacuum pump to take samples and has an analog recorder for pH; while the other takes samples using a tube pump and records pH on a digital printer.

The samplers were trial-run in the field for 1 year to check their reliability and to make necessary improvements. The precision of the pH sensors of both types were found quite satisfactory. After continuous maintenance free operation for one week, the maximum difference in the measurement with pH standard solution was found to be 0.3 pH.

However, depending on the installation site, large sized floating substances sometimes became entangled on the electrode. The shape of the electrode as well as its holder had to be made as smooth as possible.

Concerning the dimension of the sampler, its maximum O.D. was 530 mm so that it could be fitted in standard manholes of 600 mm I.D. However, in many cases, steps in the manholes reduced the available space. Therefore, the O.D. of the commercial model sampler is recommended to be 450 mm maximum. Also, since the battery and the sampled water made the monitor heavy, it became necessary to make it a lighter one. The target weight was set to be 30 kg. After making these improvements, commercial products have been marketed from this year.

b. Stationary type automatic sampler

This type of automatic sampler operated and controlled by a signal from a pH sensor was designed to be permanently installed at the inlet sewer or grit chamber of a sewage treatment plant as well as at the grit chamber of a pumping station.

Since commercial power supply is available at these installation sites, a refrigerator to preserve the samples was designed to be built into the sampler, and also where necessary, output for telemetering signals to indicate pH measurements and sampling was provided. 24 sampling bottles of 500 ml were provided and the sample size was made selectable, between 1 bottle and 4 bottles. The pH sensor used is almost the same as Type I shown in Fig. 5, the difference being that the glass electrode is cleaned with ultrasonic waves.

Field tests were conducted for more than 6 months by installing the automatic samplers at the grit chamber of a municipal sewage treatment plant as well as at a grit chamber of a sewer system into which large quantities of tannery wastewater flow.

At the former site, the flow velocity was about 2 to 3 m per second and large size floating substances became entangled severely. The error of measurements when the electrode was covered by these substances reached as much as 1 pH. However, the measurements could be easily restored to normal through removal of the large size trash that entangled the electrode. The growth of slime on the electrode was very slight due to the rapid flow. As in the portable type sampler, it is important to shape the electrode in such a way that large size floating substances do not become entangled on it. The effect of cleaning the electrode with ultrasonic waves was not clear because only a little slime adhered during the period without cleaning.

The field test at the grit chamber into which tannery wastewater flowed was conducted under a severe condition. Sometimes a thick scum formed on the surface of the grit chamber because the capacity of the chamber was too large for the actual flow. This gave rise to several troubles, namely, the sampling tube tended to get clogged, fats and oils adhered to the pH electrode, and so on. Therefore, the field test is, still being continued to make improvements in the sampler.

3. EVALUATION OF THE PERFORMANCE OF pH SENSOR - EVALUATION OF DETECTORS FOR AUTOMATIC PROCESS CONTROL

Availability of suitable automatic water quality analyzers with enough reliability and precision is an essential factor in automatic treatment process control. Whereas, adequate information regarding reliability and performance under the field conditions are not always necessarily available. The data provided by manufacturers usually refers to results with clean water, and rarely refers to the actual field conditions.

For this reason, the Ministry of Construction began investigating the reliability and performance of these instruments when used under practical field conditions.

Beginning this year, a pH meter was selected as the first object of investigation. The investigation includes establishment of the methodology to evaluate these detectors.

Following is the current test protocol for the pH sensor evaluation.

- (1) Participating manufacturers shall supply 2 units of their products, respectively.
- (2) All instruments shall be installed under identical conditions. And the installation shall be done by the manufacturers, themselves.
- (3) Out of the 2 units, one shall be operated continuously without any maintenance work until its measurement error exceeds a predetermined level (Test I).
- (4) The other unit shall be operated during the test period with standard maintenance works, that is, cleaning, calibrating and so forth at periodic intervals (Test II).
- (5) Total period of the experiment shall be longer than 2 times the period to run Test I, or 3 months.
- (6) Manual analysis shall be done at least once everyday.
- (7) At the end of Test I, pH standard solution shall be measured to determine the measurement error.

- (8) Prior to each calibration operation in Test II, pH standard solution shall be measured to determine the measurement error.
- (9) The user or a third party shall perform the manual analysis and regular maintenance work; and repairs, if any, shall be entrusted to the manufacturer.
- (10) The tests shall be conducted at least 2 places, one under favourable conditions for the measurement and the other under adverse conditions.
- (11) Using data obtained from the experiment, such as zero drift, span drift, difference of measurement from manual measurement, change in response, executed maintenance work, and so forth, the relationship between accuracy and maintenance, or frequency of maintenance to obtain data with a certain level accuracy shall be examined.

Experiments to evaluate the performance of pH meters are scheduled for September 1981 at a municipal sewage treatment plant with the participation of 4 different manufacturers. The initial series of experiments shall be conducted at a grit chamber for dry weather flow, and the subsequent ones are scheduled in a primary settling tank or aeration tank. In the service area of this treatment plant, numerous Chinese restaurants are located, disposing off fats and oils which render difficult conditions for automatic measurement.

The cleansing methods employed in the 4 different versions of instruments include one with air bubbles, one with water jet, and two with ultrasonic waves.

4. AUTOMATIC MONITOR FOR SEWAGE TREATMENT PLANT EFFLUENT

4.1 INSTALLED STATUS OF AUTOMATIC MONITORS FOR ORGANIC SUBSTANCE

In conspicuously developed regions, effluent standards which regulate only concentrations are not always adequate to curb the increasing water pollution; so in 1979, the Water Pollution Control Act was amended to implement a total loading regulation system. The regions where this Act is applied are currently the Tokyo Bay, Ise Bay, and Seto Inland Sea basins; and the pollutant covered by the system is limited to COD load (manganese method). By this act, the specified sources of pollution which discharge more than 400 cubic meters of wastewater were made obliged to install automatic monitors giving data that are highly correlated with manually analyzed COD, and to make continuous recording of the load. The designated instruments for this purpose are the UV photometer, COD meter, TOC meter, and TOD meter.

The sewage treatment plants which were subject to the above obligations numbered 67 around the Tokyo Bay region, 48 around the Ise Bay region, and 124 around the Seto Inland Sea region, that is a total of 239 sewage treatment plants. And the instruments installed are 229 UV photometers in 204 sewage treatment plants, 32 COD meters in 32 plants, and 5 TOC meters in 5 plants. Plants having multiple drainage outlets are obliged to install such instrument at each outlet.

Of the total installed instruments, UV meters account for the largest part. Since the UV photometer requires less maintenance and is rather inexpensive as compared to other instruments, its usage is recognized so far as its data have good correlation with the manually analyzed COD.

Generally speaking organic substances do not have strong absorption characteristics against the UV light of 253 nm wavelength, which is used in the measurement, but sewage which has undergone biological treatment contains substantial amounts of refractory organic substances which absorb UV light with a wavelength of that vicinity. Therefore, a relatively favourable relationship between UV absorption and COD seems likely to be attained.

However, if the effluent is colored due to industrial wastewater and so forth, the UV photometer may not give an appropriate index. In this case, a COD meter or some other instrument is used.

The total number of polluters which are obliged to take automatic measurements under the total loading regulation system is about 4000, out of which about 80% employ UV photometers. The enforcement of this type of obligatory automatic measurements is considered to be very effective.

4.2 DEVELOPMENT OF AUTOMATIC TOTAL PHOSPHORUS MONITOR

The total loading regulation system, as mentioned earlier, is enforced in limited areas with a limited parameter. However, there are several stagnant water bodies, like, lakes, etc., whose water quality is gradually deteriorating. So, in the future, not only implementation of more stringent effluent, concentration standards, but expansion of the total loading regulation system is expected. Especially concerning nutrients, national environmental standards and effluent standards are not yet established, but the control of eutrophication of stagnant water bodies is coming up as one of the most important topics in water pollution control policy. In the near future national standards are likely to be established.

Removal of nutrients, especially phosphorus removal in sewage treatment is keenly awaited. Although technological and economic problems still remain unresolved, a couple of treatment plants, which discharge their effluents into lakes, already have or have a plan to construct phosphorus removal facilities. Under these circumstances, it can be expected that in the future automatic measurement of total phosphorus in sewage will be on schedule. Therefore, technological research to develop an instrument for this purpose has begun.

The most important part in analyzing the total phosphorus is the decomposition of organic phosphorus into orthophosphate. Therefore, automatic monitoring of phosphorus is being studied placing emphasis on decomposition of organic phosphorus. The following three decomposition methods are being studied:

- (1) Decomposition by adding potassium persulfate and sulfuric acid to the sample, and heating the solution.

(2) Decomposition by UV irradiation

(3) Decomposition by electrolysis in the presence of sodium chloride

Method 1 is used in manual analysis which will be the official analytical method for the expected water quality standards, and a couple of manufacturers have already manufactured prototype models of this instrument based on this method. In Method 2, it is already known that complete decomposition with UV irradiation only is impossible, so that addition of oxidizing agents, heating, and other means are being examined. And, as to Method 3, the decomposition of organic phosphorus is considered to be done by chlorine and oxygen which is generated by electrolysis. This method is reported to be effective in decomposing the organic phosphorus contained in sea water.

The phosphorus in secondary effluents exists mostly as dissolved ortho-phosphate. Taking this into account, investigation is being continued to establish the most appropriate decomposition method for the automatic analyzer, and production of prototype models, field tests and assessment are scheduled for 1982.

**PILOT PLANT STUDY
FOR
TREATMENT OF COMBINED FISH-PROCESSING
AND
DOMESTIC WASTE
IN
MAKURAZAKI CITY**

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1. INTRODUCTION

Makurazaki city is situated in southern Kyushu and as a port has performed important duty as a southern ocean fishery base, especially for the landing of bonito and mackerel.

The city has a great number of medium and small size factories manufacturing dried bonito, the flakes of which are a popular seasoning in Japan. The factories are scattered over about 130 locations in the downtown area. As the wastewater discharged from those factories increased, the water pollution in small rivers, drains and the port significantly worsened. Therefore Makurazaki city planned to receive these fish processing wastes into its municipal wastewater treatment plant to improve water pollution in public water bodies.

Fish processing wastewater is high in BOD and Nitrogen compared with domestic wastewater, and fluctuates hourly both in quality and quantity. Accordingly, pilot plant experiments were conducted to determine the design and operational conditions of activated sludge process to obtain the effluent BOD of 20 mg/l or less.

2. INVESTIGATION OF WASTEWATER

2.1 Fish Processing Waste

Investigation was made of the quantity and quality of the wastewater from 4 factories selected according to the size of each factory. The manufacturing process of dried bonito is shown in Fig. 1.

As shown in Fig. 1, the main processes, in which wastewater is discharged from the bonito drying factories are; defrosting, cutting and boiling. These factories run during the daytime only. As the concrete floors and instruments are cleaned at the end of operations in each factory, the wastewater from the cleaning work is also discharged from these factories. As for the wastewater from boiling process, only the overflow is discharged during operation, and the concentrated solution remaining in caldrons is usually recovered and used as the material for manufacturing seasonings. However, the comparatively dilute solution remaining in the caldrons is not recovered and is discharged on weekend.

The characteristics of the three types of processing wastewaters shown in Fig. 1 are as follows:

Wastewater from defrosting process: At the early stage of discharge, the wastewater temperature was especially low at 3 - 4°C. The BOD gradually increased from 350 mg/l and finally reached about 3000 mg/l during discharge. Its quantity was about 53% of the total wastewater.

Wastewater from cutting process: The suspended solids in this wastewater was around 1000 mg/l and a large amount of bloody water was included. The BOD was 2000 - 5000 mg/l. Its quantity was 43% of the total wastewater.

Wastewater from boiling process: The suspended solids in this wastewater was around 2000 mg/l, the BOD was 1000 - 5000 mg/l, and the normal hexane extract substance was 500 mg/l. Compared with the wastewater from other processes, the concentration of this wastewater was highest, but its quantity was only 4% of the total wastewater.

Fig. 1 Dried bonito manufacturing process and water supply & discharge system

Table 1 Average wastewater quality in each process

Process Factory	BOD (mg/ℓ)					SS (mg/ℓ)	COD _{Mn} (mg/ℓ)
	Defrost- ing	Cutting	Boiling	Washing floor	Average	Average	Average
Factory A	806	3,360	7,110	-	1,970	714	1,440
Factory B	731	2,370	7,170	1,420	1,610	293	1,130
Factory C	578	1,770	6,380	4,350	1,570	776	1,300
Factory D	392	4,010	13,940	4,550	1,760	434	1,210
Average	623	2,270	8,190	-	1,720	542	1,260

Table 1 shows the average wastewater qualities investigated according to the each process of 4 factories. The ratio of BOD/COD_{Mn}/SS was about 3/2/1.

In order to project the fluctuation in the influent quality coming into a wastewater treatment plant, the quality and quantity of wastewater were investigated in drains which accepted fish processing wastewater (Fig. 2).

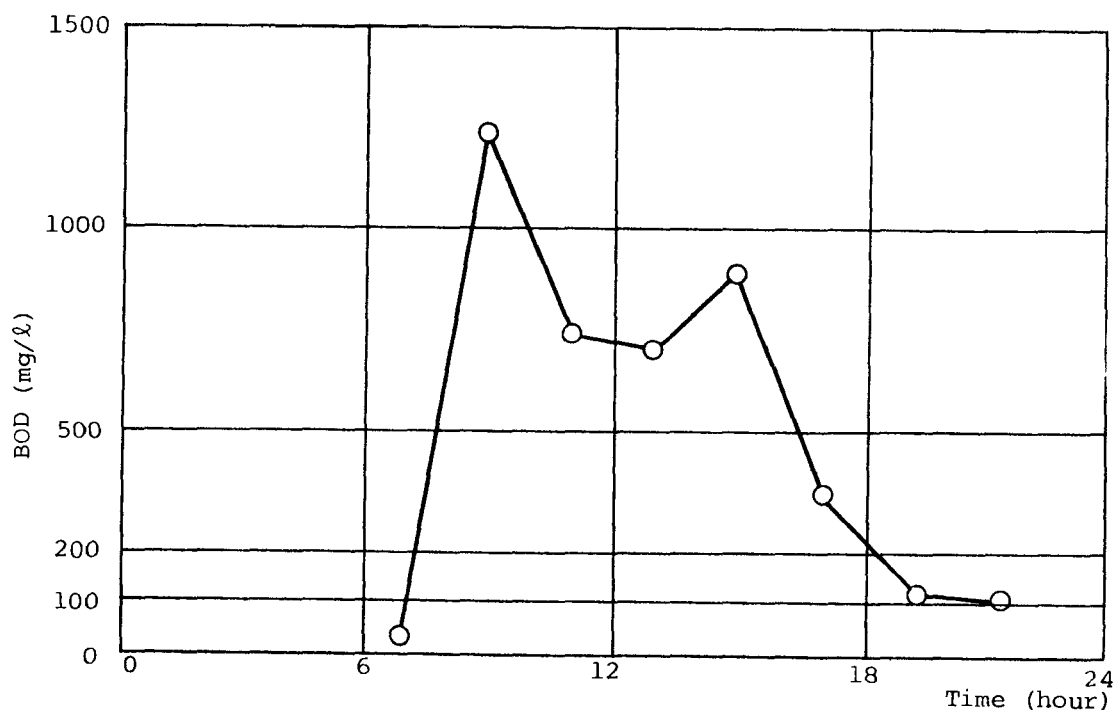


Fig. 2 Fluctuation of BOD

During the operation hours of the factories, from 8 a.m. to 4 p.m., the BOD was 730 - 1460 mg/l (averaging 960 mg/l), but at other times it was 200 mg/l.

2.2 Estimation of Combined Wastewaters

The quantity and quality of the combined wastewaters were estimated as shown Table 2.

Fish processing wastewater was estimated to account for 26% of the total inflow at the start of joint wastewater treatment, and for its influent quality, the BOD was 500 - 600 mg/l and the Kjeldahl nitrogen was 100 - 150 mg/l in most cases.

Table 2 Estimation of combined wastes

	1st stage	Final stage
<u>Quantity</u> (m ³ /day)		
Domestic Wastes		
Av.	3,618	15,200
Daily Max.	4,576	19,200
Hourly Max.	6,479	27,200
Industrial Wastes		
Av.	777	1,270
Dailt Max.	1,224	2,000
Hourly Max.	3,672	6,000
Total		
Av.	4,395	17,200
Daily Max.	5,800	21,200
Hourly Max.	10,151	33,200
<u>Quality</u> (mg/l)		
Domestic Wastes		
BOD	175 mg/l	175
SS	158 mg/l	158
Industrial Wastes		
BOD	1,715 mg/l	1,715
SS	556 mg/l	556
Total		
BOD	500	330
SS	240	200

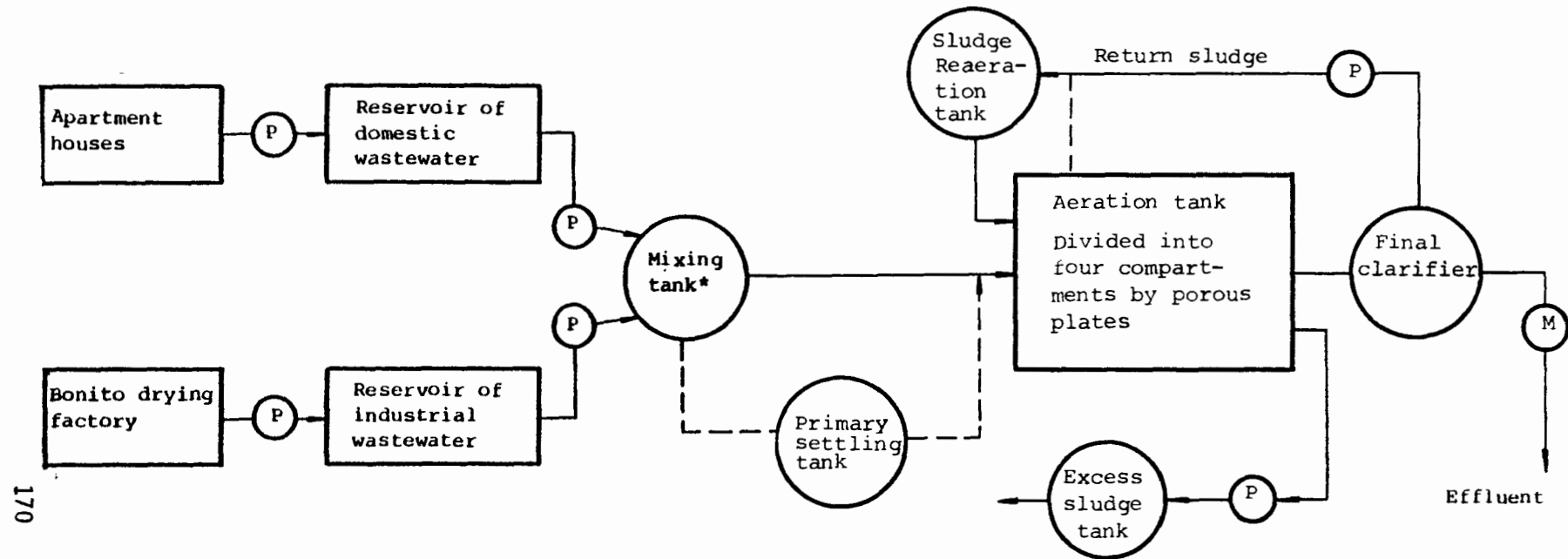
3. EXPERIMENT AND ANALYSIS

3.1 Pilot Plant and Experimental Procedures

Experiments of combined treatment of fish processing waste and domestic waste were conducted for about 1 year using an activated sludge process pilot plant.

The flow diagram of the pilot plant is shown in Fig. 3. Specifications of major facilities are shown in Table 3.

The pilot plant was operated in two modes of activated sludge process and recycled nitrification and denitrification process. In the experiments, it was planned to completely nitrify the effluent by the addition of alkali in order to assure the effluent BOD at less than 20 mg/l, since the influent nitrogen concentration was relatively high. Further, the recycled nitrification and denitrification process was studied in summer season to minimized the alkali dose rate.



* Also serving as pre-aeration tank

(P) : Pump
(M) : Flowmeter

Fig. 3 Flowchart of pilot plant

Table 3 Specifications of major facilities

Name of facility	Specifications		
Reservoir of domestic wastewater	Effective volume	2.5 m ³	
Reservoir of industrial wastewater	Effective volume	700 ℓ	
Reservoir of influent	Effective volume	200 ℓ	
Mixing tank (preaeration tank)	Effective volume	50 ℓ	
Primary settling tank	Diameter	0.6 m	
	Effective depth	1.6 m	
Aeration tank	Effective depth	Width	Length
	1.08 m x 0.7 m x 2.0 m (Divided equally into four compartments by porous walls)		
Final clarifier	Diameter	0.5 m	
	Effective depth	1.5 m	
Return sludge reaeration tank	Effective volume	200 ℓ	

Table 4 shows the operational conditions of the pilot plant study.

Table 4 Operational conditions of pilot plant

No.	Mode of operation	Temperature (°C)	Aeration time (hr)	Return sludge ratio (%)	BOD loading (BOD-kg / MLSS-kg.day)	MLSS (g/l)	SVI (-)	SRT (days)	Effluent pH
1	Activated sludge process	8.9~20.7 (14.0)	15.3~25.5 (21.2)	50	0.17~0.71 (0.38)	1.3~2.0 (1.7)	360~710 (570)	7~17 (11)	
2	Activated sludge process	11.6~18.1 (14.5)	19.5~27.3 (23.8)	100	0.21~0.42 (0.30)	1.9~2.7 (2.2)	250~400 (290)	5~17 (12)	
3	Activated sludge process	11.5~23.6 (16.3)	19.7~36.7 (24.3)	200	0.08~0.75 (0.38)	1.1~2.8 (1.8)	130~530 (230)	8~64 (16)	5.9~7.5 (6.8)
4	Recycled process	11.5~16.4 (14.6)	22.4~25.5 (23.9)	200	0.14~0.40 (0.22)	1.4~4.3 (2.7)	130~640 (290)	5~179 (33)	5.7~7.5 (6.9)
5	Recycled process	12.2~16.8 (13.2)	46.8~51.4 (48.5)	100	0.08~0.25 (0.12)	1.3~1.9 (1.6)	340~730 (520)	29~32 (32)	6.3~7.0 (6.6)
6	Recycled process	11.5~15.5 (13.2)	23.8~50.0 (35.8)	100	0.04~0.19 (0.11)	0.9~5.2 (3.4)	89~320 (190)	16~51 (28)	7.0~7.4 (7.2)
7	Recycled process	18.5~23.5 (21.7)	21.8~27.3 (23.8)	100	0.08~0.30 (0.17)	2.5~4.5 (3.6)	210~340 (250)	8~22 (13)	6.8~7.3 (7.0)
8	Activated sludge process	8.2~15.7 (12.4)	22.9~25.9 (23.0)	100	0.11~1.24 (0.17)	2.4~4.9 (3.7)	62~240 (140)	10~16 (12)	5.8~7.5 (6.7)
9	Activated sludge process	11.5~15.9 (13.4)	24.3~25.7 (25.2)	100/150	0.24~1.13 (0.17)	2.1~3.4 (2.8)	69~98 (86)	14~21 (18)	6.3~7.8 (6.8)

Figures in parentheses show the average values.

The major experimental procedures were as follows:

- (1) The domestic sewage and the bonito processing wastewater, which had been collected in their respective storage tanks, were homogenized in a mixing tank and fed to the primary settling tank.
- (2) For the fluctuation in the influent BOD loading, the influent BOD in the daytime (9 a.m. - 5 p.m.) was set at about 1000 mg/l, and the influent BOD in the nighttime (5 p.m. - 9 a.m.) was at about 400 mg/l.
- (3) To fix the wastewater temperature inside the aeration tank, the cooling water was circulated inside the tank through the cooling tubes.
- (4) Slaked lime milk was used to control pH in the aeration tank. If this milk was allowed to be kept for a long period, it was gradually converted to calcium carbonate. Therefore, it was prepared immediately before the dosing and added directly into the aeration tank.
- (5) At the early stage, the SVI became 500 or more, so that the raw wastewater was directly sent into an aeration tank bypassing the primary tank. After this, the SVI temporarily fell to 200 or less, but then rose again. To improve this condition, 30 mg of kaoline per liter of influent was dosed as an inorganic component.

The pilot plant experiment was started with the MLSS of 2500 mg/l and the aeration time of 16 hrs, and these operational conditions were modified according as the operational performance of the pilot plant.

- (1) Operation for acclimation of biological sludge (12 January - 10 February)

The seed sludge from the existing wastewater treatment plant, was introduced into the pilot plant for the start of sludge acclimating operation. The wastewater temperature was around 10°C, and the MLSS was 2500 mg/l when the initial operation was started. The influent BOD was 250 mg/l and the effluent BOD obtained was 20 mg/l

or less. The BOD loading was 0.17 kg/kg·MLSS/day and the SVI was 200. However, the SVI later rose to 400 - 600 and the MLSS fell to 1500 - 2000 mg/l. Therefore, the BOD loading in the tank became 0.24 kg/kg·MLSS/day. At the same time, the carry-over of biological sludge was observed and the effluent BOD exceeded 100 mg/l, but the dissolved BOD was maintained at 5 mg/l or less.

(2) Case 1 (10 February - 8 March)

The experiment was intended to maintain the MLSS of 3000 mg/l at the aeration time of 16 hrs. The SVI was as high as 400 - 600 and it was impossible to maintain the MLSS of 300 mg/l. Therefore the aeration time was altered to 24 hrs. Though extreme carry-over of biological sludge was not found, the effluent BOD remained at 30 - 60 mg/l. To investigate the cause of the relatively high effluent BOD, a survey was made of BOD and nitrogen compound distributions in the aeration tank. As a result, it was found that nitrite was liable to accumulate in the aeration tank due to the oxidation of ammonia and organic nitrogen.

(3) Case 2 (10 March - 29 March)

The higher dissolved BOD concentration in the influent brought about an MLVSS-MLSS ratio of about 0.9, so that the SVI considerably increased. Therefore, in order to maintain lower SVI, the influent was made to bypass the primary settling tank and introduced directly into the aeration tank. As a result, SVI fell to around 250 and the MLSS attained 1800 mg/l, although the effluent BOD did not become 20 mg/l or less. Furthermore, filamentous microorganisms in the aeration tank disappeared, and instead, ciliata became the predominant species. The effluent BOD from the final clarifier was found to be higher than that of mixed liquor supernatant in the aeration tank because of the incomplete nitrification.

(4) Case 3 (30 March - 8 June)

For the purpose of lowering the apparent influent concentration in the aeration tank, the sludge return rate was increased from 50% to 100 - 200% and, at the same time, the SRT was controlled to depress the nitrification. When the sludge return rate was adjusted to 100% and the SRT was set at 10 days, the BOD loading

in the aeration tank increased with the decrease in MLSS. Accordingly, the effluent BOD was not improved. Further, as the wastewater temperature was 16 - 19°C, nitrification tended to still continue. The sludge return rate was altered to 200% and the SRT was readjusted to 12 days. As a result, the SVI fell to 150 and the MLSS rose to 2500 mg/l. At this time, the effluent BOD was maintained at 20 mg/l or less. Then, as the settleability of the sludge was improved, the sludge return rate and the SRT were altered to 100% and 40 days respectively. Consequently, the effluent BOD and ammonia became 10 mg/l and 0.1 mg/l respectively, but the pH was lowered to 5.8. Therefore, 200 mg/l of slaked lime was added to adjust the pH to 7.0.

(5) Case 4 (9 June - 6 July)

In order to reduce the amount of lime milk required for nitrification, the recycled nitrification and denitrification process was applied. In this case, the first compartment of the aeration tank was arranged to be in an anoxic condition (DO: 0.2 mg/l or less). The wastewater temperature was set at 14°C and, in order to maintain MLSS high, the SRT of 30 days was used for this operation. The sludge return rate was set at 200% to increase the alkaline production in the anoxic compartment as much as possible. Filamentous microorganisms were gradually generated in the tank, which resulted in the raise of SVI. Therefore, this experiment was discontinued halfway.

(6) Case 5 (July - 27 July)

In application of the recycled nitrification and denitrification process, it was considered that the BOD loading to the aeration tank should be reduced for successful operation. Accordingly, aeration time was set at 48 hrs. As, however, the SVI failed to drop to the desired value and remained at around 500, 30 mg per litre of kaoline was dosed into the tank. Fifteen days from the start of kaoline dosing, the SVI fell to 350 and the MLSS became 1900 mg/l. The effluent BOD was about 10 mg/l. The nitrification of the effluent reached nearly 100% by dosing the mixed liquor with 100 - 150 mg/l of lime milk.

(7) Case 6 (28 July - 5 October)

In the previous experiments of the recycled process, the first compartment of the aeration tank was arranged to be in an anoxic state. In order to reduce the dosing rate of lime milk, the second compartment was also made to be in an anoxic condition. Further, as the MLSS began to rise, the aeration time was gradually shortened to 48 hrs, 40 hrs and 32 hrs. As a result, the amount of lime milk required for dosing was decreased to 50 mg/l, and the effluent BOD reached a stable state around 20 mg/l. At this time, the wastewater temperature was 12 - 14°C.

(8) Case 7 (6 October - 31 October)

The wastewater temperature was raised to 20°C, and the aeration time was shortened to 24 hrs, but the effluent BOD was kept to 1.6 - 13.5 mg/l and nitrification proceeded almost completely with addition of around 40 mg/l of slaked lime.

(9) Case 8 (1 November - 28 November)

In Case 8 and subsequent experiments, the operational mode was changed to the activated sludge process with lime milk addition. In Case 8, investigation was made to stabilize the effluent quality of BOD 20 mg/l under 24 hour aeration and wastewater temperature of 12 to 14°C. As a result, 300 mg/l of dosing slaked lime brought about an effluent BOD of 11.7 mg/l and a nitrification rate of 95%.

(10) Case 9 (29 November - 17 December)

The effect of the influent quality fluctuation upon the effluent BOD was investigated. For this purpose, the influent BOD was set at 1000 mg/l for the daytime (9 a.m. - 5 p.m.), and at 400 mg/l for the nighttime (5 p.m. - 9 a.m.). The pilot plant was operated at 25 hr aeration time and dosing rate of about 250 mg/l of slaked lime. As a result, the effluent BOD attained 3.1 - 11.6 mg/l.

3.2 Results

3.2.1 Analytical Results

Table 5 shows the overall analytical results of the pilot plant experiments.

3.2.2 Influent Quality

The influent BOD was 200 - 1000 mg/l, averaging 600 mg/l approximately. The ratios of BOD to other wastewater quality indices become higher as BOD increases, but within the range of BOD from 500 to 700 mg/l, the ratios are as follows:

Table 5 Analytical results

178

	T-BOD (mg/l)		SS (mg/l)		KJ-N (mg/l)		NH ₄ ⁺ -N (mg/l)		NO ₂ ⁻ -N (mg/l)	NO ₃ ⁻ -N (mg/l)	Alkalinity (mg/l)
	In.	Eff.	In.	Eff.	In.	Eff.	In.	Eff.	Eff.	Eff.	In.
1	284~1150 (547)	18~171 (56)	51~148 (89)	6~64 (25)							
2	407~963 (656)	10~95 (46)	92~803 (202)	8~49 (28)			49.0~117 (90.7)	10.5~41.5 (27.0)	6.7~17.5 (11.7)	0~12.5 (4.2)	
3	236~1500 (615)	16~153 (62)	80~266 (153)	10~73 (35)			18.4~210 (69.7)	0~39.4 (10.4)	3.3~31.4 (14.2)	0~10.5 (3.4)	150~349 (260)
4	286~725 (544)	7~69 (24)	96~540 (221)	5~85 (24)	90.7~152 (118)	0.7~23.7 (5.5)	27.8~81.0 (48.5)	0~15.3 (2.6)	0.03~17.0 (3.9)	1.2~56.8 (15.3)	180~417 (319)
5	179~593 (384)	5~12 (7)	165~513 (240)	6~13 (12)	92.0~142 (117)	0.4~1.6 (0.9)	30.5~60.5 (42.3)	0.1~2.1 (0.6)	0.3~4.6 (1.6)	0.6~8.3 (4.8)	287~386 (347)
6	188~1060 (538)	3~35 (14)	178~720 (313)	2~32 (7)	84.0~171 (129)	0.3~48.6 (16.4)	20.9~102 (60.6)	0.1~45.8 (10.9)	0.01~1.58 (0.69)	2.1~15.9 (6.2)	242~409 (324)
7	301~968 (593)	2~14 (5)	163~735 (310)	3~31 (9)	102~154 (126)	0.6~24.2 (5.3)	46.0~116 (84.4)	0.1~22.7 (3.8)	0.04~2.0 (0.62)	6.5~15.5 (10.1)	251~401 (328)
8	355~885 (628)	2~23 (12)	193~377 (270)	2~22 (9)	50.1~193 (126)	0.3~23.9 (6.0)	15.3~162 (79.8)	0~22.9 (1.9)	0.14~15.5 (7.9)	5.4~45.2 (26.7)	147~343 (292)
9	311~800 (494)	3~12 (6)	120~308 (204)	2~9 (5)	69.7~142 (104)	0.1~6.1 (1.5)	43.9~78.7 (59.6)	0~2.8 (0.7)	0.8~4.5 (2.3)	35.7~39.9 (31.8)	160~340 (274)

Figures in parentheses show the average values.

To COD _{Mn}	2.8
To Suspended Solids	2.16
To Total Nitrogen	5
To Total Alkalinity	1.8

The ratio of BOD:N:P was 40:8:1. Compared with ordinary municipal wastewater, the nitrogen content was comparatively high. Further, the values of wastewater quality indices other than BOD, in most cases, were as follows:

COD _{Mn}	: About 200 mg/l
Suspended Solids	: About 250 mg/l
Alkalinity	: 250 - 350 mg/l
Nitrogen	: Both nitrite and nitrate were scarcely present in the wastewater. Kjeldahl nitrogen was 100 - 150 mg/l and about a half of it was accounted for by ammonia.
Phosphorus	: The total phosphorus was 10 - 20 mg/l and 60 - 80% of the phosphorus was orthophosphate.
Normal Hexane Extract Substance	: 20 mg/l on the average. When the overflow from the caldrons came into the wastewater, 300 mg/l was reached due to the increased oil and fat contents.

3.2.3 Operational Conditions of Aeration Tank

When a primary settling tank was installed and the effluent from the tank was introduced into an aeration tank, the MLVSS/MLSS ratio became 90% or more. Because of relatively low content of inorganic matter, the settleability of activated sludge was poor. Therefore, the influent was made to bypass the primary settling tank and to flow directly into the aeration tank. As a result, the MLVSS/MLSS ratio went down to 80%, and SVI was lowered from 500 to 200. After this, the settleability of sludge became worse again, so that 30 mg of kaoline per liter was introduced into the mixed liquor. As a result, SVI was improved. SVI was lowered when $S_a \cdot t$ [S_a : MLSS (g/l)],

t: aeration time (hours)] became high. SVI was correlated with $Sa \cdot t$ and wastewater temperature as follow:

$$SVI = 922.1 - 17.5 (WT) - 7.3 (Sa \cdot t) \dots\dots\dots(1)$$

Where,

WT = Wastewater temperature ($^{\circ}C$)

Operational condition under the stable state were as follwos:

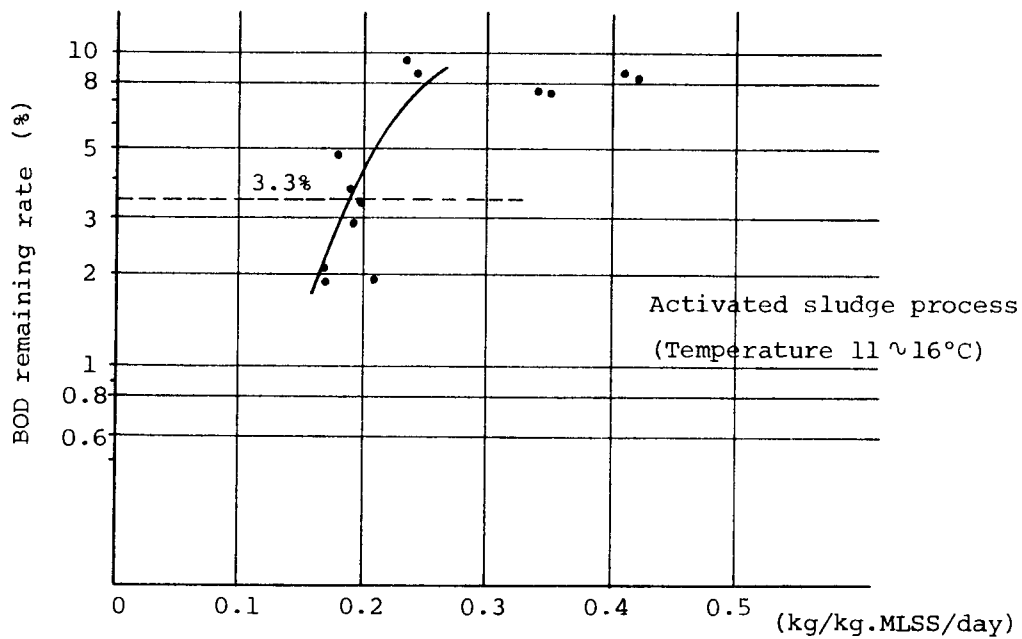
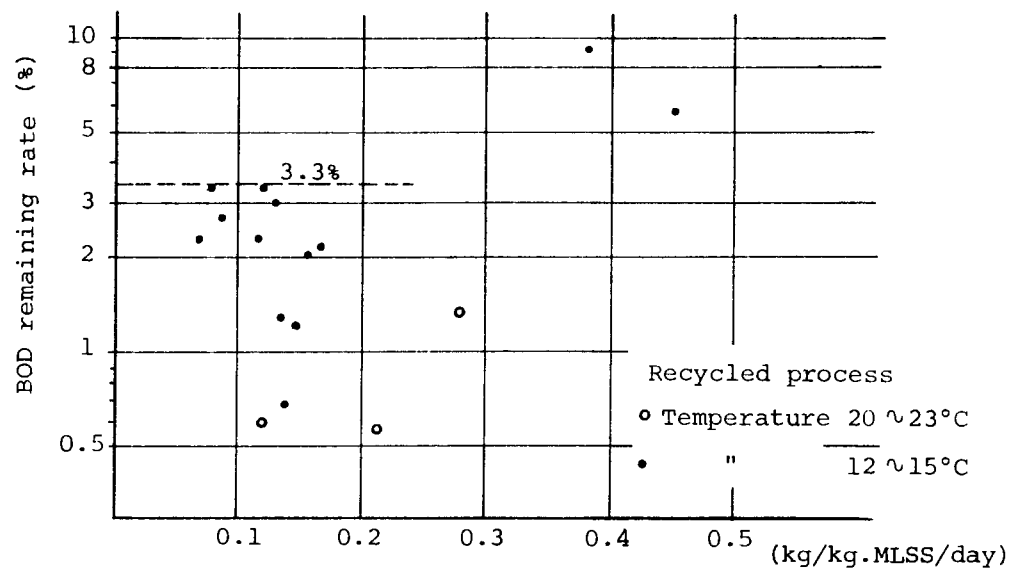
The 90% reduction of dissolved BOD was completed in the first compartment of the aeration tank, when the activated sludge process was applied. The nitrification showed almost the same tendency as the reduction of dissolved BOD.

When the recycled nitrification and denitrification process was applied with two anoxic compartments, 60% of dissolved BOD was reduced at the end of the first compartment and 90% of dissolved BOD was reduced at the end of the second compartment. And nitrate became about 0.1 mg/l at the end of the first compartment, about 5 mg/l at the end of the second compartment and about 12 mg/l at the end of the third compartment and about 14 mg/l at the end of the fourth compartment of the aeration tank.

In either the activated sludge process or the recycled nitrification and denitrification process, the oxygen utilization rate was estimated at 1.1-1.2 times the BOD removed. As for the activated sludge microorganisms, when the effluent quality was favorable, ciliatea became the predominant species. Especially when SVI and the MLVSS/MLSS ratio were comparatively low, aschelminthes appeared. Further, rhizopodes, especially Arealla were found when the nitrification was progressing favorably.

3.2.4 BOD loading vs. Efficiency

The relationships between BOD loading and BOD remaining rate are shown in Fig. 4. In winter season, the BOD loading was required to be 0.2 kg/kg.MLSS/day or less for the activated sludge process and 0.1 kg/kg.MLSS/day for the recycled process in order to get the BOD remaining rate of 3.3%, or the effluent BOD of 20 mg/l.



BOD-SS loading

Fig. 4 BOD-SS loading vs. BOD remaining rate

3.2.5 Alkali Balance for Nitrification

The alkalinity which is consumed when organic nitrogen and ammonia are converted to nitrate are theoretically 3.57 mg/l and 7.14 mg/l respectively. The average content of Kjeldahl nitrogen in the influent was 125 mg/l, of which about 1/2 was ammonia. The alkalinity of the influent was 323 mg/l. Accordingly, in the activated sludge process, the shortage in alkalinity was calculated as follows:

$$(125 \times 0.5 \times 3.57 + 125 \times 0.5 \times 7.14) - 323 = 346 \text{ (mg/l)}$$

In addition, the residual alkalinity, which was required to maintain the pH of effluent around 7, was about 50 mg/l. Total alkalinity of 400 mg as CaCO_3 is equivalent to slaked lime of 300 mg approximately. Therefore the actual dosed amount of slaked lime (= 300 mg/l) coincided with the calculated one. In the recycled process which was applied at the influent temperature of around 20°C, the slaked lime dosed was only 40 mg/l. The relationship between the actual and the calculated amounts of alkali consumption are shown in Fig. 5.

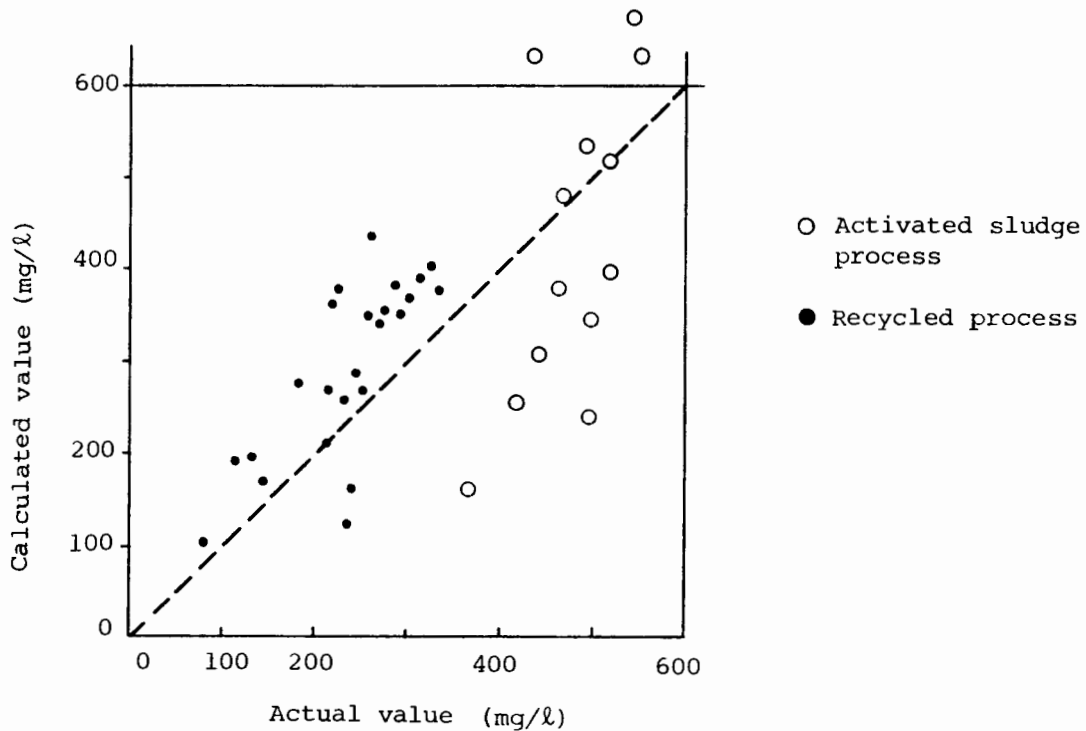


Fig. 5 Comparison of alkali consumptions

3.3 Analysis

In analyzing the results of the pilot plant experiments, the following multiple regression model was used.

$$Y = a_0 \times X_1^{a_1} \times X_2^{a_2} \times \dots \times X_n^{a_n} \dots \dots \dots (2)$$

Where, Y : dependent variable
 $X_1 \sim X_n$: independent variables

The data used in the multiple regression analysis were mostly the operational results of the activated sludge process. Table 6 shows the single correlation coefficients of the major operational parameters of the activated sludge process. WT, MLSS, BOD-SS loading, F/M, and Sa-t have large correlation coefficients to BOD remaining rate, and these values are more than 0.74. On the contrary, since aeration time and influent BOD were not altered much during the experiments, their correlation coefficients to BOD remaining rate were rather small.

Table 6 Single correlation coefficient

	R . T	W . T	IN BOD	IN BOD COD	MLSS	MLVSS	BOD-SS	OUT BOD	SVI	R	SRT	F/M	IN- BOD T/S	BOD remain- ing rate	Sa . t
R . T	1	0.46	0.43	0.10	-0.27	-0.39	0.47	0.41	-0.20	0.00	0.10	0.58	0.09	0.17	-0.22
W . T	0.46	1	-0.11	0.04	-0.87	0.10	0.85	0.73	0.26	0.00	-0.16	0.83	0.23	0.75	-0.86
IN BOD	0.43	-0.11	1	0.50	0.18	-0.32	0.26	0.27	0.03	-0.23	-0.09	0.32	0.26	0.12	0.19
IN $\frac{\text{BOD}}{\text{COD}}$	0.10	0.04	0.50	1	-0.23	-0.09	0.41	0.33	-0.05	0.51	0.30	0.37	0.45	0.12	-0.24
MLSS	0.27	-0.87	0.18	-0.23	1	-0.34	-0.87	-0.82	-0.45	-0.17	0.03	-0.79	-0.49	0.89	1
MLVSS	-0.39	0.10	-0.32	0.09	-0.34	1	-0.09	0.23	0.65	-0.17	-0.10	-0.16	-0.05	0.33	-0.36
BOD-SS	0.47	0.85	0.26	0.42	-0.87	0.01	1	0.92	0.40	0.04	-0.10	0.97	0.46	0.82	-0.87
OUT BOD	0.41	0.73	0.27	0.33	-0.82	0.23	0.92	1	0.60	0.13	0.08	0.85	0.47	0.91	-0.81
SVI	-0.20	0.26	0.03	0.05	-0.45	0.65	0.40	0.60	1	-0.58	-0.10	0.22	0.24	0.64	-0.46
R	-0.05	0.00	-0.23	0.51	-0.17	-0.17	-0.04	-0.13	-0.58	1	0.43	0.01	0.31	0.06	-0.18
SRT	0.07	0.16	-0.09	0.30	-0.03	0.10	-0.09	0.08	-0.10	0.44	1	-0.11	0.23	-0.10	-0.03
F/M	0.58	0.83	0.32	0.37	0.79	-0.16	0.97	0.85	0.22	0.10	-0.11	1	0.44	0.74	-0.78
IN-BOD T/S	0.09	0.23	0.26	0.45	-0.49	0.05	0.46	0.47	0.24	0.31	0.23	0.44	1	0.42	-0.49
BOD re- maining rate	0.17	-0.75	-0.12	0.12	-0.89	0.33	0.82	0.91	0.64	0.06	-0.10	0.74	0.42	1	0.89
k'	0.16	0.84	-0.30	0.19	-0.89	0.15	0.78	0.57	0.19	0.30	-0.17	0.74	0.31	0.70	-0.89

Where, RT: Aeration time

R: Return sludge ratio

 $k' = -\{\log(\text{OUT-BOD}/\text{IN-BOD})\}/\text{RT} \times \text{MLSS}$

WT: Mixed liquor temperature

T/S: Total/Soluble

Sa . t: $\text{RT} \times \text{MLSS}$

BOD-SS: BOD-SS loading

3.3.1 BOD Remaining Rate

For BOD remaining rate, the following equation was obtained.

$$\text{BOD}_{\text{R.R.}} = 8.923 \times (\text{WT})^{-0.566} \times (\text{Influent BOD})^{0.467} \times (\text{Sa} \cdot t)^{-1.71} \dots\dots\dots (3)$$

(multiple correlation coefficient: 0.941)

Where,

- BOD_{R.R.} : BOD remaining rate (%)
- WT : Wastewater temperature in aeration tank (°C)
- Sa : MLSS (g/l), and
- t : Aeration time (hours).

Fig. 6 shows the relationship between the measured and the calculated values of BOD remaining rate using the equation.

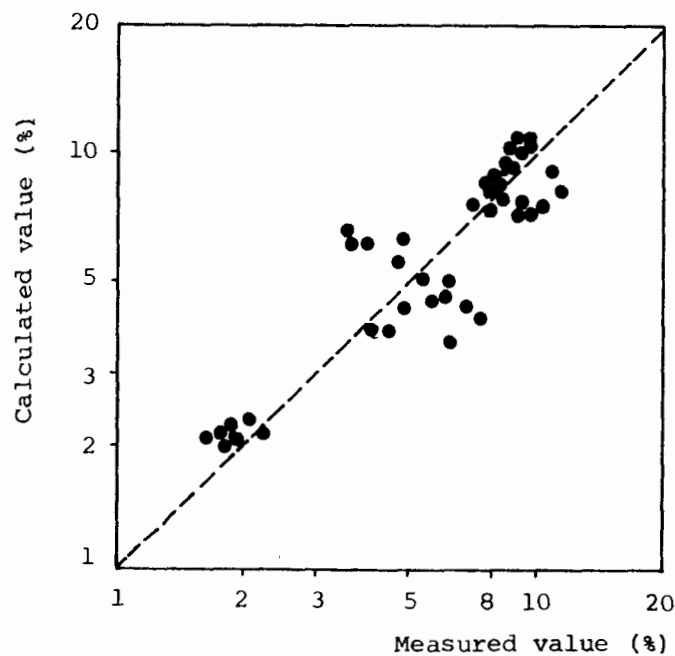


Fig. 6 BOD remaining rate

Fig. 7 shows the relationship between BOD remaining rate and $\text{Sa} \cdot t$ in regard to the summer water temperature (20°C) and to the winter one (14°C) when influent BOD is 600 mg/l.

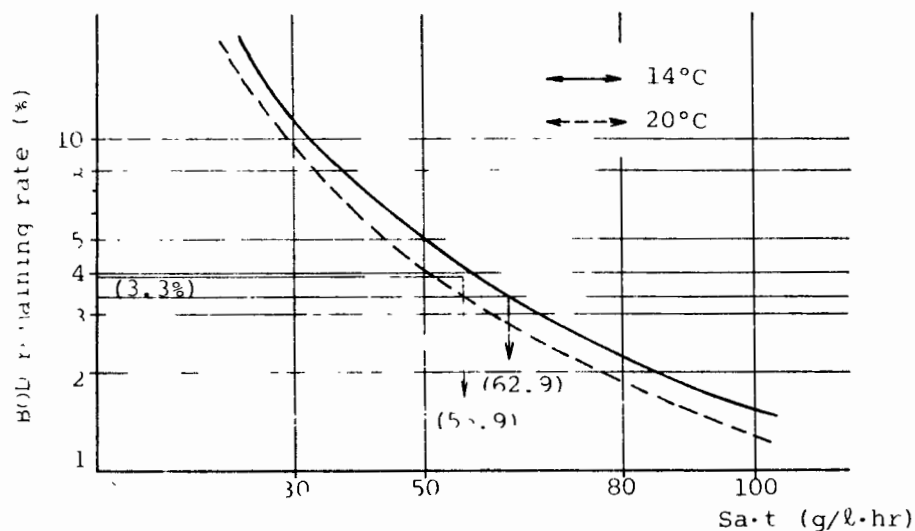


Fig. 7 BOD remaining rate vs. Sa·t

This figure shows in order to obtain the BOD remaining rate of 3.3%, namely, effluent BOD is 20 mg/l, Sa·t should be kept more than 62.9 g/l·hr and more than 55.9 g/l·hr to water temperatures of 14° and 20°C, respectively.

Based upon this, the relationship between aeration time and BOD remaining rate for winter season is shown in Fig. 8.

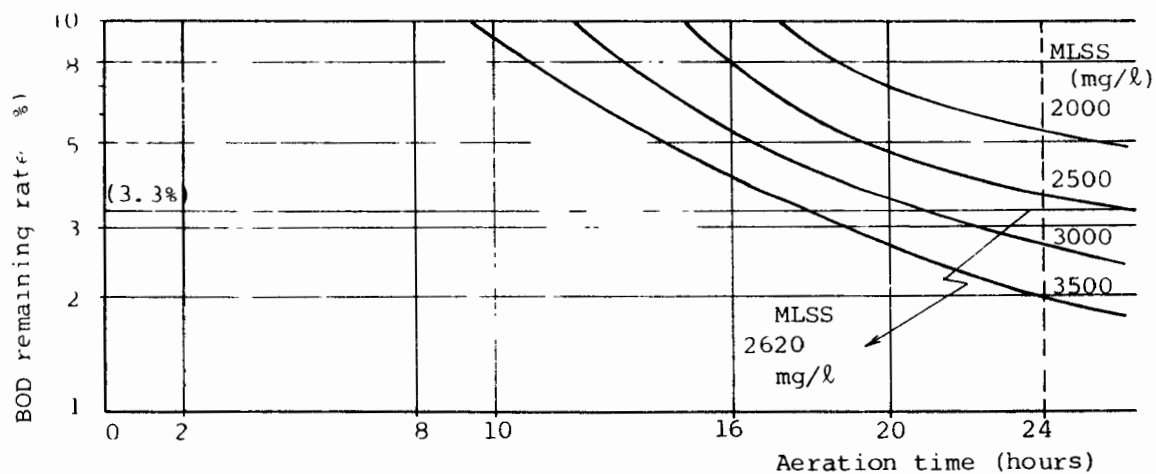


Fig. 8 BOD remaining rate vs. aeration time when water temperature is 14°C

It is known from this figure that in order to obtain the effluent BOD of 20 mg/l in winter, it is required to keep MLSS at some 2620 mg/l for the aeration time of 24 hrs.

3.3.2 Nitrification Rate

For the nitrification rate, the following equation was obtained to the variables of mixed liquor temperature, $S_a \cdot t$ and influent total nitrogen.

$$1 - \text{N.R.} = 18.578 \times (\text{WT})^{-1.901} \times (S_a \cdot t)^{-2.834} \times (\text{Influent nitrogen})^{2.263} \quad \text{..... (4)}$$

($\gamma=0.899$)

Where,

N.R. : Nitrification rate (%)

The relationship between the measured and the calculated values of (1 - nitrification rate) is shown in Fig. 9.

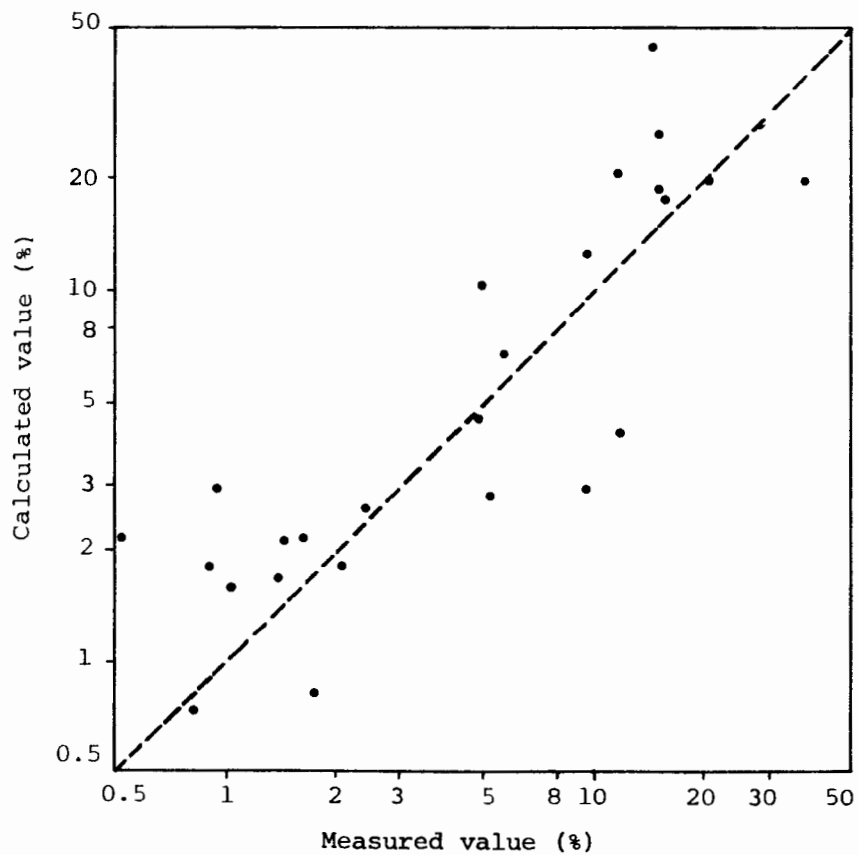


Fig. 9 (1 - Nitrification rate)

Fig. 10 shows the relationship between $Sa \cdot t$ and $(1 - N.R.)$ at temperatures of 14°C and 20°C , and influent total nitrogen concentration of 130 mg/l .

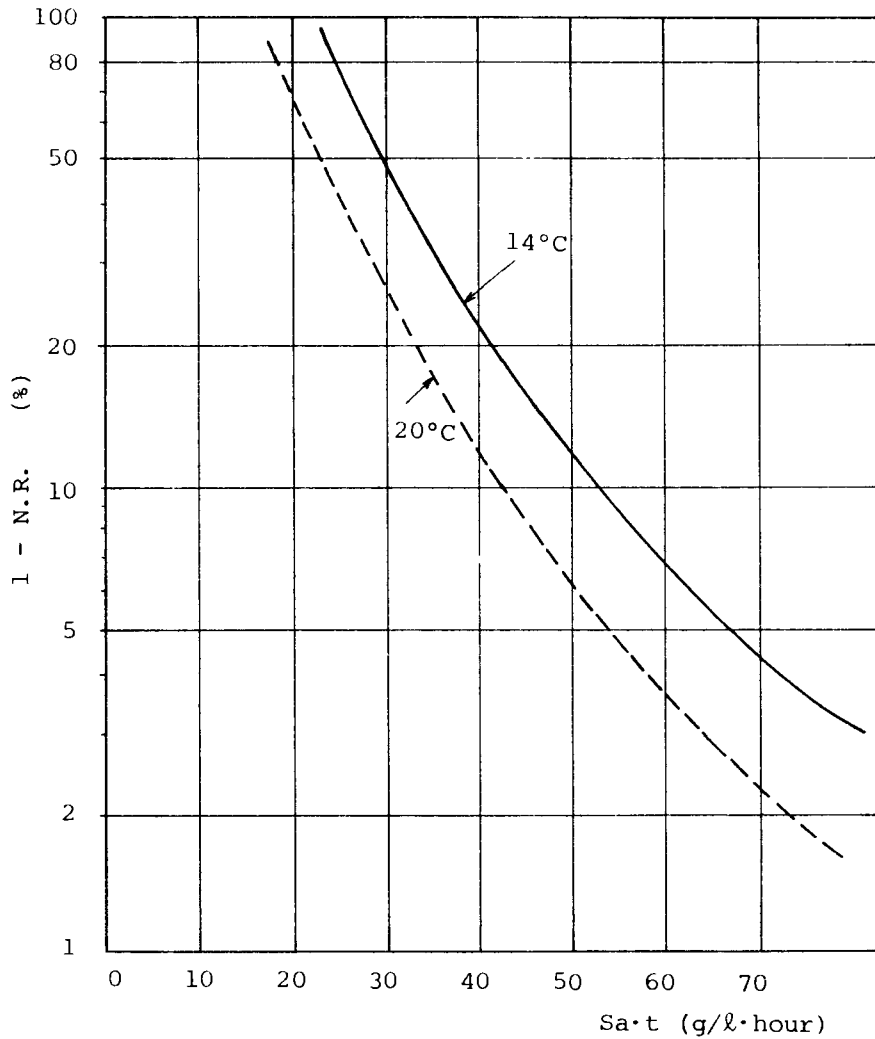


Fig. 10 $(1 - N.R.)$ vs. $Sa \cdot t$

From this figure, it is found that $Sa \cdot t$ required to obtain the nitrification rate of 90% is $52.5 \text{ g/l} \cdot \text{hr}$ in winter. Accordingly, it is known from figures 7 and 10 that nitrification rate can reach 90% if only $Sa \cdot t$ exceeds the value that satisfies the BOD remaining rate of 3.3%.

3.3.3 Oxygen Demand

The relationship between mixed liquor temperature and oxygen utilization rate (O_2 UR) is shown in Fig. 11.

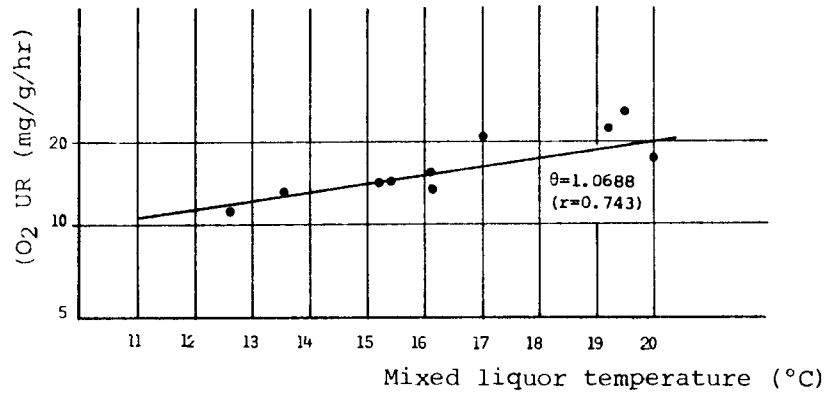


Fig. 11 Mixed liquor temperature vs. O_2 UR

From this figure, O_2 UR is expressed in the

$$O_2 \text{ UR} = 20.1 \times (1.0688)^{(T-20^\circ\text{C})} \dots\dots\dots (5)$$

Fig. 12 shows the relationship between O_2 UR and BOD removal at the water temperature of 14°C.

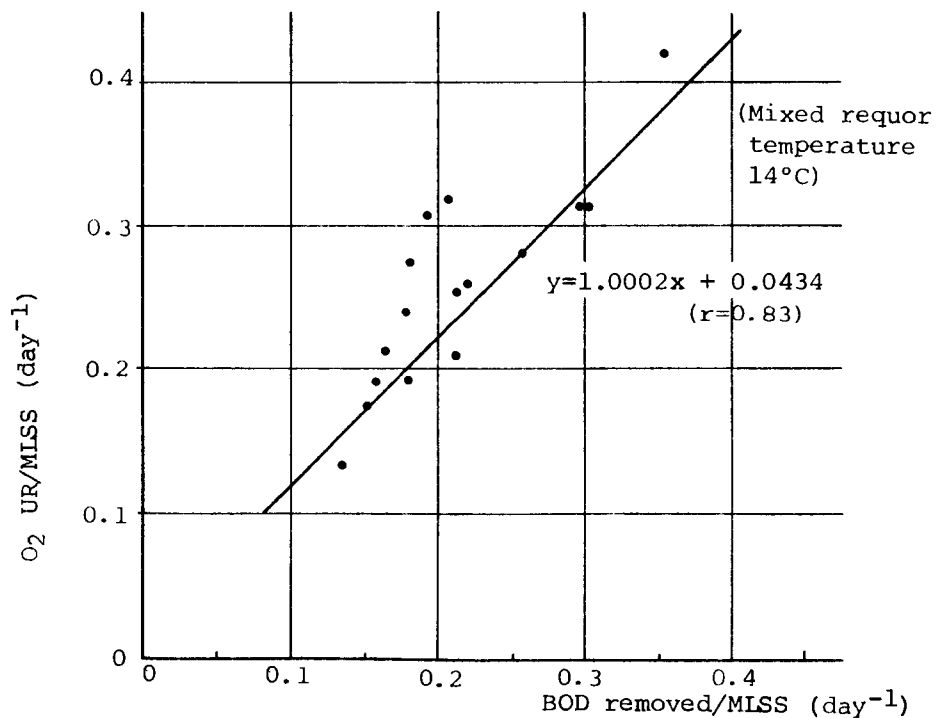


Fig. 12 O_2 UR, vs. BOD removed

3.3.4 Sludge Production

Since the excess sludge produced was not expressed well in multiple regression equation, the data were processed through the following equation.

$$\Delta S = a \cdot L_r - b \cdot S_a' \dots\dots\dots(6)$$

Where, ΔS : Sludge production (kg/day)

L_r : BOD removed (kg/day)

S_a' : MLSS in the system (kg)

a : Proportion of L_r that is synthesized and converted to new cell

b : Endogenous respiration rate per day.

Using this equation, Fig. 13 was obtained, which shows the relationship between BOD removed and sludge production.

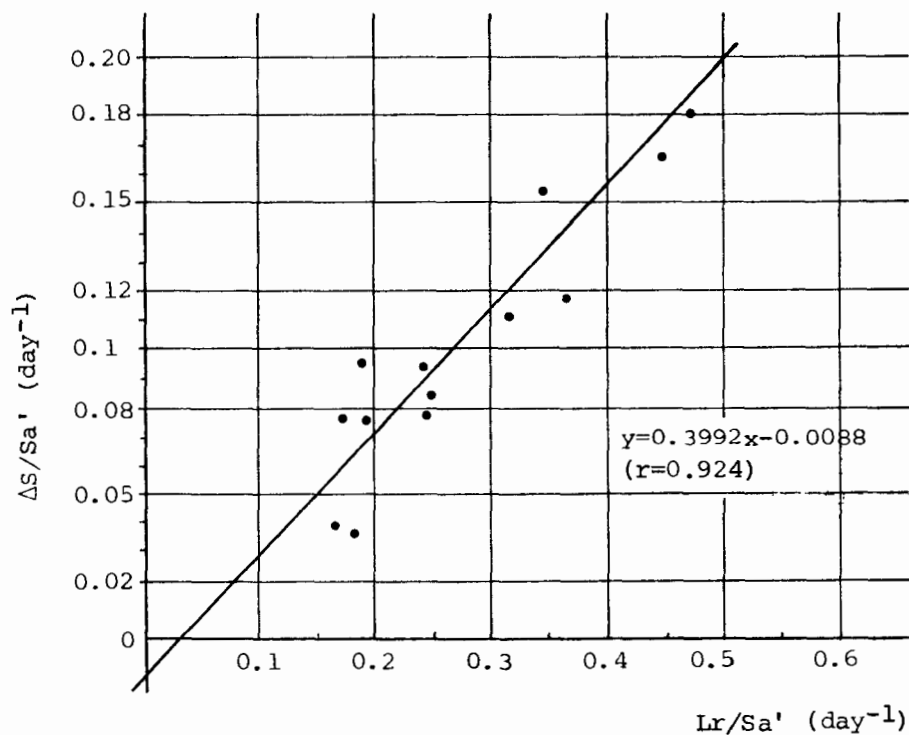


Fig. 13 Relationship between BOD removed and sludge production

3.3.5 Sludge Thickening Ratio in Final Clarifier

It was considered that the sludge thickening characteristics and the return sludge concentration could be judged by thickening ratio (= Return Sludge concentration/MLSS). As MLSS rose, the thickening ratio tended to fall. When MLSS was around 3000 mg/l, the thickening ratio became 1.8 - 2.2.

For solid loading, following equation was obtained.

$$\begin{aligned} \text{Solid loading (kg/m}^2\text{/day)} &= -81.97 \cdot (\text{thickening ratio}) \\ &+ 190.5 \dots\dots\dots (7) \end{aligned}$$

Fig. 14 shows this relationship.

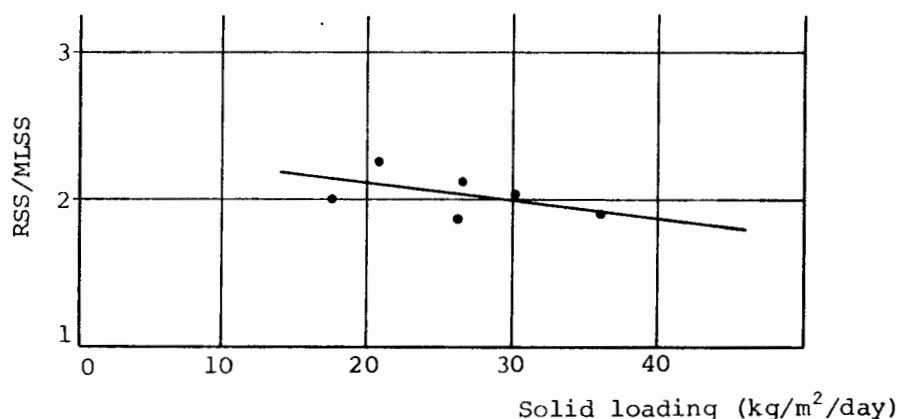


Fig. 14 Solid loading vs. thickening ratio

A relation between thickening ratio and SVI was not found clearly.

3.3.6 Effluent Characteristics

An investigation was made on the relation between effluent BOD and ammonia and nitrite. When the effluent nitrite and ammonia were 1 mg/l or less, the BOD of the suspended solids portion of the effluent (hereinafter refers to (T-S) BOD) was 10 mg/l or less. On the other hand, when the effluent nitrogen was nitrite type containing the nitrite of 15 mg/l or more, (T-S) BOD to suspended solids ratio was nearly 1.0. When the nitrogen in effluent was ammonia type and the ammonia concentration was 10 mg/l or more, (T-S) BOD/SS ratio = 2.0 - 2.5. In either case, (T-S) BOD mostly became 20 mg/l.

Figures 15 and 16 show the relation between effluent SS and effluent (T-S) BOD.

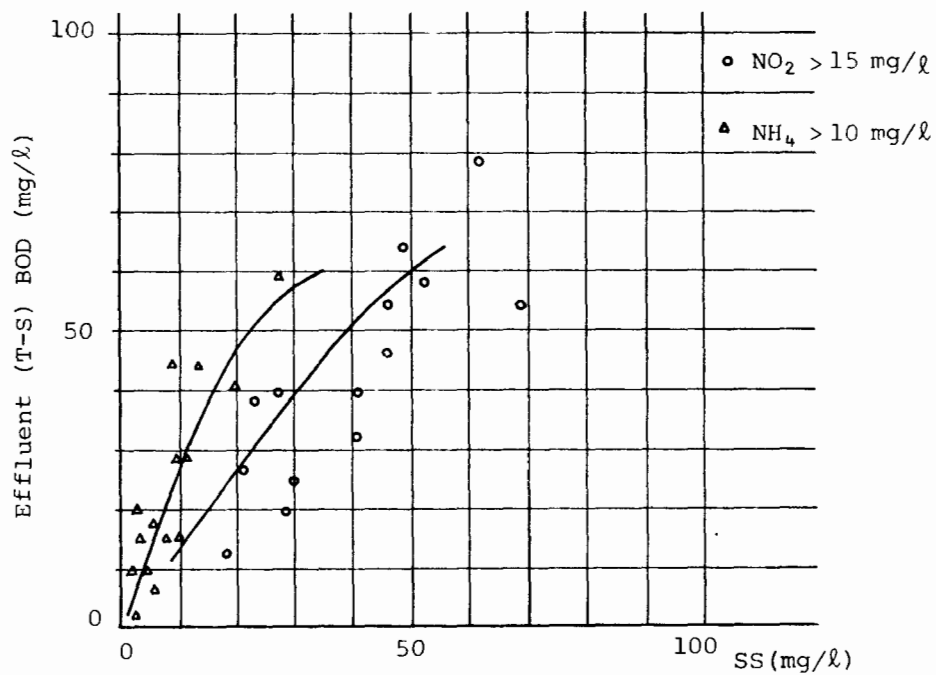


Fig. 15 Effluent SS vs. Effluent BOD in SS

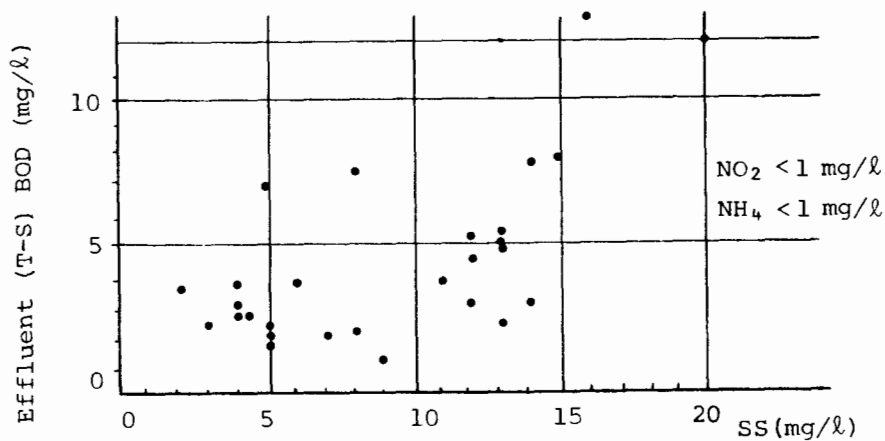
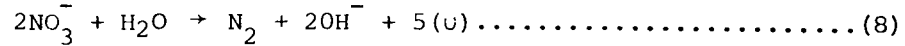


Fig. 16 Effluent SS vs. Effluent BOD in SS

3.3.7 Nitrogen Removal in Recycled Process

The relation between nitrogen removal and BOD removal in Recycled Nitrification and Denitrification Process was analyzed.

The denitrification reaction is expressed as follows:



Therefore, the theoretical amount of BOD required for 1 mg of nitrate removed is:

$$\frac{5 \times 16}{2 \times 14} = 2.86$$

Fig. 17 shows the relation between the nitrogen removed and BOD removed in the denitrification compartments of recycled process.

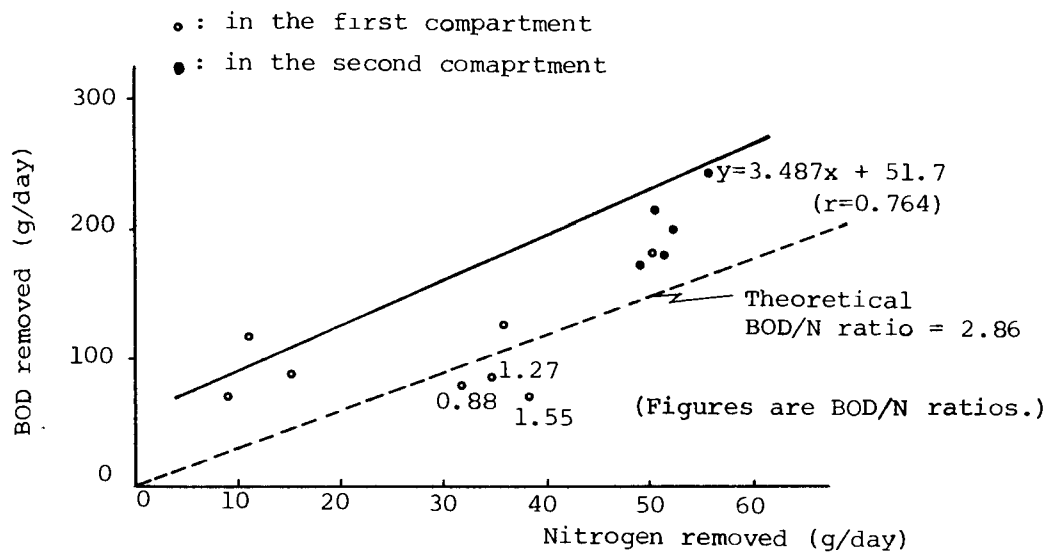


Fig. 17 Nitrogen removed vs. BOD removed

For influent BOD_L/N ratio ≥ 2 , the following equation was obtained.

$$\text{BODr} = 3.487 \text{ Nr} + 51.7 \dots \dots \dots (9)$$

Where,

Nr : Nitrogen removed (g/day)

BODr: BOD removed (g/day)

The slope 3.487 indicates the BOD consumed for denitrification, which is about 20% higher than the theoretical value of 2.86.

The constant 51.7 indicates that there existed a portion of BOD removed by other than denitrification.

This suggests that the initial adsorption of BOD by micro-organisms might occur. Fig. 18 shows the relation between the amount of nitrification and BOD removed.

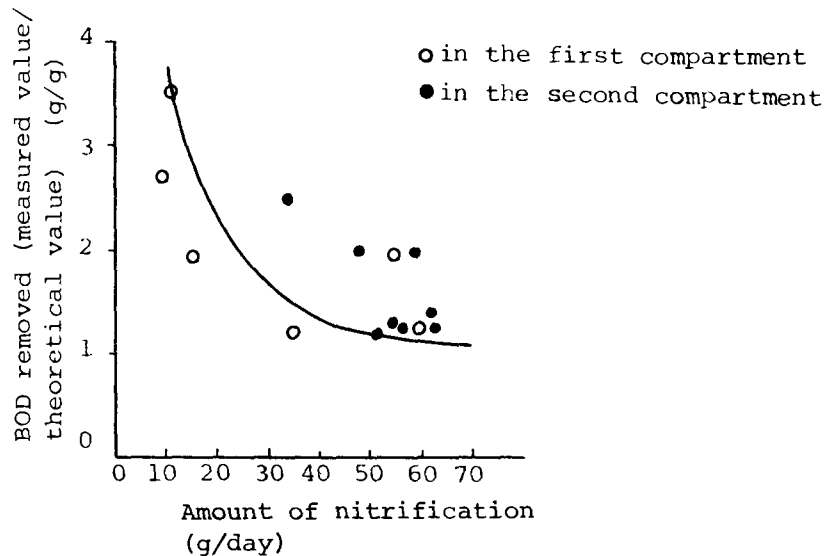


Fig. 18 Amount of nitrification vs. BOD removed

The ratio of the actual BOD removed to the theoretical value approaches unity as the amount of nitrification increases. In other words, it shows that BOD is removed in accordance with the theory when nitrification progresses, and that BOD removal is mostly due to the reaction other than denitrification when nitrification does not much progress.

The nitrogen removal rate could be expressed in terms of aeration time rather than $Sa \cdot t$ as shown in Fig. 19.

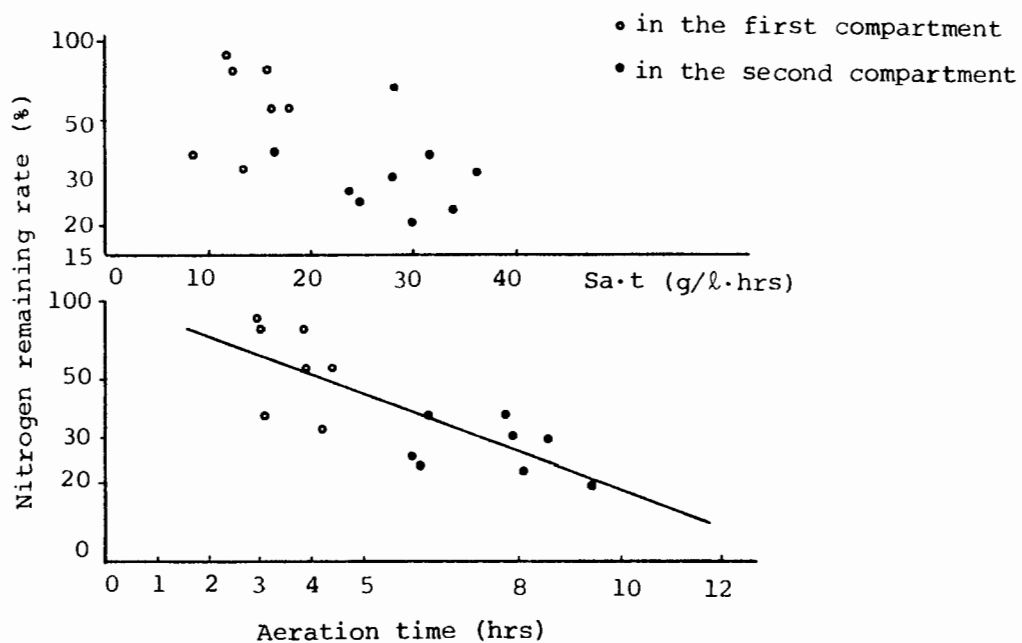


Fig. 19 Nitrogen remaining rate vs. Sa·t and aeration time

From Fig. 19, nitrogen remaining rate is obtained as follows:

$$\log (\text{Nitrogen R.R.}) = -0.072t + 2.003 \dots\dots\dots (10)$$

$$(r = 0.78)$$

Where,

t : Aeration time (hours)

Nitrogen R.R.: Nitrogen Remaining Rate (%)

4. CONCLUSION

In the pilot plant experiments, it was found that the complete nitrification of the effluent was required by adding alkali to the aeration tank in order to assure effluent BOD of 20 mg/l through the biological treatment of the combined wastewater having a BOD of 600 mg/l and a nitrogen concentration of 100 - 150 mg/l.

Design temperature of the influent in winter season was set at 14°C.

Under this condition, the following design and operational concepts were recommended to obtain the stable effluent BOD of 20 mg/l.

- (1) The flowchart of the combined wastewater treatment plant is proposed as shown in Fig. 20.
- (2) An aeration tank should be designed to have 4 divided compartments. And the first one or two compartments should be able to be kept under anoxic condition in case of the recycled nitrification-denitrification process.
- (3) In winter season when the wastewater temperature is 14°C or less, the plant is to be operated in the activated sludge process mode of 24 hr-aeration and the effluent pH should be controlled by adding a maximum 300 mg/l of slaked lime.

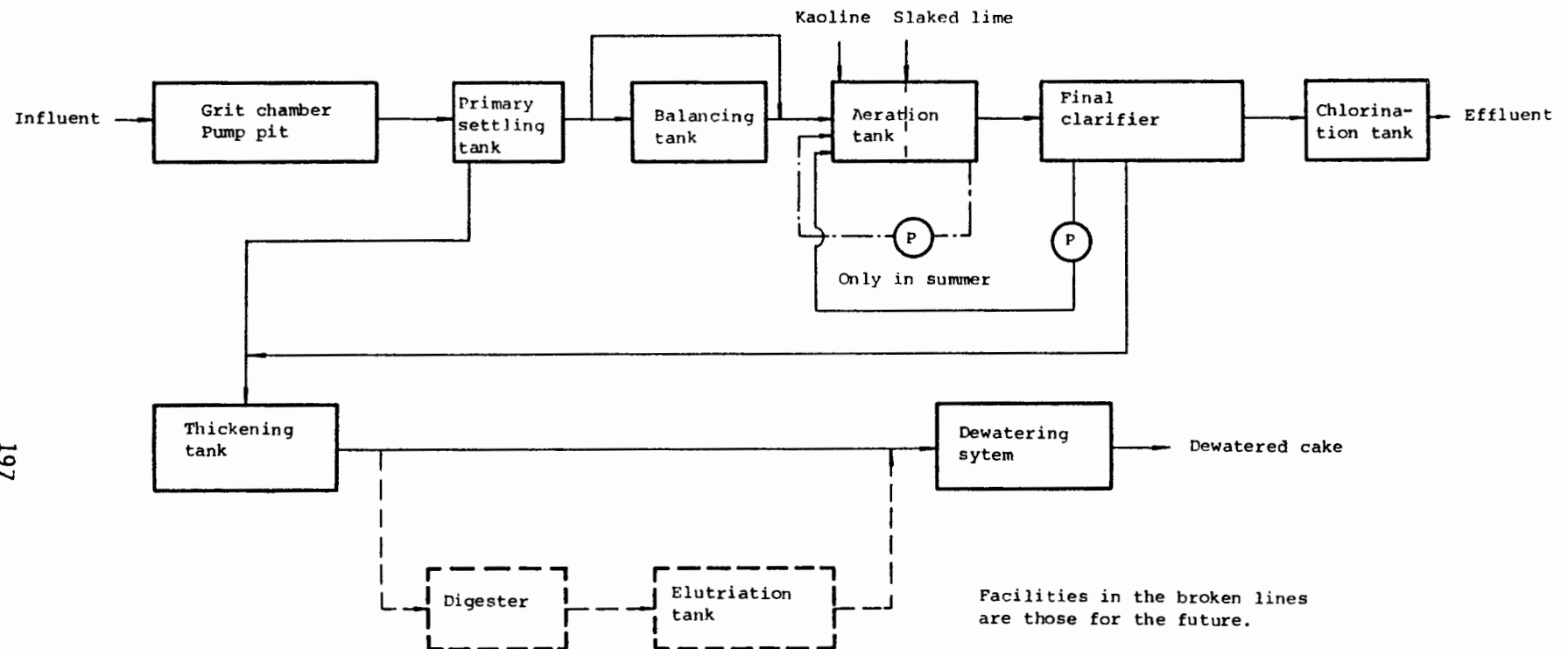


Fig. 9 Flowchart of the combined treatment plant

- (4) It is possible to reduce the lime addition rate to 40 mg/l if the plant is operated in the recycled nitrification denitrification mode of 32 hr-aeration time, in winter season.
- (5) In summer season when the influent temperature of 20°C or more, the plant is to be operated in the recycled nitrification-denitrification mode of 16 hr-aeration time with the addition of 40 mg/l of slaked lime.
- (6) An overflow rate of 15 m³/m²/day should be adopted for the final clarifier because of high organic content of the activated sludge and its resultant poor settling characteristic.
- (7) It is preferable to add kaoline to the aeration tank in order to improve the settling characteristic of the activated sludge.

**Eighth US/JAPAN Conference
on
Sewage Treatment Technology**

TECHNICAL EVALUATION OF DEEP WELL BIOLOGICAL PROCESS

October 13-14, 1981

Cincinnati, Ohio USA

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1. TECHNOLOGY EVALUATION SYSTEM

The Ministry of Construction has a technology evaluation system for promoting actual use of new and practical technologies for public works developed by private companies. In this system, the Ministry of Construction publicly announces a particular technology to be evaluated and the goals to be achieved by that technology, and invites private companies to submit their own development techniques for this evaluation. After applications are received, the applicants are required to conduct an experiment for about one year to provide data to determine whether the goals are achieved by their techniques. Then, the Ministry of Construction will evaluate the results of the experiment, and the results of any additional experiment that the Ministry may specify. This evaluation is conducted by a committee consisting of engineers from the Ministry of Construction and local governments, and college professors specializing in each particular technological field.

A public announcement was made in June 1979 regarding the project evaluating "Deep Well Biological Process for Sewage Treatment". Two companies, I and K, applied for this evaluation, and in fiscal year 1980 the Ministry of Construction evaluated the techniques of these two companies. The evaluation was completed in July 1981, and the results of the evaluation will be described in this paper.

The conclusions of the evaluation are summarized as follows:

Both of the techniques attained the publicly announced goals, and judging from the treatment capacity and the actual operation, they reached the stage of practical use as a municipal sewage treatment technology.

- (1) These were practical techniques with a treatment capacity of 1,000 m³/day or more.
- (2) These techniques were capable of meeting the legal requirements for biochemical oxygen demand(BOD) and suspended solids(SS) in effluents. The requirements are 20 mg/l for BOD and 70 mg/l for SS.

- (3) In the operation of facilities under this process, there was no marked difference between these techniques and the conventional methods such as the conventional activated sludge process (hereinafter referred to as "conventional method").
- (4) As for the operation and maintenance cost, these techniques did not differ greatly from the conventional method.
- (5) These techniques had simplicity in taking countermeasures to prevent possible detrimental effect on the surrounding environment.
- (6) Much less land area was required for the aeration tank in this process, compared with the conventional method.

2. OUTLINE OF TECHNOLOGY TO BE EVALUATED

The technology selected as the object of the evaluation is called the "Deep Well Biological Process for Sewage Treatment".

The deep well biological process is a type of activated sludge process with a deep well shaped aeration tank of about 50 - 150 m depth which utilizes a high hydrostatic pressure for improvement in the dissolution rate and utilization ratio of oxygen in the air and, at the same time, reduces the land area required for the sewage treatment facilities.

2.1 PRINCIPLES

As shown in Fig. 1, the aeration tank of this process is a deep well type aeration tank having a depth to over a hundred meters.

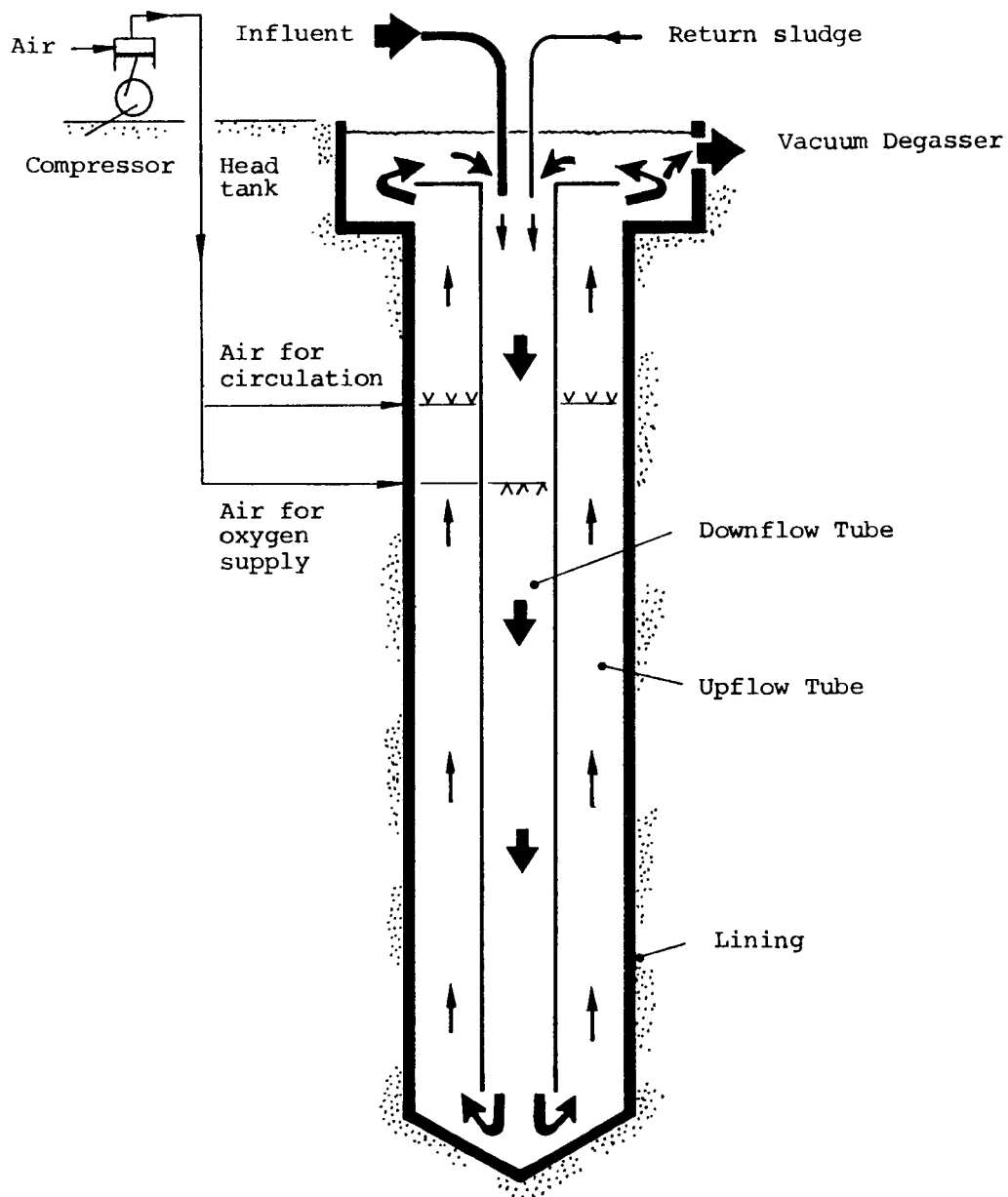


Fig. 1-a Deep well aeration tank of Company I

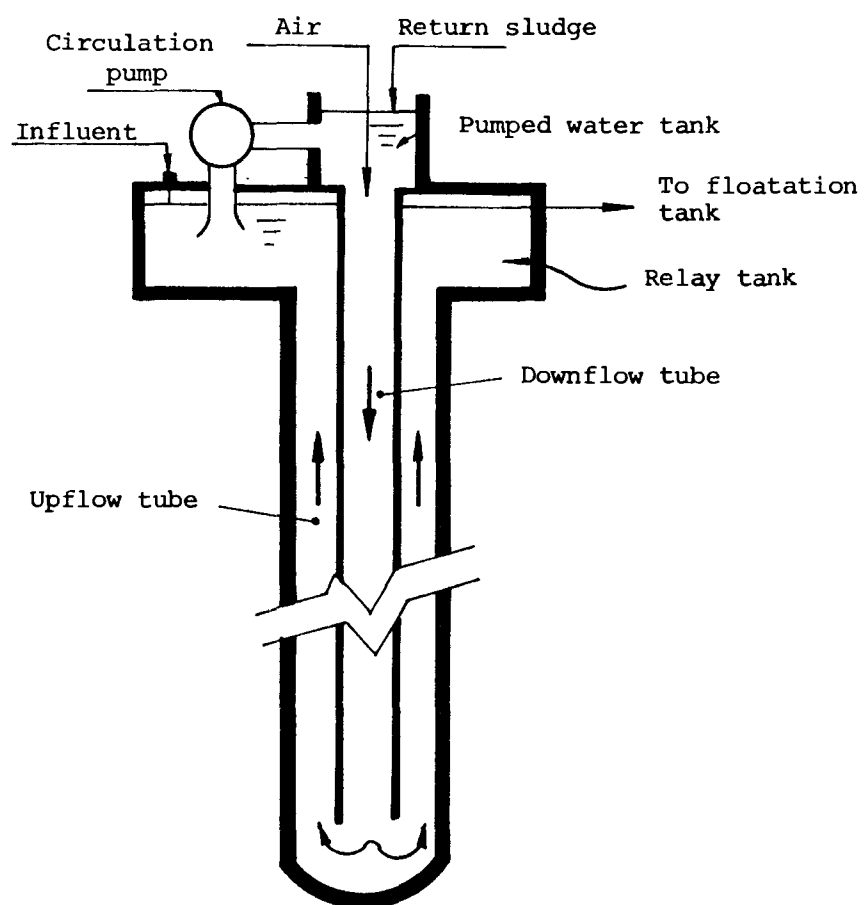


Fig. 1-b Deep well aeration tank of Company K

This aeration tank is composed of an upflow tube, a downflow tube, and head tank. The head tank is provided for releasing spent air from the aeration tank, through which the mixed liquor is circulated.

Two companies, I and K, applied for technical evaluation of the deep well biological process. The circulation systems of two companies were different from each other.

Company I employed an "air-lift system" for circulation, in which the mixed liquor is circulated by injecting air into the upper portion of the upflow tube.

Company K employed a "pump system", in which a recirculation pump is installed between the relay tank and the downflow tube.

2.2 SYSTEM AND FUNCTIONS OF PLANT

2.2.1 System

Fig. 2 shows the total system of a sewage treatment plant employing the deep well biological process in comparison with the conventional method. (Fig. 2-a shows the plant of Company I, and Fig. 2-b, that of Company K. In the figures and tables, the plants of Companies I and K are referred to as "a" and "b", respectively.)

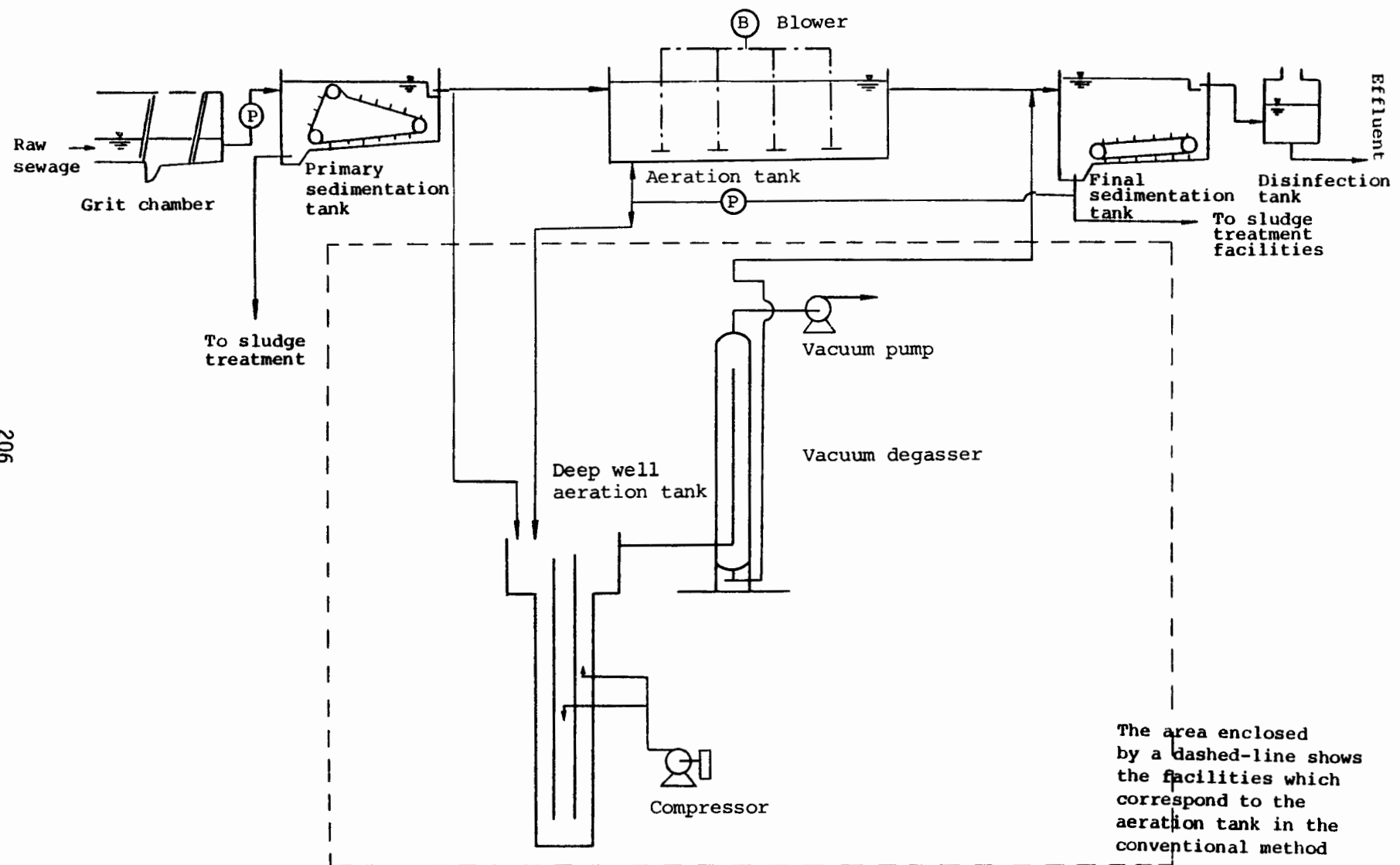


Fig. 2-a System diagram of Company I's process compared with the conventional process

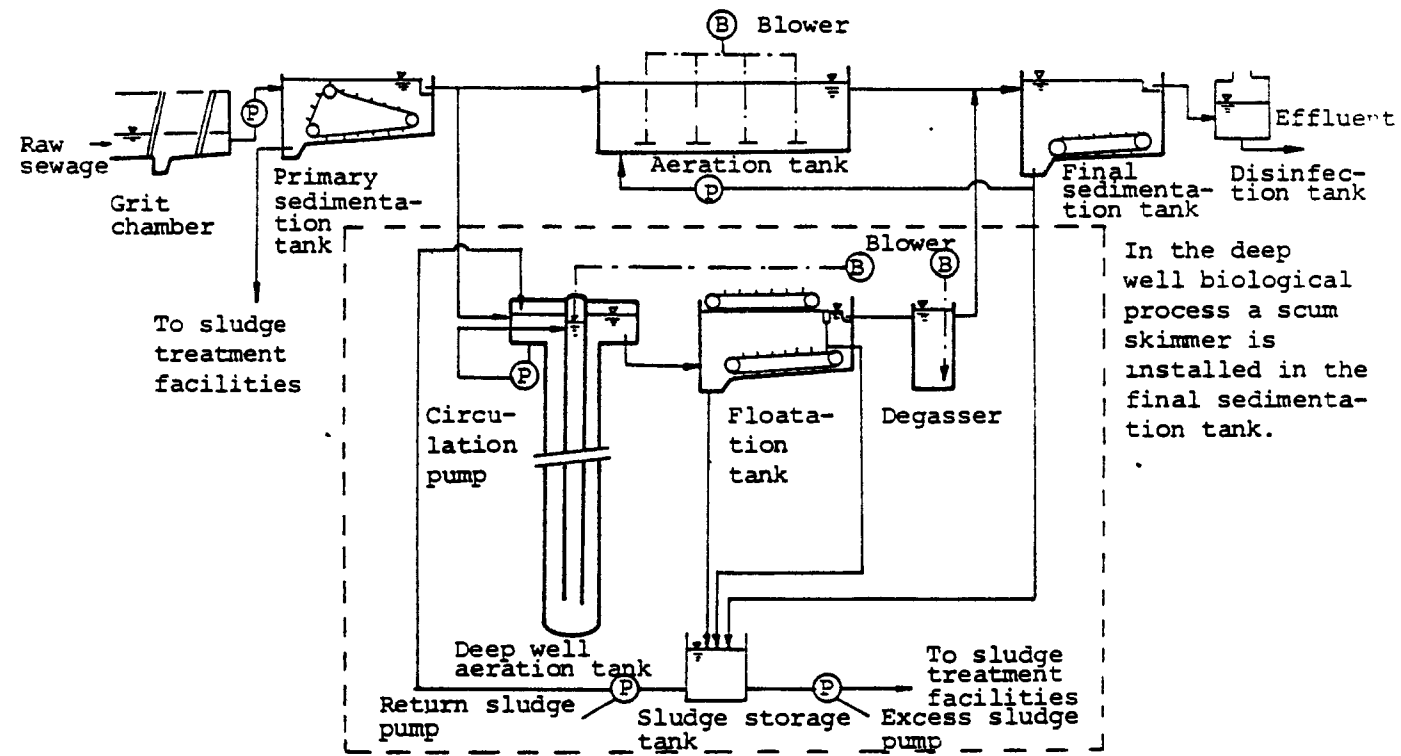


Fig. 2-b System diagram of Company K's process compared with the conventional process

2.2.2 Deep Well Aeration Tank

The aeration tank of this process consists of a deep well type aeration tank and a head tank; the deep well aeration tank being composed of a downflow tube and an upflow tube. (In the system of Company K, the tank corresponding to the head tank is called the "relay tank".)

In the system of Company I, the mixed liquor in the tank is circulated at a velocity of 0.7 - 1.5 m/sec by the air lift produced by injection of air into the upper portion of the upflow tube. The influent flows into the head tank where it is mixed with return sludge, and flows down the tube. Air is also injected into the downflow tube. The velocity of the downward flow is higher than that of rising bubbles. Bubbles are, therefore, carried downward with the downward flow, and their diameter is consequently reduced. When the bubbles reach the bottom of the tank, they move into the upflow tube and rise up the tube to the head tank, and conversely increase their diameter. As the injection of air into the downflow tube is made at 40 m or more below the surface, the specific gravity in the upper portion of the upflow tube becomes smaller than that in the upper portion of the downflow tube, generating the air-lift effect. In this way, the liquor is repeatedly circulated for biological treatment. The mixed liquor is then transferred to the solids separation system.

In the system of Company K, the deep well aeration tank is composed of a deep-well tank comprising a downflow tube and a upflow tube, and of a relay tank. The mixed liquor in the aeration tank is circulated by a recirculation pump. The influent which flows into the relay tank is mixed with return sludge and flow downs the tube at a velocity of 1 - 2 m/sec, and through the recirculating pump. In this system, air is injected into the downflow tube, and bubbles rise up to the relay tank through the upflow tube. The mixed liquor that has been treated bilogically is transferred to the solids separation system.

2.2.3 Solids Separation System

The mixed liquor transferred from the deep well aeration tank contains large quantities of supersaturated dissolved gas and fine bubbles; therefore, a part of the sludge comes to the surface of the liquor and a part of the sludge precipitates. The following combinations of methods are considered as viable solids separation systems.

- a) Deep well aeration tank -- Vacuum degasser -- Final sedimentation tank
- b) Deep well aeration tank -- Mechanical degasser -- Final sedimentation tank
- c) Deep well aeration tank -- Floatation tank -- Final sedimentation tank
- d) Deep well aeration tank -- Floatation tank -- Degasser -- Final sedimentation tank
- e) Deep well aeration tank -- Floatation tank

Company I employed combination(a). The solids separation equipment of Company I is shown in Fig. 3-a.

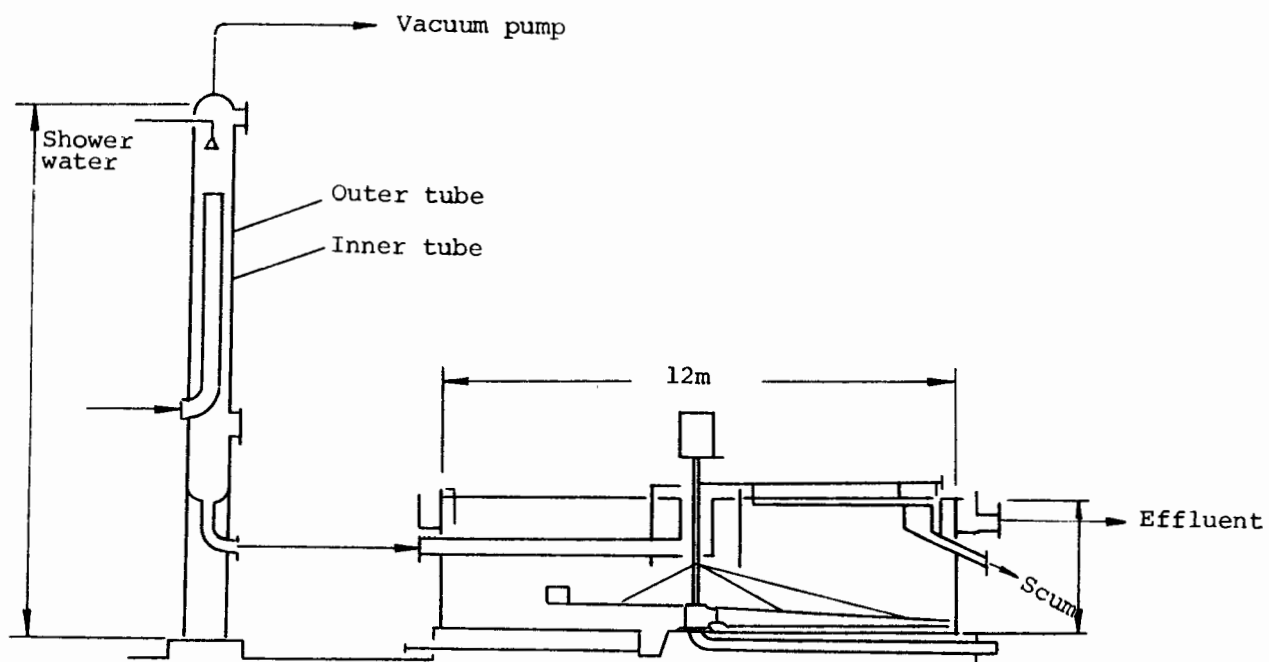


Fig. 3-a Solid separation facilities of Company I's process (vacuum degasser -- final sedimentation tank)

The vacuum degasser is composed of an upflow tube and a downflow tube, and the prescribed degree of vacuum (0.3 atm abs.) is applied by a vacuum pump. The fine bubbles in the mixed liquor are released from the liquor through the vacuum degasser. The mixed liquor is transferred from the deep well aeration tank to the final sedimentation tank through the vacuum degasser by siphonage due to the difference in the liquid levels between the head tank and the final sedimentation tank. The solid-liquid separation is conducted in the final sedimentation tank. A part of the separated sludge is returned to the deep well aeration tank.

Company K, on the other hand, employed combination(d). The solid-liquid separation system of K Company is shown in Fig. 3-b. The mixed liquor is sent to the floatation tank shown in Fig. 4 for solid-liquid separation.

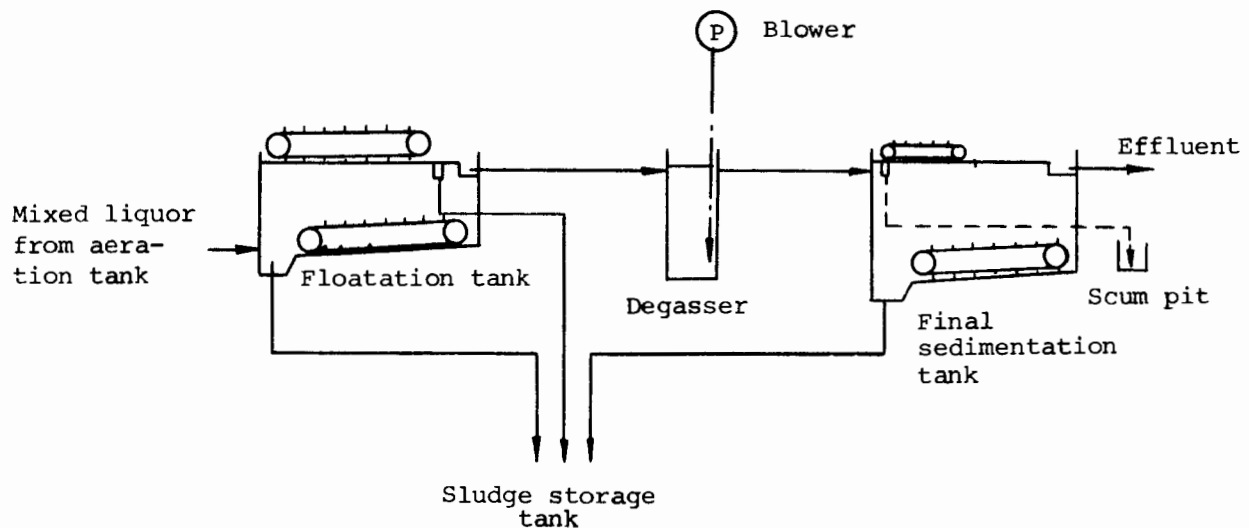


Fig. 3-b Solid separation facilities of Company K's process (floatation tank -- degasser -- final sedimentation tank)

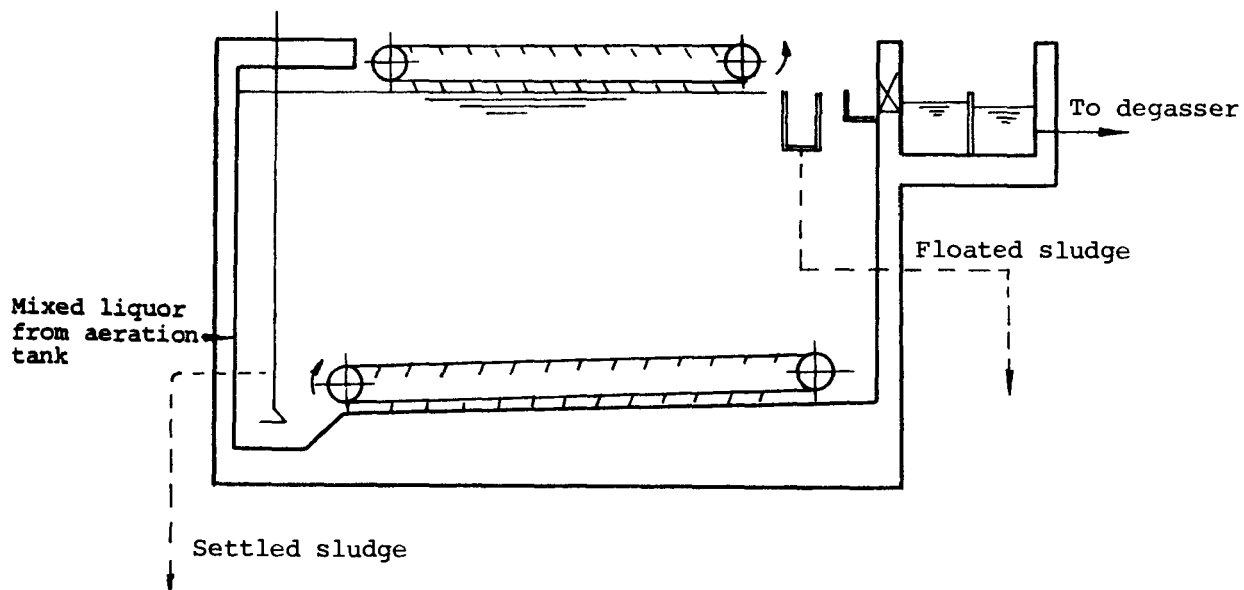


Fig. 4 Floatation tank of Company K's process

The floatation tank has sludge collectors which are operated continuously and the collected sludge is returned to the aeration tank. The solid concentration of the floated sludge is very high, which makes it possible to maintain high MLSS in the aeration tank.

In the effluent from the floatation tank, suspended solids having extremely fine bubbles on their surface still remain, and therefore the liquor is aerated and mixed in the degasser to separate the bubbles and make the sludge precipitates readily. In the degasser, a diffuser as shown in Fig. 5 is provided for diffused aeration.

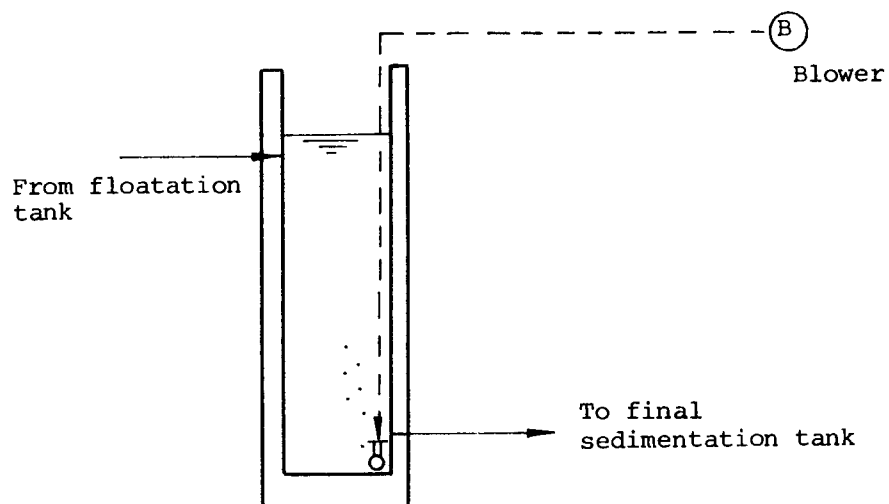


Fig. 5 Degasser

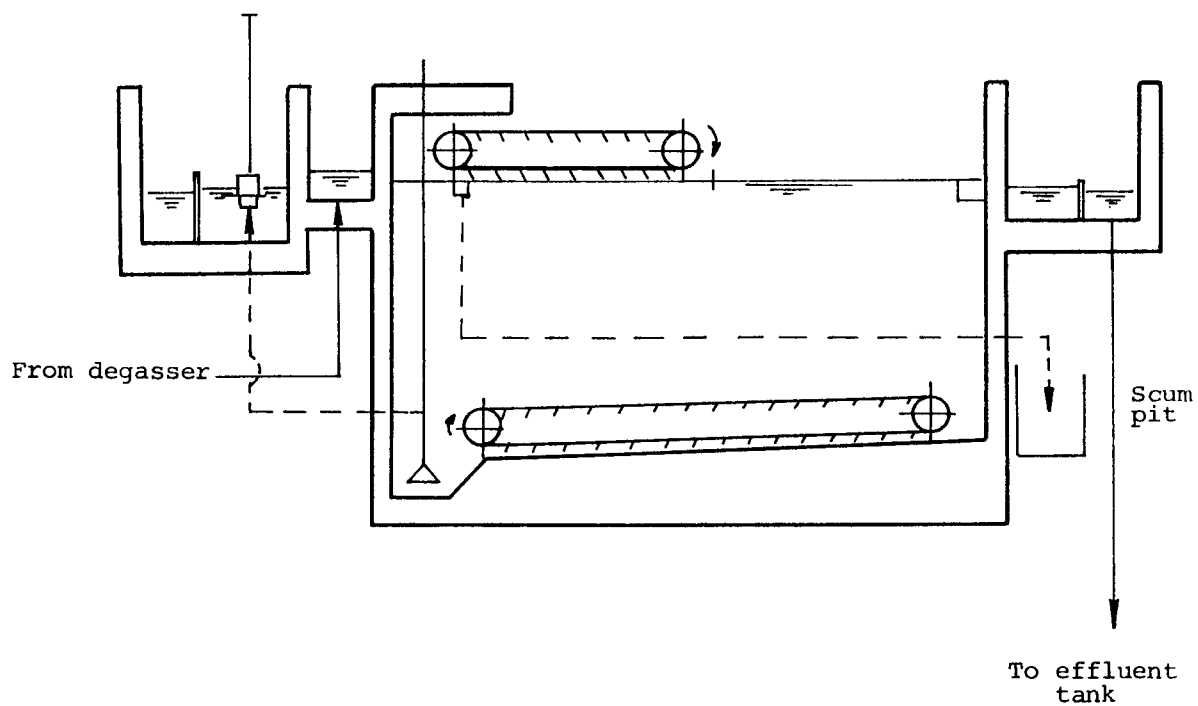


Fig. 6 Final sedimentation tank of Company K's process

After passing through the degasser, the liquor is transferred to the final sedimentation tank for the separation of the remaining sludge from the liquor. The final sedimentation tank is of the same type as that used in the conventional activated sludge process, but a scum collector as shown in Fig. 6 is provided for the removal of scum.

3. EVALUATION METHOD

3.1 PURPOSE OF DEVELOPMENT

The conventional activated sludge process has been commonly used for municipal wastewater treatment in Japan. This is because the conventional activated sludge process has a variety of advantages, although there is still room for improvements such as in the structure of the aeration tank, oxygen transfer rate, mixing method, and so forth.

On the other hand, there are many cases where the conditions of land utilization are too severe to obtain a large area for a sewage treatment plant or special efforts must be made to controlling detrimental effects on the surrounding environment.

It was considered that the deep well biological process was capable of overcoming such disadvantages, particularly in the reduction of the required site area, the ease of applying countermeasures to odor and other environmental problems and probably in reduction of the aeration cost.

On the other hand, difficulty in solid-liquid separation was expected, and in addition, it seemed necessary to further examine construction methods, safety, and durability of the facilities.

Therefore, the characteristic of this process have been investigated and further development of this technique is proposed.

3.2 GOALS TO BE ACHIEVED

The goals to be achieved by this process were set as follows:

Subject: The development of the deep well biological process for sewage treatment.

Goals to be Achieved

- (1) To have a treatment capability of 1000 m³/day or more.
- (2) To be able to meet legal requirements for the biochemical oxygen demand(BOD) and suspended solids(SS) of the secondary effluent.

- (3) To be simple in operation and maintenance.
- (4) To be low in operation and maintenance costs.
- (5) To have simplicity in taking countermeasures against detrimental effects on the surrounding environment.

3.3 SCOPE OF EVALUATION

The prerequisites for the sewage and sewage treatment facilities to be used for the evaluation of this technique are as follows:

- (1) The sewage used in the experiment shall be ordinary sewage composed of domestic wastewater and institutional wastewater (from offices, hospitals, and other business facilities), and it shall be municipal sewage that can be treated and biodegraded by the conventional activated sludge process.
- (2) The facilities in the treatment plant other than those within the scope of this evaluation shall be designed according to the "Sewerage Facilities Design Guide" and shall have the functions required by the legal regulations.
- (3) The deep well biological process facilities shall be constructed with sufficient safety and accuracy.

Under the above prerequisites, the evaluation was made within the range of the sewage treatment process from the inlet to the deep well aeration tank to the final sedimentation tank.

3.4 EXPERIMENTAL FACILITIES

The evaluation was made based on the data of the experiments carried out by the applicants using experimental facilities installed in municipal sewage treatment plants. When the data was not sufficient, the evaluation committee required the applicants to conduct specific additional experiments.

The dimensions of the main units of the experimental facilities are shown below. The major mechanical equipments are shown in Table 1.

Experimental facilities of Company I

Deep well aeration tank: Aeration tank with a diameter of 1 m, a depth of 100 m, and an effective capacity of 100 m^3 (including a head tank).

Vacuum degasser: Degassing tank with a diameter of 1 m, a height of 13.6 m (from the ground), and a capacity of 6 m^3 .

Final sedimentation tank: Circular sedimentation tank with a diameter of 12 m, and a depth of 3 m.

Experimental facilities of Company K

Deep well aeration tank: Aeration tank with a diameter of 0.8 m, a depth of 100 m, and a capacity of 57.5 m^3 (including a head tank).

Floatation tank: Floatation apparatus with a width of 4 m, a length of 7.5 m, a depth of 3.5 m, and an effective capacity of 84 m^3 .

Degasser: Rectangular tank with a width of 1.5 m, a depth of 4 m, and an effective capacity of 4 m^3 .

Final sedimentation tank: Rectangular sedimentation tank with a length of 10 m, a width of 4 m, a depth of 3 m and an effective capacity of 120 m^3 .

Table 1-a Specifications for major equipments of Company
I's facilities (design treatment capacity:
2400 m³/day)

No.	Equipments	Specification	No. of equipments installed
1	Influent pump	2.5m ³ /min × 12m × 11kW	2
2	Compressor	250Nm ³ /hr × 7kg/cm ² G × 35.9kW	1
3	Head tank	1.5 ^W _m × 6.0 ^L _m × 4 ^H _m	1
4	Deep well aeration tank	1.0 [∅] _m × 100 ^H _m	1
5	Vacuum degasser	1.0 [∅] _m × 13.6 ^H _m	1
6	Vacuum pump	0.49Nm ³ /min × 550mmHg × 2.2kW	2
7	Return sludge pump	2.5m ³ /min × 10m × 15kW	1
8	Sludge collector of final sedimentation tank	0.4kW	1
9	Final sedimentation tank	12 [∅] _m × 3 ^H _m	1
10	Scum tank	1.3 [∅] _m × 1.0 ^H _m	1
11	Scum pump	0.1m ³ /min × 9m × 0.75kW	1
12	Effluent tank	1.67 [∅] _m × 1.97 ^H _m	1
13	Effluent pump	2.5m ³ /min × 15m × 11kW	1

Table 1-b Specifications for major equipments of Company
K's process (design treatment capacity:
1000 m³/day)

No.	Equipment	Specification	No. of equipments installed
1	Deep well aeration tank	$0.8^{\phi}_m \times 100^H_m$	1
2	Relay tank	$2^W_m \times 2.5^L_m \times 3^H_m$	1
3	Floatation tank	$4^W_m \times 7.5^L_m \times 3.5^H_m$	1
4	Degasser	$1.5^W_m \times 1^L_m \times 4^H_m$	1
5	Final sedimentation tank	$4^W_m \times 10^L_m \times 3^H_m$	1
6	Sludge storage tank	$1^W_m \times 4^L_m \times 2.4^H_m$	1
7	Effluent tank	$2^{\phi}_m \times 2^H_m$	1
8	Influent pump	$1.5\text{m}^3/\text{min} \times 10\text{m} \times 7.5\text{kW}$	1
9	Recirculation pump	$8\text{m}^3/\text{min} \times 5.5\text{m} \times 15\text{kW}$	1
10	Sludge return pump	$1.5\text{m}^3/\text{min} \times 20\text{m} \times 11\text{kW}$	1
11	Effluent pump	$1.5\text{m}^3/\text{min} \times 5\text{m} \times 3.7\text{kW}$	1
12	Aeration blower	$1\text{Nm}^3/\text{min} \times 0.5\text{kg}/\text{cm}^2 \times 3.7\text{kW}$	1
13	Degasser blower	$1\text{m}^3/\text{min} \times 0.5\text{kg}/\text{cm}^2 \times 3.7\text{kW}$	1
14	Sludge collector in floatation tank	0.4kW	1
15	Sludge collector in final sedimentation tank	0.4kW	1

3.5 EXPERIMENTAL METHOD

3.5.1 Characteristics of Influent

The experimental facilities were built into the existing sewage treatment plants. The properties of the corresponding drainage areas were as follows:

Drainage area of the STP where the experimental facilities of Company I was installed.

Sewage drainage system: Separate system with partially combined system

Population: About 280,000

Average flow: 166,900 m³/day

Drainage area of the STP where the experimental facilities of Company K was installed.

Sewage drainage system: Separate system

Population: About 10,000

Average flow: 2,200 m³/day

The properties of influents are shown in Table 2.

Table 2-a Characteristics of influent of facilities of Company I

Item	Raw sewage	Influent to deep well aeration tank*
Characteristics of sewage	Most of the sewage is made up of domestic wastewater, but partially includes industrial wastewater. Further, the supernatant from the nightsoil treatment facilities is always discharged into the sewage.	
pH	7.1 ~ 7.4	7.7 (7.4 ~ 8.3)
SS (mg/l)	125 ~ 142	126 (67 ~ 292)
BOD (mg/l)	105 ~ 152	99 (51 ~ 189)
COD (mg/l)	95 ~ 127	69 (40 ~ 113)

*Effluent from primary sedimentation tank.

Table 2-b Characteristics of influent of facilities of Company K

Item	Raw sewage	Influent to deep well aeration tank*
Characteristics of sewage	Domestic sewage	
pH	7.6 (7.4 ~ 7.6)	7.4 (7.3 ~ 7.5)
SS (mg/l)	175 (163 ~ 218)	120 (106 ~ 146)
BOD (mg/l)	159 (123 ~ 179)	92 (76 ~ 102)
COD (mg/l)	107 (100 ~ 114)	64 (56 ~ 81)

*Effluent from primary sedimentation tank

3.5.2 Experimental Conditions

The experimental conditions are shown in Table 3, and the sampling stations for water quality analysis and parameters analyzed are shown in Table 4.

Table 3-a Experimental conditions for Company I's process

Exp. No.	Experimental period	Flow m ³ /day	Flow fluctuation m ³ /hr	Aeration time, hr	Air supply Nm ³ /hr		MLSS mg/l	Return sludge ratio %	BOD-SS loading kg/kg SS day	BOD volume loading kg/m ³ day
					upflow tube	downflow tube				
D-1	1979. 4/6 ~4/11	1320	55 constant	1.8	50	30	4380 (4000~4630)	55 constant	0.27 (0.21~0.33)	1.19 (0.94~1.89)
D-2	1979. 4/15 ~4/22	2400	100 constant	1.0	50	30	3560 (3310~3990)	60 constant	0.74 (0.62~0.91)	2.64 (2.20~3.19)
D-3	1979. 4/23 ~5/18	1840	30 ~ 100	1.0~3.3	50	30	3230 (2240~3940)	78 (60~200)	0.94 (0.69~1.28)	2.95 (2.47~4.44)
D-4	1979. 8/13 ~8/23	1840	30 ~ 100	1.0~3.3	50	30	3570 (2880~4260)	78 (60~200)	0.54 (0.34~0.89)	2.04 (1.37~2.81)
D-5	1979. 8/24 ~8/30	1520	30 ~ 80	1.3~3.3	50	30	3490 (3190~3750)	110 (88~230)	0.43 (0.30~0.48)	1.49 (1.06~1.75)
D-6	1979. 8/31 ~9/5	1840	30 ~ 100	1.0~3.3	50	30	4020 (3500~4400)	91 (70~230)	0.44 (0.29~0.55)	1.77 (1.22~2.18)
E-1	1979. 10/11 ~10/21	2400	100 constant	1.0	50	30	1990 (1670~2250)	15 constant	1.05 (0.67~1.47)	2.08 (1.34~2.74)
E-2	1979. 10/22 ~10/25	3000	125 constant	0.8	50	30	1790 (1480~1970)	12 constant	1.49 (1.26~1.91)	2.60 (2.43~2.82)
E-3	1979. 10/26 ~11/4	3600	150 constant	0.67	50	30	1760 (1580~1980)	10 constant	1.78 (1.51~2.09)	3.14 (2.38~3.85)
E-4	1979. 11/5 ~11/20	2400	50 ~ 150	0.67~2.0	50	30	1490 (1230~1810)	15 (10~30)	2.21 (1.64~3.13)	3.24 (2.48~3.85)
E-5	1979. 12/4 ~12/27	1840	30 ~ 100	1.0~3.3	50	30	2480 (1790~3430)	33 (25~83)	1.14 (0.44~1.89)	2.68 (1.42~3.60)
F-1	1980. 1/18 ~1/31	1840	30 ~ 100	1.0~3.3	50	30	2830 (2260~3170)	39 (30~100)	1.12 (0.83~1.42)	3.02 (2.23~3.60)
F-2	1980. 2/1 ~3/5	947~1940	30 ~ 145	0.69~3.3	50	30	2910 (1990~3620)	78 (34~167)	1.39 (0.71~2.06)	3.96 (2.44~5.90)
S*	1980. 9/26 ~10/5	2240	50 ~ 130	0.77~2.0	50	30	2260 (2030~2550)	21 (15~40)	0.98 (0.65~1.33)	2.19 (1.41~2.93)

*Performance confirmation exp.

Table 3-b Experimental conditions for Company K's process

Exp. No.	Experimental period	Flow m ³ /day	Flow fluctuation	Aeration time (hr)	Air supply Nm ³ /hr	MLSS (mg/l)	Return sludge ratio %	BOD-SS loading kg/kg SS day	BOD volume loading kg/m ³ day
Run I-1 *1	1980 3/1~3/21	1000	41.7 m ³ /hr constant	1.38	30	2820 1440~3890	65 61~95	0.41 0.29~0.87	1.2 0.76~1.6
2	5/10~ /30	600	25 m ³ /hr constant	2.3	18	3060 1810~3770	44 34~62	0.24 0.11~0.54	0.70 0.34~1.3
3	6/4~6/14	1000	41.7 m ³ /hr constant	1.38	18	3020 2120~4040	50 34~65	0.56 0.31~1.1	1.7 1.0~2.4
4	3/22~3/23	1000	12.4~81.3 m ³ /hr	1.38 0.71~4.64	30	3600	75~242	0.42	1.5
5	6/15~6/25	1000	13.5~88.1 m ³ /hr	1.38 0.65~4.26	18	3760 3100~3950	42 34~59	0.53 0.24~0.71	2.0 0.9~2.4
6	12/1~12/20	1060	24.0~58 m ³ /hr	1.30 0.99~2.40	12	3940 3600~4100	44 38~48	0.43 0.37~0.50	1.7 1.4~1.9
Run IV *2	10/11~10/23	1000	22.3~59.5 m ³ /hr	1.38 0.97~2.58	12	3240 1710~3970	50 47~54	0.45 0.35~0.83	1.45 1.3~1.6

*1: Applicant's experiment

*2: Performance confirmation experiment

Table 4-a Parameters analyzed in the experiment of Company I's process

		Influent		Effluent		Head tank	Effluent from vacuum degasser	Return sludge	Excess sludge	Frequency
Parameters		T	S	T	S					
Water quality	pH	0		0						Once/day
	SS	0		0						Once/day
	BOD	0	0	0	0					Once/day
	CODMn	0	0	0	0					Once/day
	CODcr	0	0	0	0					2-3 times/week
	TOC		0		0					Once/day
	Kj-N	0		0						Once/week
	NH ₄ ⁺ -N	0		0						Once/week
	NO ₂ ⁻ -N		0		0					Once/week
	NO ₃ ⁻ -N		0		0					Once/week
	T-P	0		0						Once/week
	PO ₄ ³⁻		0		0					*Once/day
Sludge	M-alkalinity	0		0						*Once/day
	Transparency	0		0						Once/day
	Temperature	0		0						Twice/day
	DO	0		0		0				Twice/day
	SS					0		0		Once/day
	VSS	0				0		0		Once/day
	SV						0			Once/day
	SVI						0			Once/day
	SSV						0			Once/day

T: Total

S: Soluble

*: Additional parameters on performance confirmation experiment

Table 4-b Parameters analyzed in the experiment of Company K's process

Parameters	Influent (effluent from primary sedimenta- tion tank)	Effluent	Mixed liquor in aeration tank	Return sludge	Excess sludge	Settled sludge withdrawn from floatation tank	Floated sludge in floatation tank	Sludge withdrawn from final sedimenta- tion	Frequency times/day
	Total Soluble	Total Soluble	Total						
Water quality	pH	0	0						3
	SS	0	0	0					1
	BOD	0 0	0 0						1
	CODMn	0 0	0 0						1
	TOC	0	0						1
	Kj-N	0	0						1
	NH ₄ ⁺ -N	0	0						1
	NO ₂ ⁻ -N	0	0						1
	NO ₃ ⁻ -N	0	0						1
	T-P	0	0						1
	PO ₄ ³ -P	0	0						1
	M-alkalinity	0	0						1
	Transparency		0						1
	Temperature	0	0	0					3
Sludge	DO		0	0					3
	SS				0	0	0	0	1
	VS				0	0			1

For the sampling of the influent, effluent, and return sludge, automatic samplers were used, and respective 24-hour samples were analyzed as the flow proportional composite samples. The analytical methods used were the glass-fiber filtration method specified in JIS K 0102 for SS, and other methods described in "Analytical Methods for Sewage Water Quality" compiled by the Japan Sewage Work Association.

The sewage flow, return sludge, and excess sludge withdrawal were measured by electromagnetic flowmeters.

As to the feed flow rate to the experimental facilities, two types of flow patterns were employed, that is, a fixed flow rate, and a fluctuating flow rate that was proportional to the influent rate of the corresponding treatment plant. This fluctuation pattern in flow rate is shown in Fig. 7.

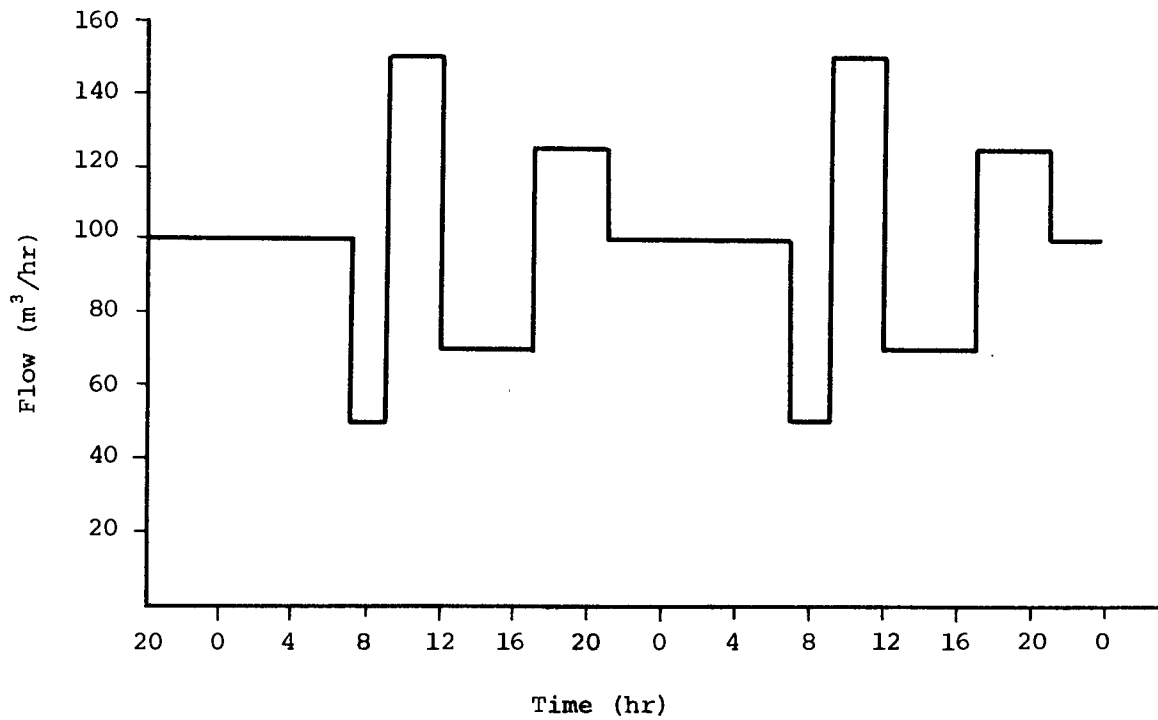
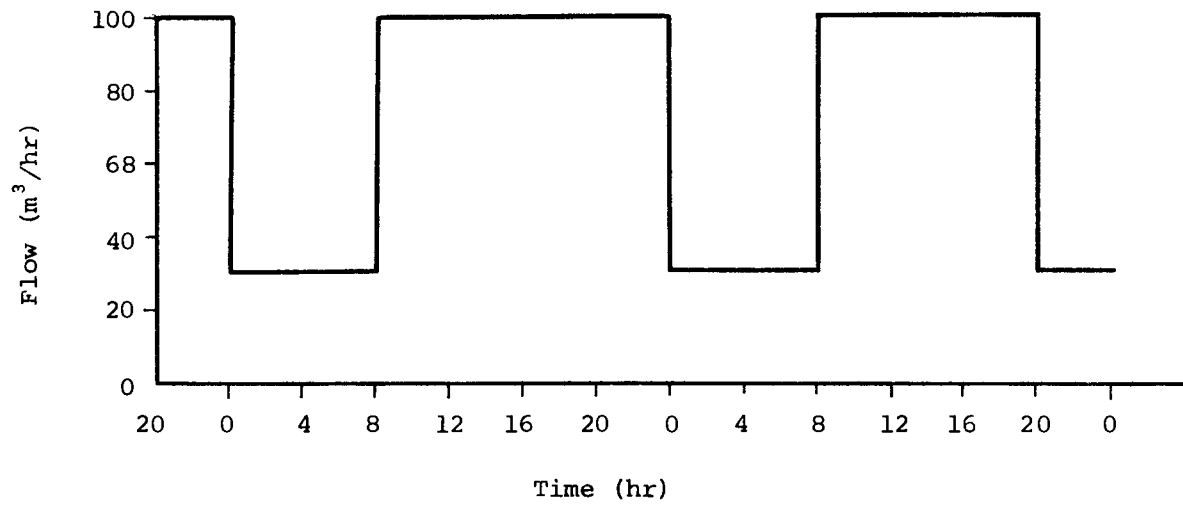


Fig. 7-a Flow fluctuation pattern (To be adjusted manually)
for Company I's process

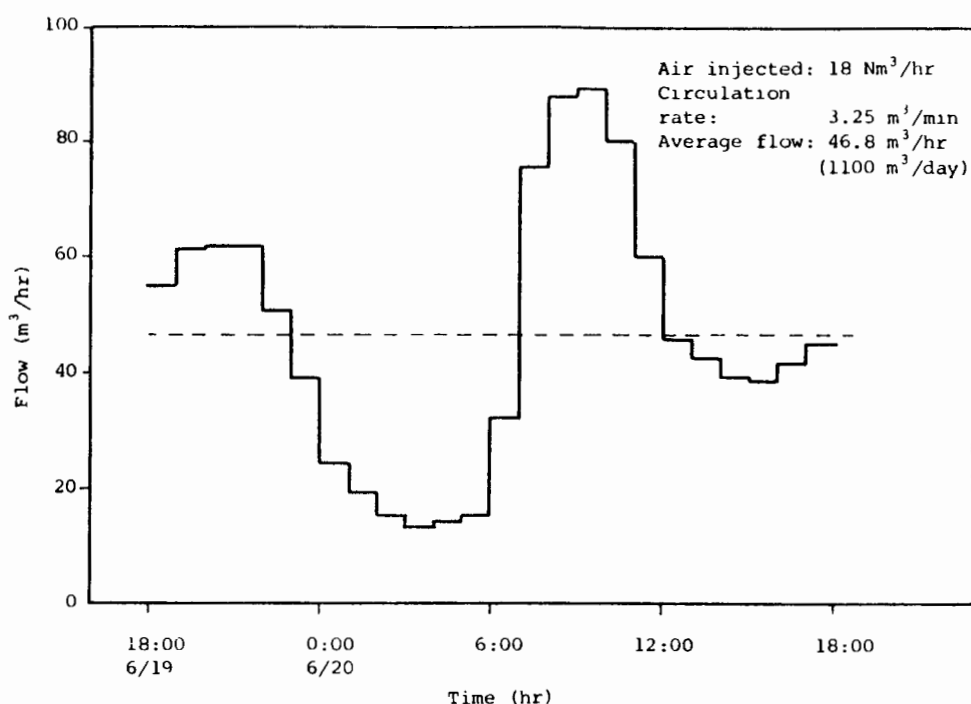


Fig. 7-b Flow fluctuation pattern for Company K's process

As for the experiment No. E-4 of Company I, the flow rate fluctuation pattern often found in a typical small scale treatment plant was employed as shown in Fig. 7-a.

The return sludge rate was determined so that the MLSS concentration in the head tank or relay tank reached the predetermined value.

The vacuum degasser was controlled so that the vacuum at the top of the tank was maintained at a level of 0.3 atm abs. For the recirculation of the mixed liquor with a pump, the circulation was conducted at a circulation rate of 3.7 m³/min and at a downflow velocity of 0.9 m/sec.

3.5.3 Performance Confirmation Experiment

The performance confirmation experiment for confirming practicability was conducted using the same experimental facilities as that in other experiments. The water quality of the influent during the performance confirmation experiment is shown in Table 5, and

the experimental conditions are shown in Table 6. Further, the fluctuation pattern of the flow rate is shown in Fig. 8.

Table 5-a Water quality of the influent to the aeration tank, Company I's process

Parameter	Influent water quality
pH	7.63 (7.51~7.71)
SS (mg/l)	112 (80~162)
BOD (mg/l)	97.9 (63~131)
COD (mg/l)	71.0 (55~88)

Table 5-b Water quality of the influent to the aeration tank, Company K's process

Parameter	Influent water quality
pH	7.1 (6.8~7.8)
SS (mg/l)	77 (50~114)
BOD (mg/l)	80.8 (72.1~90.1)
COD (mg/l)	43.0 (36.5~48.5)

Table 6-a Experimental conditions for Company I's process

Exp. No.	Experimental period	Flow m ³ /day	Flow fluctuation m ³ /hr	Aeration time (hr)	Air supply Nm ³ /hr		MLSS (mg/L)	Sludge return rate m ³ /hr	BOD-SS loading kg/kg SS day	BOD volume loading kg/m ³ day
					upflow	downflow				
S	1980. 9/26 ~10/5	2240	50~130	0.77~2.0	50	30	2260 (2030~2550)	20 21.4%	0.98 (0.65~1.33)	2.19 (1.41~2.93)

Table 6-b Experimental conditions for Company K's process

Exp. No.	Experimental period	Flow m ³ /day	Flow fluctuation m ³ /hr	Aeration time (hr)	Air supply Nm ³ /hr	MLSS (mg/L)	Return sludge rate m ³ /hr	BOD-SS loading kg/kg SS day	BOD volume loading kg/m ³ day
Performance confirmation experiment	1980 10/11~10/23	1000	22.3~59.5	1.3 0.97~2.58	12	3240 1710~3970	20.8 Return ratio 50%	0.45 0.35~0.83	1.45 1.3~1.6

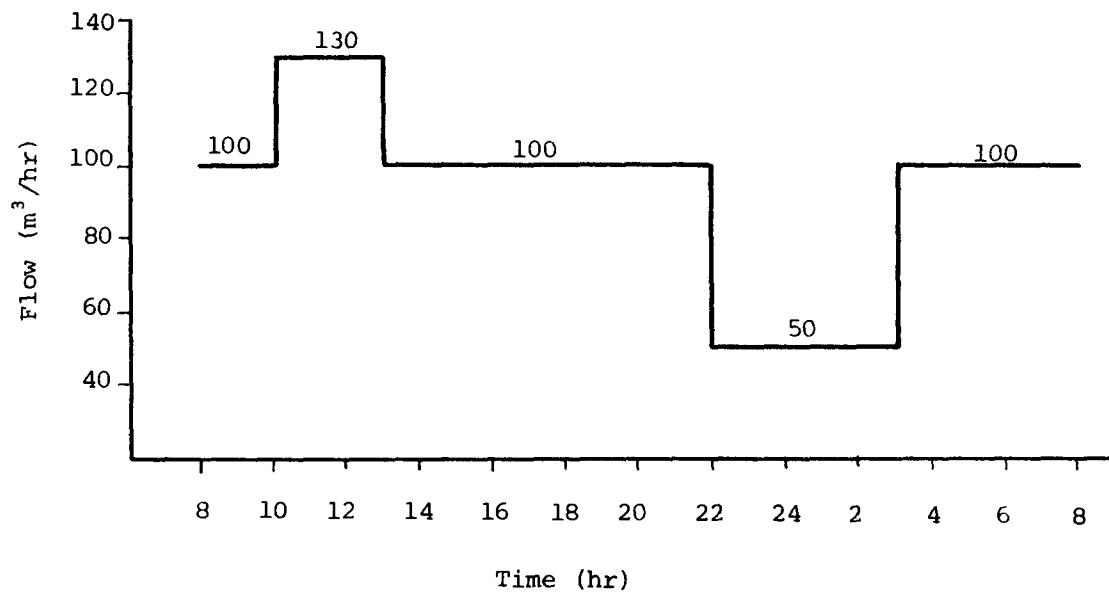


Fig. 8-a Flow fluctuation pattern for performance confirmation test of Company I

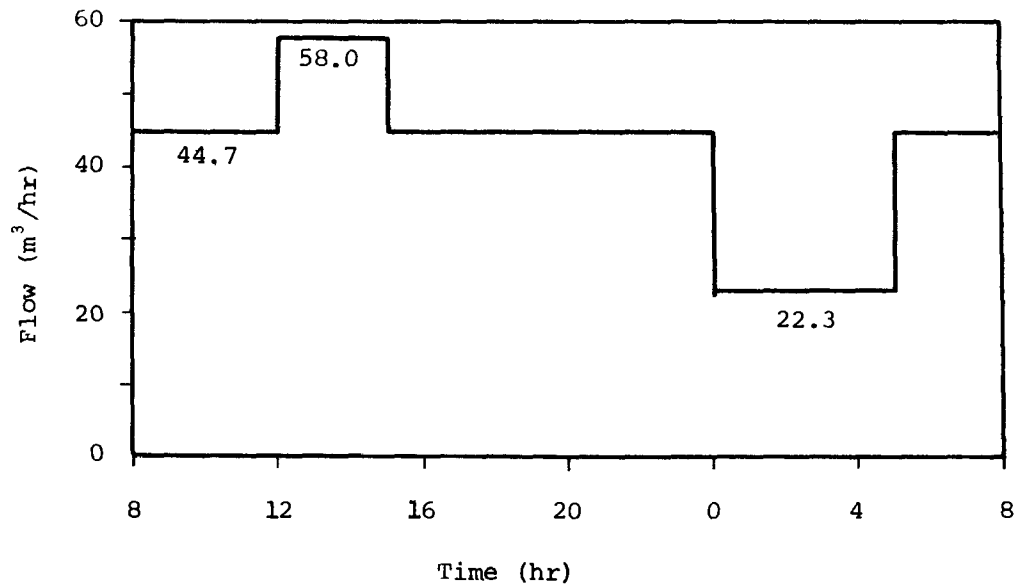


Fig. 8-b Flow fluctuation pattern for performance confirmation test of Company K

4. EVALUATION OF FACILITIES

4.1 OPERATING SIMPLICITY

This process is a modification of the conventional activated sludge process, and it is no more difficult to operate than the conventional method. The operation procedure and inspection method in this process is determined by each applicant.

4.2 DURABILITY

The material used for the deep well aeration tank is steel or reinforced concrete. The steel will cause no serious problem as long as the corrosion rate over its lifetime is taken into consideration in the design, and anti-corrosion paint is applied to the draft portion of the tank.

The reinforced concrete also causes no problem, as the conditions for the concrete are not much different from those for other similar structures.

Japan is subject to frequent earthquakes which often cause great damage to structures. Therefore, earthquake proof design for the deep well aeration tank was investigated using kinetic analysis. It was found that there would be no difficulty in designing earthquake proof structures for the deep well aeration tank.

4.3 SAFETY

The facilities except for the aeration tank and the vacuum degasser are the same as those used in the conventional method. It is possible to secure safety in operation of the aeration tank by taking preventive measures, such as installing fences around the tank to prevent people falling in. Safety of the degasser can be assured by designing the structure according to the regulations described in the "Labor Safety and Sanitation Law".

4.4 INFLUENCE ON SURROUNDING ENVIRONMENT

It is possible to control noise pollution due to the compressor, pump, and other apparatus by taking ordinary preventive measures.

Deodorization is not difficult as the area open to the air and the air volume in this process are less than those in the conventional method, and the odor concentration is of almost the same degree as in the conventional method. Prevention of aerosol dispersion is easy as the area open to the air is small.

4.5 CONSTRUCTION METHOD

Each applicant has established their own construction method. Although, previously there were no municipal wastewater treatment plants employing this process in Japan, several industrial wastewater treatment plants using this process have been already constructed. Judging from these experiences, there will be no problem in the construction of actual municipal wastewater treatment plants.

5. EVALUATION OF TREATMENT PERFORMANCE

5.1 DESIGN CAPACITY

The design flow of the experimental facilities of Company I is 2,400 m³/day. In the performance confirmation experiment conducted under the condition that the daily average flow was 2,240 m³/day (maximum hourly rate: 130 m³/hr, minimum hourly rate: 50 m³/hr), the effluent water quality met the legal standards in which BOD was 20 mg/l and SS was 70 mg/l or less. Accordingly, the goal of the effluent water quality was achieved.

The design flow of the experimental facilities of Company K is 1,000 m³/day. The performance confirmation experiment conducted at a flow rate of 1,000 m³/day (maximum hourly rate: 58 m³/day, minimum hourly rate: 22.3 m³/day) showed that the effluent quality could meet the standards. Therefore, it can be said that the goal for the effluent water quality was accomplished by plants with capacities set as a goal.

5.2 AERATION PERFORMANCE

Fig. 9 shows the vertical distribution of dissolved oxygen (DO) in the downflow tube of the aeration tank when clear water is used.

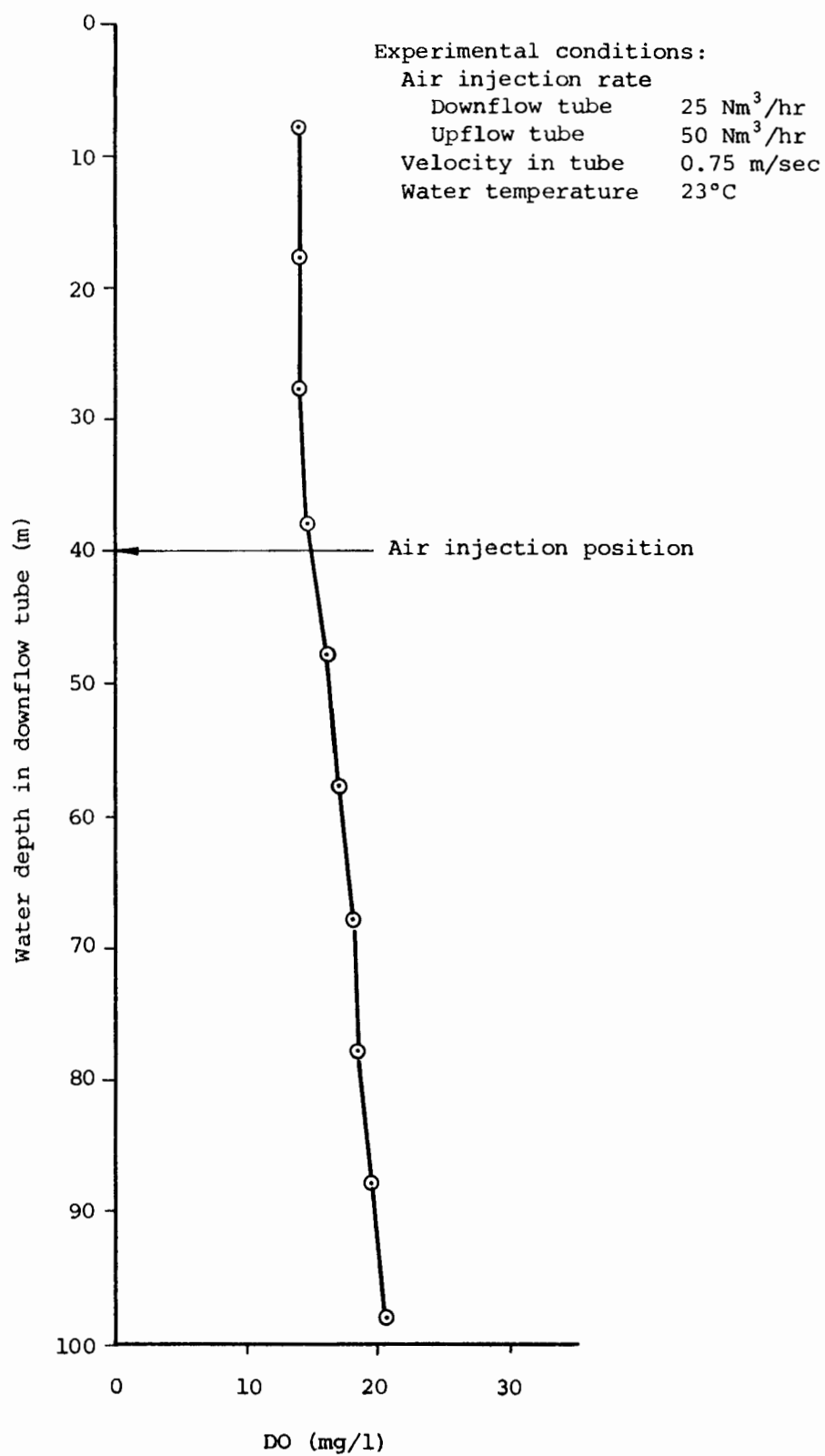


Fig. 9-a DO distribution in deep well aeration tank of Company I
(clean water experiment)

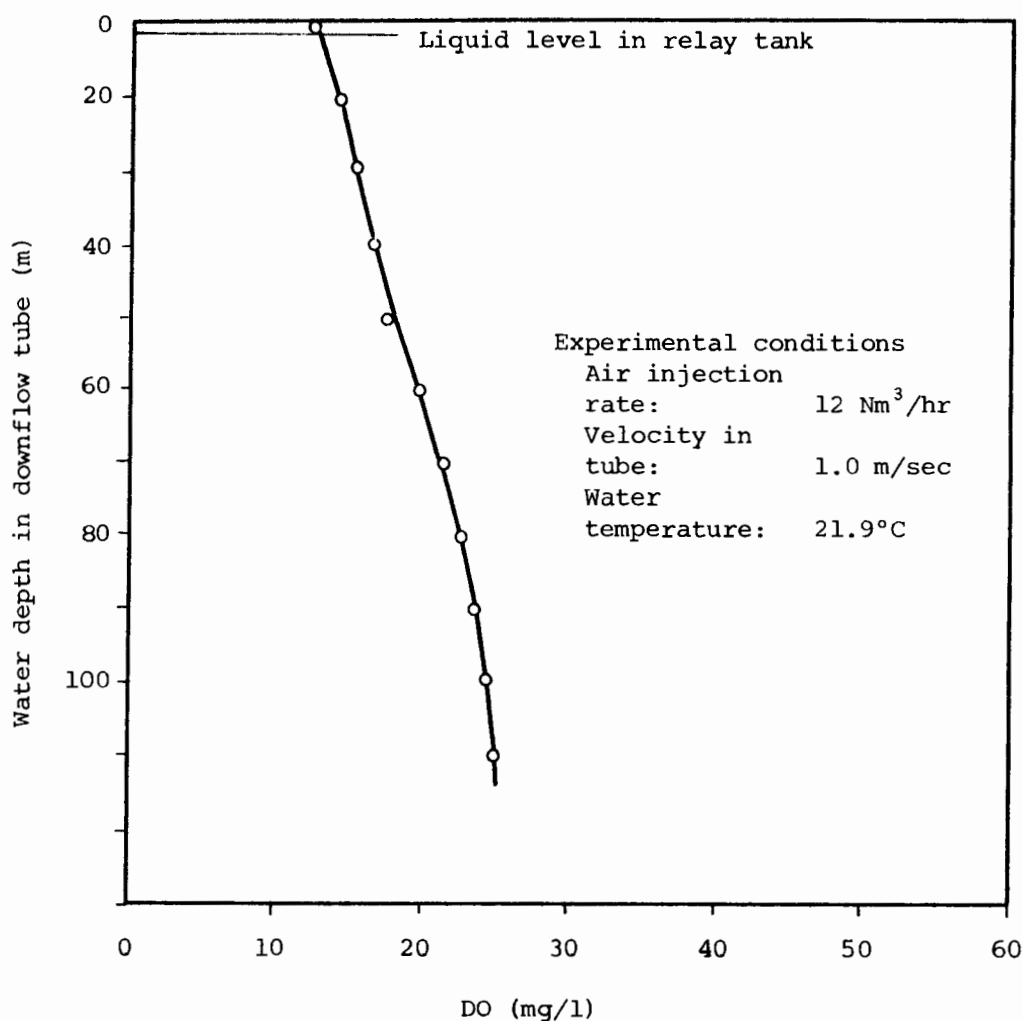


Fig. 9-b DO distribution in deep well aeration tank of Company K (clean water experiment)

According to the distribution of DO, the oxygen transfer efficiency is 71% for Company I and 79% for Company K. As to the air volume for aeration, in Company I's experimental plant, the air was injected into the downflow tube at 30 Nm³/hr, and the ratio of removed BOD to oxygen by this air was 1:0.8. Additional air was injected into the upflow tube at 50 Nm³/hr in order to maintain stable recirculation of the liquor. Fig. 10-a shows the time variations of DO in the head tank, and BOD and SS of the effluent during the performance confirmation experiment. During 24 hours, DO in the head tank was 4 mg/l or more, which means sufficient aeration was accomplished.

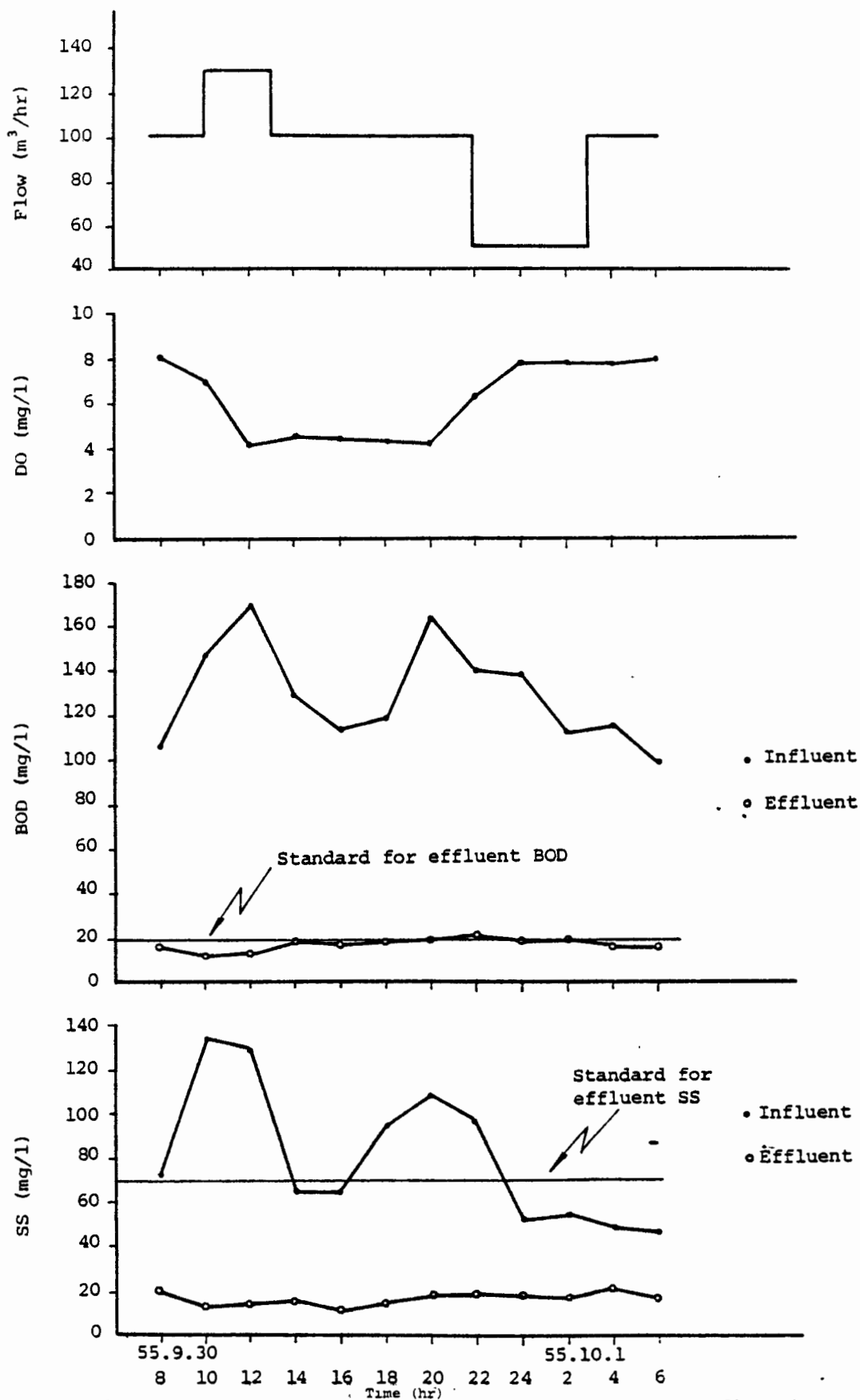


Fig. 10-a Variation of water quality (Performance confirmation experiment of Company I's process)

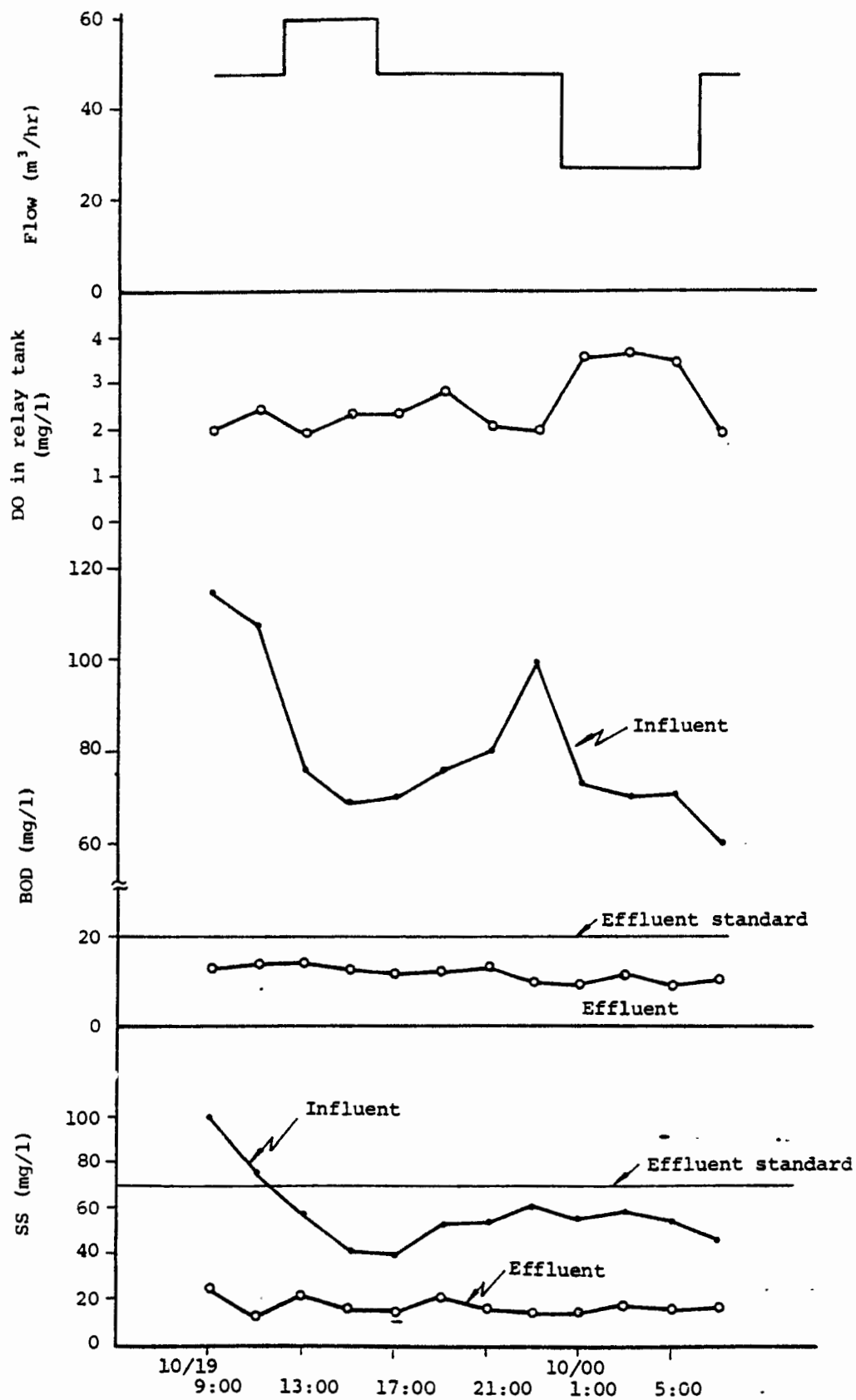


Fig. 10-b Variation of water quality (Performance confirmation experiment of Company K's process)

In the performance confirmation experiment of Company K, the air supply was kept at 12 Nm³/hr based on the results of the experiments conducted by the applicant. The time variations of similar parameters are shown in Fig. 10-b. During 24 hours, DO in the relay tank was 2 mg/l or more, and the aeration was sufficient for the operation.

The vertical profile of DO in the downflow tube is shown in Fig. 11. From this DO distribution, it can be seen that sufficient DO concentration was maintained and an aerobic condition was provided throughout the tank.

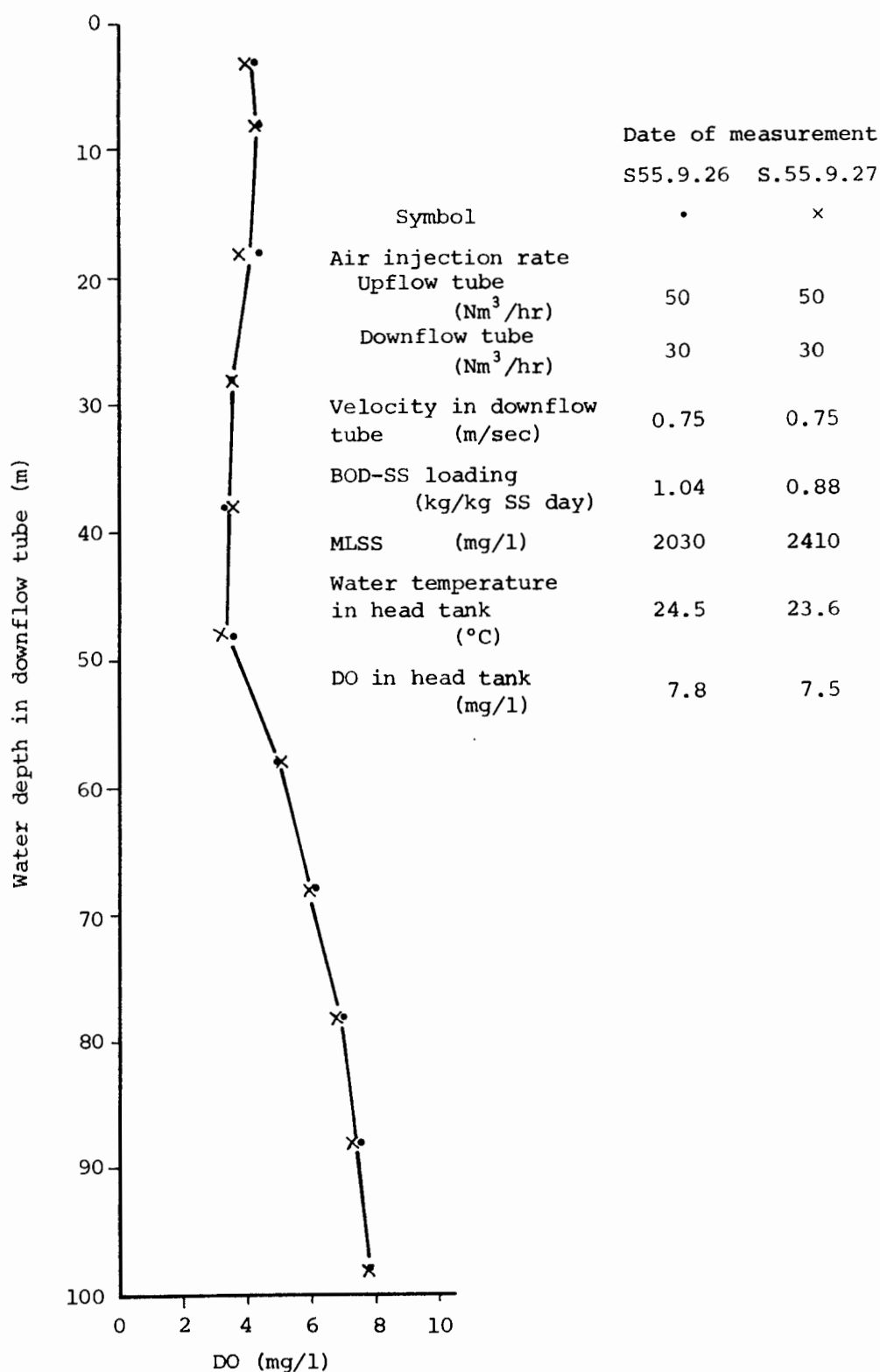


Fig. 11-a DO distribution in deep well aeration tank of Company I (regular operation)

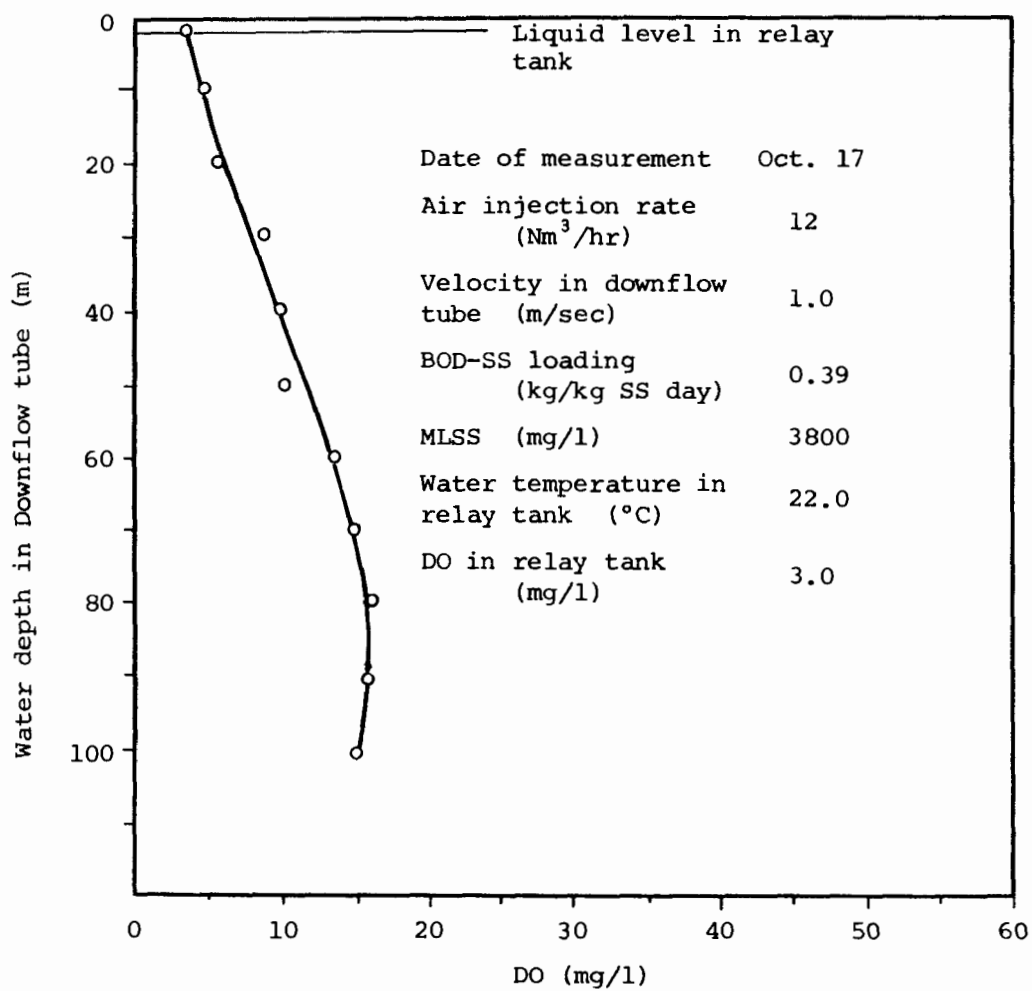


Fig. 11-b DO distribution in deep well aeration tank of Company K (regular operation)

As for the microorganisms in the activated sludge of the deep well biological process, Ciliata such as Vorticella, Opercularia and Aspidisca were predominant, and higher organisms such as Rotaria and Phabdolaimus were also present. The biota in the deep well biological process does not differ greatly from that in the conventional method.

5.3 WATER-QUALITY IMPROVEMENT PERFORMANCE

5.3.1 BOD and SS

(1) Effect of BOD Loading

The relationship between BOD volume loading and effluent BOD is shown in Fig. 12, and the relationship between BOD volume loading and BOD removal is shown in Fig. 13.

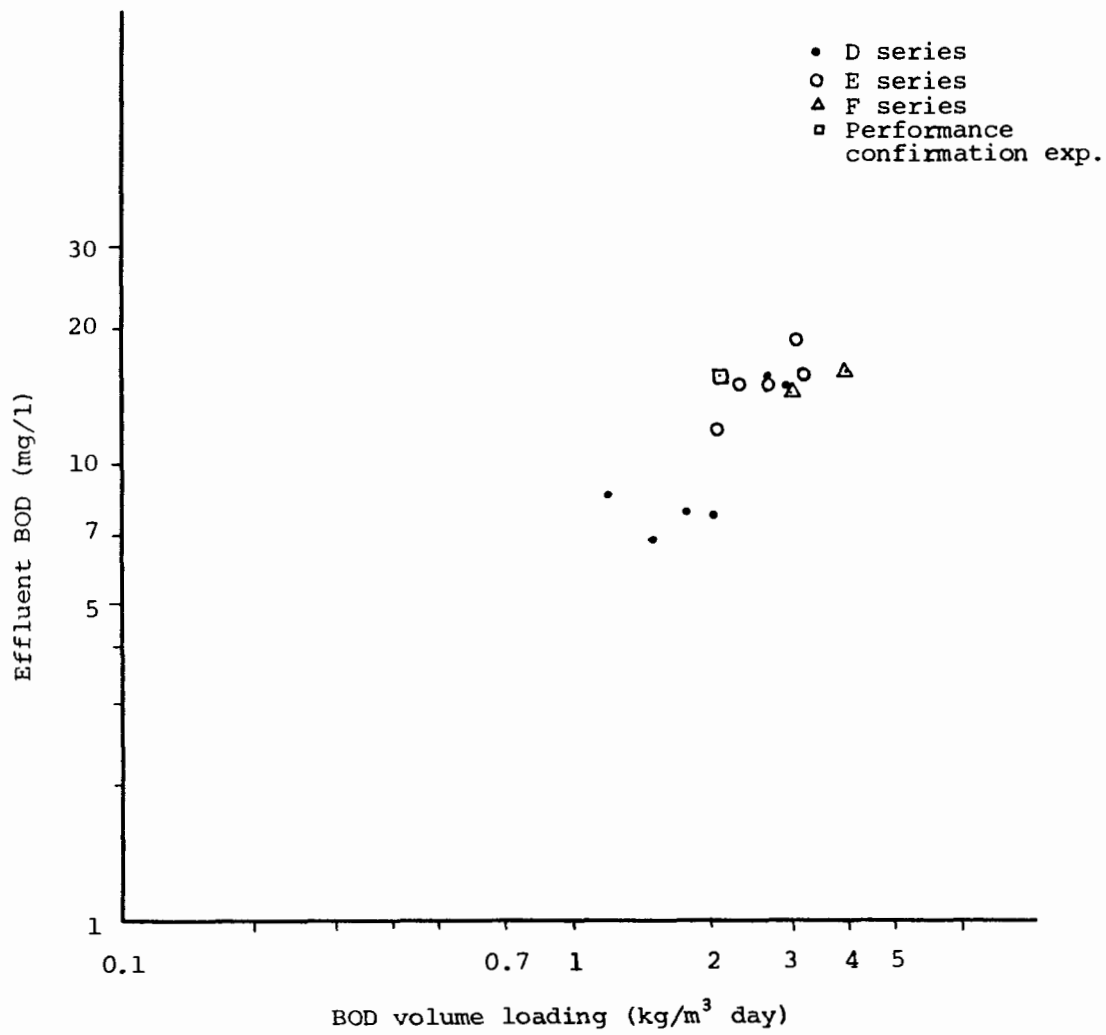


Fig. 12-a BOD volume loading and effluent BOD in Company I's process

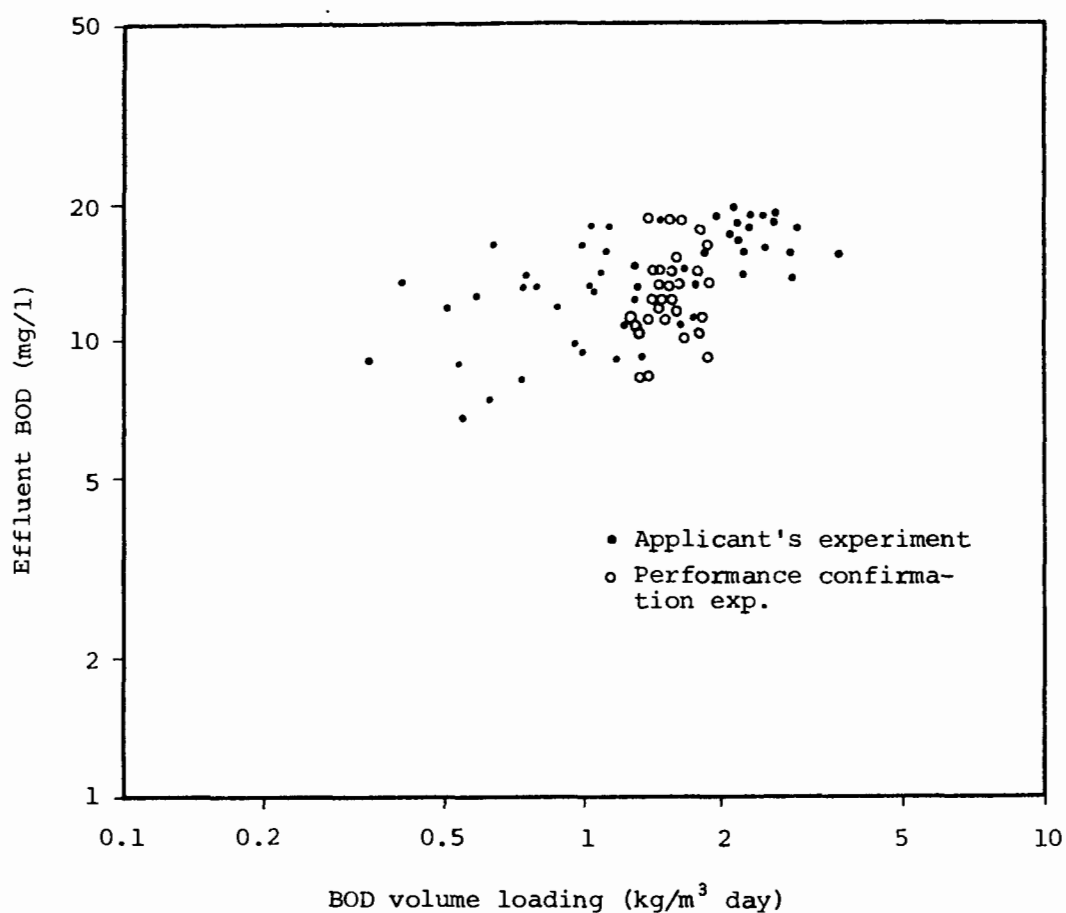


Fig. 12-b BOD volume loading and effluent BOD in Company K's process

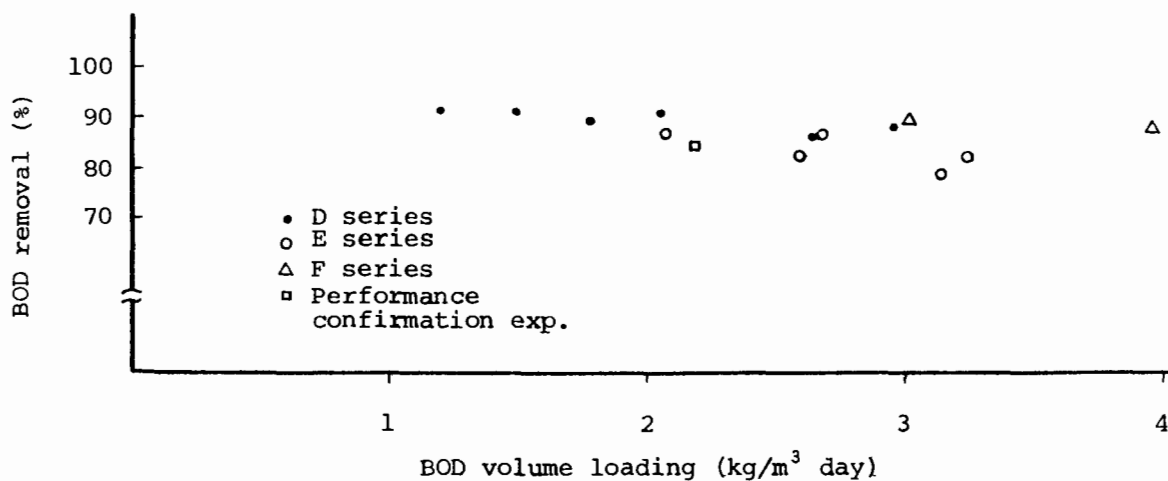


Fig. 13-a BOD volume loading and BOD removal in Company I's process

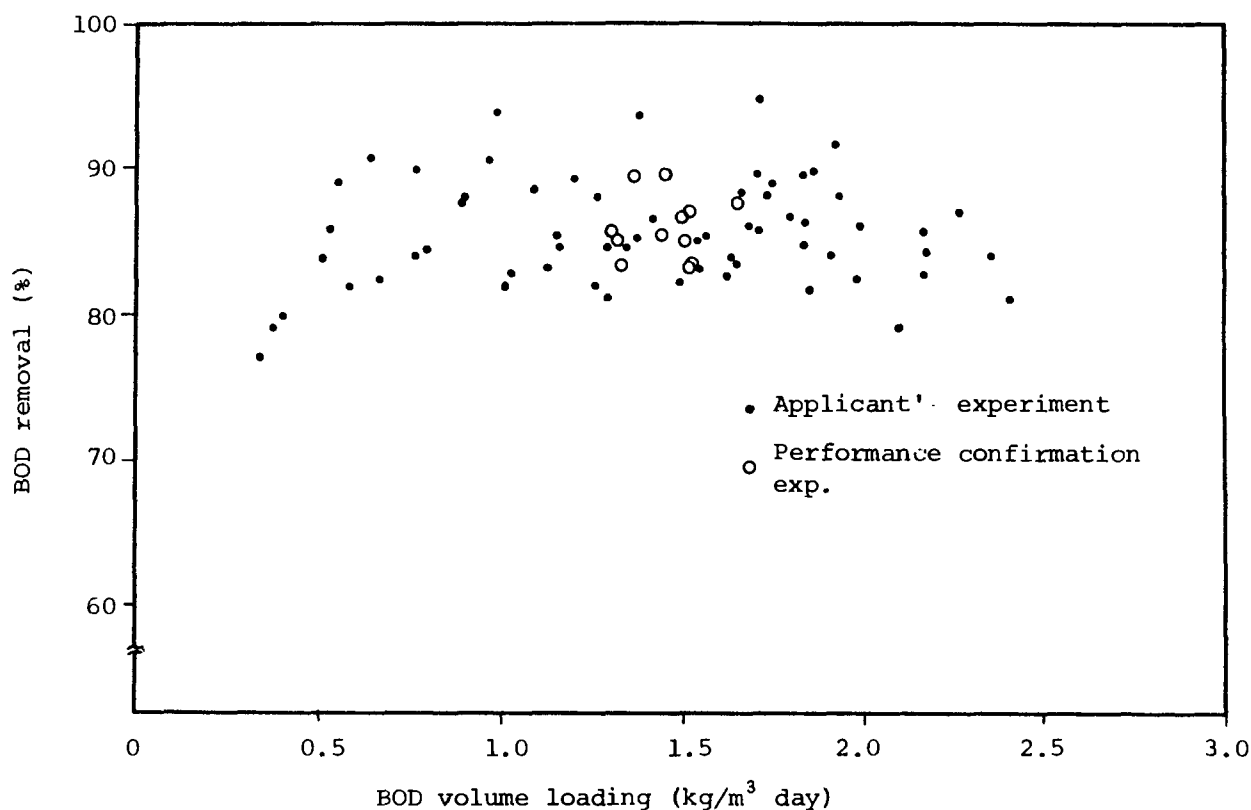


Fig. 13-b BOD volume loading and BOD removal in Company K's process

In this experiment, BOD volume loading was set at 1.3 - 1.6 kg/m³ day and 1.2 - 4.0 kg/m³ day for the facilities of Companies K and I, respectively. BOD-SS loading was set at 0.4 - 0.8 kg/kg SS day for K plant and at 0.3 - 2.2 kg/kg SS day for I plant. In these ranges of loading, both facilities could meet the goal of the development on effluent quality. Therefore, in the deep well biological process, as compared with the conventional method (BOD volume loading: 0.3 - 0.8 kg/m³ day), high BOD volume loading and almost the same or more BOD-SS loading than that in the conventional method (BOD-SS loading: 0.2 - 0.4 kg/kg SS day) can be applied to the plant. The relationship between BOD-SS loading and effluent BOD is shown in Fig. 14.

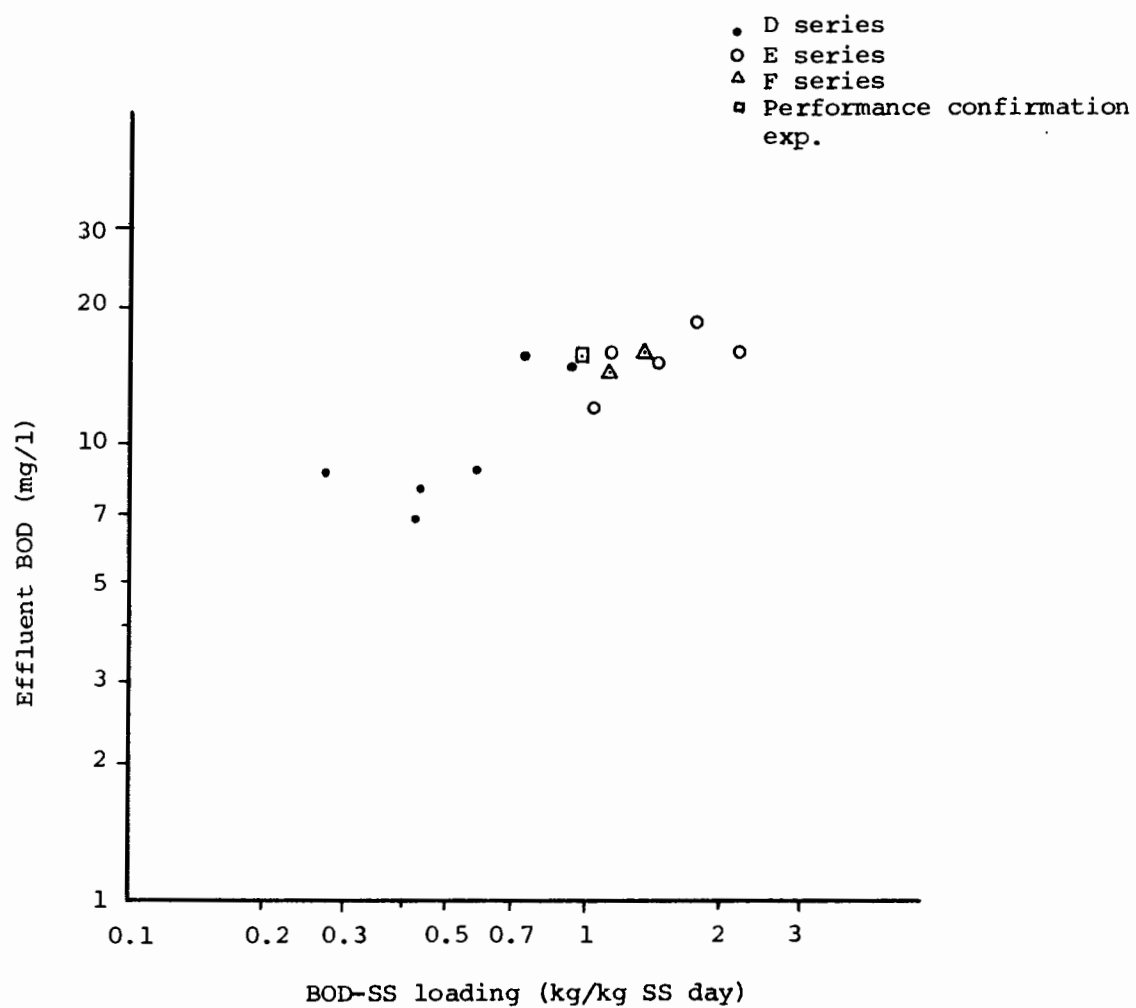


Fig. 14-a BOD-SS loading and effluent BOD in Company I's process

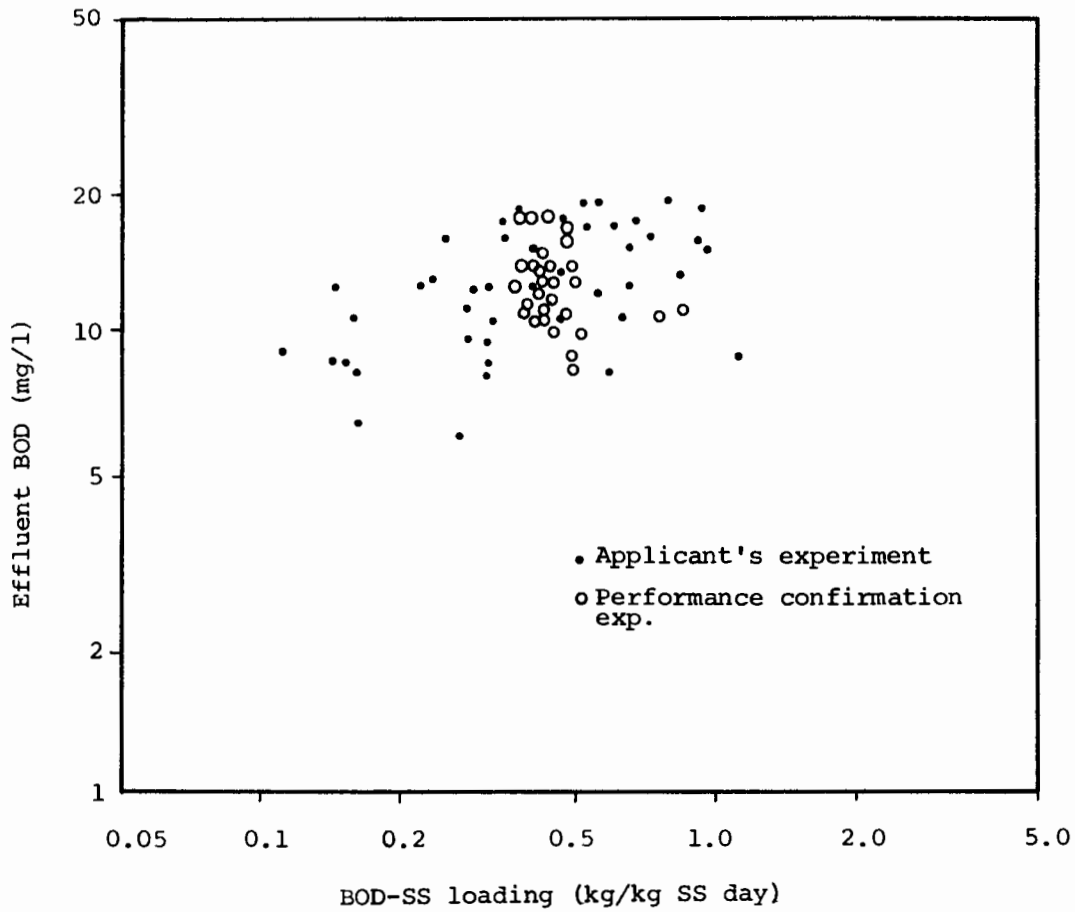


Fig. 14-b BOD-SS loading and effluent BOD in Company K's process

(2) Removal of SS

Table 7 shows the overflow rate, weir loading, and effluent SS in this experiment. In any case, effluent SS was within the standard, and there was no problem in applying overflow rate and weir loading equivalent to those for a ordinary secondary sedimentation tank.

Table 7-a Overflow rate and weir loading VS effluent SS, Company I's process

Exp. No.	Overflow rate (m ³ /m ² day)		Weir loading (m ³ /m day)		Effluent SS (mg/l)	
	Average	Range	Average	Maximum	Average	Range
D-1	13.2	Constant	37.5 (Constant)	-	24.3	20~28
2	24.0	Constant	68.2 (Constant)	-	28.0	19~36
3	18.4	7.2~24.0	52.3	68.2	23.7	7~37
4	18.4	7.2~24.0	52.3	68.2	15.9	10~23
5	15.2	7.2~19.2	43.2	54.5	12.1	10~16
6	18.4	7.2~19.2	52.3	68.2	16.0	14~18
E-1	24.0	Constant	68.2 (Constant)	-	9.0	6~19
2	30.0	Constant	85.2 (Constant)	-	12.0	10~17
3	36.0	Constant	102 (Constant)	-	17.0	11~24
4	24.0	12.0~36.0	68.2	102	16.9	9~23
5	18.4	7.2~24.0	52.3	68.2	20.0	13~28
F-1	18.4	7.2~24.0	52.3	68.2	15.4	10~25
2	16.0	7.2~34.8	45.5	98.9	16.4	10~23
S	22.4	12.0~31.2	63.6	88.6	15.2	11~21

Table 7-b Overflow rate VS effluent SS, Company K's process

Run No.		Run II-1	Run II-2	Run II-3	Run II-5	Run II-6	Performance confirmation experiment
Overflow rate (m ³ /m ² day)	Average	25	15	25	25	26.5	25
	Peak	27	18	30	53	35	35
Weir loading (m ³ /m day)	Average	250	150	250	250	106	100
	Peak	270	180	300	530	139	140
Effluent SS (mg/l)		34 (24~45)	25 (16~34)	30 (20~40)	33 (23~43)	18 (12~26)	18 (11~25)

(3) Stability in Treatment

The daily variations in BOD and SS are shown in Fig. 15. The flow rate, BOD and SS of the feed fluctuated considerably, but the effluent quality remained fairly stable.

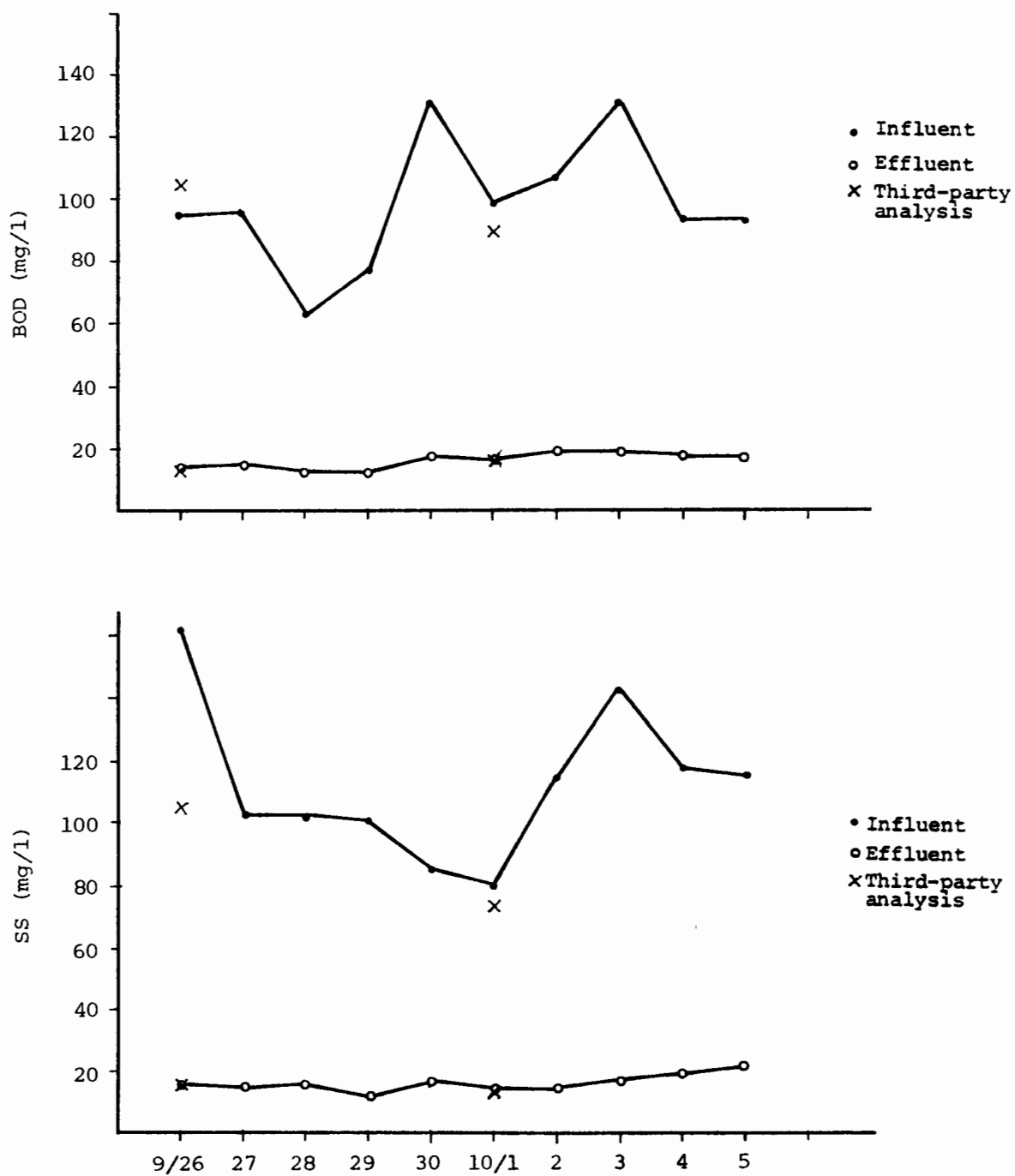


Fig. 15-a Daily variation in BOD and SS (performance confirmation experiment of Company I's process)

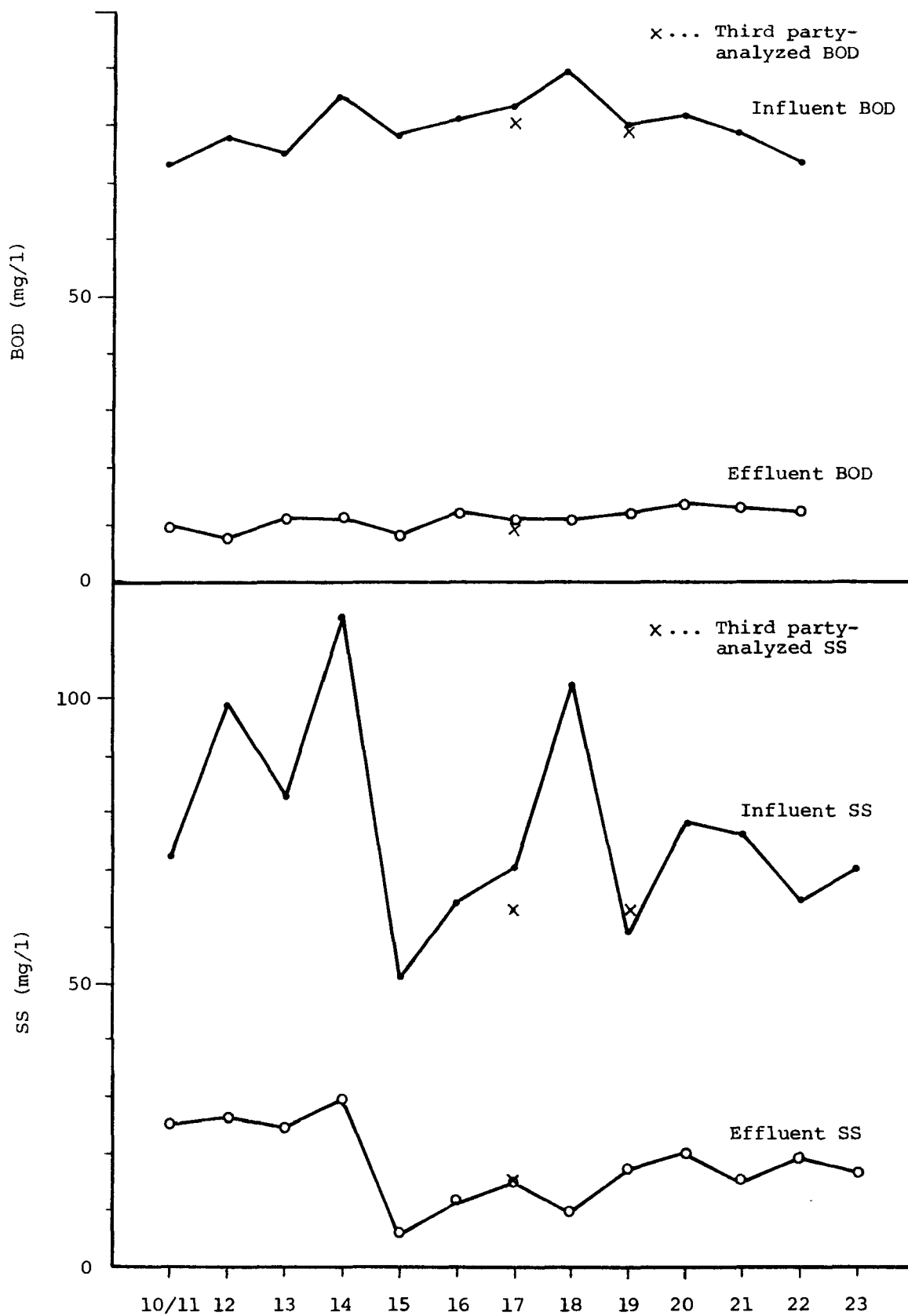


Fig. 15-b Daily variation data (performance confirmation
experiment of Company K's process)
250

Fig. 16 shows the water quality data in winter when the performance of the activated sludge process is liable to be lowerd. Although the loading varied considerably, the effluent quality also remained relatively stable.

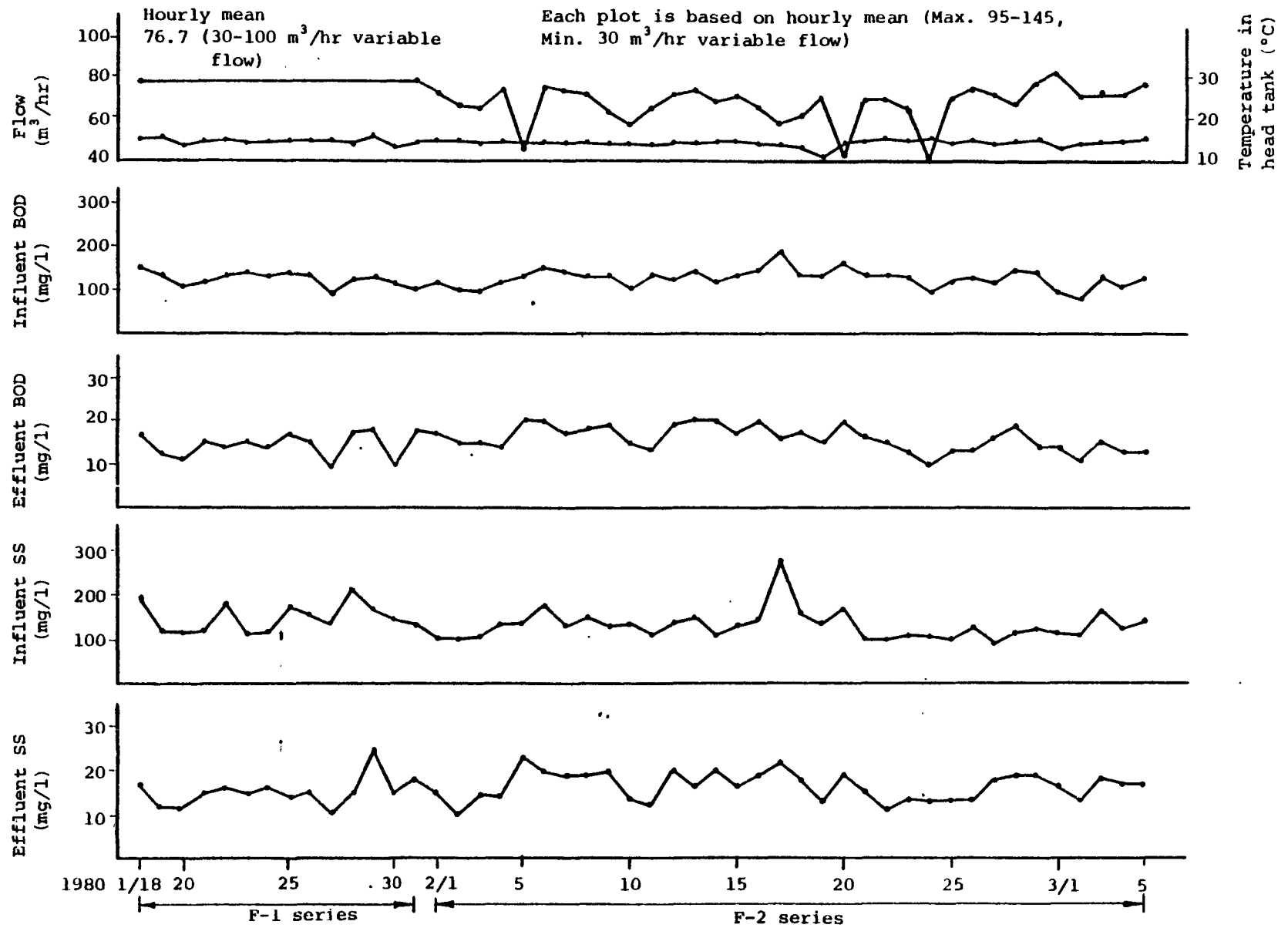


Fig. 16-a Winter experimental data of Company I's process

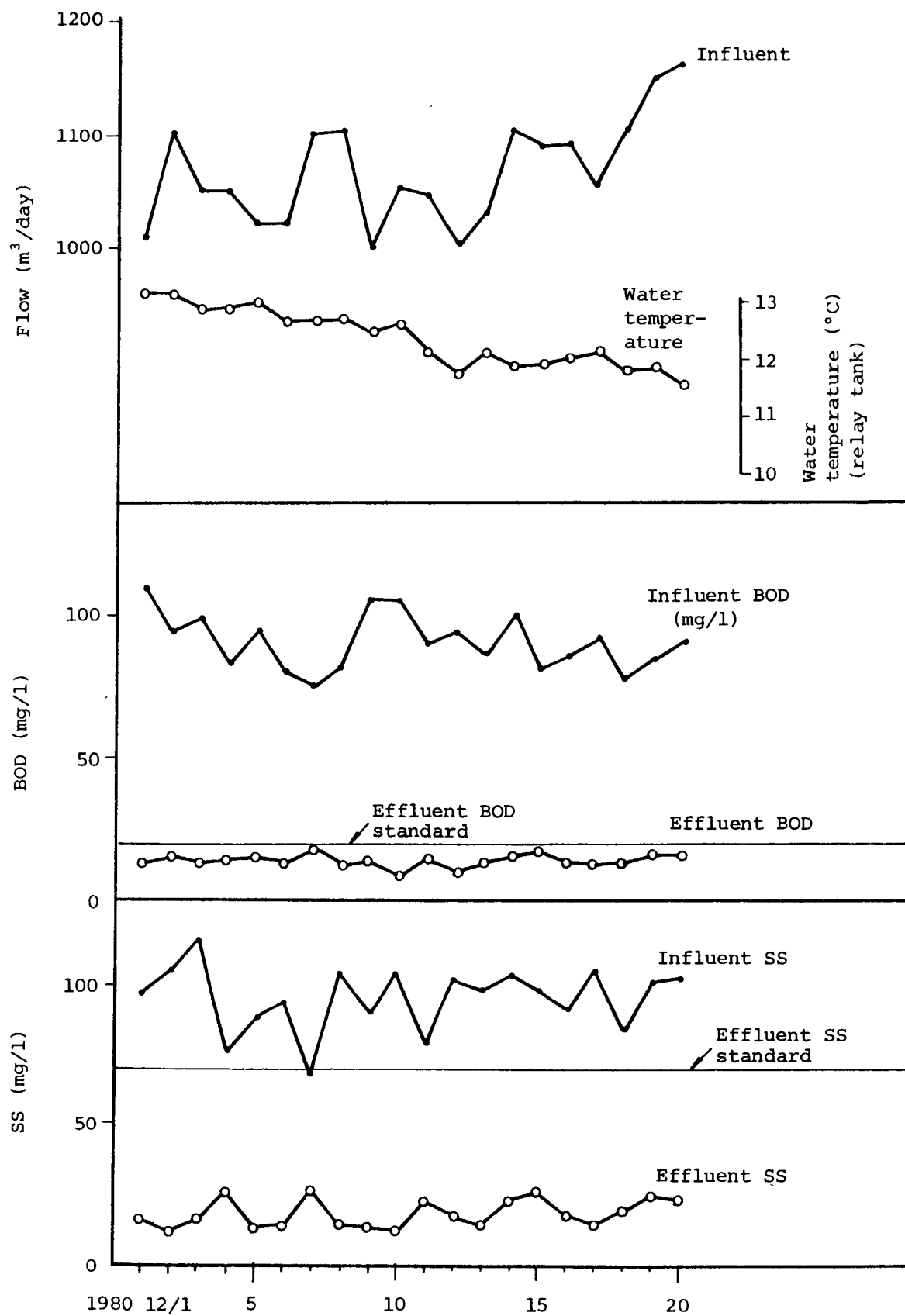


Fig. 16-b Winter experimental data of Company K's process

5.3.2 Transparency of Effluent

The transparency of the effluent during the experiment is shown in Table 8. It was rather lower than the transparency of the effluent in the conventional activated sludge process.

Table 8-a Transparency of effluent, Company I's process

Exp. No.	Effluent transparency (cm)
D-1	24.5 (21.6~27.0)
2	18.9 (15.0~26.0)
3	21.4 (14.5~39.0)
4	28.4 (19.5~40.5)
5	34.1 (26.0~44.0)
6	25.1 (20.0~27.0)
E-1	38.9 (26.0~50.0)
2	35.3 (28.0~41.5)
3	25.8 (16.0~41.0)
4	23.3 (17.0~35.0)
5	23.3 (16.0~33.0)
F-1	23.5 (19.5~32.0)
2	23.4 (17.5~29.3)
S	25.4 (22.8~28.2)

Table 8-b Transparency of effluent, Company K's process

Run No.	II-1	II-2	II-3	II-5	II-6	Performance confirmation exp.
Transparency (cm)	20 (19~21)	22 (20~24)	21 (19~22)	20 (18~21)	23 (20~26)	24 (23~26)

5.3.3 Others

The results of the analysis for nitrogen, phosphate, and COD (Mn method) in the influent and effluent during the experiment are shown in Table 9.

Table 9-a Water quality of influent and effluent, Company I's process

		Influent	Effluent
D series	Water temperature	22.2 (19.7~27.7)	22.5 (17.4~27.8)
	COD _{Mn}	69.3 (57.3~83.3)	24.2 (19.8~29.4)
	Kj-N	43.7 (34.4~56.8)	33.2 (25.3~46.3)
	NH ₄ ⁺ -N	29.2 (22.6~41.8)	27.4 (17.1~42.2)
	NO ₂ ⁻ -N	0.49 (0.21~1.06)	5.03 (0.70~10.9)
	NO ₃ ⁻ -N	0.31 (0.14~0.50)	1.37 (0.46~3.39)
	T-P	6.93 (6.00~8.10)	3.63 (3.00~4.57)
E series	Water temperature	21.1 (17.9~22.8)	21.5 (18.9~22.8)
	COD _{Mn}	64.7 (59.2~79.0)	27.0 (22.1~31.8)
	Kj-N	47.6 (40.7~59.6)	42.3 (38.6~50.9)
	NH ₄ ⁺ -N	38.1 (35.2~45.2)	36.6 (30.0~45.2)
	NO ₂ ⁻ -N	0.2 (0.15~0.25)	0.99 (0.53~1.73)
	NO ₃ ⁻ -N	0.26 (0.19~0.30)	1.51 (0.75~2.03)
	T-P	7.67 (7.27~8.36)	5.53 (5.09~6.46)
F series	Water temperature	14.0 (13.9~14.0)	14.0 (13.8~14.2)
	COD _{Mn}	78.2 (76.8~79.5)	29.2 (28.5~29.8)
	Kj-N	58.5 (55.9~61.1)	46.7 (43.0~50.4)
	NH ₄ ⁺ -N	43.8 (40.6~47.0)	41.5 (37.0~46.0)
	NO ₂ ⁻ -N	0.26 (0.09~0.42)	0.44 (0.35~0.52)
	NO ₃ ⁻ -N	0.27 (0.21~0.32)	1.33 (0.87~1.38)
	T-P	8.77 (7.88~9.65)	5.31 (4.80~5.81)
Performance confirmation experiment	Water temperature	23.1 (21.5~24.0)	23.7 (22.0~24.5)
	COD _{Mn}	71.0 (55.0~88.0)	24.6 (20.0~28.0)
	Kj-N	42.7 (29.1~50.6)	34.4 (25.4~41.3)
	NH ₄ ⁺ -N	31.2 (20.6~37.5)	29.6 (18.9~34.1)
	NO ₂ ⁻ -N	0.10 (0.04~0.26)	0.19 (0.03~0.34)
	NO ₃ ⁻ -N	0.27 (0.12~0.39)	0.87 (0.13~1.22)
	T-P	8.42 (7.19~9.81)	4.99 (4.00~6.10)
	PO ₄ ³⁻	18.1 (13.8~23.5)	13.3 (9.40~18.5)

Unit: °C for water temperature and mg/l for other parameters

The range of variation in Series D, E and F is the mean value of variations in each experiment.

Table 9-b Water quality of influent and effluent, Company K's process

Parameters		Water temperature (°C)	COD _{Mn} (mg/l)	Kj-N (mg/l)	NH ₄ ⁺ -N (mg/l)	NO ₂ ⁻ -N (mg/l)	NO ₃ ⁻ -N (mg/l)	T-P (mg/l)	PO ₄ ³⁻ -N (mg/l)
Applicant's experiment (Run II-2 Constant flow)	Influent	23.2	55.7 (50~61.4)	35.1 (32.1~38.1)	18.7 (17.2~20.2)	0.02 (0.0~0.05)	0.66 (0.29~1.03)	7.44 (6.7~8.2)	5.42 (4.4~6.1)
	Effluent	23.3	21.0 (18.1~23.9)	17.9 (14.0~21.9)	7.31 (3.68~10.9)	1.01 (0.35~1.67)	5.81 (1.59~10.0)	4.53 (3.9~5.16)	3.90 (3.3~4.4)
Applicant's experiment (Run II-5 Variable flow)	Influent	24.3	58.7 (55.5~61.9)	34.4 (32.9~38.9)	27.8 (23.7~31.9)	tr	tr	7.4 (6.9~7.9)	5.3 (4.4~6.2)
	Effluent	24.4	20.5 (19.0~22.0)	24.1 (23.1~25.1)	22.0 (20.9~29.1)	0.30 (0.22~0.38)	1.5 (1.2~1.8)	5.3 (4.9~5.7)	4.3 (3.9~4.9)
Performance confirmation experiment (Run IV)	Influent	22.5	43.0 (39.4~46.6)	29.3 (28.2~30.4)	20.0 (19.2~20.8)	tr	tr	5.3 (4.5~6.1)	3.9 (3.4~4.4)
	Effluent	22.3	14.1 (12.2~16.0)	19.3 (18.6~20.0)	17.9 (17.3~18.5)	0.24 (0.16~0.32)	1.7 (1.2~2.2)	3.73 (3.27~4.19)	3.2 (2.7~3.7)

5.4 SOLID-LIQUID SEPARABILITY

The mixed liquor from the deep well aeration tank, as compared with that in the conventional activated sludge process, contains a great quantity of supersaturated dissolved gas and fine bubbles. Accordingly, the solid-liquid system of Company I has a vacuum degasser between the aeration tank and the final sedimentation tanks. The installation of a vacuum degasser, makes it possible to employ a final sedimentation tank of ordinary design to separate solids from the mixed liquor. The results of the measurement for SVI and sludge setting velocity of the degassed sludge are shown in Table 10-a.

Table 10-a Sludge settlability, Company I's process

Run No.	MLSS	Return sludge conc.	SVI	SSV	Settling velocity
	(g/l)	(g/l)		(ml/g)	(m/hr)
D-1	4.4	12.9	128	-	-
2	3.6	9.6	109	84	1.9
3	3.2	8.6	177	93	1.7
4	3.6	8.9	134	73	2.2
5	3.5	8.2	80	54	2.9
6	4.0	9.2	73	54	2.5
E-1	2.0	15.8	122	92	2.5
2	1.8	15.2	93	70	3.5
3	1.8	16.6	81	57	3.6
4	1.5	14.2	95	64	3.6
5	2.5	11.6	126	78	2.3
F-1	2.8	11.4	121	82	2.2
2	2.9	9.7	115	87	1.9

The solid separation system of Company K consists of a floatation tank, degasser and final sedimentation tank.

(1) Floatation tank

Table 10-b shows the results of the operation of the floatation tank. Fig. 17 shows the relationship between effluent SS and the overflow rate in the floatation tank, and Fig. 18 shows the relationship between effluent SS and the solid loading.

The floated sludge had a high solid concentration at 13,300 mg/l on the average, and the SS removal was about 98%.

Table 10-b Sludge settlability (performance confirmation experiment), Company K's process

Parameter		Daily average	Hourly maximum	Hourly minimum
Operating conditions	*Flow (m ³ /day)	1558~1740	1900~1958	1044~1238
	MLSS (mg/l)	3330~3970	-	-
	Overflow rate (m ³ /m ² day)	56~62.1	67.9~69.9	37.3~44.2
	Settling time (hr)	1.16~1.3	1.02~1.06	1.63~1.93
	Solid loading (kg SS/m ² day)	185~231	-	-
SS in floatation tank effluent (mg/l)		56~98 (Average 78)		
Floated sludge concentration (mg/l)		10650~15600 (Average 13300)		
SS in final sedimentation tank effluent (mg/l)		6~29 (Average 18)		

* Flow (m³/day) includes return sludge.

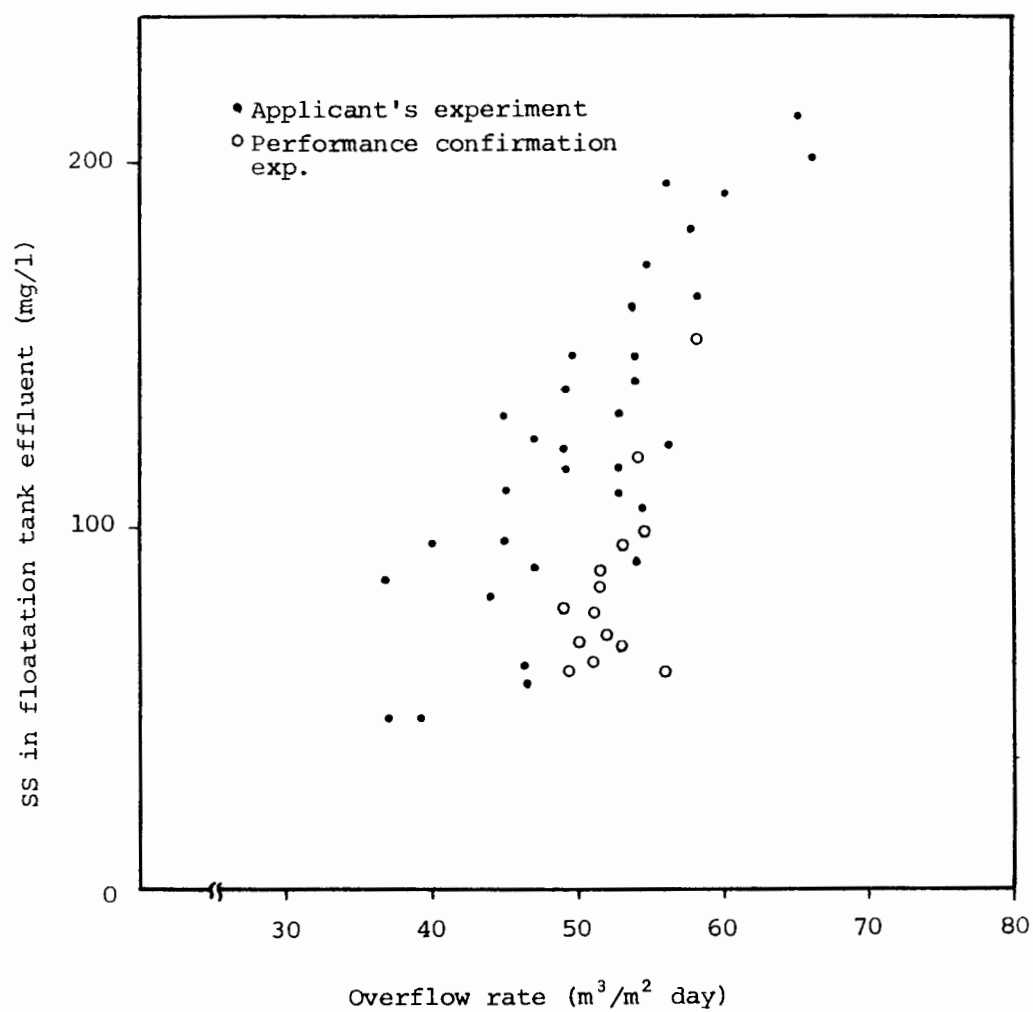


Fig. 17 Overflow rate and SS in floatation tank effluent of Company K's process

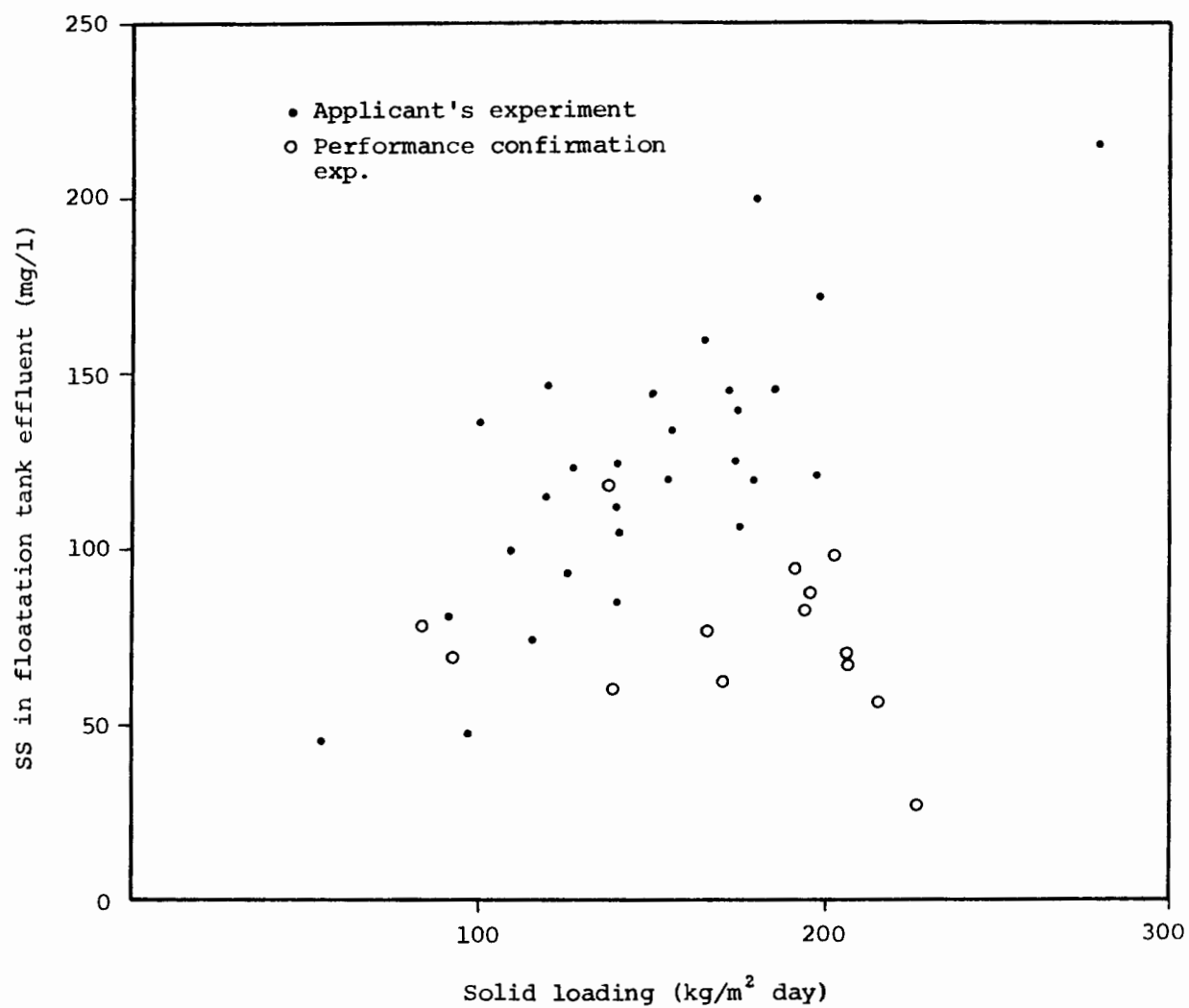


Fig. 18 Solid loading and SS in floatation tank effluent

The characteristics of the sludge in the deep well aeration tank is shown in Table 11, and the floatation curve of the sludge in the floatation tank is shown in Fig. 19. The sludge floating velocity was found to be 7.8 - 16 m/hr. Therefore, considering the high floating velocity together with the results of the solid-liquid separatability shown in Table 10-b, it is concluded that a high concentrated sludge can be obtained in a short period of time with this floatation system.

Table 11 Sludge characteristics in aeration tank of Company K's process

Parameter	Performance confirmation exp. (Run IV)	Applicant's experiment (Run II-5)
MLSS (mg/l)	3330~3970	3760
SVI	67	66
Settling velocity (m/hr)	7.8~16	4.4~12

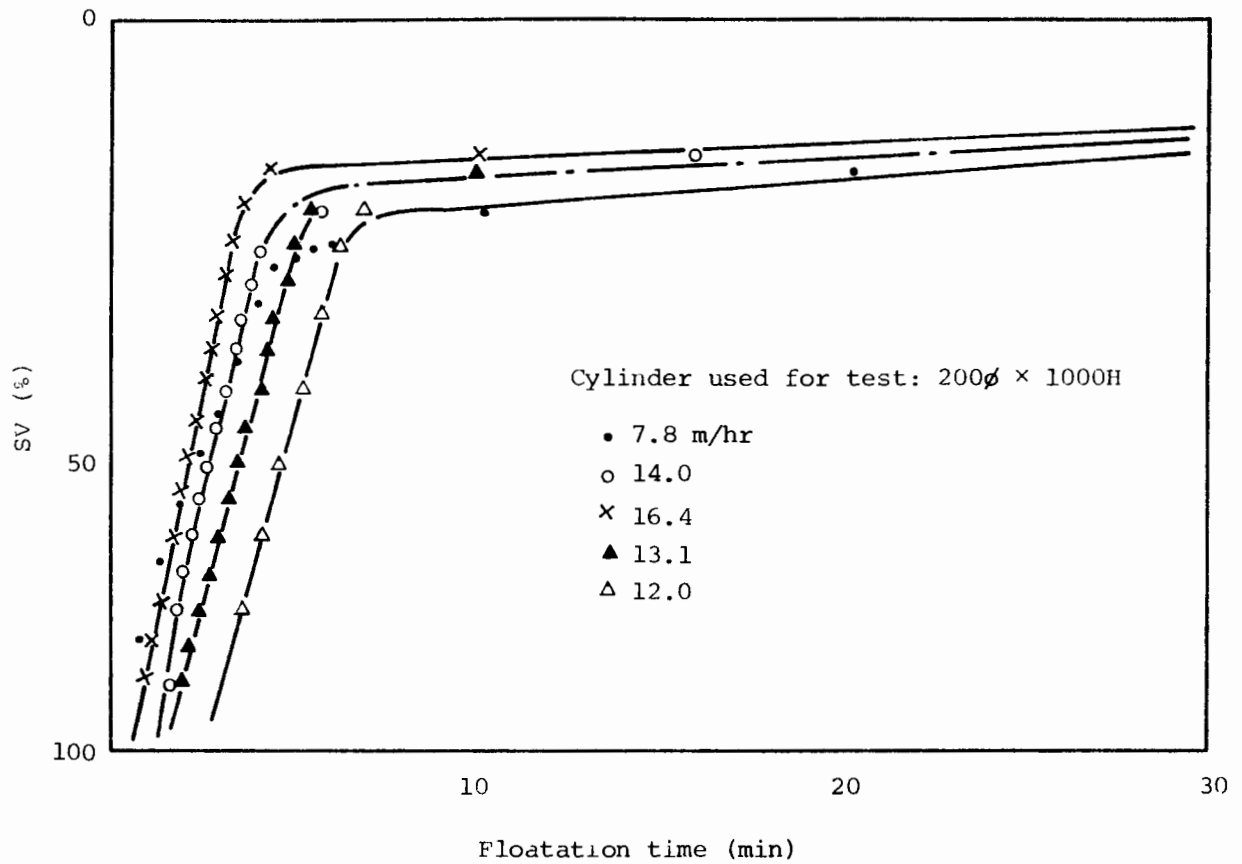


Fig. 19 Floatation curve of mixed liquor

(2) Final sedimentation tank

The relationship between the overflow rate or the flow rate and SS in the final sedimentation tank effluent is shown in Fig. 20, and the relationship between the weir loading and effluent BOD and SS is shown in Fig. 21.

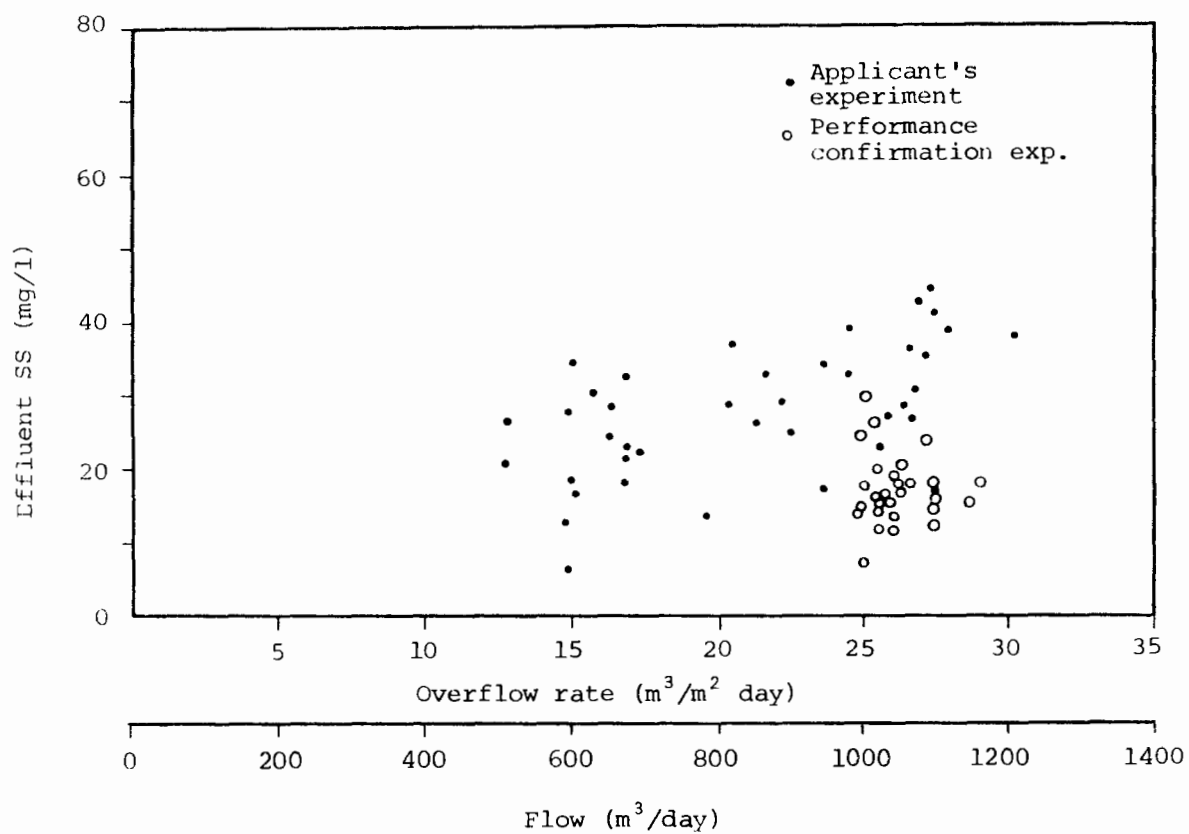


Fig. 20 Overflow rate and flow VS SS in final sedimentation tank effluent, Company K's process

In the experiment with the weir loading of $100 - 110 \text{ m}^3/\text{m} \cdot \text{day}$, effluent BOD, SS and transparency were 11.3 mg/l , 18 mg/l and 24 cm on the average, respectively. The settling curve of the sludge in the final sedimentation tank is shown in Fig. 22. Although almost all of SS in the effluent from the floatation tank settled, a part of SS floated. Therefore, it is necessary to install a scum skimmer or some other device to remove the floated SS.

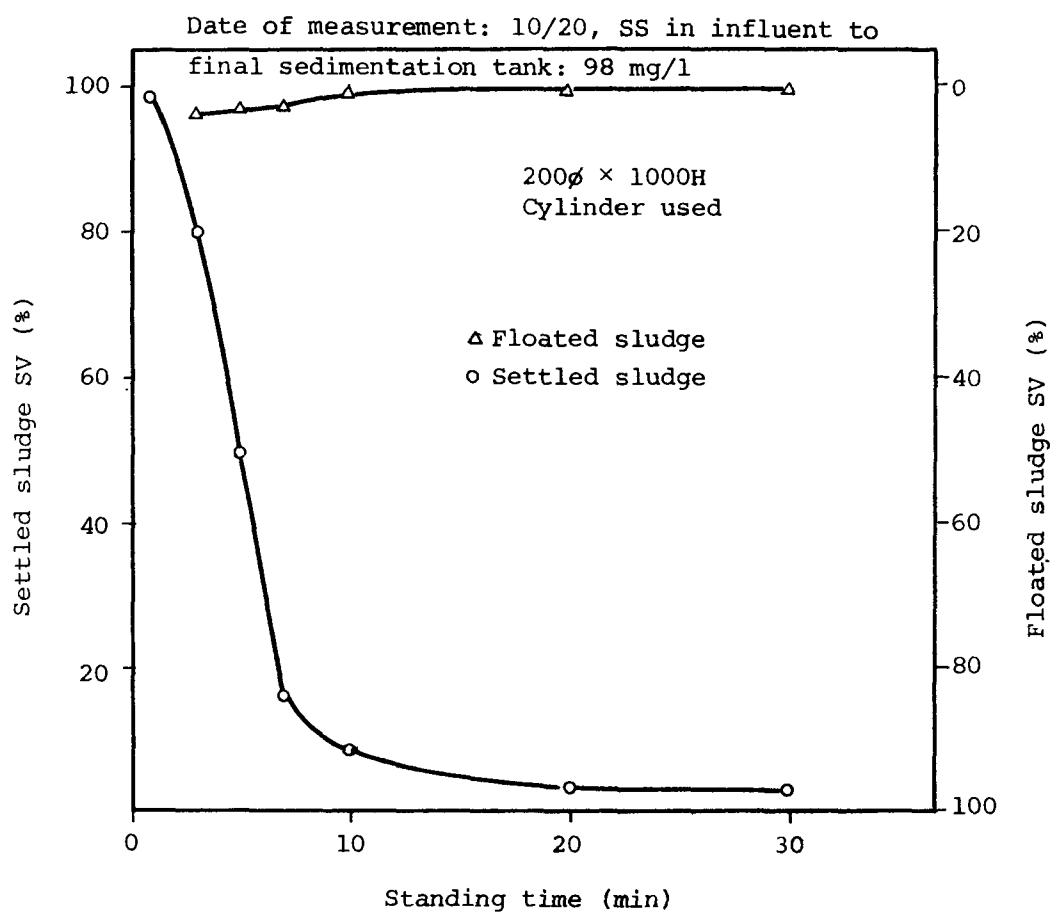


Fig. 22 Sludge settlability in secondary sedimentation tank (performance confirmation experiment of Company K's process)

5.5 TREATABILITY OF SLUDGE

5.5.1 Excess Sludge Yield

The yield of the excess sludge is shown in Table 12. It was not so different from the yield of the excess sludge in the conventional method.

Table 12-a Excess sludge yield, Company I's process

Exp. No.	Flow	Influent BOD	Influent SS	Effluent BOD	Effluent SS	MLSS	** SRT	BOD-SS loading	Inflow SS load	Outflow SS load	Excess sludge	*** Excess sludge ratio
	m ³ /day	mg/l	mg/l	mg/l	mg/l	g/l	Day	kg/kg SS day	kg/day	kg/day	kg/day	-
D-1	1320	92.2	132	8.7	24.3	4.38	14.1	0.27	168	31.7	31.0	0.227
2	2400	110	125	15.5	28.0	3.56	2.93	0.74	301	66.8	162	0.692
3	1840	124	179	15.0	23.7	3.23	2.59	0.94	329	43.1	192	0.672
4	1840	85.1	121	7.9	15.9	3.57	5.07	0.59	223	30.1	69.0	0.358
5	1520	78.1	104	6.9	12.1	3.49	16.3	0.43	159	18.9	34.2	0.244
6	1840	74.3	102	7.9	16.0	4.02	14.3	0.44	180	28.5	34.3	0.226
E-1	2400	87.2	109	12.0	9.0	1.99	3.60	1.05	261	21.3	233	0.972
2	3000	87.4	99.3	15.2	12.0	1.79	2.24	1.49	297	37.0	305	1.17
3	3600	87.2	102	18.7	17.0	1.76	2.02	1.78	369	60.0	175	0.566
4	2400	90.1	114	16.0	16.9	1.49	1.59	2.21	273	40.7	206	0.887
5	1840	112	170	15.0	20.0	2.48	2.52	1.14	309	37.4	233	0.858
F-1	1840	126	150	14.4	15.4	2.83	4.42	1.12	277	27.9	212	0.851
2	947~1940	129	132	15.9	16.4	2.91	4.69	1.39	209	25.9	158	0.863
S	2240	97.9	112	15.6	15.2	2.26	1.59	0.98	251	34.0	205	0.945

* The analysis of effluent BOD is conducted by low-temperature sterilization method.

$$** \text{ SRT} = \frac{\text{MLSS} \times \text{aeration tank volume}}{\text{excess sludge} + \text{outflow SS load}}$$

$$*** \text{ Excess sludge ratio} = \frac{\text{excess sludge}}{\text{inflow SS load} - \text{outflow SS load}}$$

Table 12-b Excess sludge yield, Company K's process

Parameter	Performance confirmation (Run IV)	Applicant's experiment (Run II-5)
Flow (m ³ /day)	1000 ~ 1100	1015 ~ 1160
Influent BOD (mg/l)	80.8	102
Influent SS (mg/l)	77	146
Effluent BOD (mg/l)	11.3	13.6
Effluent SS (mg/l)	18	33.1
MLSS (mg/l)	3240	3760
BOD loading (kg/kg SS day)	0.45	0.49
Inflow SS load (kg/day)	80.1	153
Outflow SS load (kg/day)	18.7	34.6
Excess sludge (kg SS/day)	55.8	118
Excess sludge ratio ^{*1}	0.91	1.00
SRT ^{*2} (day)	6.0	3.4

$$*1 \text{ Excess sludge ratio} = \frac{\text{excess sludge}}{\text{inflow SS load} - \text{outflow SS load}}$$

$$*2 \text{ SRT} = \frac{\text{total sludge in the system}}{\text{excess sludge} + \text{outflow SS load}}$$

5.5.2 Thickening Characteristics

Fig. 23 shows the excess sludge thickening curve. There was no particular problem in the thickning of the excess sludge.

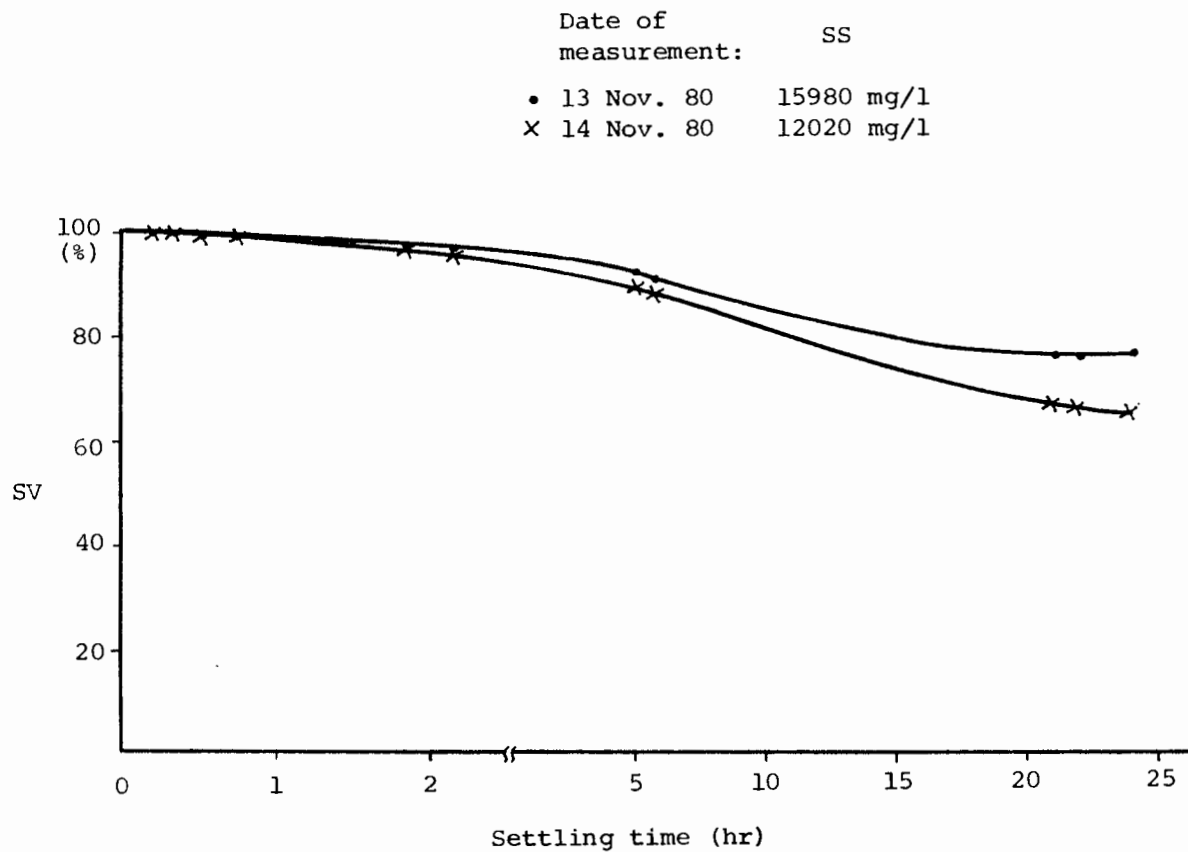


Fig. 23-a Excess sludge thickening curve, Company I's process

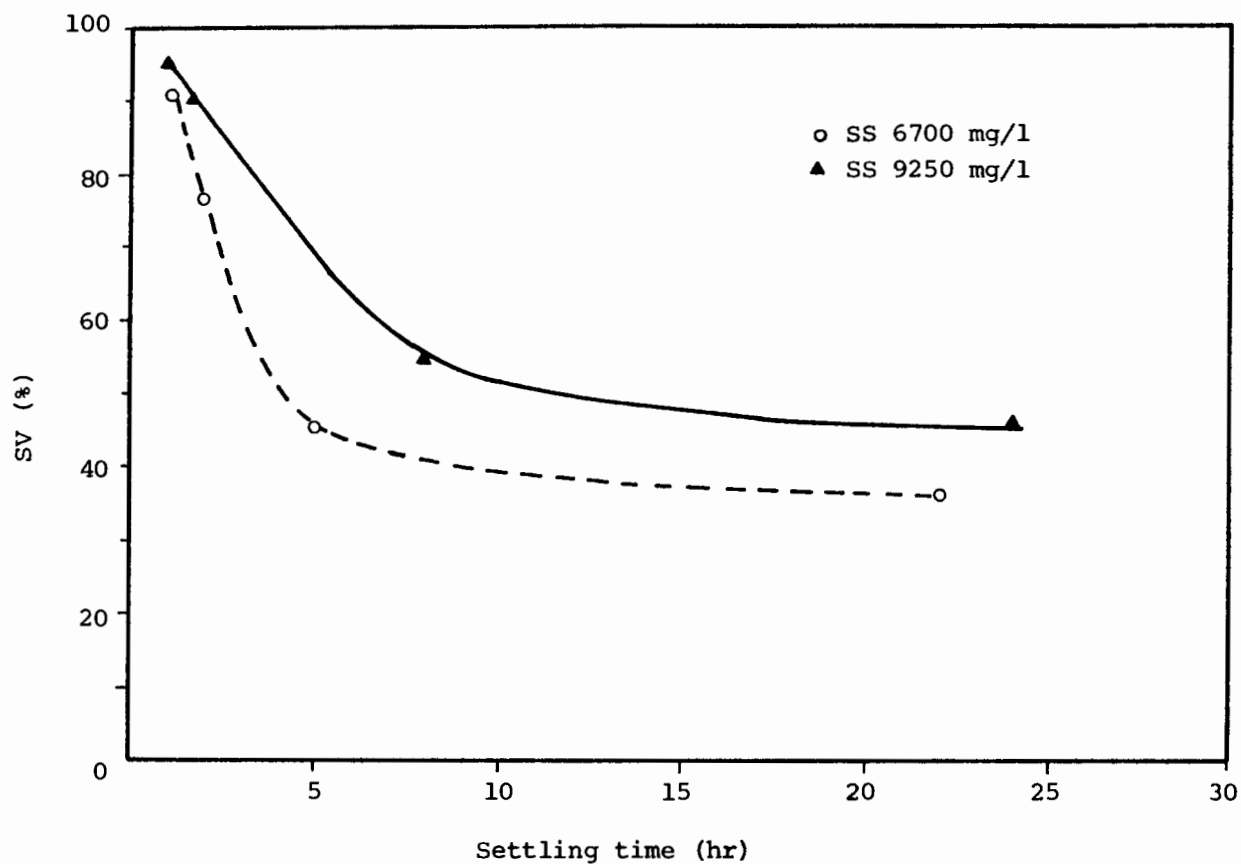


Fig. 23-b Excess sludge thickening curve, Company K's process

5.5.3 Dewaterability

The following experiments were conducted to examine the dewaterability of the excess sludge.

- (1) Vacuum filtration (leaf test)
- (2) Press filtration (leaf test)
- (3) Belt press filtration (small-sized experimental apparatus)
- (4) Centrifuge dewatering (small-sized experimental apparatus)

The results of the experiments are shown in Tables 13, 14, and 15. The dewaterability of the excess sludge in the deep well biological process was not so different from that in the conventional activated sludge process.

Table 13 Results of vacuum filtration and press filtration tests with excess sludge, Company I's process

Filtration method		Vacuum filtration		Press filtration	
		Excess sludge	Centrifuged sludge	Excess sludge	Centrifuged sludge
Concentration (wt %)		1.04	5.4	1.04	5.4
Agents dosed		$F_2Cl_3 + Ca(OH)_2$			
Dose ratio (wt %)		10+30	10+30	10+50	10+30
Filter cloth		Pyrene (A)	Pyrene (A)	Pyrene (A)	Pyrene (A)
Applied vacuum (Torr)		260	260		
Applied pressure (kg/cm ² G)	Filtration	-	-	3	3
	Pressing	-	-	10	10
Cake thickness (mm)		2.6	19.0	2.0	11.0
Sludge volume treated (m ³ /m ² hr)		0.69	0.47	0.175	0.19
Dry cake yield (kg/m ² hr)		4.9	22.6	1.84	10.8
Water content in cake (%)		81.0	84.0	64.0	70.0

Table 14 Results of dewatering experiment for excess sludge
by use of belt press, Company I's process

Filtration method	Without thickening		With thickening	
Aeration system	Conventional method	Deep well biological process	Conventional method	Deep well biological process
Dry cake Yield* (kg/m hr)	25~41	20~51	37~131	37~111
Water content in cake (%)	84.4~86.2	83.2~84.4	77.6~83.6	79.4~81.1
Coagulation agent	High cationic	→	→	→
Dose ratio (wt %)	1.35~1.61	0.98	1.10~1.47	1.0
SS recovery (wt %)	86.4	88.7~91.4	87.6~93.8	94.3~95.9

* Width of belt used 900 mm

Table 15-a Results of dewatering of excess sludge with
centrifuge, Company I's process

	Without thickening	With thickening
Concentration (wt %)	0.86	3.4
Coagulant dose ratio (wt %)	1.16	1.0
Centrifugal force (G)	3140	3140
Sludge volume treated (m ³ /hr)	3.0	1.0
Dry cake yield (wt %)	26.0	34.0
Water content in cake (%)	81~83	80~82
SS recovery (wt %)	97~99	98~99

Centrifuge used: Cylindrical conical type,
Ball diameter 250 mmφ

Table 15-b Results of dewatering of excess sludge with centrifuge, Company K's process

Parameter	Deep well biological process
Sludge volume treated (m ³ /hr)	3.5
Concentration (W/v %)	0.85
Coagulant dose ratio (wt %)	1.05
Centrifugal force (G)	3000
Dry cake yield (kg/hr)	29.5
Water content in cake (%)	83.2
SS recovery (wt %)	99

Coagulation agent used for test: Cationic polymer

6. ECONOMIC EVALUATION

For the evaluation of economical efficiency of this process, an cost estimation was conducted under the following conditions.

- (1) Influent BOD: 120 mg/l
- (2) Effluent BOD: 20 mg/l or less
- (3) Design flow: 5,000 m³/day, 50,000 m³/day and 300,000 m³/day
- (4) Two systems or more to be set up.
- (5) Scope in the estimation
 Deep well aeration tank (Companies I and K)
 Vacuum degasser (Company I)
 Floatation tank and degasser (Company K)
- (6) Geology: Geological features along the shore of Tokyo Bay shall be assumed, that is, a silt layer to the depth of 40 m and gravel layer beneath to the depth of 100 m.

6.1 CONSTRUCTION COSTS

The construction costs based on the estimation for the aeration facilities in the deep well biological process are shown in Table 16.

Table 16-a Example of estimation of construction cost of Company I's process

Design flow		5,000 m ³ /day	50,000 m ³ /day	300,000 m ³ /day	
Dimension of aeration tank		1.1 ϕ _m × 100 ^H _m	3.6 ϕ _m × 100 ^H _m	3.0 ϕ _m × 100 ^H _m	6.0 ϕ _m × 100 ^H _m
Number of systems		2	2	16	4
Material (inside/outside)		Steel pipe/ steel pipe	RC/RC	Steel pipe/ steel pipe	RC/RC
Construc- tion cost, (Unit: 1000,000 Yen)	Deep well aeration tank	261	873	4525	3427
	Vacuum degasser	21	20	256	191
	Compressor	13	63	460	460
	Total	295	956	5241	4078

Table 16-b Example of estimation of construction costs of Company K's process

Design flow		5,000 m ³ /day		50,000 m ³ /day	
Dimension of aeration tank		1.3 m ϕ \times 100 mH		2.85 m ϕ \times 100 mH	
Material for Inside aeration tube/outside tank tube		Steel pipe/ Ductile cast-iron pipe		Steel pipe/ Ductile segment	
Number of systems		2-tanks 2-systems		4-tanks 8-systems	
Construc- tion cost (1,000 Yen)		Machinery	Engineering works	Machinery	Engineering works
	Deep well aeration tank	126,000	115,000	620,000	270,000
	Floatation tank	27,000	10,000	160,000	120,000
	Degasser	15,000	5,000	24,000	6,000
	Subtotal	168,000	130,000	804,000	396,000
	Total	298,000		1200,000	

6.2 REQUIRED LAND AREA

Table 17 shows the estimated land area required for each facility

Table 17-a Required land area, Company I's process

Design flow		5000 m ³ /day	50,000 m ³ /day	300,000 m ³ /day	
Dimension of aeration tank		$1.1\phi \times 100^H_m$	$3.6\phi \times 100^H_m$	$3.0\phi \times 100^H_m$	$6.0\phi \times 100^H_m$
Number of systems		2	2	16	4
Area (m ²)	Deep well aeration tank	40	260	1904	1336
	Vacuum degasser	3.7	26.8	175	126
	Compressor air tank	5.3	38.1	71.9	71.9
	Vacuum pump	3.2	9.1	21.4	21.4
	Total	52.2	334	2172	1555

Table 17-b Required land area, Company K's process

Design flow		5,000 m ³ /day	50,000 m ³ /day
Dimension of aeration tank		$1.3 m\phi \times 100 m^H$	$2.85 m\phi \times 100 m^H$
Number of systems		2-tanks 2-systems	4-tanks 8-systems
Area required (m ²)	Deep well aeration tank	36	182
	Floatation tank	126	1260
	Degasser	36	280
	Total	198	1722

6.3 MAINTENANCE AND OPERATION COST AND REQUIRED ELECTRIC POWER

Table 18 shows the estimated power required for the aeration facilities in the deep well biological process.

Table 18-a Electric power required, Company I's process

Design flow		5,000 m ³ /day		50,000 m ³ /day		300,000 m ³ /day	
		Rated	Actual	Rated	Actual	Rated	Actual
Compressor	Kw	30	22	180	158	840	780
Vacuum pump	Kw	4.4	2.0	15	10	88	60
Return sludge pump	Kw	7.4	4.6	44	35	240	210
Total	Kw	41.8	28.6	239	203	1168	1050
Power consumed per day		KwH/day					
			686		4872		25200
Power consumed per 1 kg of removed BOD	$\frac{\text{KwH}}{\text{Removed BOD kg}}$		1.37		0.974		0.84
Power consumed per 1 m ³ of sewage	KwH/m ³		0.137		0.097		0.084

Table 18-b Electric power required, Company K's process

Design flow	5,000 m ³ /day			50,000 m ³ /day		
	Rated KW	Actual KW	Power consumed KWH/day	Rated KW	Actual KW	Power consumed KWH/day
Recirculation pump	7.5×2	5.6×2	269	30×4	27.6×4	2650
Aeration blower	3.0×1	2.0×1	48	14.3×2	9.9×2	476
Sludge collector	1.5×2	1.1×2	53	1.5×4	1.1×4	106
Degasser blower	1.65×1	0.7×1	16.8	5.0×2	3.4×2	163
Return sludge pump	5.5×1	4.0×1	96	22×2	19.0×2	912
Total	28.15	20.1	482.8	208.6	179.4	4307
Power consumed per day	483 KWH/day			4307 KWH/day		
Power consumed per 1 kg of removed BOD	0.774 KWH/removed BOD kg			0.679 KWH/removed BOD kg		
Power consumed per 1 m ³ of sewage	0.097 KWH/m ³			0.086 KWH/m ³		

From the above results of the estimation, it can be seen that the land area required for the aeration tank is much less than that for the conventional method, and the power required is almost the same as in the conventional method.

7. RESULTS OF EVALUATION

From the view points of treatment performance and the actual construction experiences, the deep well biological process developed by Companies I and K met the goals of development and has been recognized as a municipal sewage treatment technology that has already reached the stage of practical use as described below.

- (1) These were practical techniques with a treatment capacity of 1,000 m³/day or more.
- (2) These techniques were capable of meeting the legal requirements for the biochemical oxygen demand(BOD) and suspended solids(SS) in effluents.
- (3) In the operation of facilities under this process, there was no marked difference between this process and the conventional methods such as the standard activated sludge process.
- (4) As for the operation and maintenance cost, these techniques did not differ greatly from the conventional method.
- (5) These techniques had simplicity in taking countermeasures to prevent detrimental effect on surrounding environment.
- (6) Much less land area was required for the aeration tank in this process, compared with the conventional methods.

8. ADDITIONAL REMARKS

The following remarks were added to the final evaluation report.

- (1) Biochemical oxygen demand(BOD) of the effluents from the applicants' plants during the experiments for this evaluation was 20 mg/l or less.

It is necessary to consider a safety factor to meet the effluent standard in the design and operation of the deep well biological process, as it is customary done in the design and operation of the actual plants.

- (2) By the employment of the deep well biological process, the land area for the aeration can be greatly reduced. However, additional facilities for solid-liquid separation are required instead.

Further, the aeration tank is constructed with a technique different from that in the conventional method.

Accordingly, on the adoption of this process, the complete economical analysis must be carried out, with particular consideration given to the reduced land cost, the construction costs of the deep well aeration tank and solid-liquid separation facilities, and other costs.

- (3) For the design and construction of the deep well aeration tank, it is necessary to investigate thoroughly in advance: Earthquake resistance, construction method and safety depending on the geological structure, the materials to be used, and some other factors.

- (4) In this process, the removal of sand, sludge and other matter which may settle and accumulate on the bottom of the aeration tank is considered to be more difficult than in the conventional method. This must be considered in the design and operation of this plant.

**DEPHOSPHORIZATION
OF
SEWAGE BY CONTACT CRYSTALIZATION
OF
CALCIUM APATITE**

October 1981

Cincinnati, U. S. A.

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1. INTRODUCTION

As a countermeasure to the pollution of land-locked waters, such as lakes, swamps and inland bays, due to eutrophication, a variety of processes for removal of phosphorus, one of the major pollutants, have been developed.

A phosphate crystallization process which theoretically does not liberate sludge has attracted the Tokyo Metropolitan Government whose chronic headache is the lack of sites for dumping sewage sludge. In August 1978, the Tokyo Metropolitan Government installed a pilot plant and related experimental facilities at its Morigasaki Wastewater Treatment Plant to study the phosphate crystallization process. Although phosphate crystallization had already been demonstrated by a private company to be effective in treating the secondary effluent of night soil various engineering requirements and particulars had yet to be clarified in its application to the dephosphorization of sewage.

2. PRINCIPLES AND FEATURES OF DEPHOSPHORIZATION

2.1 PRINCIPLES

Phosphorus in the secondary effluent of sewage exists, for the most part, in the form of PO_4^{3-} . When charged with slaked lime (calcium hydroxide), this secondary effluent produces calcium hydroxyapatite ($\text{Ca}_5(\text{OH})(\text{PO}_4)_3$), which, though extremely low in solubility, remains dissolved and supersaturated in the secondary effluent because its precipitation velocity is quite low at a pH value of 8 to 9.

In the contact crystallization process discussed here, hydroxyapatite is crystallized and separated selectively and at a practically high speed by bringing it into contact with a liquid dephosphorization media with a low degree of supersaturation and a pH value of 8 to 9 (crystallines mainly consisting of calcium phosphate) as seeds forming crystalline nuclei.

The secondary effluent of sewage contains, in addition to phosphoric acid, 100 mg of carbonates per liter which react with calcium ions.

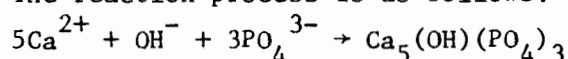
When slaked lime is injected into the secondary effluent, these carbonates are transformed into CaCO_3 or $\text{Ca}(\text{HCO}_3)_2$, which coexists with apatite in a supersaturated state to prevent the separation of the latter. Worse, CaCO_3 or $\text{Ca}(\text{HCO}_3)_2$ is liberated upon the dephosphori-

zation media to degrade the latter's dephosphorization activity.

The contact crystallization system introduced here is provided with a decarbonation process in which raw water is added to sulfuric acid in excess of its M-alkalinity (total alkalinity) equivalent (acid equivalent of CaCO_3 necessary to reduce the pH value to 5) in order to convert the carbonates into free ones, which are then aerated for expellation into the open air.

Then, slaked lime is dosed to make up Ca ions and hydroxyl ions (to a Ca^{2+} concentration of about 70 mg/lit., pH 8.5 to 8.8), and the decarbonated liquid is brought in contact with the nuclei media (dephosphorization media serving as a filter) consisting mainly of calcium phosphate for the purpose of dephosphorization.

The reaction process is as follows.



In this case, the pH value may go up more than necessary if slaked lime alone is used to increase the Ca^{2+} concentration. For this reason, part of the slaked lime does may be replaced with CaSO_4 or others.

2.2 FEATURES

The contact crystallization system has the following features.

- (1) Theoretically sludge-free, except for the sludge stemming from suspended solids in the secondary effluent.
- (2) Shorter retention period in each process, and a smaller installation space compared with the coagulation sedimentation process (phosphate precipitation process).
- (3) Use of inexpensive chemicals (sulfuric acid and slaked lime) which are readily available domestically.
- (4) Recovery of phosphorus from sewage as deposited on the dephosphorization media.
- (5) No need to reactivate the dephosphorization media, and no need to treat wastewater resulting from reactivation, unlike the ordinary ion exchange media.

3. PILOT PLANT OUTLINE

The conditions upon which the pilot plant has been designed are listed in Table 1, and the physical properties and chemical composition of the dephosphorization media* in Table 2.

Figure 1 shows the process flow sheet of the pilot plant.
(Refer to Table 1, Table 2 and Fig. 1)

Table 1. Pilot plant design conditions and specifications

Effluent to be treated		Secondary effluent of sewage treatment plant subjected to preliminary removal of SS by rapid sand filtration
Planned processing rate		100 m ³ /day
Decarbonation tank	Dimensions	0.5 m x 1 m x 2 m (H)
	Retention time	8.6 to 12.9 min.
	Aeration rate	0 to 21 m ³ /hr.
pH conditioning tank	Dimensions	0.8 m ϕ x 1 m (H)
	Retention time	5.0 min.
Filtration tank	Dimensions	1 m ϕ x 1 m (H), downward flow pressure type
	Rate of filtration	5 m/hr.
Dephosphorization tank	Dimensions	1 m ϕ x 4.5 m (H)
	Linear velocity (LV)	5 m/hr.
	Space velocity (SV)	2.5 times/hr.
	Dephosphorization media	Charge, 1.6 m ³ ; stuffing height, 2 m

Table 2 Dephosphrization media -- physical properties and chemical composition

Physical properties		Chemical composition, %							
Grain size	0.44 mm	P ₂ O ₅	34.8	H ₂ O	1.6	SiO ₂	0.5	K ₂ O	0.1
Uniformity coefficient	1.59	CaO	52.6	SO ₄	1.4	MgO	0.6	Ignition loss	1.9
True specific gravity	2.62g/cm ³	CO ₂	4.5	Na ₂ O	1.1	Al ₂ O ₃	0.3	Others	4.1

* In order to obtain high-efficiency dephosphorization media of uniform quality, phosphate rock was treated as follows. First, phosphate rock was pulverized and classified by size. Phosphate powder of a uniform size was then treated in an aqueous solution of potassium hydrophosphate and an alkaline solution of calcium chloride to form fine apatite crystallines over phosphate particles.

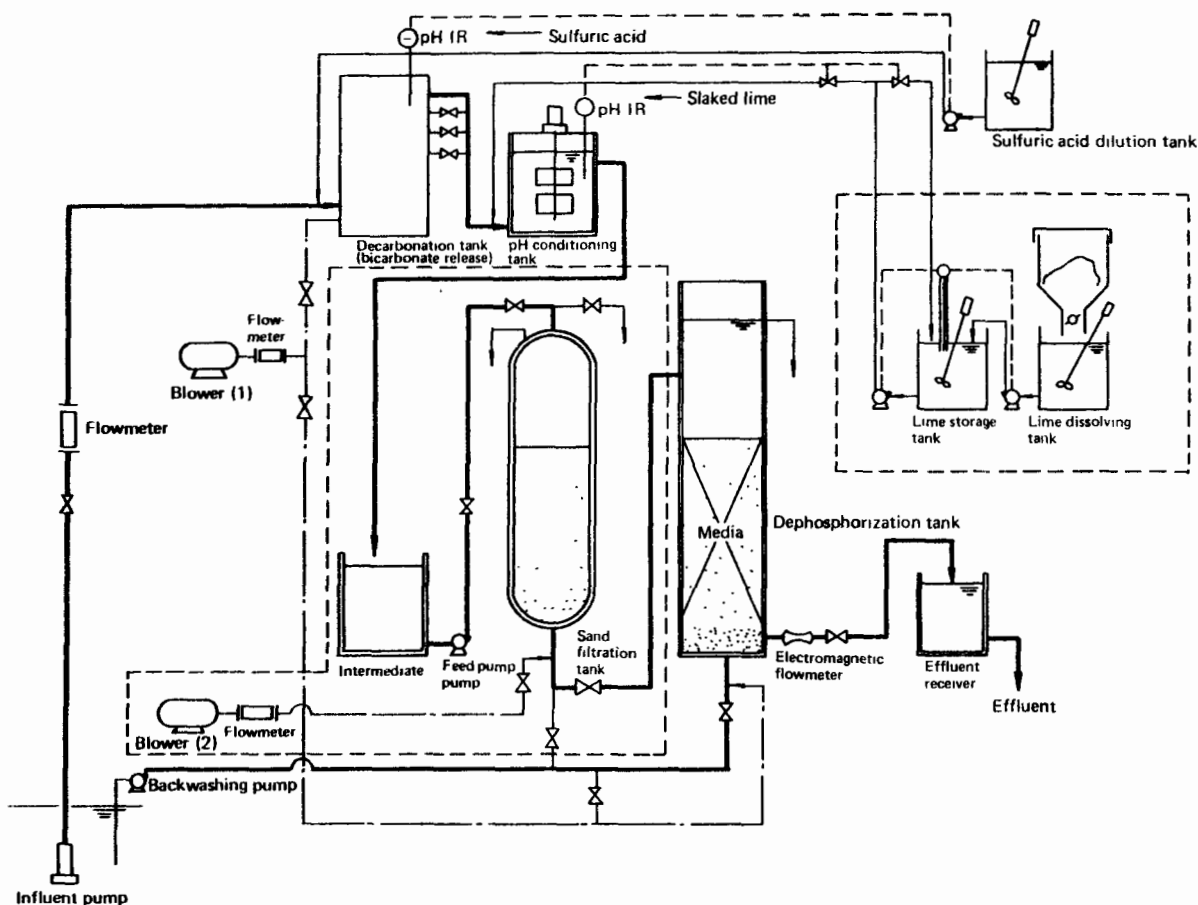


Fig. 1 Pilot plant

4. RESEARCH METHODS

An infiltrate through a high-speed sand filter of the secondary effluent available from the Morigasaki Wastewater Treatment Plant was supplied around the clock to the pilot plant for study.

1st stage (September 1978 to July 1979): The process flow used is illustrated in Figure 2. (Refer to Fig. 2)

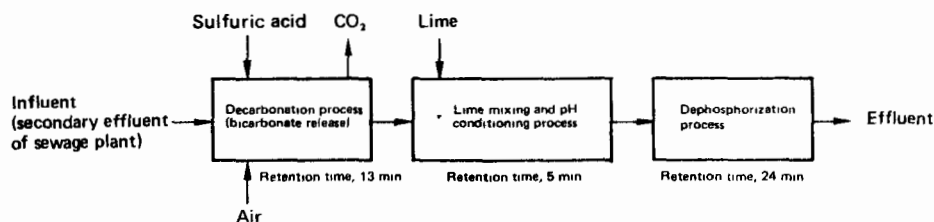


Fig. 2 Process flow for 1st stage of study

2nd stage (August 1979 to May 1980) and 3rd stage (June 1980 to March 1981): According to the results of the first stage study, an intermediate filter process was installed. Prior to each of these stages, the dephosphorization media was renewed. (Refer to Fig. 3)

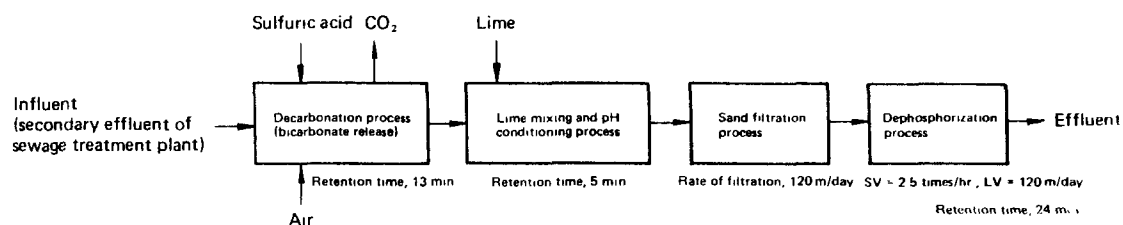


Fig. 3 Process flow for 2nd stage of study

4.1 RESULTS AND DISCUSSIONS

4.1.1 Raw Water Quality

The monthly averages of the water characteristics concerning the crystallization reaction are shown in Table 4.

Table 3 Measuring Items and methods of analysis

Measuring Item	Unit	Method of analysis
pH	—	Glass electrode method (specified in the Standard Methods for the Examination of Sewage)
Turbidity	deg.	Transmittance measuring method (specified in the Standard Methods for the Examination of Drinking Water)
Total alkalinity (as CaCO_3)	mg/lit.	Titration with N/50 HCl (specified in the Standard Methods for the Examination of Sewage)
Calcium ion	mg/lit.	EDTA titrimetric method (specified in the Japanese Industrial Standards)
Total phosphate	mg/lit.	EPA methods for chemical analysis of water and wastes (Ammonium persulfate digestion and colorimetry)
Orthophosphate	mg/lit.	EPA methods for chemical analysis of water and wastes (direct colorimetry)
Dissolved orthophosphate	mg/lit.	Ditto
Total carbon dioxide	mg CO_2 /lit.	Diaphragm type CO_2 electrode method
Free CO_2	mg CO_2 /lit.	ditto

Table 4(a) Quality of influent (1st stage of study)

Characteristic \ Date	Oct. 1978	Nov. 1978	Dec. 1978	Jan. 1979	Feb. 1979	Mar. 1979	Apr. 1979	May 1979	Jun. 1979	Jul. 1979
Water temperature °C	21.3	18.0	16.4	14.7	14.8	14.7	17.3	19.7	23.1	24.3
pH	6.65 ~ 7.14	6.60 ~ 7.14	6.95 ~ 7.15	6.75 ~ 7.10	6.82 ~ 7.09	6.72 ~ 7.05	6.45 ~ 7.00	6.63 ~ 6.85	6.70 ~ 7.10	6.72 ~ 7.10
Total Alkalinity, mg/lit.	83	86	116	93	103	97	66	54	81	72
Ca ⁺² mg/lit.	29	28	30	28	27	28	27	27	26	28
Total P mg/lit.	2.52	2.68	3.13	2.01	1.67	1.76	1.88	1.59	1.79	1.81
PO ₄ -P mg/lit.	2.41	2.43	2.93	1.93	1.59	1.69	1.79	1.52	1.67	1.75
Dissolved PO ₄ -P mg/lit.	2.38	2.41	2.93	1.85	1.53	1.65	1.74	1.49	1.65	1.72
Turbidity deg.	0.52	0.83	1.47	0.95	1.45	1.34	1.24	1.02	1.37	0.75
Total CO ₂ mg/lit.	88	88	114	94	107	101	68	58	83	78
Free CO ₂ mg/lit.	8.3	9.6	12.5	12.3	11.8	12.6	11.4	12.3	13.8	13.8

Table 4(b) Quality of influent (2nd stage of study)

Characteristic \ Date	Aug. 1979	Sep. 1979	Oct. 1979	Nov. 1979	Dec. 1979	Jan. 1980	Feb. 1980	Mar. 1980	Apr. 1980	May 1980
Water temperature °C	26.1	24.5	21.6	18.7	16.5	14.4	14.4	15.8	17.8	19.1
pH	6.76 ~ 7.13	6.75 ~ 7.00	6.70 ~ 6.99	6.75 ~ 6.94	6.70 ~ 6.94	6.75 ~ 7.05	6.88 ~ 7.17	7.00 ~ 7.12	6.80 ~ 7.15	6.83 ~ 7.15
Total alkalinity mg/lit.	62	66	60	55	61	73	99	100	80	80
Ca ⁺² mg/lit.	29	28	29	29	28	28	27	28	26	26
Total P mg/lit.	1.79	1.79	1.26	1.42	1.51	1.55	1.86	1.80	1.64	1.89
PO ₄ -P mg/lit.	1.73	1.75	1.20	1.37	1.45	1.45	1.75	1.66	—	—
Dissolved PO ₄ -P mg/lit.	1.71	1.74	1.18	1.34	1.42	1.43	1.72	1.64	—	—
Turbidity deg.	0.60	0.82	1.24	1.17	1.45	1.69	2.07	2.06	1.91	1.77
Total CO ₂ mg/lit.	68	63	63	60	66	82	112	119	85	80
Free CO ₂ mg/lit.	12.4	10.6	11.6	13.0	14.1	13.8	15.5	15.2	—	—

Table 4(c) Quality of influent (3rd stage of study)

Characteristic \ Date	Jun. 1980	Jul. 1980	Aug. 1980	Sep. 1980	Oct. 1980	Nov. 1980	Dec. 1980	Jan. 1981	Feb. 1981	Mar. 1981
Water temperature °C	23.8	23.3	23.5	22.7	20.6	18.8	16.7	14.1	13.9	16.3
pH	6.90 ~ 7.01	6.69 ~ 7.04	6.77 ~ 7.04	6.71 ~ 6.92	6.64 ~ 6.99	6.80 ~ 7.07	6.84 ~ 7.19	6.86 ~ 7.14	6.89 ~ 7.02	6.62 ~ 7.62
Total alkalinity mg/lit.	83	66	58	60	72	80	83	93	94	82
Ca ⁺² mg/lit.	28	29	30	30	30	30	30	29	28	27
Total-P mg/lit.	1.90	1.37	1.28	1.43	1.49	1.57	1.66	1.87	1.72	1.64
Turbidity deg.	1.26	1.35	0.84	0.84	1.09	0.93	1.02	1.58	2.04	2.30
Total carbonate mg/lit.	87	70	60	81	74	85	76	102	108	84

The M-alkalinity governing the sulfuric acid requirement for the decarbonation process varied in the range of 54 to 115 mg/lit. with 75 mg/lit. as an average; it decreased in summer, but increased in winter. It was also low when the nitrification was active or when it was rainy.

The total phosphorus in the raw water was in the range of 1.5 to 2.0 mg/lit. for most months and showed an annual average of 1.62 mg/lit. in 1980, suggesting that the raw water used was representative of secondary effluents so far as the phosphorus concentration is concerned.

In passing, the annual average of the total-phosphorus concentrations of the secondary effluents from the eight waste water treatment plants within the wards of Tokyo is in the range of 1.13 to 3.14 mg/lit. or 1.92 mg/lit. on the average, and phosphorus is present, primarily in the form of dissolved orthophosphate.

Calcium ions which are effective in the crystallization reaction remained almost constant at about 30 mg/lit. throughout the test stages.

The carbonates which affect the crystallization reaction were present at a level of 60 to 120 mg/lit. in terms of total carbonate, and changed almost in proportion to the change in M-alkalinity.

4.1.2 Decarbonation

In the decarbonation process, raw water was acidified with sulfuric acid, and was aerated to liberate carbonates, which were then subjected to air stripping. The injection rate of sulfuric acid was held at 150 mg/lit. up until March 1979, but from April 1979 was controlled to attain a pH value of 3.

In order to improve the decarbonation efficiency, the aeration rate was changed little by little during the study period, and the tanks were modified as well. Figure 4 shows the changes in the monthly averages of total-carbonate concentrations in the influent and decarbonated water. Table 5 shows the relationship between the running conditions and decarbonation efficiency. As is clear from Table 5, the total-carbon concentration of the secondary effluent could be held low at about 10 mg/lit. by injecting air at a gas-to-liquid ratio of 3 in the plug flow system. (Refer to Fig. 4, Table 5)

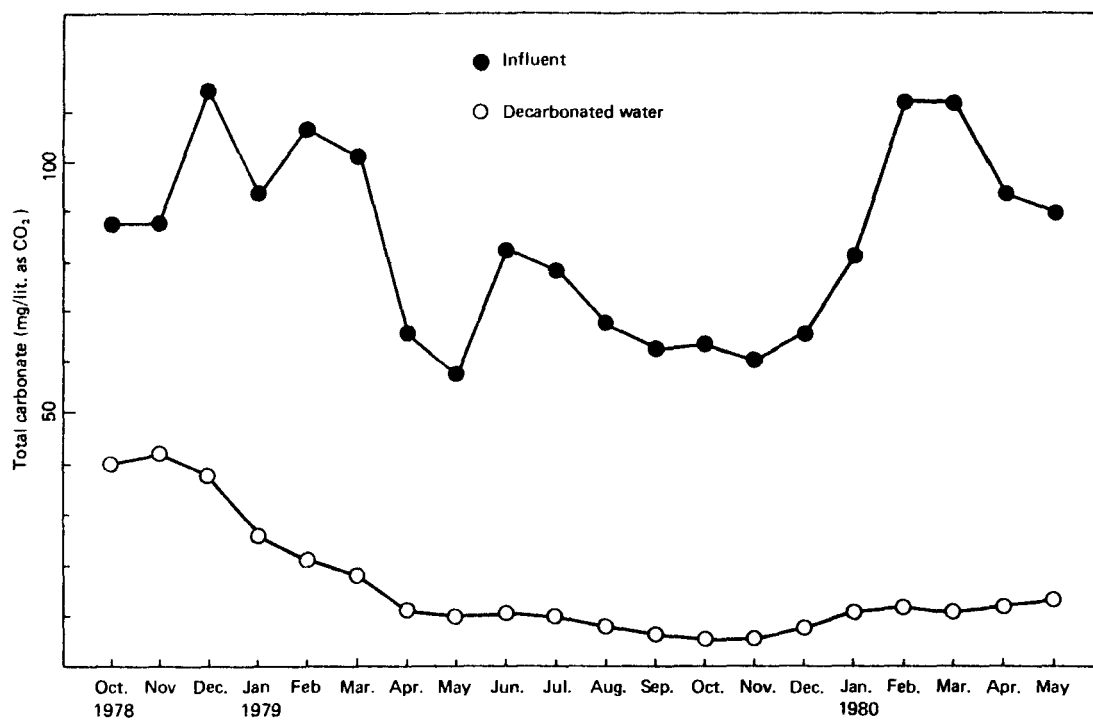


Fig. 4 Monthly averages of total carbonate in the influent and decarbonated water

Table 5 Operating conditions of decarbonation tank, and decarbonation efficiency

Period	Oct. to Nov., 1978	Dec. 1978 to Jan. 1979	Jan. 1979	Feb. to Jun., 1979	Jun. to Aug., 1979	Aug. 1979 to Mar. 1981
Type of decarbonation tank	Single-stage complete mixing				3-horizontal staggered baffle 4-turn flow	3-vertical staggered baffle 4-turn flow
Gas-liquid ratio (G/L)	1	2	3	5	3	3
Total carbonate in influent mg/lit.	90	107	108	105	73	81.2
Total carbonate in decarbonated water mg/lit.	39.9	36.0	33.8	22.9	16.6	13.9
Carbonate removal rate %	55.8	66.4	68.7	81.1	77.2	82.9

4.1.3 Dephosphorization and Reaction Products

In the first stage of the study, the changes in phosphate concentration of influent and effluent were as shown in Figure 5 and Table 6. (Refer to Fig. 5, Table 6)

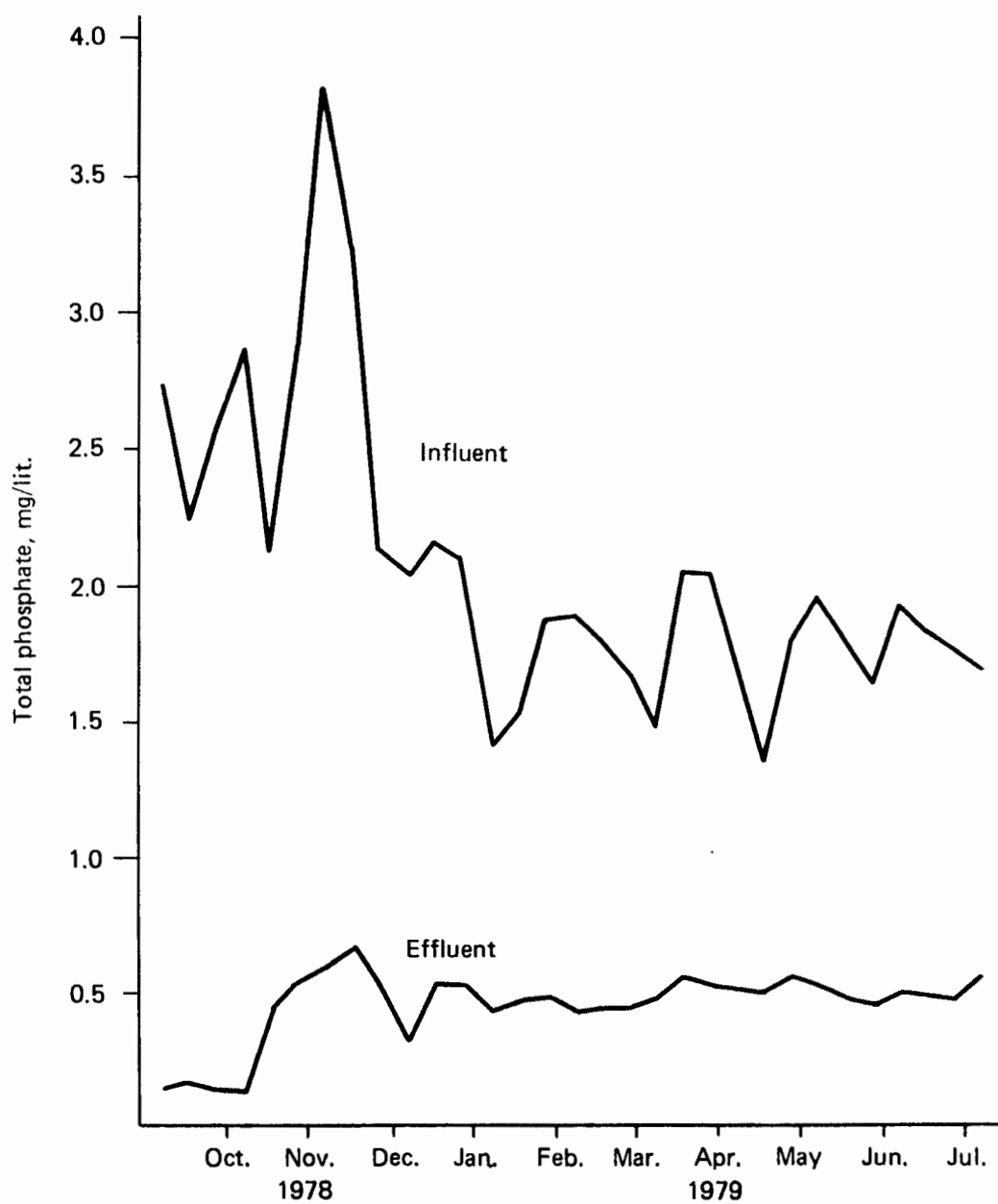


Fig. 5 Monthly changes of phosphate concentration in the first stage of study

Table 6 Phosphate concentration in the first stage of study

Sample	Characteristic	Oct. 1978	Nov. 1978	Dec. 1978	Jan. 1979	Feb. 1979	Mar. 1979	Apr. 1979	May 1979	Jun. 1979	Jul 1979
Influent	Total phosphate mg/lit.	2.52	2.68	3.13	2.01	1.67	1.77	1.88	1.59	1.79	1.82
	Dissolved orthophosphate mg/lit.	2.41	2.43	2.93	1.93	1.59	1.69	1.79	1.52	1.67	1.75
Effluent	Total phosphate mg/lit.	0.12	0.35	0.59	0.48	0.47	0.44	0.53	0.53	0.49	0.49
	Dissolved orthophosphate mg/lit.	0.07	0.28	0.51	0.45	0.42	0.40	0.51	0.50	0.45	0.48

For about one and a half months after the start of the operation, the phosphate concentration in the effluent was kept as low as 0.1 to 0.2 mg/lit. This was due to crystallization synergy, and adsorption to the surfaces of dephosphorization media, etc., in the early stages of water supply.

In December, three months later, the phosphate concentration reached 0.77 mg/lit. It was inferred that an increased amount of residual carbonates in the influent of dephosphorizing tank owing to poor decarbonation might have reacted with calcium ions to encrust calcium carbonate over the dephosphorization media thus halting the dephosphorization activity.

Later the operating conditions of the decarbonating tank were improved. As a result, the concentration had been stabilized at about 0.5 mg/lit. from January 1979 till the end of the 1st stage of the study in July 1979. To check the surface changes of the dephosphorization media, the chemical composition of flakings from the dephosphorization media were analyzed.

The results were as shown in Table 7. It is evident that the surfaces of the dephosphorization media were covered mainly by calcium carbonate. (Refer to Table 7)

Table 7 Chemical composition of reaction products

Sample	Chemical composition*	
	Apatite	Calcium carbonate
Fresh dephosphorization media	100 %	0
Dephosphorization media after three months of use	22.0 %	59.3 %

*Note: Calculated from the analyses of P and Ca

In the first stage study it was found that calcium carbonate produced in slaked lime and water served as a barrier to crystallization. In the second stage study, a sand filter was applied before the dephosphorization process to remove calcium carbonate, and the dephosphorization media was renewed.

Figure 6 and Table 8 show the phosphate concentrations in the influent and effluent in the second and third stages of the study. (Refer to Fig. 6, Table 8)

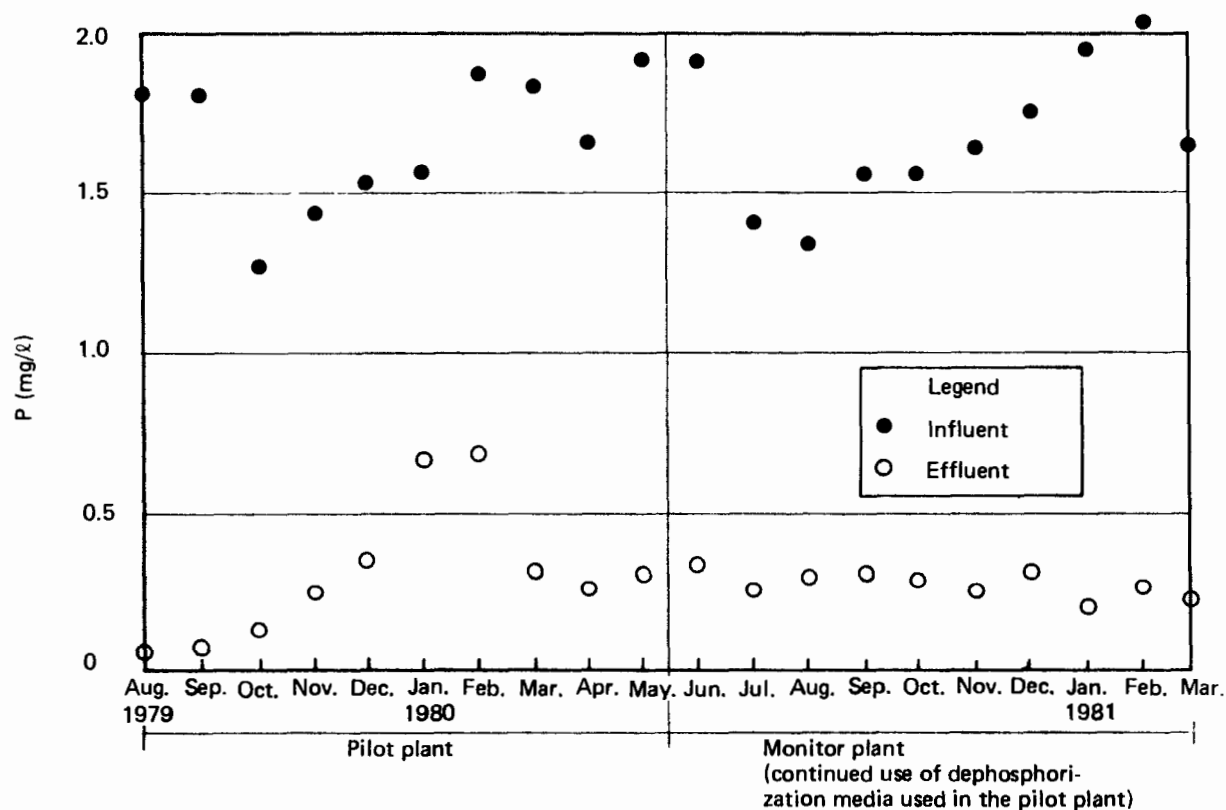


Fig. 6 Phosphate concentrations in influent and effluent (Aug. 1979 to Mar. 1981)

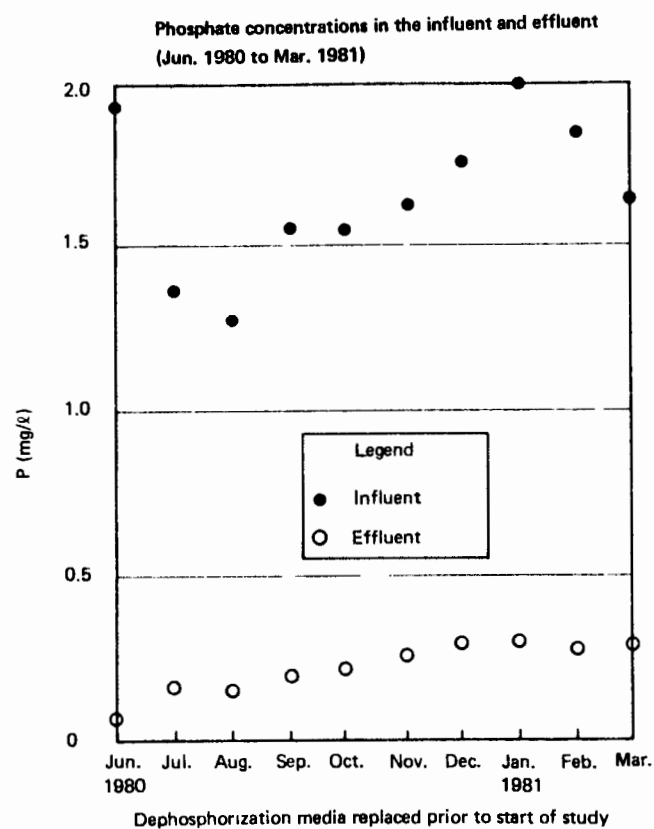


Fig. 6 (continued)

Table 8 Phosphate concentrations in influent and effluent (Aug. 1979 to Mar. 1981)

		Aug. 1979	Sep. 1979	Oct. 1979	Nov. 1979	Dec. 1979	Jan. 1980	Feb. 1980	Mar. 1980	Apr. 1980	May 1980
Influent	Total phosphate (mg/l)	1.79	1.79	1.26	1.42	1.51	1.55	1.85	1.80	1.64	1.91
	Total orthophosphate (mg/l)	1.73	1.75	1.20	1.37	1.45	1.45	1.74	1.66	1.54	1.83
	Dissolved orthophosphate (mg/l)	1.71	1.74	1.18	1.34	1.42	1.43	1.71	1.64	1.54	1.81
Effluent	Total phosphate (mg/l)	0.04	0.07	0.11	0.23	0.34	0.65	0.67	0.29	0.26	0.29
	Total orthophosphate (mg/l)	0.02	0.05	0.08	0.21	0.30	0.59	0.64	0.26	0.25	0.27
	Dissolved orthophosphate (mg/l)	0.01	0.04	0.08	0.20	0.29	0.57	0.61	0.25	0.23	0.25

		Jun. 1980	Jul. 1980	Aug. 1980	Sep. 1980	Oct. 1980	Nov. 1980	Dec. 1980	Jan. 1980	Feb. 1980	Mar. 1980
Influent	Total phosphate (mg/l)	1.90	1.37	1.28	1.54	1.55	1.62	1.75	2.00	1.83	1.64
	Total orthophosphate (mg/l)	1.81	1.34	1.23	1.50	1.49	1.57	1.66	1.87	1.72	1.52
	Dissolved orthophosphate (mg/l)	1.79	1.31	1.22	1.49	1.47	1.55	1.65	1.87	1.71	1.50
Effluent	Total phosphate (mg/l)	0.07	0.16	0.15	0.19	0.22	0.26	0.29	0.30	0.28	0.29
	Total orthophosphate (mg/l)	0.05	0.14	0.13	0.16	0.19	0.23	0.27	0.26	0.25	0.26
	Dissolved orthophosphate (mg/l)	0.04	0.12	0.12	0.14	0.18	0.22	0.25	0.25	0.24	0.24

The concentration of total phosphate in the effluent was kept at less than 0.1 mg/lit. for about three months after the start of the second stage study and for about one and a half months after the start of the third stage of study. Just as in the first stage, this was caused by crystallization synergy and adsorption to the surface of the dephosphorization medium. Compared with the first stage, the phosphate concentration in the effluent was low. This was because the inflow of calcium carbonate and other contaminants into the dephosphorization tank was reduced sharply to maintain the performance integrity of the dephosphorization media for an extended period.

After four months, the total phosphate concentration had increased to 0.2 mg/lit., and five months later, it had increased to 0.3 mg/lit. Subsequently, however, the total phosphate concentration remained unchanged. This concentration level was lower than 0.5 mg/lit., showing the effectiveness of the improvement of the decarbonation process and the installation of the sand filtration process.

With the supply of influent, the leakage of phosphate increases gradually and finally reaches an equilibrium state. During the March-May period of 1979, when the process was deemed to be stabilized, the total phosphate concentration was 0.18 to 0.34 mg/lit. or 0.28 mg/lit. on the average, which seems to represent the performance of the contact crystallization process.

In January, the effluent showed a high phosphate concentration. This was because the decarbonation tank was run at a pH value of 3.5, which reduced the injection of slaked lime necessary for neutralization to cause insufficiency in the calcium ions required for dephosphorization. When the pH value was reset to 3, the phosphate concentration level was reestablished.

To investigate the properties of the reaction products deposited on the dephosphorization media, elementary analysis of surface layers of the dephosphorization media was conducted using an X-ray microanalyzer. It was found that the Ca/P ratio became larger after use than it was before. It was thus inferred that

the increase in Ca might have been caused by the formation of calcium carbonate.

Also, a chemical analysis of solids in the backwashings of the dephosphorization tank was conducted in order to evaluate the properties of reaction products. Calcium carbonate was found to be 35%, or half as much as that recorded in the first stage of the study, thus proving the effectiveness of sand filtration tank installation.

4.1.4 Sludge and its Disposal

Phosphate removed in the contact crystallization system makes the dephosphorization media grow, but causes no sludge. But the contact crystallization system produces sludge in the form of solids in the sand filtration tank and backwashings in the dephosphorization tank.

In the first stage of the study, sludge was developed from the dephosphorization tank alone. It was generated at a rate of 4.2 g/m^3 of effluent. In the second stage of the study, the sludge was generated from the sand filtration tank at a rate of 4.9 g/m^3 and from the dephosphorization tank at a rate of 0.45 g/m^3 , or about 5 g/m^3 in toto, which is far smaller than in any other method.

These wastes should preferably be handled from the viewpoint of the entire sewage treatment facilities, including the primary and secondary treatment, and may well be turned back to the primary settling tank. The increase in the loadings due to return sludge is calculated as shown in Table 9. (Refer to Table 9)

Table 9 Effects of backwashings

	Backwashing wastes per m ³ of effluent			Increase in loading* (%)		
	Water (m ³)	SS (g)	Phosphate (g)	Water (m ³)	SS (g)	Phosphate (g)
Backwashings from rapid sand filter	0.05	4.9	0.05	5	3.3	1.25
Backwashings from dephosphorization tank	0.01	0.45	0.05	1	0.3	1.25
Total	0.06	5.35	0.1	6	3.6	2.5

Note*: Calculated on presupposition that the influent into the primary sedimentation tank contains 150 mg-SS/lit. and 4 mg-phosphate/lit.

4.1.5 Phosphorus Balance

According to the measurements taken during the March to May period of 1979 when the dephosphorization became stabilized, the phosphorus balance was determined as shown in Figure 7. It was found that 77% of influent phosphorus was fixed in the dephosphorization tank, 16% carried away with the effluent, and about 7% removed in the form of backwashings. (Refer to Fig. 7)

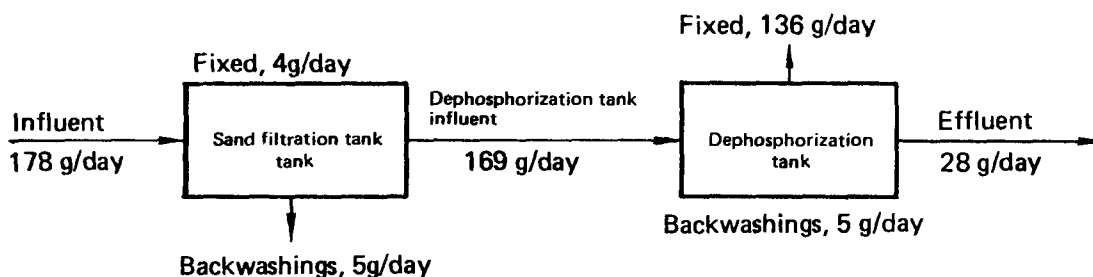


Fig. 7 Phosphate balance (Mar. to May, 1980)

Phosphorus fixed and removed in the dephosphorization tank is regarded to be in the form of calcium hydroxyapatite ($\text{Ca}_5(\text{OH})(\text{PO}_4)_3$). Namely, the solids to be accumulated upon the dephosphorization media will be 5.4 times as heavy as the phosphorus to be removed. The gravimetric growth of the dephosphorization media for a year of plant operation is calculated as follows.

$$136 \text{ g/day} \times 5.4 \times 365 \text{ days} = 268 \text{ kg/year}$$

This increase is 13.4% to 2,000 kg of dephosphorization media. No visible volumetric increase was noticed for about 10 months of operation, however. The theoretical volumetric increase is calculated to be about 4% in terms of grain size if the dephosphorization media particles are assumed to be spherical.

4.1.6 Chemical Injection Requirements

The chemicals used in the contact crystallization system are sulfuric acid for the decarbonation process and slaked lime for the pH conditioning process.

Slaked lime injection is required for the purpose of generating the calcium ions necessary for maintaining a proper crystallization velocity and also for obtaining a pH value suitable for crystallization.

It was found in basic studies that decarbonation is carried out smoothly when the pH value is maintained at 4.5.

The pH value, however, was set at 3 because the process was designed to use slaked lime alone for generating calcium ions, and the injection rate of sulfuric acid became 140 mg/lit. on the average, or an overdose of 50 to 60 mg/lit.

According to the surveys of the factors affecting dephosphorization (which will be discussed later), it was found that slaked lime was not the only usable calcium ion source.

According to practicality and economics, chemical injection may be modified as follows.

- a. Injection of sulfuric acid for adjustment of pH to 4 and for decarbonation
Alkalinity equivalent + 10 to 15 mg/lit.
The pH value is adjusted with slaked lime to 8.5.
A shortage of calcium ions is made up by inexpensive calcium sulfate.
- b. Injection of slaked lime
15 to 25 mg/lit. (depending on alkalinity)
- c. Injection of calcium sulfate (total calcium ion make-up:
40 mg/lit.)
100 to 120 mg/lit.

By applying these to the representative quality of the secondary effluent from the Morigasaki Wastewater Treatment Plant, the injection rates are calculated as follows.

. Sulfuric acid	: 80 mg/lit.
. Slaked lime	: 15 mg/lit.
. Gypsum	: 120 mg/lit.

4.1.7 Treatment System and its Control

The pilot plant findings suggest that the practical plant have the following treatment system and control.

Figure 8 shows a flow sheet for the suggested plant.
(Refer to Fig. 8)

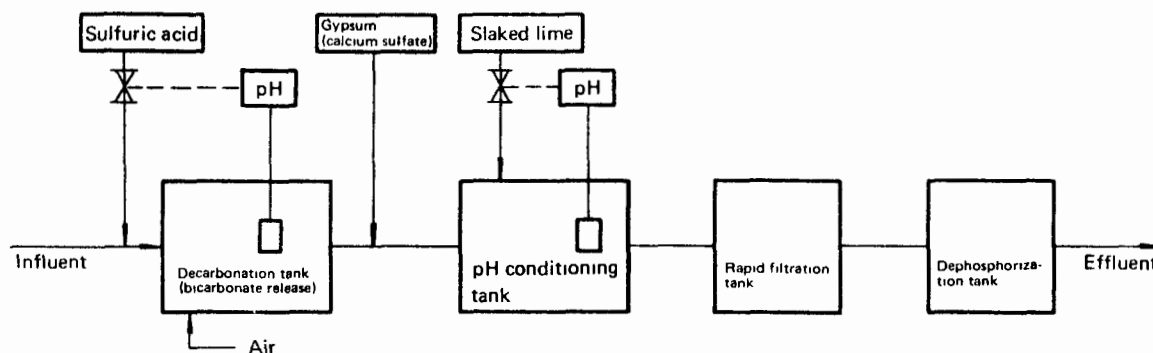


Fig. 8 New process flow

a. Control of sulfuric acid injection

Control of sulfuric acid injection rate according to the pH value of the decarbonation tank.

b. Control of aeration

Proportional flow control of air into the decarbonation tank according to a preset gas/liquid ratio.

c. Control of calcium injection

The calcium generation rate is dependent on the influent flow rate and phosphate concentration. Each individual Waste-water treatment plant has a phosphate concentration of its own, which does not change greatly. Thus, it is possible to preset the make-up requirement according to a preliminary survey. Namely, a gypsum solution is subjected to proportional flow control, and slaked lime is injected so as to control the pH conditioning tank at a preset pH value.

d. Water flow rate control

The flow rate of water through the sand filtration tank and dephosphorization tank should preferably be fixed.

e. Backwashing of sand filtration tank and dephosphorization tank

Backwashing is automatically carried out by making use of an interval timer or by detecting a head loss.

5. FACTORS INFLUENCING DEPHOSPHORIZATION

In the contact crystallization system, the factors that are considered influential to the dephosphorization reaction include pH, calcium ion concentration, carbonate ion, other coexistent ions, water temperature, and contact time.

Batch tests were conducted to evaluating these factors, followed by continuous process tests using a small-scale monitor plant annexed to the pilot plant.

5.1 MONITOR PLANT EQUIPMENT

The monitor plant has the same configuration and functions as the pilot plant, except that its 10 cm-diameter acrylic resin dephosphorization reactor is filled with dephosphorization media to a height of 1 m and that its size is reduced one-hundredth that of the pilot plant.

Three series of monitor plants were installed in parallel with the pilot plant. Each series was designed to be operable independent of the others to enable three different tests to be conducted simultaneously.

The process flow of the monitor plant is as illustrated in Figure 9. (Refer to Fig. 9)

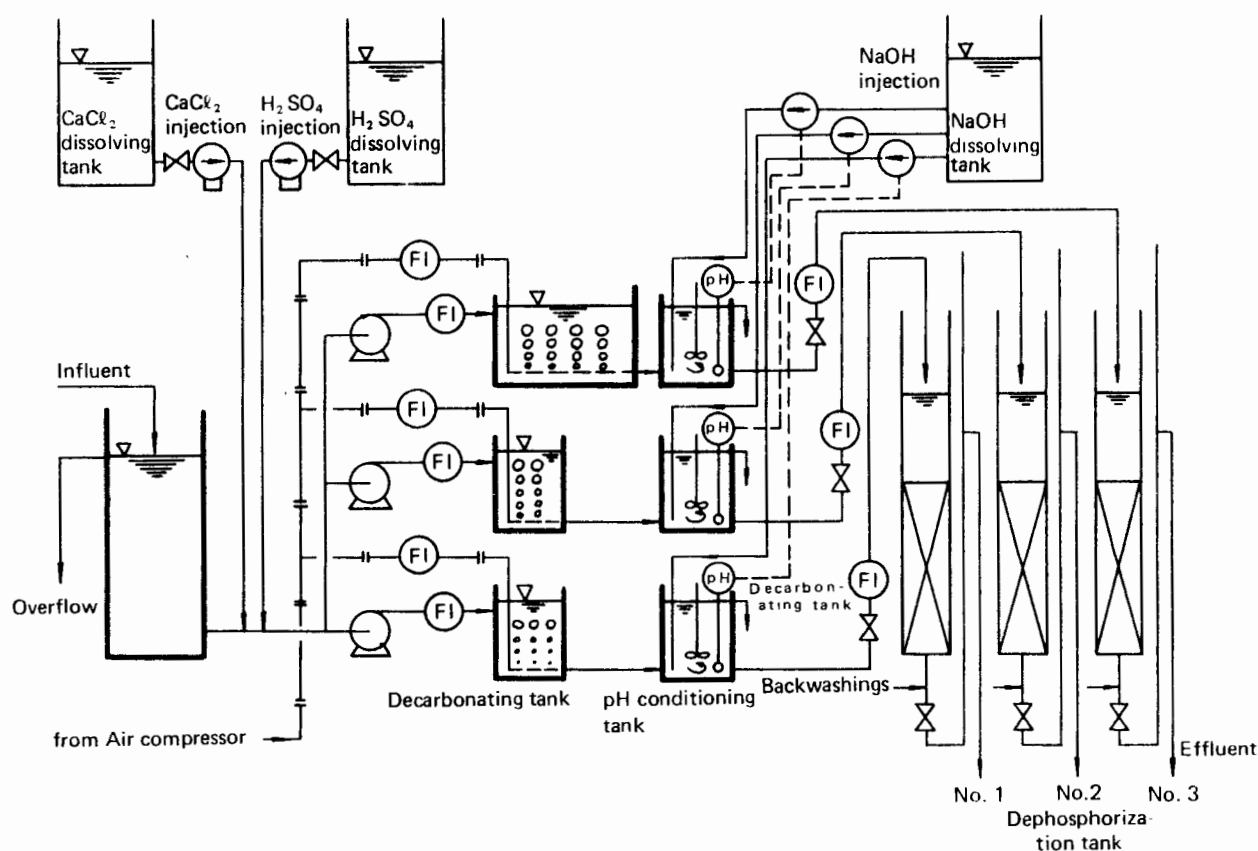


Fig. 9 Monitor plant process flow

5.2 EFFECTS OF CARBONATES

Upon empirical corroboration that the presence of carbonate ions has a significant bearing upon the velocity of the reaction between phosphate ions and calcium ions to produce sediment, Zoltek (JWPCF, Nov. 1974) and Ferguson (JWPCF, Apr. 1973) proposed the following empirical formula.

$$K_0 = \frac{k}{(\text{CO}_{3T})^n}$$

Where, CO_{3T} : Total-carbonate concentration (mols/m^3)

k: Proportional constant

n: Exponent (1.00 to 1.28)

K_0 : Dephosphorization velocity coefficient
($\text{m}^{3n-2}/\text{mol}^{n-1} \cdot \text{h}$)

According to this formula, it is found that the rate at which calcium phosphate is produced is in inverse proportion to the carbonate ion concentration; namely, that the presence of carbonate ions seriously affects dephosphorization.

On the other hand, if calcium ions are injected into a liquid in which phosphate and carbonate coexist in order to crystallize apatite, apatite and calcium carbonate will crystallize concurrently to cover up the surfaces of, and hence degrade the activity of, the dephosphorization media. This fact was confirmed during the first stage of study on the pilot plant.

For these reasons, the following tests were conducted to further clarify the effect of carbonate concentration upon dephosphorization.

5.2.1 Relationships between Carbonate and Effluent Phosphate Concentrations

By making use of the monitor plant, the carbonate concentration in the influent was changed in six steps over a range of 10 to 50 $\text{mg-CO}_2/\text{lit.}$, and the influent was run at a space velocity (SV) of 5/hr. to compare the respective phosphate concentrations in the effluent. (Refer to Fig. 10) As shown in Figure 10, there

	Test conditions	Passing time (H)
Test 1	Concentration of carbonate in the influent 50mg/l, 30mg/l, 20mg/l	700
Test 2	40mg/l, 20mg/l, 10mg/l	700

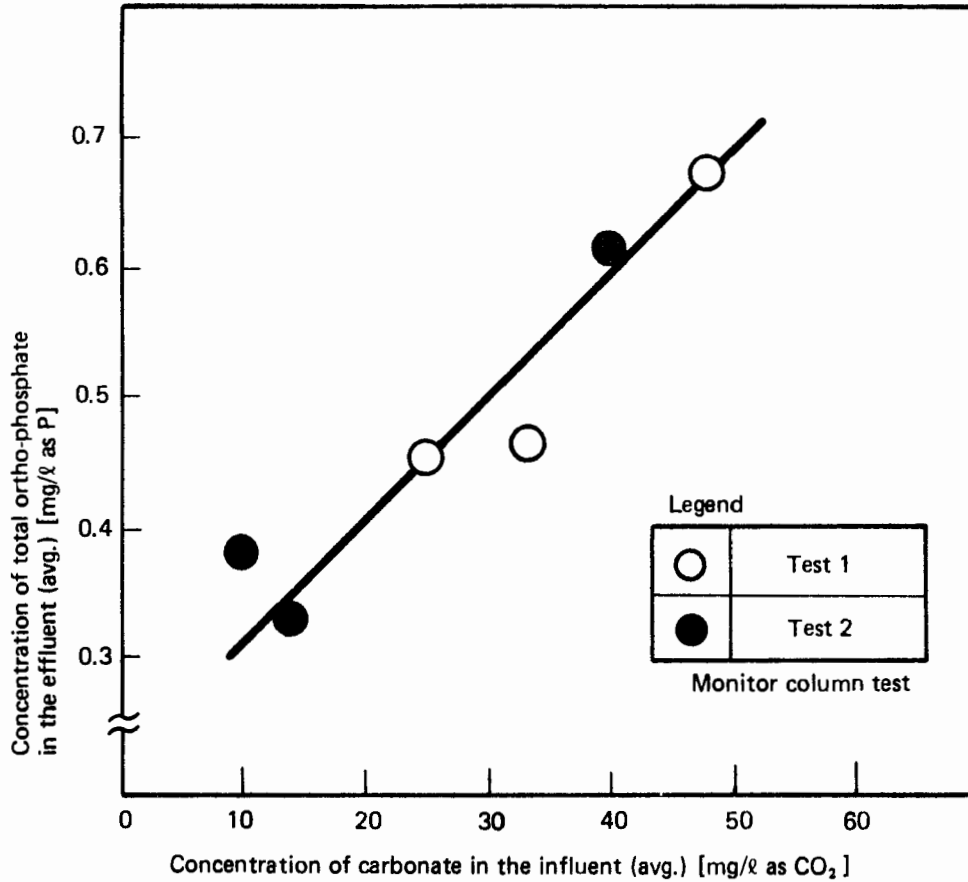


Fig. 10 Relationship between carbonate concentration in the influent and phosphate concentration in the effluent

was a proportional relationship between the phosphate concentration in the dephosphorization tank influent and that in the effluent; it was confirmed that the phosphate concentration in the effluent could be reduced linearly with increase in the degree of decarbonation. It was also found from Figure 10 that it is necessary to decarbonate the influent to about 10 mg-CO₂/lit. in terms of CO₂ if the phosphate concentration in the effluent is to be reduced to about 0.3 mg-P/lit.

5.2.2 Effects of Non-decarbonated Carbonates

While it was found, as discussed in 5.2.1, that the omission of the decarbonation process detrimentally effects the effluent, a dephosphorization process without the decarbonation step (hereinafter referred to as the direct process) was examined using the monitor plant to investigate the effects of pH and calcium ion concentration in the influent of the dephosphorization tank upon dephosphorization.

In the direct process, the effects of carbonates in the influent are inevitably present, and the experimental conditions were formulated accordingly.

In the direct process (1), the influent was run into the dephosphorization tank after its pH value was adjusted to 9.5 with slaked lime. This high pH value was intended to increase the crystallization velocity of apatite to counteract the decline in crystallization velocity due to carbonates, and at the same time to minimize the supply of Ca^{2+} because a high ratio of CO_3^{2-} present at this level of pH value made the development of calcium carbonate easier.

In the direct process (2), the pH value was set at 9.0 to defy the formation of calcium carbonate.

The decline in the crystallization velocity of apatite was improved by injecting calcium chloride to set Ca^{2+} concentration at 60 mg/lit.

The dephosphorization media charged into the monitor plant column was fetched from the dephosphorization media being used in the pilot plant.

In the direct process, the influent was the same as the filtrate used in the pilot plant that was obtained by filtering the secondary effluent of the Morigasaki Wastewater Treatment Plant through a sand filter.

In a standard process taken as a control, the influent for the pilot plant dephosphorization tank was employed.

The test period and test conditions are listed in Table 10. (Refer to Table 10)

Table 10 Test conditions

Test No.	Name		Test period	Flow velocity		Design pH	Ca ²⁺ injection rate (mg/l)	Chemicals used
				SV, 1/hr.	LV, m/hr.			
Control	Standard process Dephosphorization tank		Feb 6 – Mar. 31	2.5	2.5	8.5	50	H ₂ SO ₄ Ca(OH) ₂
			Apr 1 – Mar. 16	2.5	2.5	8.5	60	
1	Direct process (1)	Dephosphorization tank	Feb. 6 – Mar. 31	2.5	2.5	9.5	30	Ca(OH) ₂
		Sand filter		5.0	5.0	—	—	
2	Direct process (2)	Dephosphorization tank	Apr. 1 – May 16	2.5	2.5	9.0	60	CaCl ₂ Ca(OH) ₂
		Sand filter		5.0	5.0	—	—	

Figure 11 and Table 11 show the test results of direct process (1), direct process (2) and the standard process. (Refer to Fig. 11, Table 11)

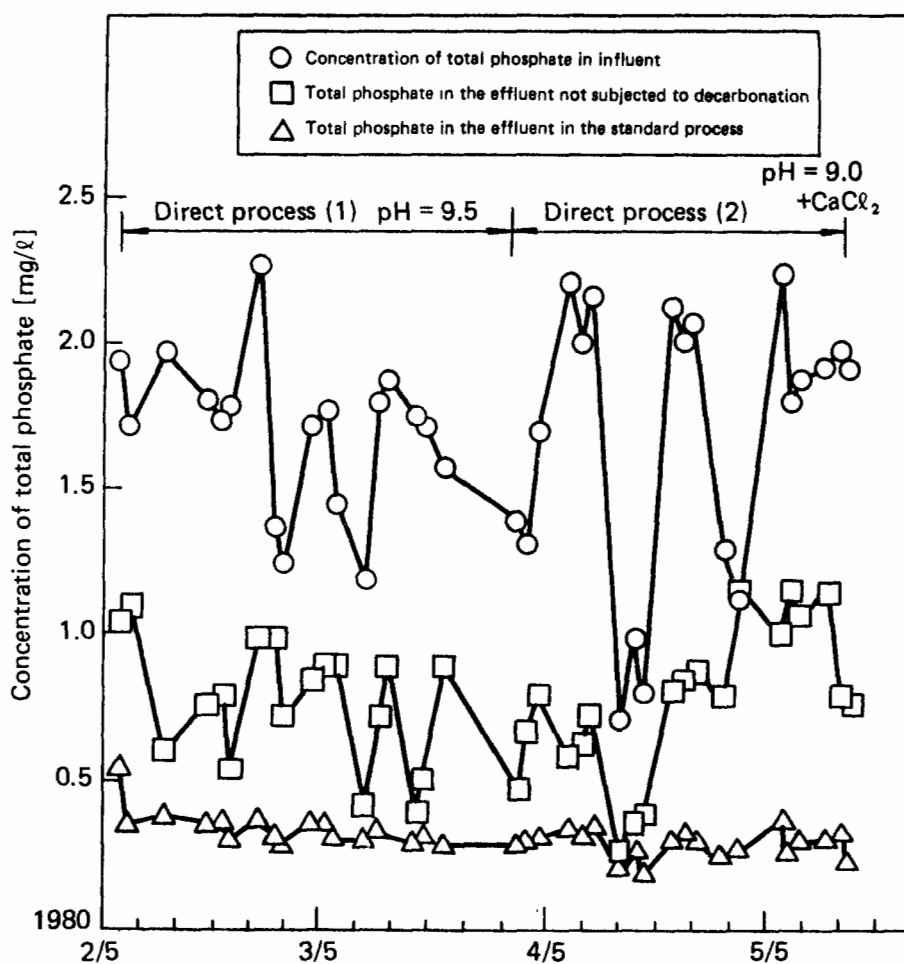


Fig. 11 Comparison of phosphate concentration in the effluent for standard process and direct process

Table 11 Phosphate concentrations in the sand-filtered influent and effluent in both processes

Test No.	Test period	Influent	Standard process		Direct process	
			Filtrate of influent through sand filter	effluent	Filtrate of influent through sand filter	effluent
1	Feb. 6 – Mar. 31	1.19 ~ 2.24 *(1.70)	1.13 ~ 2.16 (1.65)	0.28 ~ 0.37 (0.32)	0.27 ~ 1.36 (0.96)	0.38 ~ 1.10 (0.77)
2	Apr. 1 – May 16	0.82 ~ 2.18 (1.64)	0.66 ~ 2.02 (1.53)	0.21 ~ 0.35 (0.29)	0.60 ~ 1.85 (1.19)	0.35 ~ 1.15 (0.75)
Remarks		Filtrate of secondary effluent through a quick sand filter	Decarbonated with pH value of influent adjusted at about 3.0, and then adjusted with Ca(OH)_2 to a pH value of about 8.5.		Direct injection of chemicals into the influent (1) Ca(OH)_2 pH \approx 9.5 (2) CaCl_2 Ca(OH)_2 pH \approx 9.0	

Note: The values in parentheses denote the average values within the test period.

In direct process (1) where the pH value was set at 9.5, the phosphate concentration was higher and more changeable compared with the standard process, making dephosphorization unstable. This was because despite a high pH value set for an increased susceptibility to crystallization of apatite, coexistent carbonates reduced dephosphorization rate to increase the phosphate concentration in the effluent. The formation of calcium carbonate increased the volume of SS. (Refer to Table 12)

Table 12 SS* discharged from both processes

Item	Standard process		Direct process (1)		Direct process (2)	
	Sand filter	Dephosphorization tank	Sand filter	Dephosphorization tank	Sand filter	Dephosphorization tank
SS discharged from each process [g/m ³]	5.0	0.29	13.4	1.05	8.61	0.40
SS discharged from the entire process [g/m ³]	5.29		14.5		9.01	

* SS per m³ of effluent

In direct process (2) in which the pH value was set at 9.0 with Ca^{2+} set at 60 mg/lit., the SS volume was reduced, but the phosphate concentration in the effluent remained almost the same as direct process (1), making dephosphorization unstable as with direct process (1).

Unless decarbonation is performed, carbonates degrade dephosphorization efficiency regardless of the pH value and Ca^{2+} conditions.

5.2.3 Effects of carbonate ions in the liquid upon dephosphorization reaction

Batch tests were conducted to examine the effects of carbonate ions in the liquid upon dephosphorization reaction. Monopotassium phosphate was added to deionized water to make an aqueous solution of 10 mg/lit. in terms of phosphate. Then, carbonate ions were added in steps over a range of 10 to 1,000 mg/lit. This was followed by an addition of dephosphorization media in a powder form at a rate of 1,000 mg/lit.

The mixture was stirred for 2 to 20 hrs., and the phosphate ion concentration of the filtrate was measured.

The relationship between the carbonate ion injection and dephosphorization efficiency was as shown in Figure 12.

(Refer to Fig. 12)

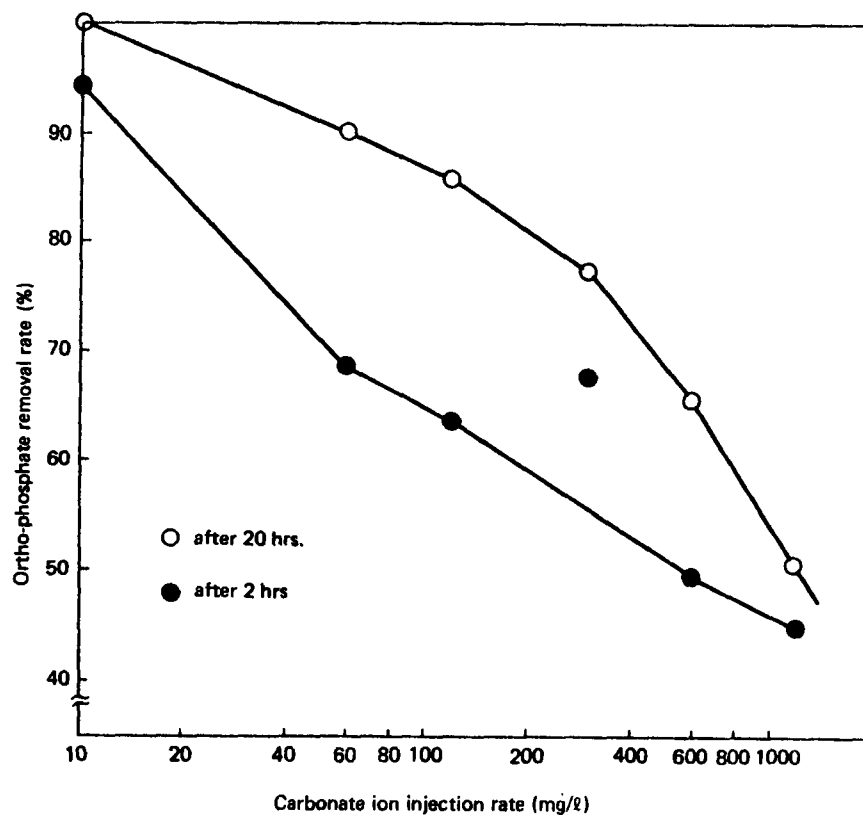


Fig. 12 Effect of carbonate ions (CO_3^{2-}) on the removal of ortho-phosphate

As carbonate ions coexist, it is evident that they block the dephosphorization reaction.

These experiments demonstrated that carbonates have a severe negative effect upon dephosphorization, dictating decarbonation as a process essential to dephosphorization.

5.3 EFFECTS OF CALCIUM IONS

With increase in the addition of Ca^{2+} , the degree of supersaturation is increased thus increasing the crystallization reaction velocity. The effects of Ca^{2+} addition upon dephosphorization were investigated by making use of the monitor plant.

The secondary effluent of the Morigasaki Wastewater Treatment Plant used as an influent for the monitor plant originally contained about 30 mg/lit. of Ca^{2+} , and was added with Ca^{2+} at a rate of 0, 20, and 50 mg/lit. in the form of calcium chloride. The pH value was adjusted to 8.5 with caustic soda, and the influent was run for about 700 hrs. The results were as shown in Figure 13. (Refer to Fig. 13)

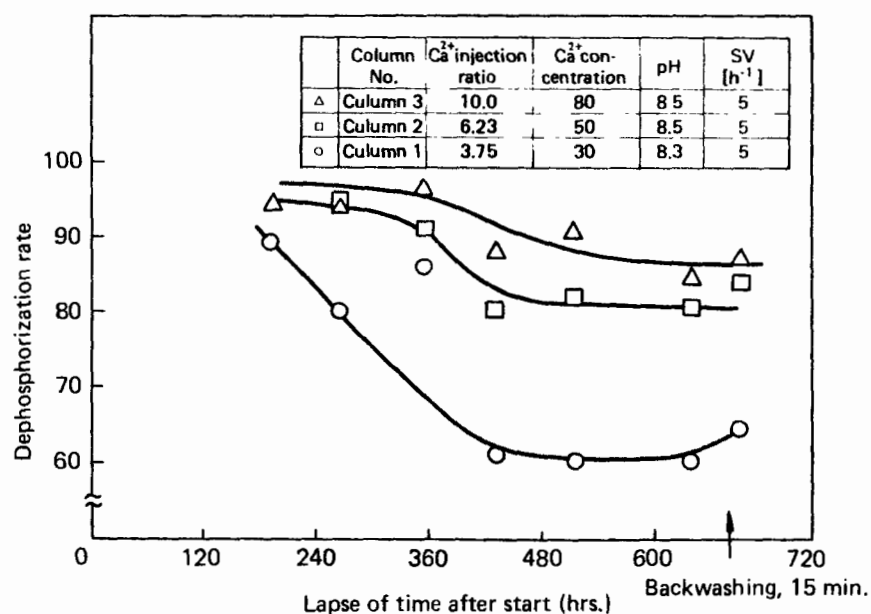


Fig. 13 Effect of calcium ions on the dephosphorization

In every case, the dephosphorization efficiency decreased over time, and leveled off to an equilibrium point.

Without the injection of Ca^{2+} , the reduction in dephosphorization efficiency was noticeable; within 400 hrs., the efficiency was reduced to 60%. When Ca^{2+} was injected at a rate of 20 and 50 mg/lit., the larger the injection rate, the higher the ultimate equilibrium point of dephosphorization. In the tests, even when the Ca^{2+} injection rate was 20 mg/lit., the dephosphorization efficiency could be maintained at as high a level as 80%.

5.4 EFFECTS OF pH

A liquid containing phosphate was adjusted to various pH values, added with dephosphorization media in a powder form, and stirred for the purpose of conducting batch tests to investigate the effects of pH value on dephosphorization. The results were as shown in Figure 14. The dephosphorization efficiency increased with increase in the pH value; in the tests, the maximum dephosphorization efficiency was achieved at a pH value of around 9.0. (Refer to Fig. 14)

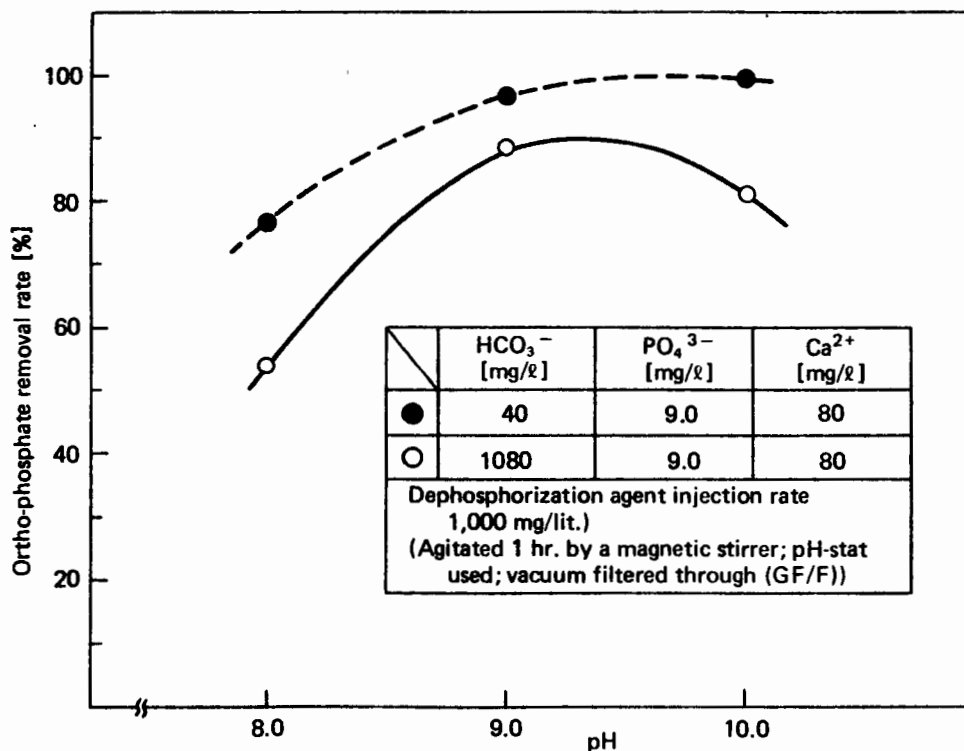


Fig. 14 Effect of pH value on ortho-phosphate removal rate

When a large amount of carbonates was contained in the liquid, the dephosphorization efficiency reduced in a high pH range, demonstrating that the carbonates have an overpowering effect upon the dephosphorization efficiency.

5.5 EFFECTS OF OTHER IONS

The effects of F^- , Cl^- and SO_4^{2-} were investigated according to batch tests. The relationships between the anion injection rates and dephosphorization efficiency were as shown in Figure 15.

(Refer to Fig. 15)

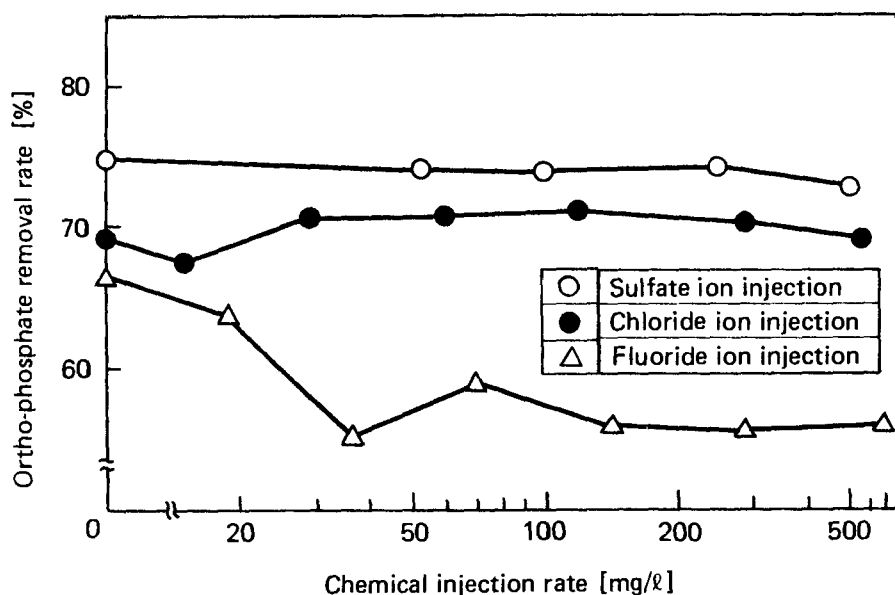


Fig. 15 Effect of sulfate ions, chloride ions and fluoride ions on the removal of ortho-phosphate

It was found that these anions have little effect upon the dephosphorization reaction in the contact crystallization process.

5.6 EFFECTS OF WATER LOADING (SV)

The effects of the change in the flow rate of influent upon the dephosphorization efficiency of the contact crystallization process treating the secondary effluent of the waste water treatment plant were investigated. In the pilot plant, the space velocity was set at a constant value of about 2.5 times/hr. In the practical plants, however, the space velocity is changeable. Thus, the tests were conducted with respect to space velocities of 2.5, 5.0 and 10.0 times/hr. for a running time of 700 hrs., to investigate the effects of water loadings

upon the dephosphorization efficiency. The results were as shown in Table 13. (Refer to Table 13)

Table 13 Influence of water loadings

Space velocity (SV = 1/hr.)	Mean value of total phosphate concentrations in the effluent (mg/l as P)	Mean value of phosphate removal rates (%)
2.5	0.32 * (0.16 ~ 0.45)	85.9
5	0.34 (0.19 ~ 0.57)	84.7
10	0.53 (0.20 ~ 0.86)	74.5

* The values in parentheses show the range from the minimum to the maximum value.

When the space velocity was set at a design value of 2.5, the phosphate concentration in the effluent was held low, and dephosphorization could be carried out stably for an extended period. But when the space velocity was set at 5.0, the phosphate concentration in the effluent changed over a wide range detrimental to dephosphorization, though its average was almost the same as when the space velocity was 2.5.

When the space velocity was set at 10.0, the phosphate concentration in the effluent increased unfavorably. It was thus concluded that the space velocity can be increased to twice the design value.

6. SUMMARY

The results of the two and a half year study are summarized below.

- (1) The contact crystallization process has a dephosphorization efficiency comparable to the aluminum sulfate coagulation sedimentation process.
- (2) The contact crystallization process is practically stable and easy to operate.
- (3) The amount of sludge generated is practically negligible.
- (4) The contact crystallization process necessitates the decarbonation process.
- (5) The contact crystallization process is superior to the coagulation process from the economic viewpoint if the disposal of sludge is to be considered.

ADVANCED TREATMENT PROJECT FOR EUTROPHICATION CONTROL IN LAKE BIWA

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1. OUTLINE OF LAKE BIWA

Lake Biwa, located in the Kinki area, one of the industrial and cultural centers in Japan, is the largest lake in Japan. Its surface area is 674 Km^2 , or about one-sixth of the total area of Shiga Prefecture in which the lake exists, and is about 1/120 as large as Lake Superior bordering on both the United States and Canada.

Lake Biwa has a storage capacity of about 27,500 million m^3 , or about 80 years' worth of drinking water for 3 million people in Osaka City.

It is not clear when the lake was born, but it is said that the lake may have been created by a major crustal movement about 4 million years ago.

Lake Biwa consists of North Lake and South Lake, and is said to have been named after the "Biwa", a traditional Japanese stringed instrument, which it resembles.

Lake Biwa receives no less than 437 major rivers, but empties into the Seta River alone. Really, Lake Biwa is a natural dam.

The water running into the lake is conveyed by complex currents, mixed with lake water, and empties into the Seta River on the south of the lake after 10 to 15 years of detention.

Table 1-1 shows the principal particulars of Lake Biwa.

Table 1-1 Outline of Lake Biwa

Area	673.9 km ²	
Circumference	235 km	
Water depth	Max.	103.58 m
	Avg. North lake	43.0 m
	Avg. South lake	4.0 m
Storage capacity	27,500 mil. m ³	
Water temperature	Max.	27°C
	Min.	6°C
Catchment area	3,848 km ²	
Annual rainfalls	Max.	2,656 mm (in 1896)
	Min.	1,352 mm (in 1939)
	Avg.	1,920 mm (in 1894~1967)
Annual discharge	Max.	9,100 mil. m ³ (in 1896)
	Min.	3,000 mil. m ³ (in 1939)
	Avg.	5,300 mil. m ³ (in 1896~1939)

The functions of Lake Biwa as a water resource are manifold as summarized below.

(1) Water source

With its ample yet stable supply of water unaffected seasonally, Lake Biwa is vital to the 13 million citizens living in the Kinki area. Fig. 1-1 shows the water uses of Lake Biwa.

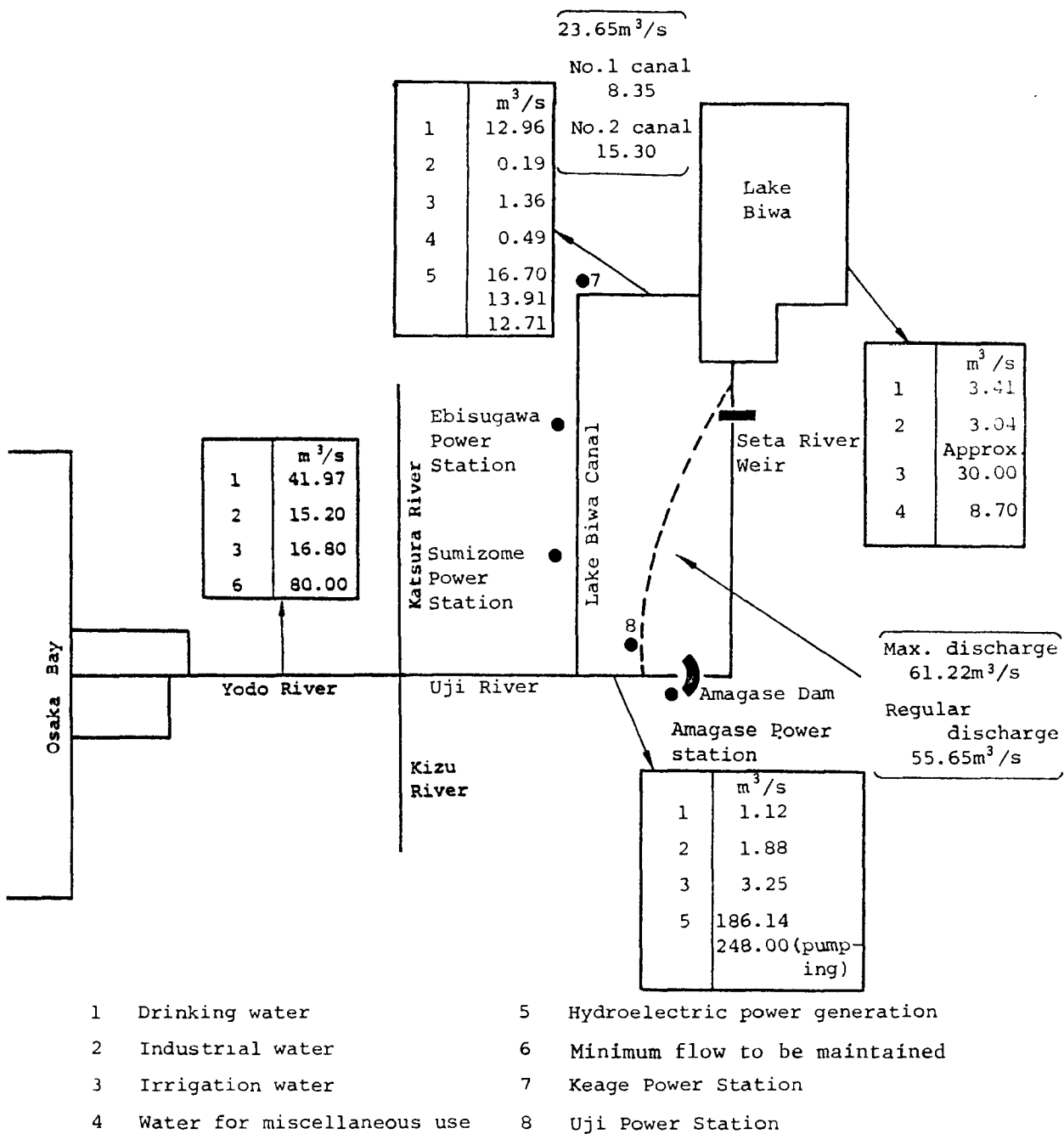


Fig. 1-1 Water use of Lake Biwa

(2) Fishing grounds

About 40 species of freshwater shellfish and about 60 species of freshwater fish inhabit Lake Biwa. In fact, Lake Biwa is a natural freshwater aquarium.

Commercial catches of freshwater products amount to several thousand million yen a year. 95% of freshwater pearls are exported to the Middle and the Near East and other countries. In addition, Lake Biwa accounts for about 70% of young sweet fishes distributed throughout the country.

(3) Energy source for hydropower station

There are five hydropower stations using the water from Lake Biwa.

(4) Tourist resources

There are many prehistoric remains around Lake Biwa, which together with beautiful scenes of the lake attract visitors throughout the year.

(5) Subject of academic study

Lake Biwa is said to be the third or fourth oldest lake in the world. It has a complex geological structure, and sinks about 2 mm every year.

Lake Biwa is a treasure-house of animals and plants, and its floor contains relics of the prehistoric ages. Accordingly, Lake Biwa is treasured as a subject of academic studies.

As Lake Biwa is playing an important role as a water resource, the central Government instituted the Lake Biwa Comprehensive Development Project in 1972 for the purposes of water quality conservation, protection of the natural environment, flood control, effective use of water resources, etc.

2. EUTROPHICATION IN LAKE BIWA

From around 1965, the water quality of Lake Biwa started to deteriorate. In 1972 to 1973, Lake Biwa's water quality was critical. Since then, the lake has been reeling between short recovery and relapse. Table 2-1 shows the morbid changes of Lake Biwa due to eutrophication.

Table 2-1 Transition of Lake Biwa

Stage	Year			Symptoms	Morbidity and troubles	Remarks	Estimated water quality	
		North lake	South lake				T - N	T - P
I	Latter half of 1950's	1955 to 1963		Clean and clear	nil		mg/l 0.15	mg/l 0.004
		1955 to 1963	1955 to 1960				~ 0.20	~ 0.007
II	First half of 1960's	1959 to 1966		Pollution set in.	Failure of filter		0.17	0.005
		1960 to 1966	1959 to 1963				~ 0.23	~ 0.010
III	First half of 1960's	1962 to 1968		Pollution progressed. Bottom quality also got fouled.	Frequent filter trouble. Decline of Corbiculidae.	P loading reached a critical level. Closterium proliferated.	0.18	0.006
		1964 to 1968	1962 to 1968				~ 0.24	~ 0.010
IV	Latter half of 1960's	1967 to 1971		Pollution aggravated violently.	Trouble of city water filter system; smelly Decline of tourist population.	Pollution apparent to the majority of Shiga citizens. Staurostrum proliferated.	0.20	0.009
		1967 to 1970	1967 to 1971				~ 0.26	~ 0.013
V	First half of 1970's	1970 to 1975		Pronounced pollution; worst water quality recorded.	Spread of odors. Further decline of shellfish.	Wide-spread concern over environmental problems.	0.20	0.009
		1970 to 1974	1971 to 1975				~ 0.25 South lake (0.23)	~ 0.012 South lake (0.010)
VI	Latter half of 1970's	1972 to ~		Remarkable pollution	South lake bathing beach closed. Red tides developed.	Most of Shiga citizens had grave concern about the lake pollution.	in 1975 0.28 South lake (0.33)	in 1975 0.010 South lake (0.015)

In Stage I, the water was still comparatively clear, and the phytoplanktons were changing from Diatomeae to Closterium. No particular evil symptoms were found, and the north lake showed T-N of 0.15 to 0.20 mg/l and T-P of 0.004 to 0.007 mg/l.

In Stage II, the pollution set in. Closterium bloomed on and off, clogging up the filter system for the city water.

The north lake showed T-N of 0.17 to 0.23 mg/l, reaching nearly the critical level of eutrophication. T-P was 0.005 to 0.010 mg/l.

In Stage III, the pollution worsened, and Closterium and Staurostrum became rampant. As a result, filter troubles were frequent. In addition, the bottom quality was aggravated, reducing DO to affect fishery. The Corbiculidae began to decrease. In the north lake, T-N rose a little to 0.18 to 0.24 mg/l from the level in stage II, but often exceeded 0.20 mg/l. T-P was in the range of 0.006 to 0.011 mg/l.

In Stage IV, the water pollution was exacerbated further, and became apparent to more than half of the people living in Shiga Prefecture. Closterium was overwhelmed by Staurostrum, and NO_3^- -N increased in both the surface and bottom of the lake.

The city water filter system failed frequently, and the lake water became smelly, detracting from the tourist value of the lake; the number of the tourists visiting the lake began to fall, accordingly. In the north lake, T-N became 0.20 to 0.26 mg/l., and T-P 0.009 to 0.013 mg/l. On the other hand, the south lake registered T-N of 0.21 to 0.29 mg/l and T-P 0.009 to 0.011 mg/l.

In Stage V, the water pollution became pronounced, and was accompanied by a high incidence of filter troubles and complaints lodged against foul tap water. Anomalous growth of Diatomeae and Chlorophyceae was seen on and off.

In the north lake, T-N was in the range of 0.2 to 0.25 mg/l. while T-P was in the range of 0.009 to 0.012 mg/l. On the other hand, the south lake showed T-N of 0.23 to 0.32 mg/l and T-P of 0.010 to 0.016 mg/l.

In Stage VI, the water pollution was visibly quite clear. Pre-dominant among the phytoplanktons was Pediastrum. Red tides occurred, and part of the bathing area in the south lake had to be closed. Fig. 2-1 shows the zones affected by red tides in May to June, 1977 to 1978.

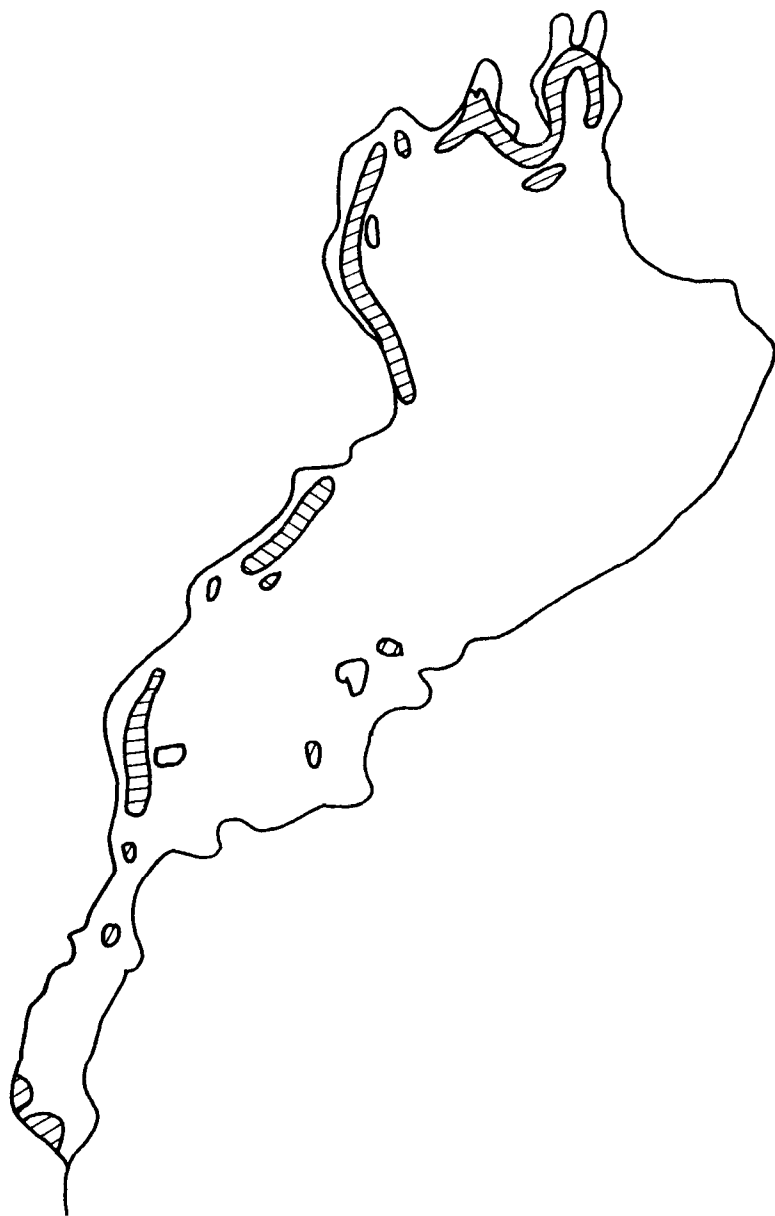


Fig. 2-1 (a) Major red tide affected zones during
the period from May to June, 1977

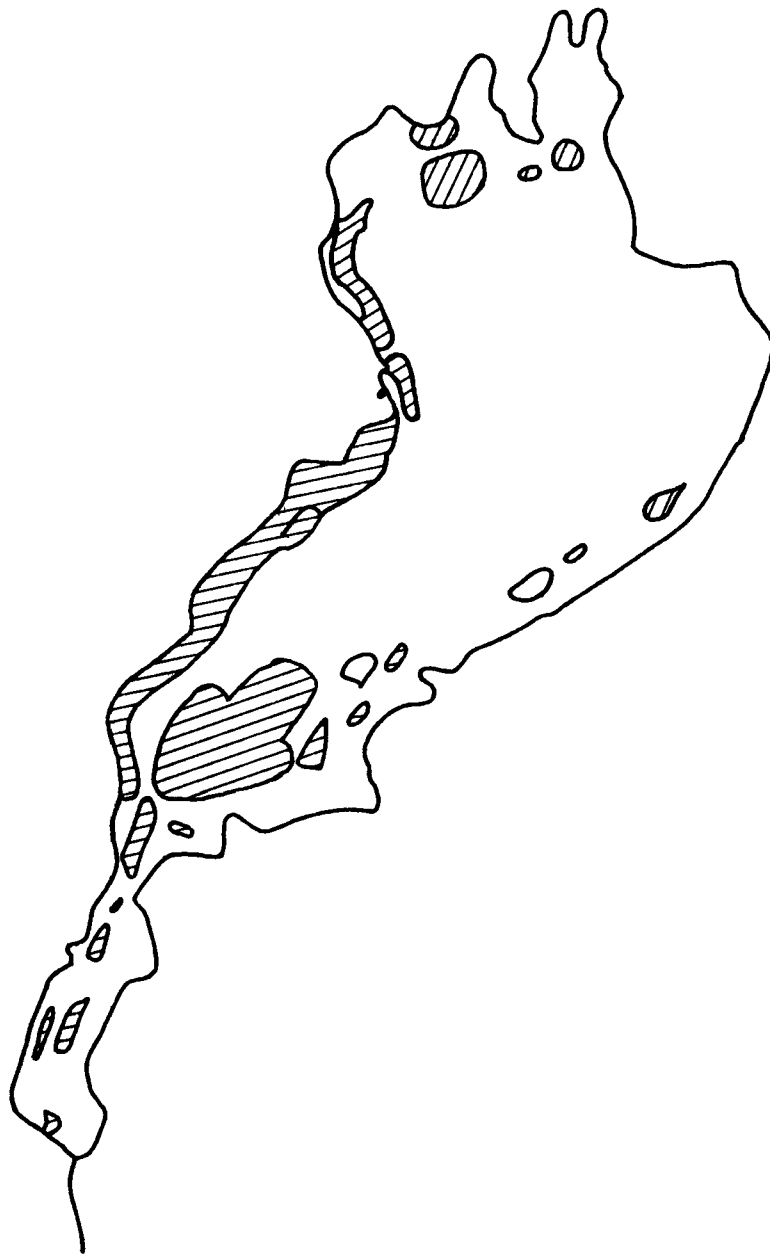


Fig. 2-1 (b) Major red tide affected zones during the period from May to June, 1978

In 1977, the red tides were seen at twenty-one places, and in 1978 they spread to thirty-nine places.

Table 2-2 shows estimated N-and P-loadings in the catchment area of Lake Biwa.

Table 2-2 N- and P-loadings generated in the catchment area of Lake Biwa

(Kg/day)

	N						P					
	1960	1965	1970	1975	1980	1985	1960	1965	1970	1975	1980	1985
North lake	12,627	13,370	15,342	16,875	19,397	21,429	778	1,187	1,804	1,716	2,096	2,481
South lake	2,336	2,493	3,673	4,579	5,797	7,227	229	352	696	613	822	1,058
Lake Biwa	14,963	15,863	19,015	21,454	25,194	28,656	1,007	1,539	2,500	2,329	2,918	3,539

(Source: Shiga P.G. Estimation)

In 1980, the sources of pollution loadings were as shown in Table 2-3; domestic waste and industrial wastes accounted for 58% of total N-loadings and 81.6% of P-loadings, demonstrating how significant human activities were to the pollution of the lake.

Table 2-3 Sources of loadings in 1980

Source	N		P	
	Loadings	Ratio	Loadings	Ratio
Domestic wastes	kg/day 8,893.5	% 35.3	kg/day 1,362.7	% 46.7
Industrial wastes	5,719.0	22.7	1,018.4	34.9
Agricultural and livestock farming wastes	4,736.5	18.8	329.7	11.3
Nonpoint source	5,845.0	23.2	207.2	7.1
Total	25,194.0	100.0	2,918.0	100.0

(Source: Shiga P.G. Estimation)

Domestic wastes was responsible for 35.3% of total N-loadings and 46.7% of total P-loadings.

Of the P-loadings due to domestic wastes, 39% was accounted for by night soil, 38% by phosphorus syndets, and the remainder by miscellaneous wastes in 1975.

Of the total generated loadings, about two thirds are expected to enter Lake Biwa. The relationship between these influent loadings and plankton is as shown in Fig. 2-2.

[North lake]

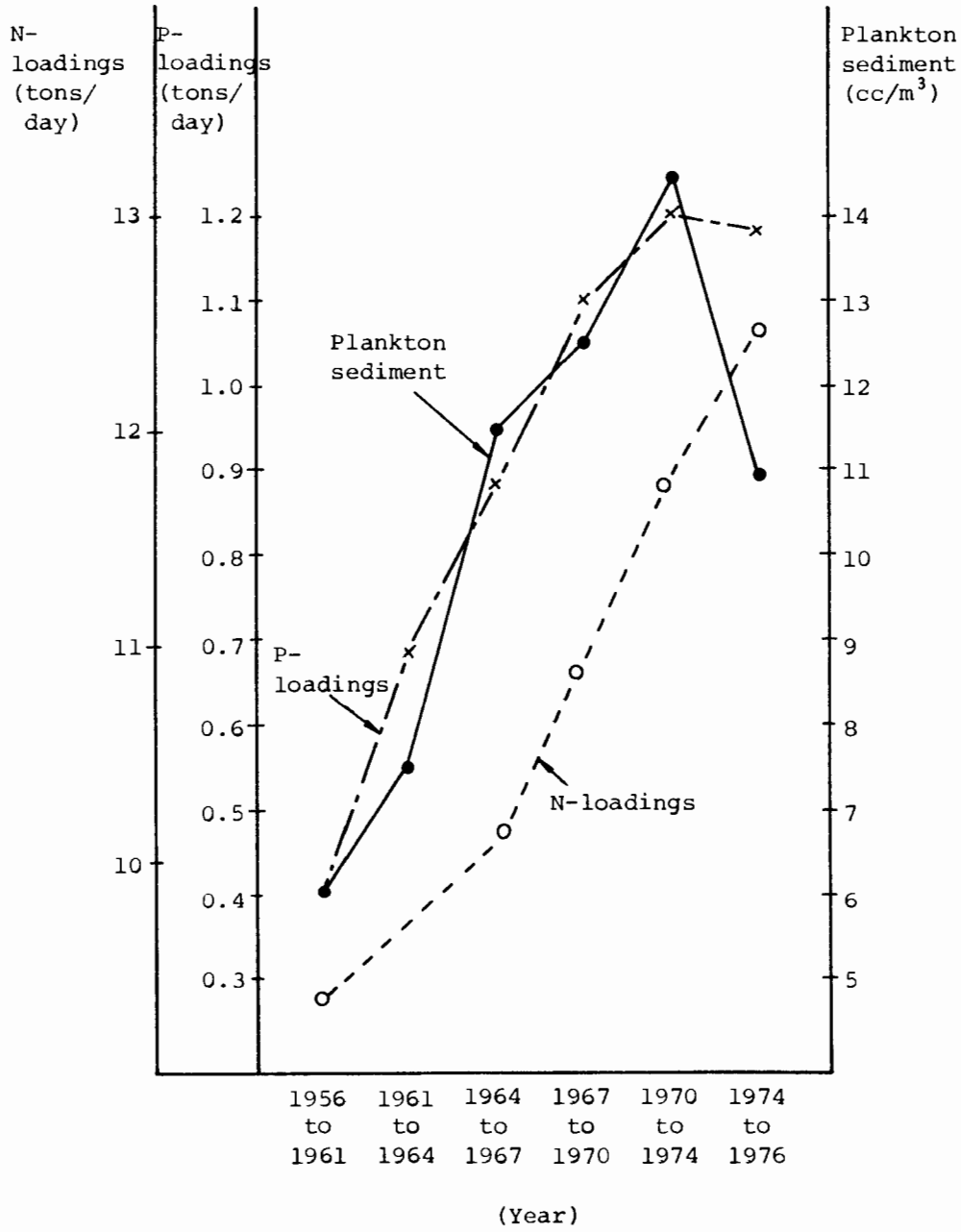


Fig. 2-2 (a) Influent loadings vs. plankton

[South lake]

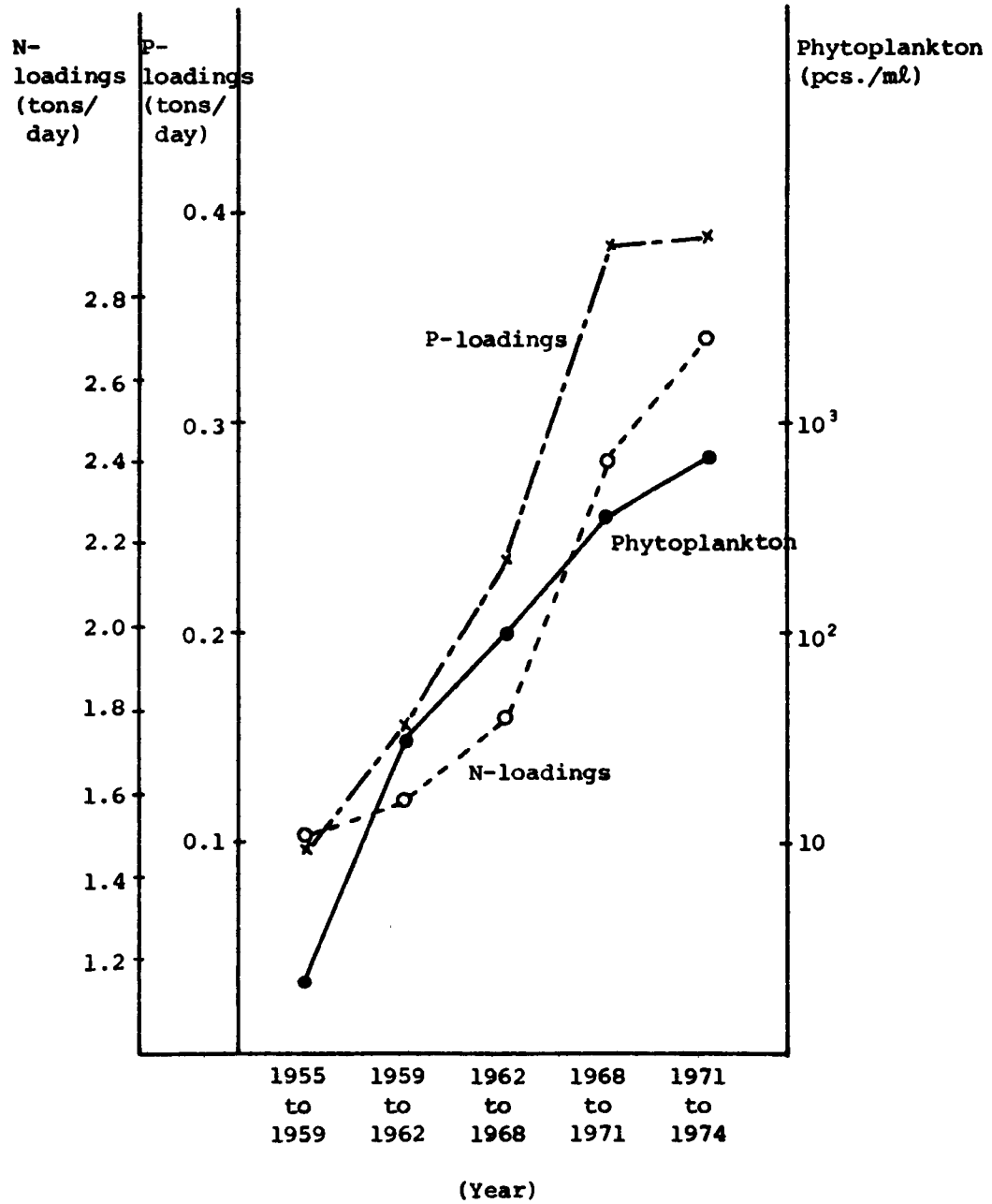


Fig. 2-2 (b) Influent loadings vs. plankton

According to a forecast of the lake water quality based on the estimated N-and P-loadings shown in Table 2-2, it is estimated that in 1985, the eutrophication of Lake Biwa will be even more aggravated, and that the north lake will show T-N of 0.31 mg/l and T-P of 0.017 mg/l while the south lake will have T-N of 0.40 mg/l and T-P of 0.023 mg/l.

The Shiga Prefectural Government, concerned about the situation, promulgated in 1979 the Regulations Concerning Eutrophication Control in Lake Biwa with a view to recovering the lake water quality to the 1965 to 1970 level, and initiated various measures against eutrophication.

These measures were aimed to achieve the target values shown in Table 2-4 by reducing the estimated N-and P-loadings in 1985 by about 25% and 50%, respectively.

Table 2-4 Target water quality for Lake Biwa

	(mg/l)	
	T-N	T-P
North lake	0.25	0.010
South lake	0.30	0.015

(Source: Shiga P.G.)

The implementation programs of the regulations are wide-varied. Among them are effluent controls for the municipal wastewater treatment plants, works and other establishments, and the first-ever prohibition in Japan of the use of phosphorus syndets, obtaining the attention of the public.

According to the effluent standards set for the municipal wastewater treatment plants, T-N is temporarily set at 20 mg/l, and T-P is 0.5 mg/l for new sources and 1 mg/l for existing sources.

As regards the phosphorus syndets, their sale and use in Shiga Prefecture is totally banned. In addition, the citizens in Shiga Prefecture are also forbidden from accepting syndets as offerings from outside the prefecture. The announcement of this tight control became a controversial issue at the time.

3. LAKE BIWA REGIONAL SEWERAGE PROGRAM

The Shiga Prefectural Government works out a comprehensive Lake Biwa regional sewerage program in 1971, and undertook a sewerage construction project according to the program.

The promulgation of the Regulations Concerning Eutrophication Control in Lake Biwa gave an impetus to the construction of sewage works indispensable for the reduction of N- and P-loadings for which domestic wastes and industrial effluents are most responsible.

According to the comprehensive Lake Biwa sewerage program, Shiga Prefecture is divided into five blocks - four regional sewerage systems and the Ohtsu municipal sewerage system. As regards Oki Island within Lake Biwa, a specific environmental protection municipal sewerage system has already been adopted. An outline of the comprehensive sewerage program is given in Fig. 3-1 and Table 3-1.

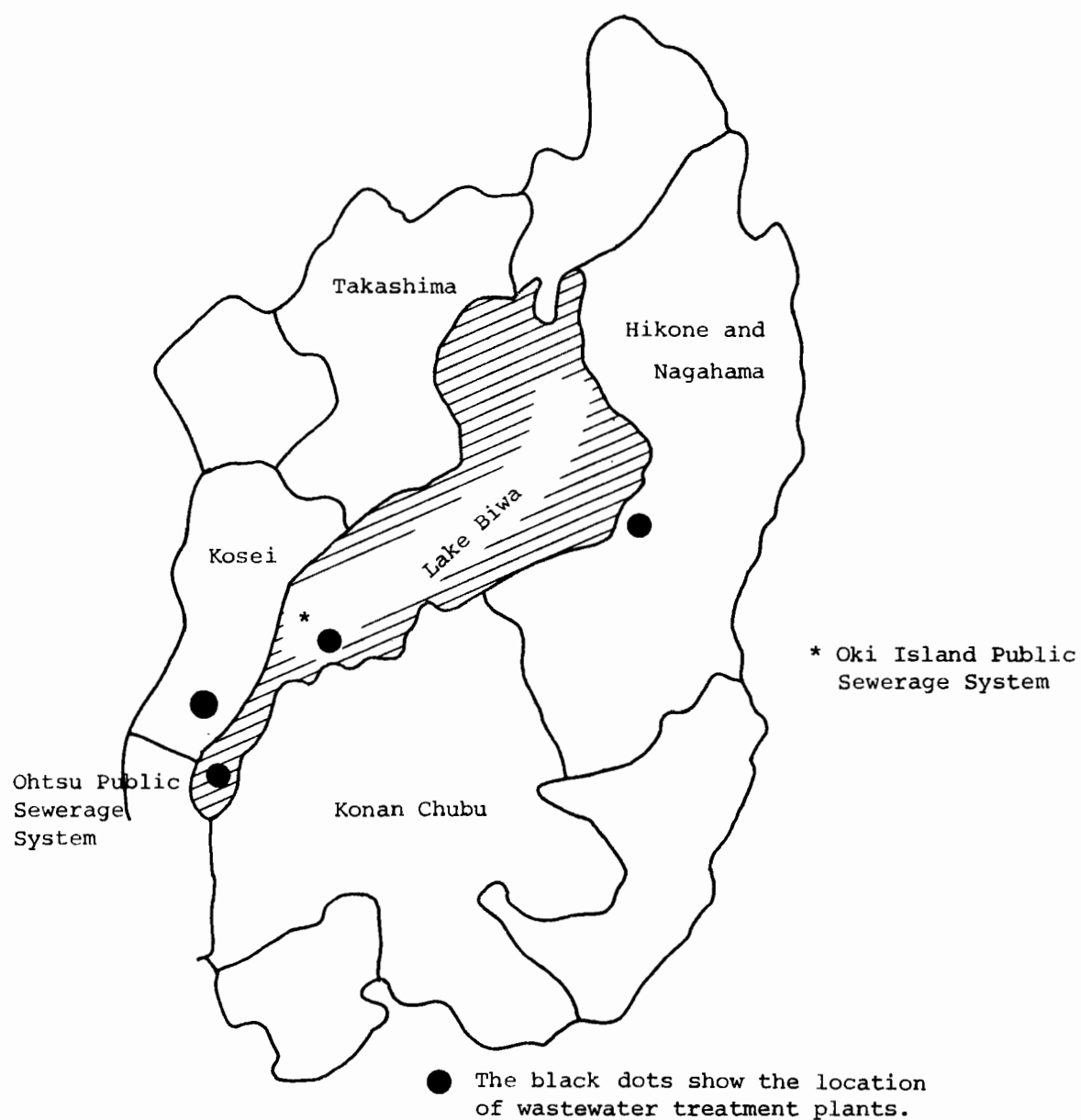


Fig. 3-1 Comprehensive sewerage system in Shiga

Table 3-1 An overview of sewerage program in Shiga Prefecture

Item	Lake Biwa Regional Sewerage System				Ohtsu Municipal Sewerage System	Oki Island Municipal Sewerage System
	Konan Chubu	Hikone and Nagahama	Kosei	Takashima		
Planned Service Area	approx. 25,500 ha	approx. 12,700 ha	approx. 2,600 ha	Pending	approx. 1,338 ha	approx. 8.7 ha
Planned served population	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	approx. 420 m ³ /day
Sewer system	Separate system				Partly combined system	Separate system
Aggregate 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
Area of 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 municipalities concerned	5 cities and 14 towns	2 cities and 17 towns	1 city and 1 town	5 towns	1 city	1 city

The Ohtsu municipal wastewater treatment plant has been in service since 1969, and is now removing phosphorus by the conventional activated sludge process combined with secondary alum precipitation.

Four regional sewerage systems are planned, each with a wastewater treatment plant. The wastewater treatment plant for the Konan Chubu sewerage system (hereinafter referred to as the Konan Chubu Purification Center) will be installed on an about 73 ha of reclaimed land on the shore of Lake Biwa, and will process about a million m^3 of wastewater a day. In 1977, the reclamation work was started, and the construction of the treatment facilities was started in 1978.

Originally, the treatment facilities were planned with the conventional activated sludge process. As the Regulations Concerning Eutrophication Control in Lake Biwa were established, the design of the facilities was modified for nutrients removal. It is expected that the Konan Chubu Purification Center will start its service in 1982.

For the wastewater treatment plant of the Kosei sewerage system (hereinafter referred to as the Kosei Purification Center), design of the wastewater treatment facilities including the nutrients removal processes was completed in 1980, and construction is now under way.

Land acquisition for the wastewater treatment plant for the Hikone Nagahama sewerage system is in progress.

A master plan for the Takashima sewerage system is being studied.

At Oki Island within the lake, an oxidation ditch having a capacity of 420 m^3/day is now under construction.

4. RESERACH AND DEVELOPMENT OF ADVANCED WASTEWATER TREATMENT TECHNOLOGY

In Japan, research and development of advanced wastewater treatment technology started toward the end of the 1960s.

In 1971, the Public Works Reserach Institute of the Ministry of Construction installed a 250 m^3/day pilot plant within the premises of the Shitamachi Sewage Treatment Plant, Yokosuka, about 50 miles southwest of Tokyo, in order to initiate a study on a lime precipitation process for phosphorus removal of secondary effluent. Since then, the Public Works Research Institute has been playing a key role for the basic studies of physico-chemical processes, biological denitrification processes, etc.

In 1972, the Japan Sewage Works Agency was established. Its Research and Technology Development Division has been promoting surveys for the implementation of advanced wastewater treatment technology.

In 1974, the Japan Sewage Works Agency, together with the Sewerage and Sewage Purification Department of the Ministry of Construction, and the Shiga Prefectural Government, constructed the Lake Biwa Advanced Wastewater Treatment Pilot Plant with a capacity of 500 m³/day for the purpose of studying the implementation of advanced wastewater treatment processes applicable to the Lake Biwa regional sewerage systems. This pilot plant was located within the Ohtsu Municipal Wastewater Treatment Plant near the south lake of Lake Biwa. It was composed of: (1) alum precipitation process for phosphorus removal; (2) biological nitrogen removal process; (3) suspended solid removal process using an up/down flow filtration system; and (4) dissolved organic matter removal process using an up/down flow activated carbon adsorption column. Given in Fig. 4-1 and Table 4-1 are the principal particulars of the major facilities.

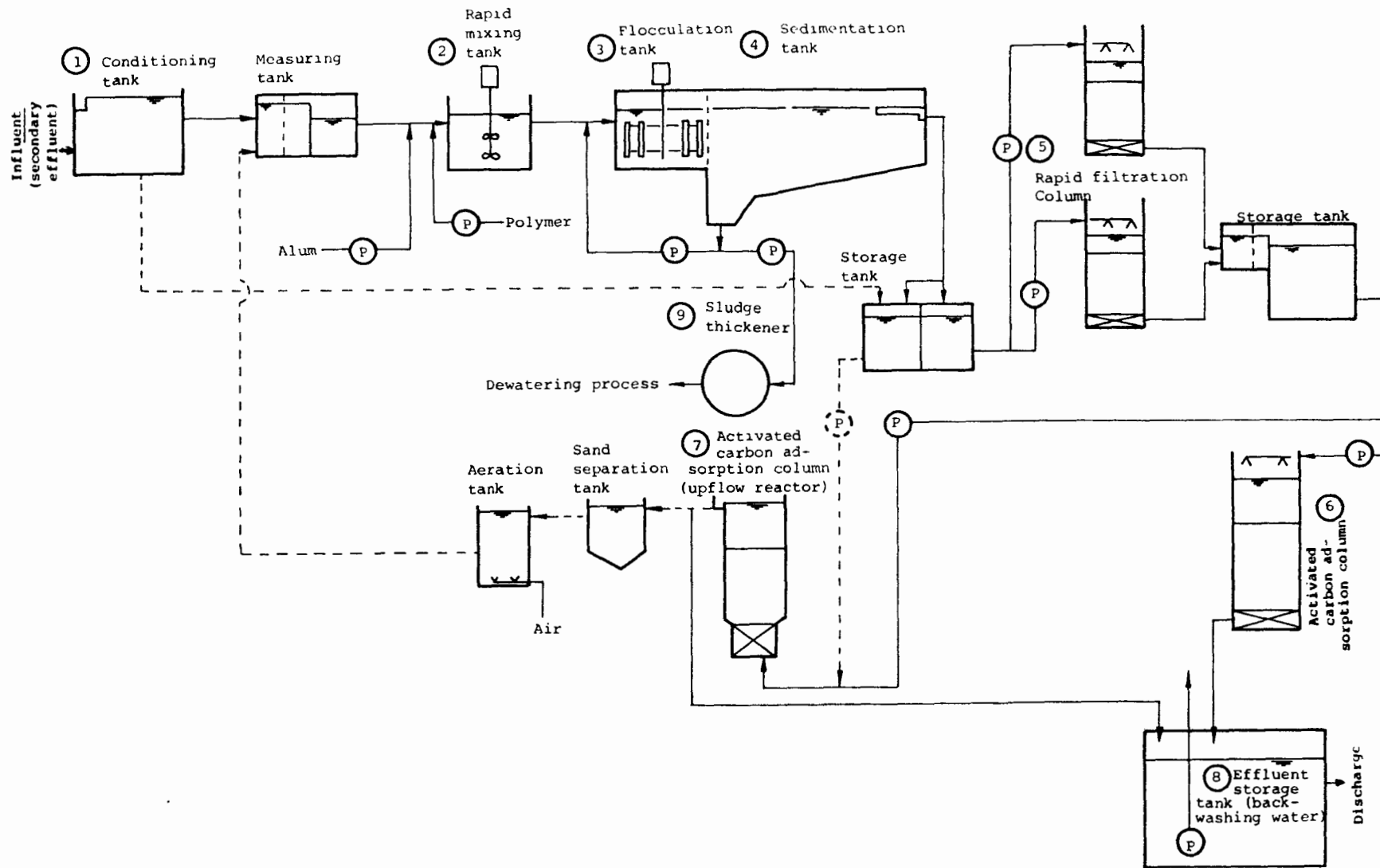


Fig. 4-1 Lake Biwa Tertiary Treatment Pilot Plant flow sheet

Table 4-1 Pilot Plant Biological Nitrogen Removal Process

Process	Reaction	Capacity (m ³ /day)	Influent
Separate stage nitrification in modified aeration tank with fine sand addition process (SSNP)	Nitrification	450	Secondary effluent
Fluidized bed denitrification process (FBDP)	Denitrification	450	Nitrified liquor (nitrified secondary effluent)
Combined carbon oxidation-nitrification process (CCONP)	Nitrification	30	Primary effluent
Packed bed denitrification process (PBDP)	Denitrification	20	Nitrified liquor (nitrified secondary effluent)
Recycled nitrification-denitrification process (RNDP)	Nitrification and denitrification	7	Primary effluent

The main purpose of the pilot research was to identify the phosphorus removal and denitrification processes feasible for eutrophication control in Lake Biwa and to clarify the design criteria for these processes.

4.1 Survey of Chemical Precipitation for Phosphorus Removal

Chemical precipitation processes are largely classified into: (1) metal precipitation and (2) lime precipitation. Of the two, the metal precipitation process was investigated because of its simple process formation and low capital and operating costs.

According to laboratory tests, alum was selected as a coagulant, and a pilot plant was installed to test alum precipitation treatment of secondary effluent. Thus, the design criteria for the process were established.

- (1) As shown in Fig. 4-2, it is found that rapid mixing can be carried out thoroughly on condition that the \bar{G} -value is about 100 sec^{-1} with the detention time in the rapid mixing tank at 5 min. It is also found that flocculation is best achieved when the \bar{G} -value is less than 50 sec^{-1} and the GT-value is about 90,000.

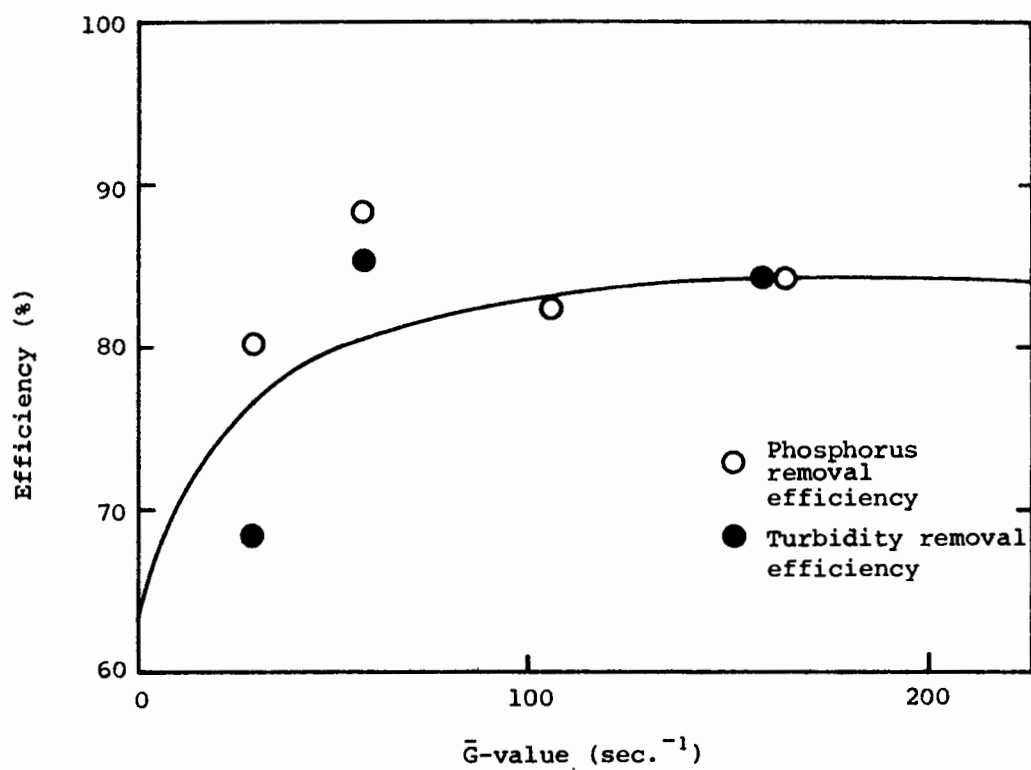


Fig. 4-2 \bar{G} -value, and phosphorus and turbidity removal efficiency (rapid mixing)

- (2) The relationship between alum addition ratio and residual phosphorus is shown in Fig. 4-3.

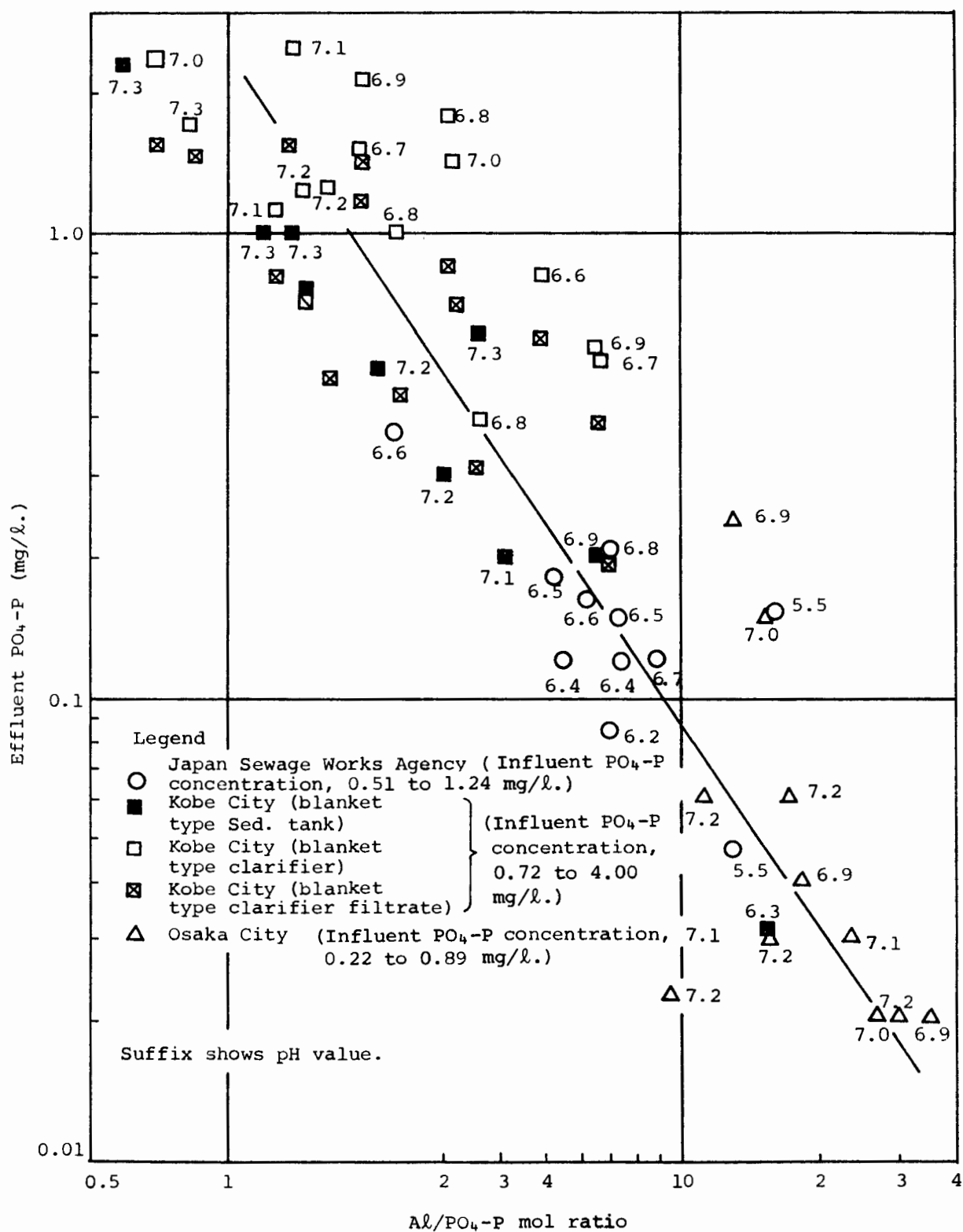


Fig. 4-3 Al/ $\text{PO}_4\text{-P}$ mol ratio and effluent $\text{PO}_4\text{-P}$

In Fig. 4-3, the results of pilot plant tests conducted in Osaka City and Kobe City are also plotted. From the figure, it is found that it is generally possible to reduce the concentration of $\text{PO}_4\text{-P}$ in the effluent below 0.5 mg/l if the $\text{Al}/\text{PO}_4\text{-P}$ mol ratio is 3 or larger. It is also found that the addition of anionic polymer as a coagulation aid at a rate of 0.2 to 0.3 mg/l improves the flocculation rate and results in the improvement of the phosphorus and turbidity removal efficiencies by about 23% and 90%, respectively.

- (3) It is found that the phosphorus removal efficiency can be increased by returning the sludge from the sedimentation tank to the flocculation tank as illustrated in Fig. 4-4.

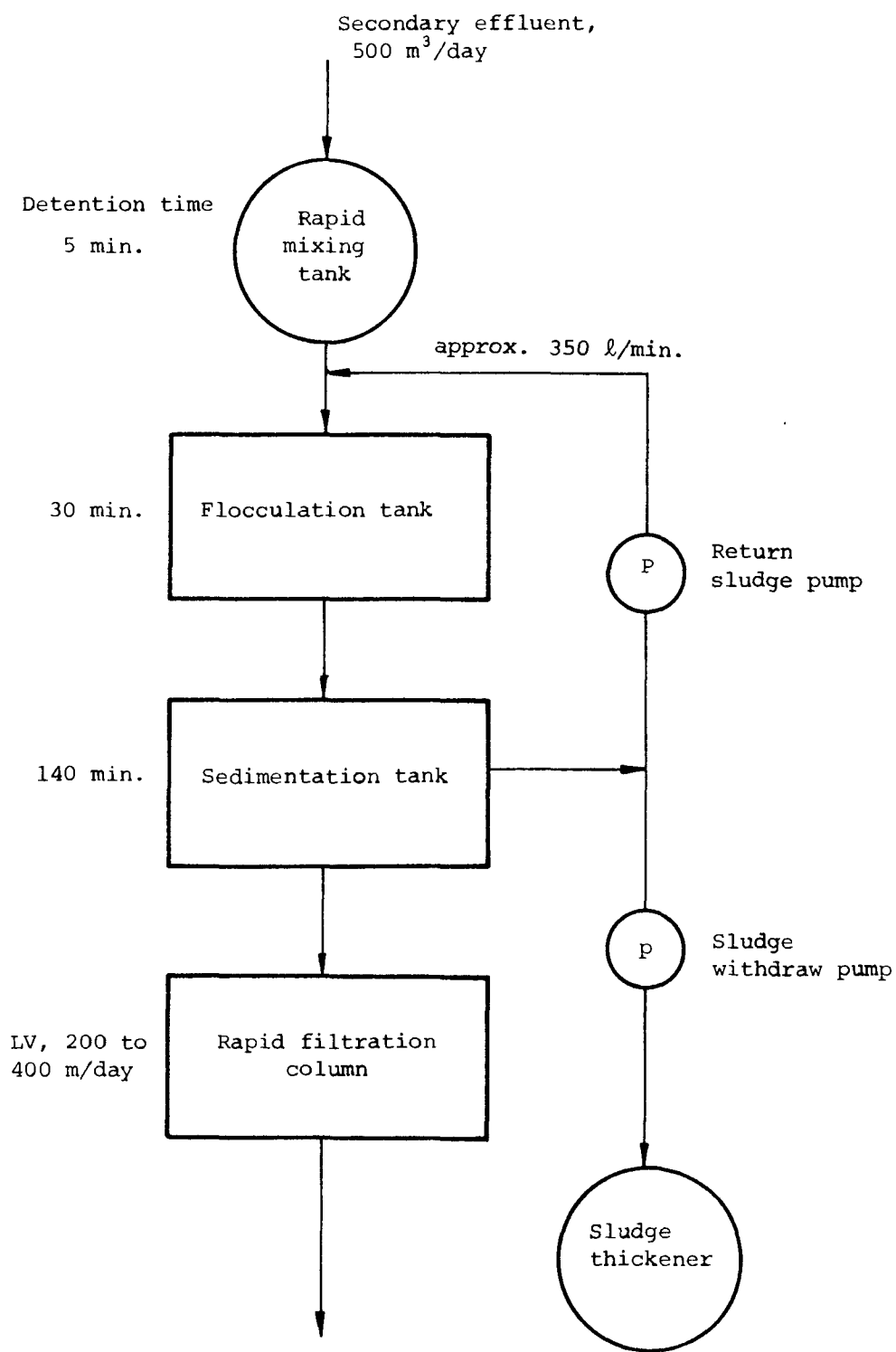


Fig. 4-4 Return of alum-precipitated sludge

The optimum condition of sludge return is that the return sludge ratio is 5%, and the concentration of return sludge is 2,000 to 3,000 mg/l.

- (4) It is found that the phosphorus concentration in the effluent can be further reduced when the alum-precipitated sludge is returned to the flocculation tank after its pH value is conditioned with sulfuric acid to about 3.5. The effect of returning the acid-treated sludge is as shown in Fig. 4-5.

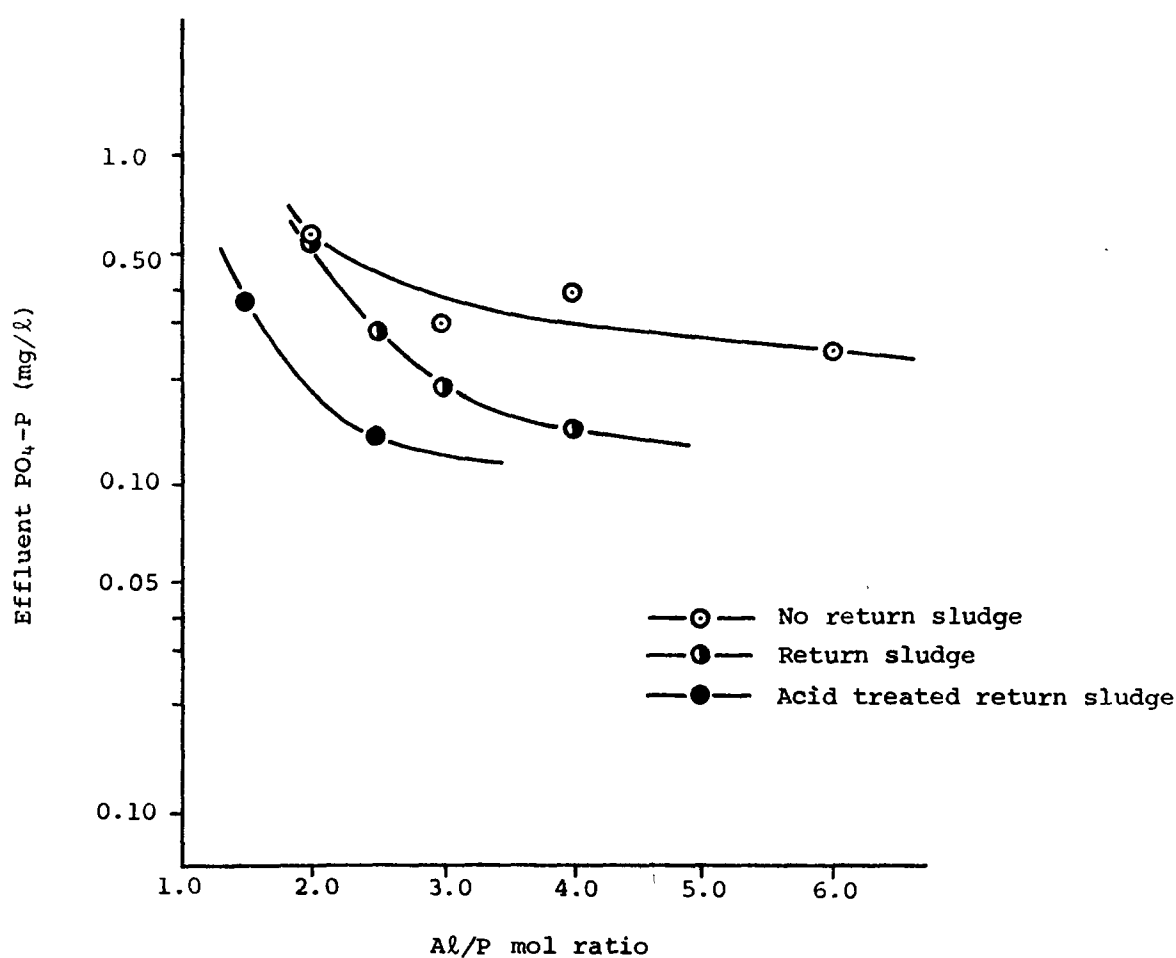


Fig. 4-5 Effect of acid treatment of alum-precipitated sludge

In addition to the surveys referred to above, at the pilot plant the precipitated sludge was investigated as to its generated volume, characteristics, settling, thickening and dewatering abilities, etc. A study was also carried out for the potentiality of sludge as a construction material, and the sludge subjected to freezing, melting and then dewatering by filter press proved that it is applicable as a roadbed material in terms of soil mechanics.

4.2 Survey of Biological Nitrogen Removal

The biological nitrogen removal process can potentially remove all forms of nitrogen, but has the following problems.

- (1) The biological process is lower in reaction rate as compared with the physico-chemical process, and is liable to be influenced by water temperature.
- (2) The alkalinity of sewage in Japan is, in general, lower than that of the sewage in the United States and Europe. Thus, it is often necessary to inject an alkaline agent for the purpose of conditioning the pH value in the nitrification process.
- (3) For the denitrification process, it is necessary to inject methanol as a hydrogen donor.

Taking account of these problems, the five biological nitrogen removal processes listed in Table 4-1 were selected and subjected to pilot plant studies.

4.2.1 Separate Stage Nitrification in Modified Aeration with Sand Addition Process (SSNP)

In this process, fine sand (ϕ 0.15 to 0.3 mm) is suspended in a nitrification tank in order to grow nitrifying bacteria on the surfaces of sand particles for the purpose of nitrification. Since SSNP can keep MLVSS high and the active surface of the biomass is large, the nitrification rate per unit volume of nitrification tank is high.

At the Ohtsu Municipal Wastewater Treatment Plant, secondary effluent was experimented with by running a turbine blade type mixing

nitrification tank (hereinafter referred to of nitrification tank with mechanical mixer) and a nitrification tank with air lift in parallel for the purpose of investigating the nitrification rate and process stability, etc.

Fig. 4-6 shows flow diagrams of the two pilot plants.

The nitrification tank with mechanical mixer and the nitrification tank with air lift were operated under the same loading conditions for the purpose of comparison. The results are as shown in Fig. 4-7.

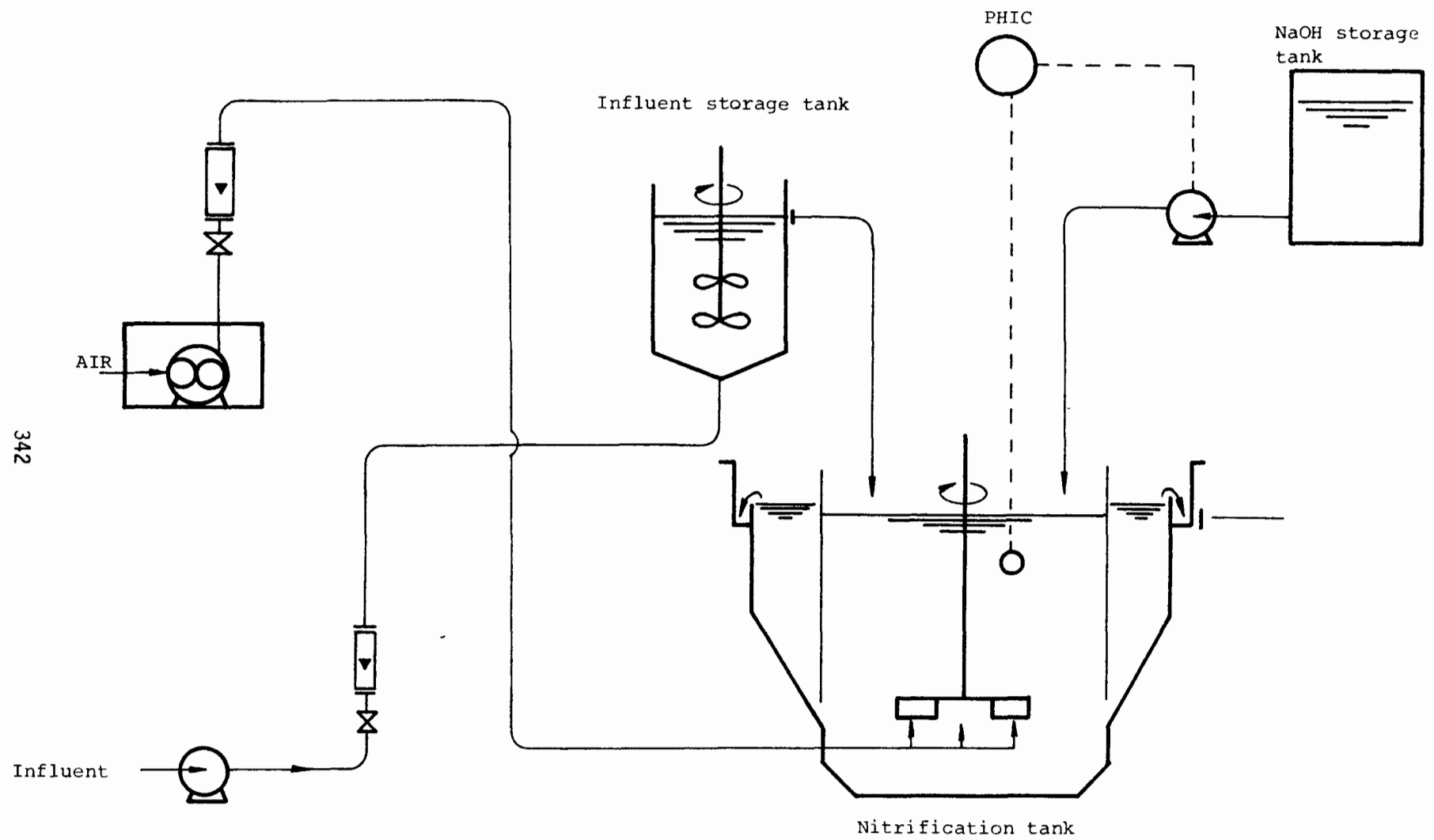


Fig. 4-6 (a) Nitrification tank with mechanical mixer (450 m³/day)

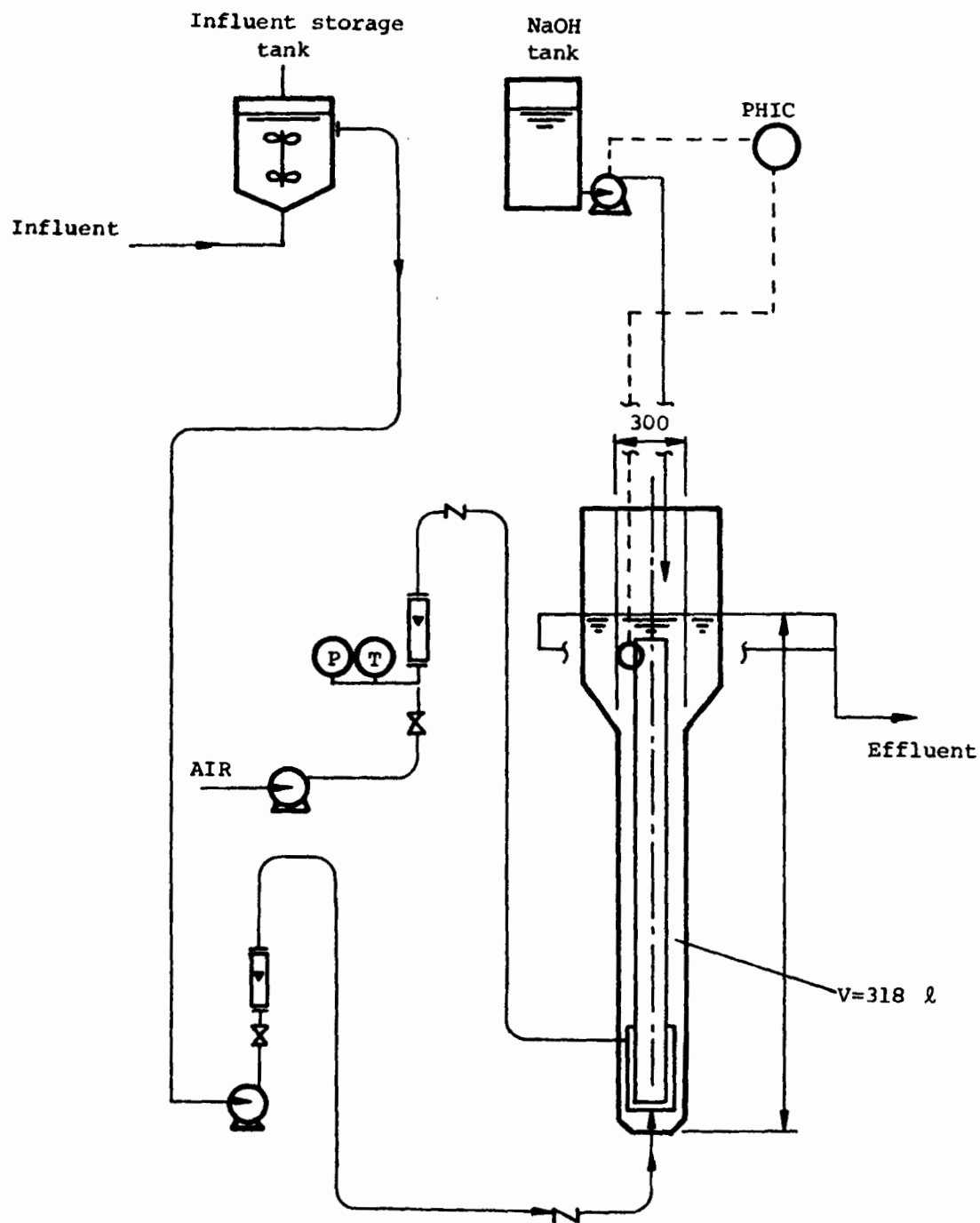


Fig. 4-6 (b) Nitrification tank with
air lift ($20 \text{ m}^3/\text{day}$)

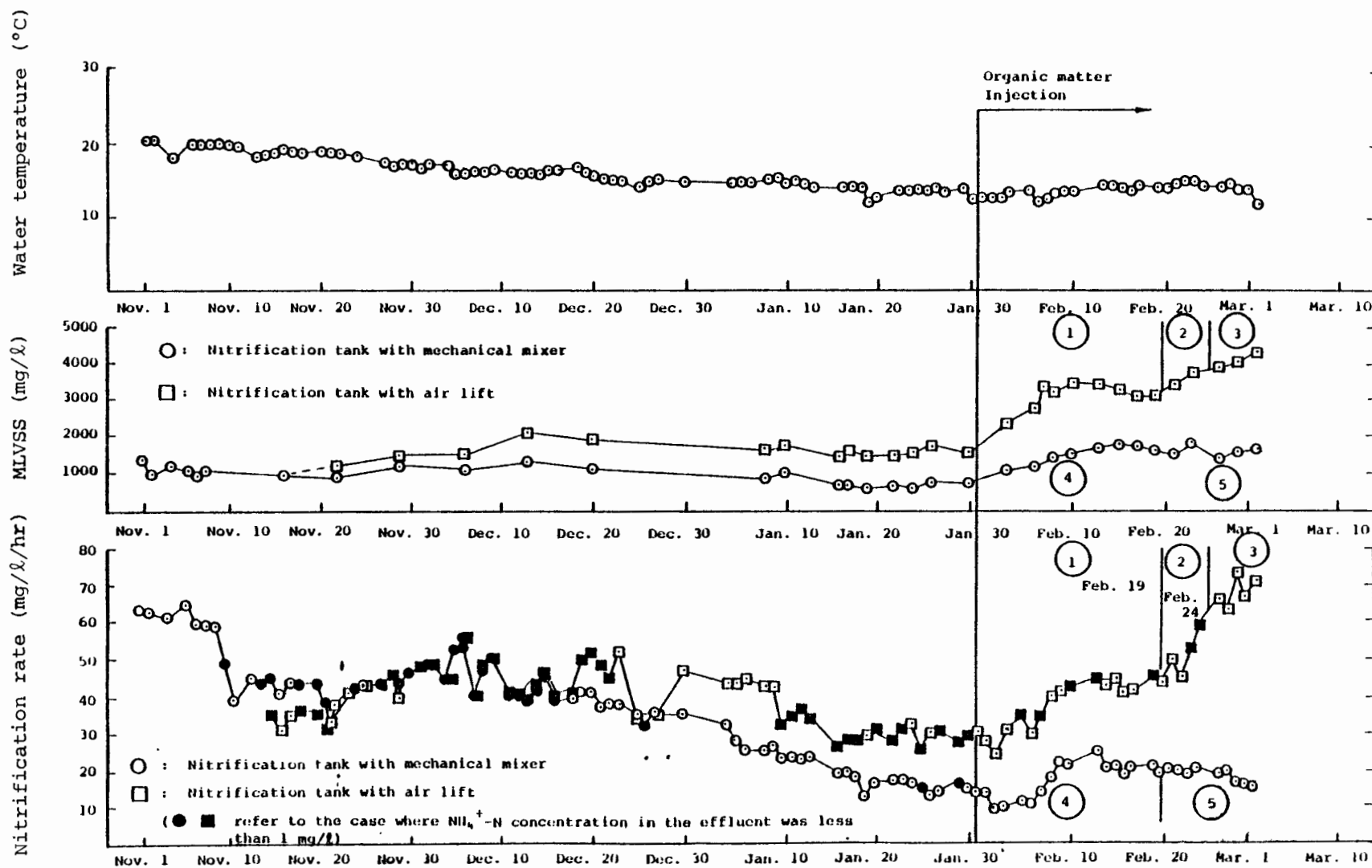


Fig. 4-7 Parallel running of nitrification tank with mechanical mixer and nitrification tank with air lift

It was found that the volume of biomass to be kept in the nitrification tank varied depending on the type of mixing. In about two months and a half after the start of operation, the MLVSS in the nitrification tank with air lift became as large again as that in the nitrification tank with mechanical mixer. As a result, the volumetric nitrification rate became about $30 \text{ g/m}^3/\text{h}$ in the nitrification tank with air lift, or about twice as high as about $15 \text{ g/m}^3/\text{h}$ in the nitrification tank with mechanical mixer.

On February 1, a test was started to investigate the effect of the concentration of organic matter on a nitrification rate. For this purpose, a solution of meat extract was injected into the influent to have a BOD concentration of 20 mg/l . (The influent BOD before conditioning was about 11 mg/l). By the addition of organic matter, the MLVSS in both tanks tended to rise, and the nitrification rate rose as well. On March 2, the MLVSS in the nitrification tank with air lift was about $4,300 \text{ mg/l}$ or about 2.6 times as much as that in the nitrification tank with mechanical mixer, and the nitrification rate in the nitrification tank with air lift was about $70 \text{ g/m}^3/\text{h}$, or about 4.6 times as much as that in the nitrification tank with mechanical mixer.

In the nitrification tank with mechanical mixer, the growth of nitrifying bacteria on the sand particles was largely governed by the turbine blade speed, and the count of nitrifying bacteria to be kept and the nitrification rate were found smaller as compared with the nitrification tank with air lift. At a water temperature of 20°C , the nitrification rate in the nitrification tank with air lift was about $60 \text{ g/m}^3/\text{h}$, which was higher than in any other nitrification processes in Table 4-1. But, the nitrification tank with air lift was found destitute of stable performance on a long-term continuous running basis.

The future study includes the controlling procedures of the nitrification capacity, and measures for protecting the nitrification tank from sand abrasion, optimum aeration system, and mixing intensity.

4.2.2 Fluidized Bed Denitrification Process (FBDP)

When sand (ϕ 0.47 to 0.59 mm) is fluidized in a reactor by the upward flow of nitrified liquor while injecting methanol, denitrifying bacteria grow on the surfaces of sand particles to form pellets.

In the FBDP, the reactor is free from clogging since the pellets are fluidized, and a high denitrification rate can be achieved. Fig. 4-8 shows a flow diagram of the pilot plant.

Table 4-2 shows an example of the performance of the FBDS in stable operation.

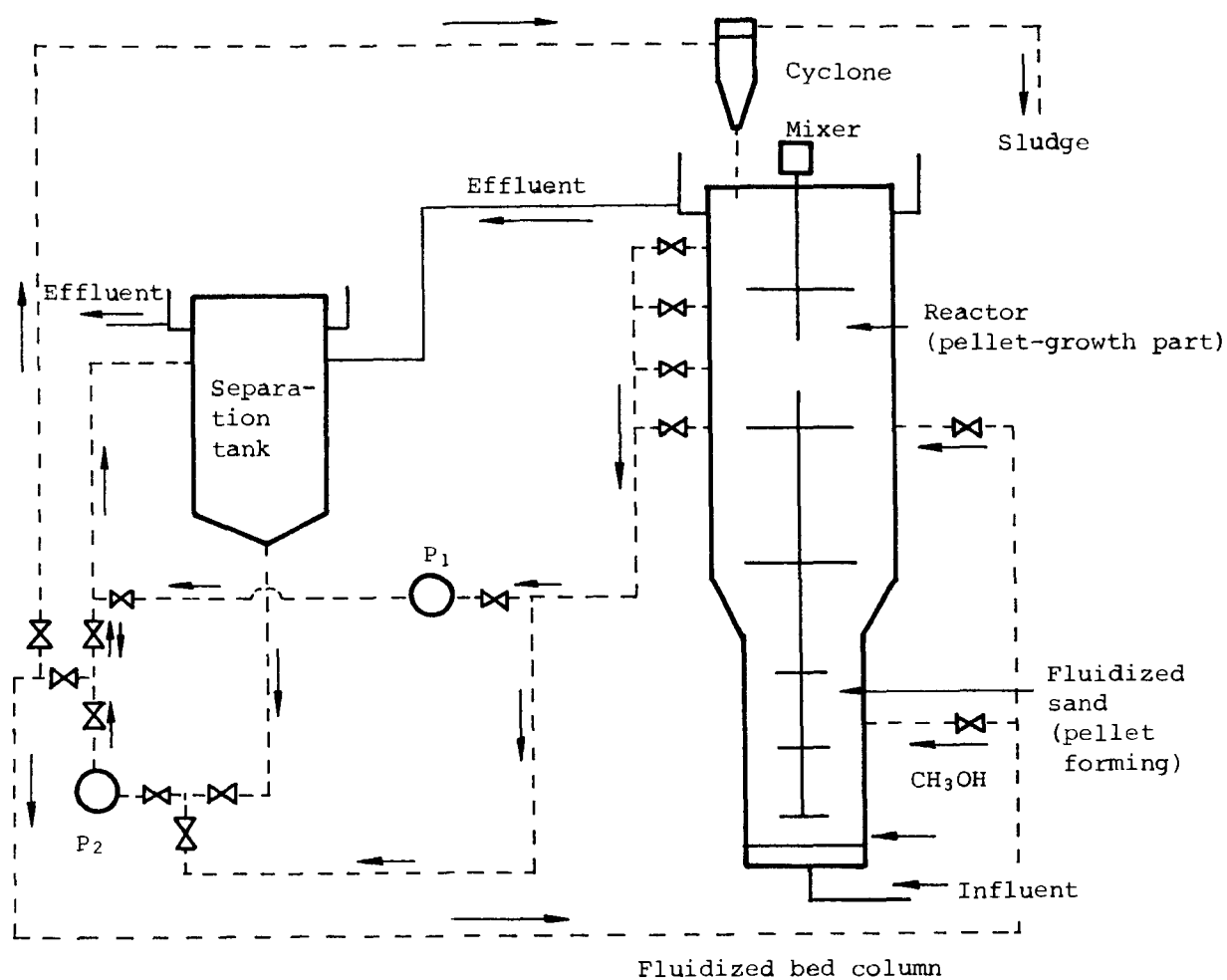


Fig. 4-8 Flow diagram of FBDP

Table 4-2 An example of FBDP operation

Date	Water temperature, °C	Flow rate (m ³ /day)	Sand size (mm)	¹ / Methanol ratio (-)	N-loadings (kg/m ³ /day)	Denitrification efficiency (%)	Denitrification rate (g/m ³ /h)	Biomass (VSS-kg)	Influent (NO ₃ ⁻ +NO ₂ ⁻) (mg/l)	Effluent (NO ₃ ⁻ +NO ₂ ⁻) (mg/l)
Feb. 2, 1978, Morning	13.0	432	0.47	5.5	24.0	99.5	99.5	20.3	11.24	0.06
Feb. 2, 1978 Afternoon	13.5	432	0.47	4.3	6.34	93.8	248	20.3	29.83	1.86

Note: ¹/_{Methanol ratio} = Injected methanol/influent (NO₃⁻+NO₂⁻)-N

Fig. 4-9 shows the changes in $\text{NO}_2^- \text{N}$ and $\text{NO}_3^- \text{N}$ in the fluidized bed referred to in Table 4-2.

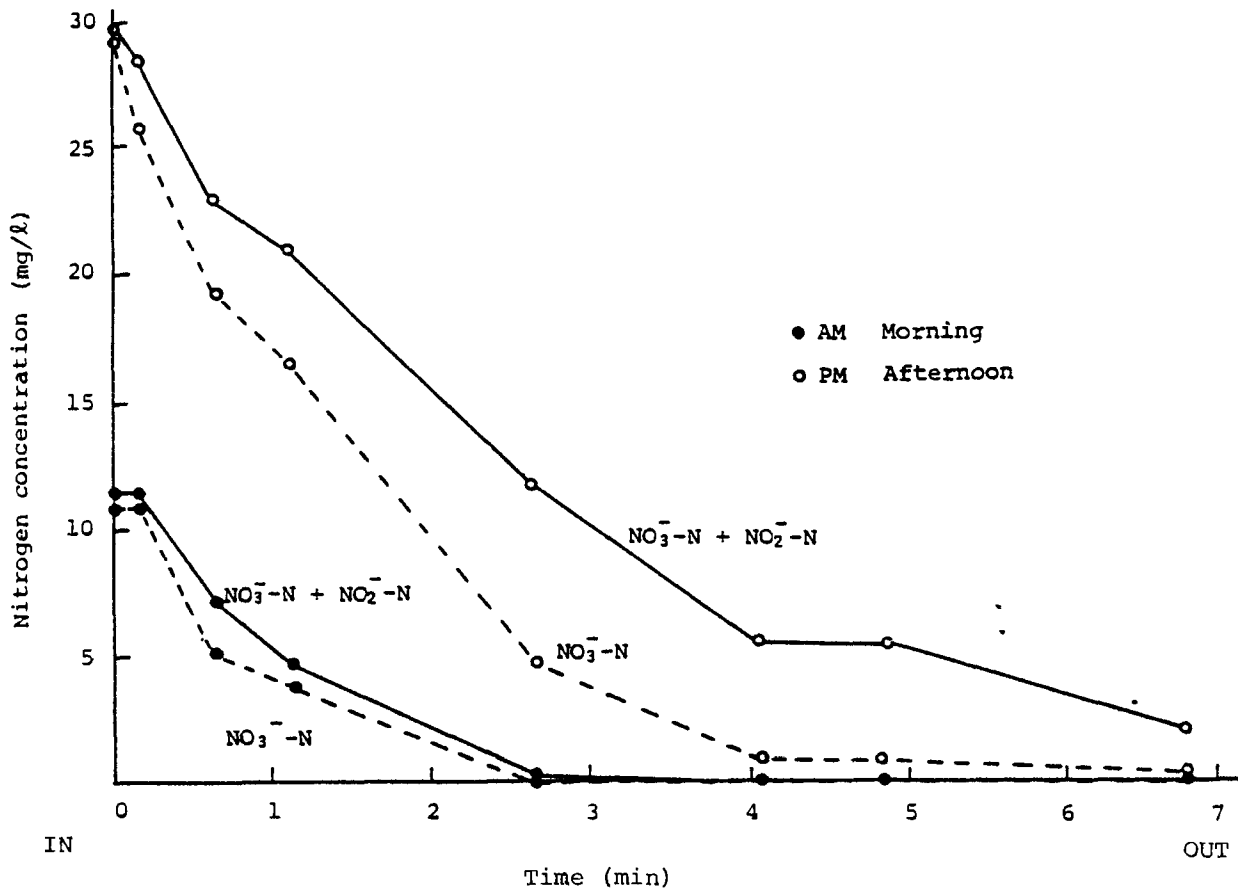


Fig. 4-9 Changes of $(\text{NO}_2^- + \text{NO}_3^-) \text{N}$ in the fluidized bed

The morning data showed a small nitrogen loading of $2.4 \text{ kg/m}^3/\text{day}$, and the denitrification was completed in about 3 min. at a denitrification efficiency of 99.5%.

In the afternoon, the nitrogen loading was about 2.6 times the morning value to $6.34 \text{ kg/m}^3/\text{day}$. In this case, the denitrification efficiency was 93.8% with the detention time of about 7 min., and the denitrification rate was as large as $248 \text{ g/m}^3/\text{h}$. In order to maintain such a high denitrification rate, it is necessary to draw grown pellets out of the column top, remove the biological encrustations from sand particles and return the naked sands to the fluidized bed. The pellet withdrawal rate was set at about 20 to 25%

of the influent flow rate. It was found that the pelletizing in the bottom of the fluidized bed could be promoted when sands were returned with incomplete removal of the encrustations.

In a steady state, the biomass to be held in the fluidized bed was about 20 kg-VSS, or a 1.2% concentration. In this case, the sand was about 200 kg (12.6%).

It was demonstrated by the pilot plant tests that the FBDP is 10 to 100 times faster in denitrification rate than the packed bed denitrification process (PBDP) and recycled nitrification-denitrification process (RNDP).

However, such important operation factors as the expansion ratio of sand and pellets, the degree of the biomass removal from sand, and biomass removal method, stability against loading changes, etc. could not be investigated thoroughly, and it was judged that the FBDP should be further upgraded.

4.2.3 Combined Carbon Oxidation-Nitrification Process (CCONP)

The CCONP is a process in which the oxidation of organic matter and nitrification of $\text{NH}_4^+ - \text{N}$ are carried out in one aeration tank. The CCONP was tested using a pilot plant fed with primary effluent of the Arakawa Wastewater Treatment Plant adjoining the Research and Training Center of the Japan Sewage Works Agency. The pilot plant was run continuously in order to investigate the relationship between SRT and nitrification efficiency, process stability, etc. Table 4-3 shows a summary of the operation results of the CCONP.

Table 4-3 Running results of CCONP

No.	Test period	Flow rate (m ³ /day)	Detention time (hrs.)	Water temperature (°C)	MLSS (mg/l)	N-loadings (kg/m ³ /day)	BOD loadings (kg/kg-MLSS/day)	Nitrifi- cation rate (mg/g- MLSS/h)	Nitrifi- cation rate (g/m ³ /h)	SRT (dys)	Nitrifi- cation efficiency (%)
1	Dec. 7, 1977 to Jan. 23 1978	20.0	12.1	10.9	3370	0.050	0.058	0.63	2.09	29.7	99.9
2	Jan. 26 to Feb. 7	30.0	8.1	9.4	3270	0.077	0.097	0.96	3.13	10.5	98.1
3	Feb. 9 to Feb. 16	36.7	6.6	10.2	3590	0.087	0.120	1.00	3.58	14.4	99.0
4	Mar. 13 to Apr. 7	36.7	6.6	12.5	3250	0.074	0.089	0.97	3.04	6.3	98.3
5	Jun. 15 to Jun. 26	58.8	3.0	25.0	3330	0.184	0.200	2.04	7.63	5.5	99.7
6	Jul. 3 to Jul. 21	60.5	2.9	26.6	2800	0.184	0.257	3.22	7.58	3.5	98.9
7	Aug. 10 to Aug. 28	60.0	3.0	28.4	2430	0.192	0.284	3.65	7.95	4.6	99.3
8	Sep. 20 to Oct. 4	43.0	4.1	23.6	2750	0.149	0.176	2.82	6.13	8.2	98.9
9	Oct. 6 to Oct. 30	43.0	4.1	20.3	4910	0.123	0.075	1.03	4.96	5.8	96.6
10	Nov. 6 to Nov. 22	41.5	4.2	17.6	3830	0.138	0.105	1.38	5.70	6.0	98.8
11	Nov. 30 to Dec. 19	41.5	4.2	14.8	4340	0.160	0.119	1.46	6.33	16.4	95.1
12	Feb. 6 to Feb. 16, 1979	42.7	4.1	12.2	4750	0.135	0.105	1.17	5.54	17.8	98.2

The tests were started in winter with the detention time set at 12 hrs. With the rise of water temperature, the detention time was reduced; in summer, the pilot plant was run with the detention time set at 3 hrs. Later, in the period from autumn to winter, the detention time was set at 4 hrs. Throughout the test period, the nitrification was carried out stably with a nitrification efficiency of 98% to 100%. Although it is claimed that, in winter when water temperature is low, stabilized nitrification is difficult unless the detention time is extended to a certain degree, it is found from Table 4-3 that a practically high nitrification efficiency can be achieved with a detention time of about 4 hrs. even in winter if the operational conditions (SRT, MLSS, etc.) are set properly.

Fig. 4-10 shows the relationship between SRT and water temperature for the data representing a nitrification efficiency of 95% or more.

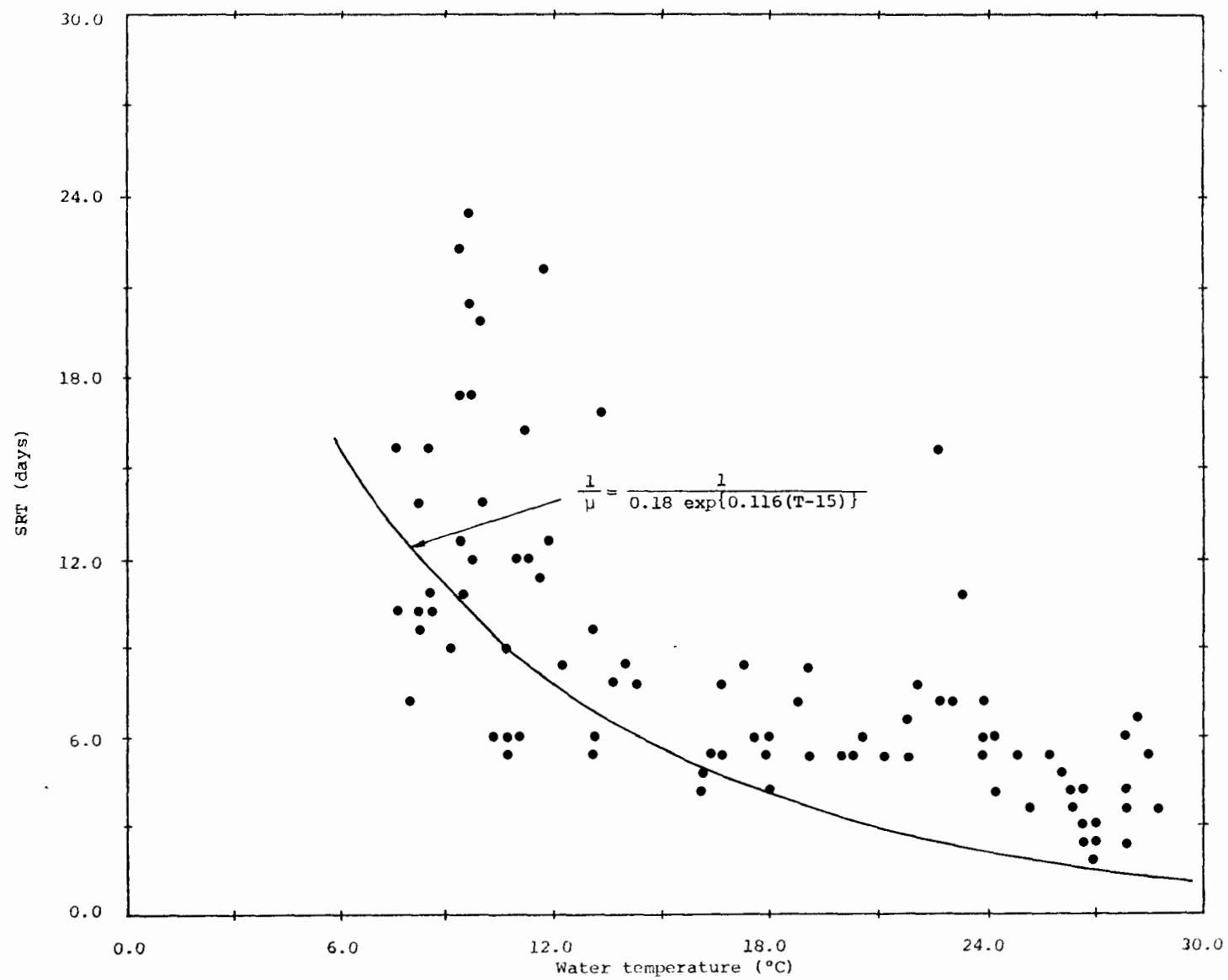


Fig. 4-10 SRT vs. water temperature

The solid line in the figure shows the reciprocal of the maximum growth rate, μ , of nitrosomonas determined by Downing, et al.⁽¹⁾ The greater part of the SRTs measured is above the reciprocal of μ calculated from the formula established by Downing, et al., suggesting that the SRT necessary for keeping nitrosomonas in the aeration tank can be determined by that formula.

The nitrification rate was about 2.5 g/m³/h at a water temperature of 10°C and about 5.4 g/m³/h at a water temperature of 20°C. Where the nitrification efficiency was 95% or more, the BOD loading was 0.05 to 0.1 kg/kg-MLSS/day at a water temperature of 10°C and 0.15 to 0.25 kg/kg-MLSS/day at a water temperature of 25°C, and was smaller as compared with the conventional activated sludge process. The test Nos. 1 through 4 refer to the case where the settling time in the final sedimentation tank was 1.2 to 2.3 hours. The effluent showed 5 to 10 mg/l in BOD, 9 to 12 mg/l in COD and 13 to 15 mg/l in SS, attaining a practical high efficiency of organic matter removal. The tests Nos. 5 through 12 refer to the case where the settling time was as short as 0.8 to 1.1 hour; because of SS carryover, the effluent quality was degraded a little. The alkalinity of the influent was about 130 mg/l, and caustic soda of the amount equivalent to an alkalinity of 50 to 90 mg/l was injected into the aeration tank for the purpose of conditioning the effluent pH value to about 6.5 to 7.3.

Finally, it was found that the CCONP could be run stably at a nitrification efficiency of more than 95% when the SRT, MLSS concentration and pH value were set appropriately.

4.2.4 Packed Bed Denitrification Process (PBDP)

When nitrified secondary effluent is run downward with the addition of methanol through a filtration column stuffed with sand of 3 to 5 mm in diameter, it is denitrified by the denitrifying bacteria grown over the surfaces of the sand particles in the filtration column. The PBDP dispenses with sedimentation tank and sludge return operation, and can remove SS by filtration.

A 20 m³/day pilot plant was employed for tests. Its flow diagram is as shown in Fig. 4-11.

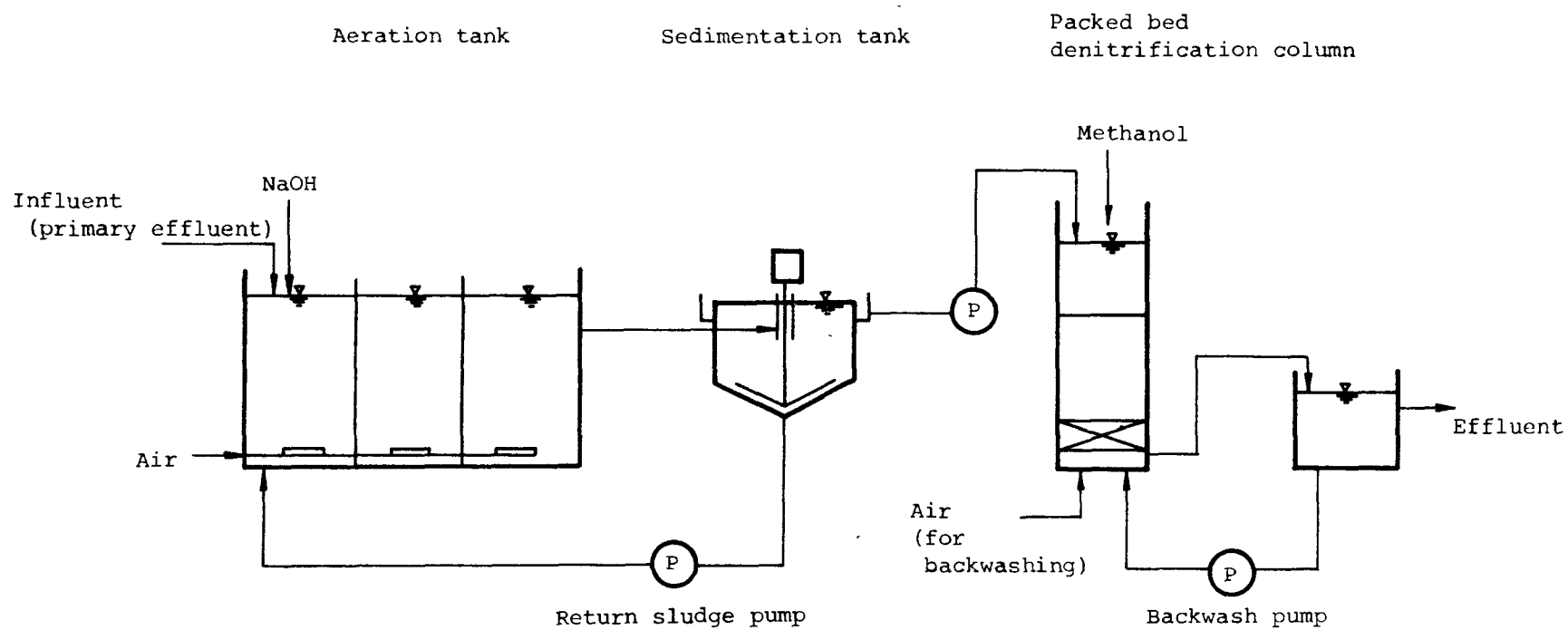


Fig. 4 11 Flow diagram of CCONP and PBDP

The packed bed (700 x 700 by 5,500 H) was stuffed 1.8 m deep with sand of 2 to 4 mm size over a gravel layer (6 to 20 mm ϕ x 400 thick). The influent used was the nitrified effluent from the CCONP. Namely, the CCONP and PBDP were combinedly operated continuously for the purpose of studying the denitrification rate, SS removal efficiency, and overall nitrogen removal efficiency, etc.

Table 4-4 shows a summary of the operation results of the PBDP.

Table 4-4 Running results of PBDP

No.	Test period	Flow rate (m ³ /day)	Detention time (min.)	Water temperature (°C)	N-loadings (kg/m ³ /day)	Volumetric loadings (m ³ /m ² /day)	Methanol ratio (-)	Denitrification rate (g/m ³ /h)	Denitrification efficiency (%)	Effluent BOD (mg/l)	Effluent COD (mg/l)
1	Jan. 20 to 31, 1978	18.0	70.4	7.8	0.502	36.7	3.8	19.8	94.7	4.1	10.9
2	Feb. 1 to 17	18.0	70.4	8.2	0.471	36.7	4.0	16.8	87.9	6.6	12.8
3	Mar. 1 to 31	18.0	70.4	10.5	0.404	36.7	5.0	15.5	91.9	14.9	14.8
4	Apr. 1 to 28	18.0	70.4	14.7	0.369	36.7	5.6	14.8	94.5	14.8	15.3
5	Jul. 24 to 28	29.0	43.7	28.2	0.582	59.2	4.5	24.9	98.3	-	16.2
6	Aug. 12 to 28	29.4	43.1	29.1	0.760	60.0	3.3	31.0	97.4	-	10.1
7	Oct. 6 to 31	29.4	43.1	19.1	0.690	60.0	3.9	28.5	99.0	6.6	8.8
8	Nov. 1 to 26	29.4	43.1	17.0	0.810	60.0	3.4	31.3	93.4	3.6	8.8
9	Dec. 1 to 20	29.4	43.1	13.2	0.870	60.0	2.6	32.4	88.5	11.4	13.1
10	Jan. 29 to 31, 1979	18.7	67.8	9.9	0.490	38.2	3.4	19.3	92.6	13.9	11.8
11	Feb. 1 to 16	18.7	67.8	12.0	0.502	38.2	3.5	20.5	96.6	4.5	9.2

The volumetric loading was set in two stages: In winter, the volumetric loading was set at 37 to 38 m³/m²/day and the detention time at 68 to 70 min; and in summer, the volumetric loading was at 59 to 60 m³/m²/day and the detention time at 43 to 44 min.

The tests Nos. 1, 2, 3 and 10 refer to data obtained when the water temperature was as low as 7.8 to 10.5°C and the influent NO₃⁻-N concentration was between 20 and 25 mg/l, but the NO₃⁻-N and NO₂⁻-N concentration in the effluent were 0.1 mg/l to 1.2 mg/l and 0.7 to 1.6 mg/l respectively, achieving as high a denitrification efficiency as 88 to 95%.

When the water temperature rose above 10°C, the denitrification efficiency improved further; with the exception of test No. 9, the denitrification efficiency was 93 to 98%. Test No. 9 showed a denitrification efficiency of 88.5%, a little lower than other tests, and this may have been attributable to the fact that the methanol ratio was as low as 2.6.

The denitrification rate was 6.67 g/m³/h at 10°C and 29.2 g/m³/h at 20°C. The packed bed as a filter showed an SS removal efficiency of 85 to 97% at a volumetric loading of 37 to 97 m³/m²/day, and the effluent SS was 2 mg/l.

The nitrogen gas generated by denitrification was removed by backwashing the filter at a water flow rate of 0.32 m³/m²/min. once every 2 to 4 hrs.

And the filter was backwashed in a mode of (air agitation, 2 min. + air and water backwashing, 10 min. + water backwashing, 3 min.) every 3 to 3.5 days in case of low volumetric loading and every 2.5 days in case of high loading.

The overall T-N removal efficiency of the combination of CCONP and PBDP remained 85% when the water temperature was as low as 8°C and when the methanol ratio was as low as 2.6. In other cases, the T-N removal efficiency was 90 to 95%; T-N in the effluent was 2 to 3 mg/l as against about 30 mg/l in the influent.

PBDP requires the addition of methanol as a hydrogen donor, but exhibits a stable, high denitrification efficiency. In addition, it dispenses with the independent filtration process. All these features have been demonstrated by the pilot plant tests.

4.2.5 Recycled Nitrification-Denitrification Process (RNDP)

In this process, a denitrification tank and a nitrification tank are combined, and the effluent from the nitrification tank is returned to the denitrification tank. Namely, the organic matter in the influent is used as a hydrogen donor for denitrification and the resultant alkalis can be used in the nitrification process. Therefore, methanol and alkali agent can be saved. The tests were conducted using a 7 m³/day pilot plant fed with the primary effluent from the Arakawa Wastewater Treatment Plant in order to investigate the recycle ratio, SRT, excess sludge volume and other operating factors.

Table 4-5 shows a summary of the operation results of the pilot plant.

Table 4-5 Operation results of RNDP

No.	Test period	Flow rate (m ³ /day)	Tank capacity (m ³)		Detention time (hrs.)	Recycle ratio (-)	Water temperature (°C)	MLSS (mg/l)	N-loadings (kg/m ³ /day)	BOD loadings (kg/kg-MLSS/day)	SRT in total system (days)	SRT in nitrification tank (days)
			Denitrification	Nitrification								
1	Mar. 10 to Apr. 4, 1978	9.8	3.0 2.0	3.2 4.2	15.2	1.9~2.0	9.5	4960		0.025	-	-
2	Apr. 5 to 28	9.8	2.0	4.2	15.2	1.7~1.9	14.9	4820	0.044	0.023	144.2	97.7
3	Jun. 15 to 28	10.1	2.0	4.2	14.7	1.4	26.0	3600	0.059	0.046	185.3	125.6
4	Jul. 1 to 14	10.1	2.0	4.2	14.7	3.0	27.4	3100	0.055	0.045	266.6	180.6
5	Jul. 15 to 28	10.1	2.0	4.2	14.7	4.6	28.4	2490	0.056	0.047	121.5	82.3
6	Aug. 10 to Sep. 30	9.3	2.0	4.2	16.0	1.6	26.1	2780	0.049	0.041	139.5	94.5
7	Oct. 1 to Dec. 19	9.3	1.0	2.1	8.0	1.6~1.7	17.1	3680	0.096	0.061	48.3	32.5
8	Jan. 17 to Feb. 16, 1979	14.9	3.0	3.2	10.0	1.5	11.0	6630	0.074	0.035	46.3	23.9

Table 4-5 (Continued)

No.	Nitrification rate (mg/g-MLSS/hr.)	Nitrification rate (g/m ³ /hr.)	Nitrification efficiency (%)	Denitrification rate (mg/g-MLSS/hr.)	Denitrification rate (g/m ³ /hr.)	Denitrification efficiency (%)	T-N removal efficiency (%)	Influent BOD (mg/l)	Influent T-N (mg/l)	BOD/T-N ratio (-)	BOD removal efficiency (%)
1	-	-	-	-	-	-	-	79.8	-	-	82.8
2	0.50	2.37	91.7	0.53	2.52	43	52	69.8	28.1	2.5	82.0
3	1.0	3.58	99.7	1.2	4.43	57	59	101.5	36.5	2.8	93.9
4	1.1	3.35	99.5	1.2	36.9	52	53	84.9	34.0	2.5	96.0
5	1.4	3.37	99.6	1.0	2.38	32	34	72.1	34.5	2.1	94.0
6	1.1	2.90	99.5	0.9	2.45	38	41	69.9	32.5	2.2	94.5
7	1.6	5.60	95.5	1.0	3.79	30	35	72.4	32.1	2.3	90.7
8	0.7	4.85	86.5	0.4	27.1	41	54	95.7	30.9	3.1	92.4

When the water temperature was as low as 11°C, the nitrification efficiency remained at 86.5%. In other cases, however, the nitrification efficiency was more than 90%. As the influent BOD/N ratio was as low as 2.1 to 3.1, the denitrification efficiency remained at 30 to 57%, and the overall T-N removal efficiency was in the range of 35 to 59%. The SRT was 24 days in winter when the pilot plant was run with the detention time at 10 hrs. and 82 to 180 days in summer when the detention time was set at 15 hrs. The SRT in summer was extremely long because little excess sludge was formed as auto oxidation of sludge progressed. In winter, the BOD removal was degraded a little, but in almost all cases, the BOD removal efficiency was more than 90%; the BOD in the effluent was 5 to 7 mg/l as against 70 to 100 mg/l, in the influent.

The nitrogen removal efficiency of the RNDP is governed by factors such as influent BOD/N ratio, recycle ratio, detention time, volumetric ratio of denitrification tank to nitrification tank, MLSS concentration, and water temperature, etc. A mathematical model of the RNDP was developed based on the pilot plant test results to identify the interrelations between these factors and the design and operational requirements of the RNDP. An analysis of the mathematical model has already been submitted in the form of a progress report. (2)

5. SURVEY OF RNDP IN FULL-SCALE FACILITIES

The operating characteristics of the RNDP were clarified according to the 7 m³/day pilot plant tests. A full-scale demonstration plant of RNDP was designed based on the test findings and began operation in March 1981, and the process stability, and the factors governing the operation and maintenance, etc. were studied.

The demonstration plant used was one train of the conventional activated sludge plant in the Arakawa Wastewater Treatment Center. The principal particulars of this train are listed in Table 5-1. Its processing capacity as a conventional activated sludge process is 17,600 m³/day. In order to convert the conventional activated sludge process into the RNDP, modifications were made as follows.

Table 5-1 Principal particulars of experimental train

		Width (m)	Length (m)	Effective depth (m)	Number of compartments	Capacity (m ³)
Primary sedimentation tank		4.3	50.0	4.0	2	1720
Denitrification tank	Fore	9.0	21.25	5.0	1	956.25
	Aft	9.0	21.25	5.0	1	956.25
Nitrification tank	Fore	9.0	21.25	5.0	1	956.25
	Aft	9.0	21.25	5.0	1	956.25
Final sedimentation tank		4.3	56.0	4.45	2	2143

- (1) The aeration tank was halved into fore and aft parts, and the diffuser was removed from the fore part, and four units of submerged mixers (15 kW x 2 units and 11 kW x 2 units) were installed instead in order to form a denitrification tank.
- (2) The aft half of the aeration tank was used intact as a nitrification tank.
- (3) A submerged pump (5,400 m³/day) was installed in the nitrification tank for recycling nitrified liquor to the denitrification tank. When increase of the recycling rate was required, a standby sludge return pump was operated to feed nitrified liquor, together with return sludge, from the final sedimentation tank back to the denitrification tank.
- (4) A system for adding methanol and alkaline agent was installed to provide against the shortages of BOD and alkalinity in the influent.
- (5) A monitoring system was installed for continuous recording of DO in both the denitrification tank and nitrification tank and pH in the nitrification tank.

A flow diagram of the modified facilities is shown in Fig. 5-1. The modified facilities (demonstration plant) were put into operation in March 1981, and will be run continuously for one year. In this report, therefore, the operating results obtained for the three months from March till June are discussed.

Table 5-2 is a summary of the running conditions during the said three-month period.

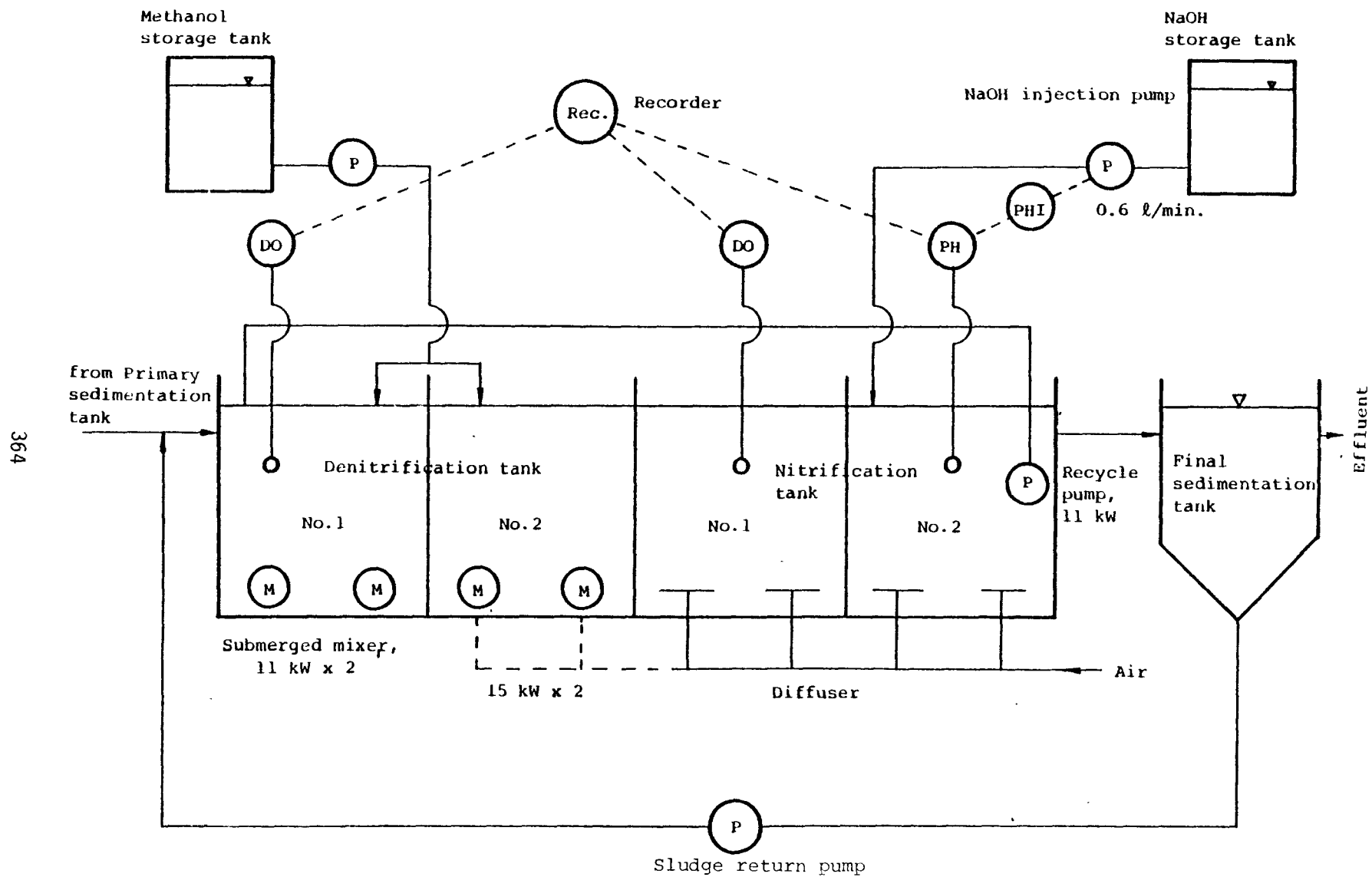


Fig. 5-1 Flow diagram of full-scale RNDP

Table 5-2 Demonstration plant running conditions

		Run 1	Run 2	Run 3	Run4
Period		Mar. 30 to Apr. 21, 1981	Apr. 22 to May 13	May 14 to Jun. 8	Jun. 9 to 29
Influent flow rate (Q) (m ³ /day)		5,000	6,000	6,000	6,000
Detention time (hrs.)	Denitrification tank	9.1	7.6	7.6	7.6
	Nitrification tank	9.1	7.6	7.6	7.6
	Total	18.2	15.2	15.2	15.2
Recycle ratio (%)	Return sludge pump	100	100	100	100
	Recycle pump	0	0	90	90
	Total	100	100	190	190
Aeration rate (×Q)		4.8	4.8	4.8~8.0	6.4~7.2
Sludge withdrawal rate (m ³ /day)		20~40	20	20	20 ~ 30
MLSS (mg/l)		2450 (1546 ~ 3820)	1980 (1710 ~ 2220)	2540 (2150 ~ 2830)	1920 (1510 ~ 2640)
SVI		114 (106 ~ 134)	110 (97 ~ 122)	127 (112 ~ 152)	120 (103 ~ 138)
SRT (days)		26 (18 ~ 42)	33 (27 ~ 39)	37 (19 ~ 48)	20 (11 ~ 30)

Table 5-3 is an analysis of the influent (effluent of the primary sedimentation tank) and the effluent quality. Of the total nitrogen (T-N) in the influent, 60 to 70% was accounted for by NH_4^+ -N, and the majority of the remainder by Org-N. NO_2^- -N and NO_3^- -N were almost naught. The BOD/N ratio of the influent was about 4, and the addition of methanol was not carried out.

Table 5-3 Quality of influent and effluent

Primary sedimentation tank effluent quality (1)

(mg/l)

	Water temperature	M-alkalinity	NH_4^+ -N	NO_2^- -N	NO_3^- -N	T-N	Dissolved T-N
Run-1 Mar. 30 to Apr. 21	15.5 (14.4~17.0)	133.0 (76.1~146.3)	15.8 (8.0~20.8)	N.D	0.08 (0.02~0.20)	22.7 (15.1~29.6)	18.3 (10.8~21.3)
Run-2 Apr. 22 to May 13	16.4 (15.5~17.2)	134.0 (127.0~146.8)	14.4 (12.5~16.4)	N.D	0.07 (0.03~0.12)	22.3 (19.8~24.8)	18.1 (14.7~19.6)
Run-3 May 14 to Jun. 8	16.6 (13.0~18.0)	129.7 (103.3~144.8)	13.1 (11.3~16.2)	N.D	0.06 (0.02~0.15)	21.9 (19.9~25.4)	16.3 (13.0~19.3)
Run-4 Jun. 9 to 29	18.4 (17.6~19.2)	130.1 (95.9~140.4)	13.7 (7.8~16.1)	N.D	0.13 (0.02~0.47)	21.0 (13.2~25.6)	17.0 (10.2~21.2)

Primary sedimentation tank effluent quality (2)

(mg/l)

	Org-N	Dissolved Org-N	COD	Dissolved COD	BOD	Dissolved BOD	SS
Run-1 Mar. 30 to Apr. 21	6.6 (6.0 ~ 9.2)	3.0 (1.9~4.8)	39.3 (32.5~45.2)	20.7 (12.0~25.1)	80.7 (56.4~96.9)	37.7 (22.0~46.9)	48 (28~71)
Run-2 Apr. 22 to May 13	7.8 (6.0 ~ 9.8)	3.6 (1.9~6.4)	39.9 (33.5~45.2)	20.7 (18.5~23.0)	83.6 (77.6~87.8)	40.2 (32.1~46.4)	53 (31~66)
Run-3 May 14 to Jun. 8	8.6 (6.8~10.2)	3.0 (1.5~6.1)	43.6 (39.3~46.2)	18.5 (16.1~20.7)	92.3 (72.0~108.5)	25.9 (14.4~45.6)	84 (77~96)
Run-4 Jun. 9 to 29	7.1 (4.2~9.4)	3.1 (1.0~5.0)	38.7 (28.6~47.4)	21.8 (12.7~43.7)	70.6 (50.6~86.2)	23.5 (16.3~30.0)	49 (29~98)

Table 5-3 (Continued)
Final sedimentation tank effluent quality (1)

	(mg/l)						
	pH	M-alkalinity	NH_4^+-N	NO_2^--N	NO_3^--N	T-N	Dissolved T-N
Run-1 Mar. 30 to Apr. 21	7.1 (6.6~7.6)	69.0 (34.1~123.6)	2.9 (0.4~9.8)	1.0 (0.2~2.4)	6.7 (2.5~8.7)	12.2 (8.1~19.5)	9.7 (7.9~11.3)
Run-2 Apr. 22 to May 13	7.2 (6.6~7.6)	62.1 (48.4~74.1)	1.2 (0.1~4.1)	0.17 (0.06~0.3)	7.7 (5.0~9.7)	10.1 (6.2~12.5)	9.9 (6.0~12.1)
Run-3 May 14 to Jun. 8	7.4 (7.0~7.7)	71.3 (60.3~93.9)	0.2 (0~1.8)	0.08 (0.01~0.15)	5.2 (3.1~6.2)	6.4 (5.4~7.3)	6.3 (5.2~7.0)
Run-4 Jun. 9 to 29	7.2 (6.8~7.4)	59.4 (50.9~68.7)	0.1 (0~0.7)	0.04 (0.01~0.14)	6.2 (4.2~7.4)	7.2 (4.9~8.6)	7.1 (4.7~8.5)

Final sedimentation tank effluent quality (2)

	(mg/l)						
	Org-N	Dissolved Org-N	COD	Dissolved COD	BOD	Dissolved BOD	SS
Run-1 Mar. 30 to Apr. 21	1.6 (0.3~6.5)	0.9 (0.6~1.1)	10.3 (9.7~11.7)	8.6 (8.0~10.0)	4.6 (3.0~8.4)	1.2 (0.7~1.9)	4.0 (2.4~5.7)
Run-2 Apr. 22 to May 13	1.0 (0.7~1.2)	0.8 (0.6~1.0)	8.6 (7.8~9.8)	7.3 (6.6~8.3)	3.5 (2.0~5.7)	1.1 (0.7~1.7)	3.4 (2.2~4.0)
Run-3 May 14 to Jun. 8	0.9 (0.6~1.4)	0.7 (0.5~1.1)	7.5 (6.0~8.9)	6.7 (5.6~8.2)	2.2 (1.6~2.8)	1.1 (0.7~2.1)	2.8 (1.4~5.0)
Run-4 Jun. 9 to 29	0.8 (0.6~0.9)	0.8 (0.6~0.9)	6.8 (6.1~8.2)	6.3 (5.4~7.2)	2.1 (0.8~4.7)	1.3 (0.4~2.0)	2.1 (1.1~5.5)

The alkalinity of the influent was about 130 mg/l, far higher than originally expected, and the pH conditioning with alkali agent was not carried out accordingly. The T-N value of the effluent was as high as 12.2 mg/l on the average in RUN 1, and the average T-N removal efficiency remained at 46.3% during the period. This may have been caused by insufficient nitrification because the acclimation period of the activated sludge was as short as about a week.

From RUN 2 on, the nitrification efficiency improved. In RUN 4, the effluent showed little or no NH_4^+ -N (0 to 0.7 mg/l) remaining.

In RUN 2, the effluent T-N was 10.1 mg/l on the average. In RUNs 3 and 4, it was below 8.6 mg/l, attaining a T-N removal efficiency of 66 to 71%. Both BOD and SS in the effluent were stabilized below 10 mg/l. Especially from RUN 2 on, they were almost less than 5 mg/l. It is inferred that this may have been due to the fact that the sludge settled fairly well with SVI of 100 to 130 and that the surface loading of the final sedimentation tank was as low as 10 to 12 $\text{m}^3/\text{m}^2/\text{day}$.

Fig. 5-2 shows an example of the composition of dissolved nitrogen in each tank.

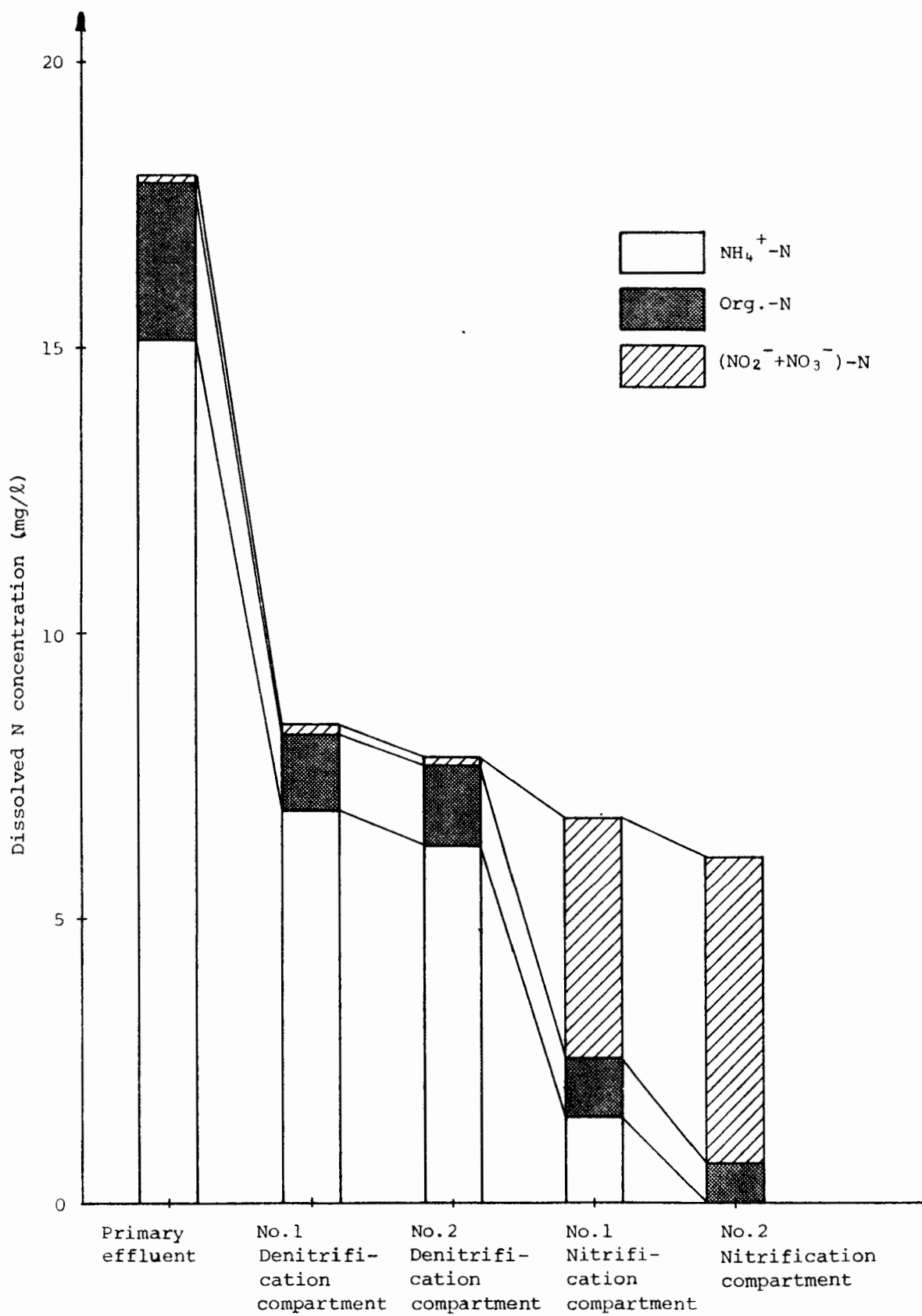
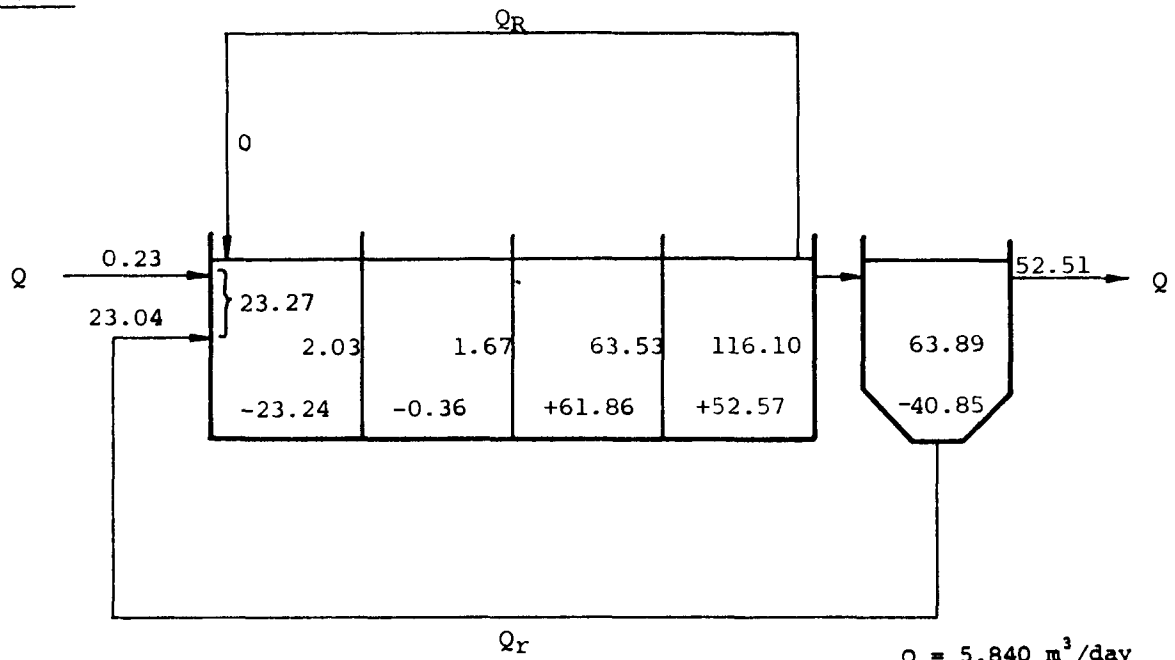


Fig. 5-2 Changes in dissolved nitrogen

The mass balance of $(\text{NO}_2^- + \text{NO}_3^-)\text{-N}$ in the RNDP is determined in Fig. 5-3.

Run 2



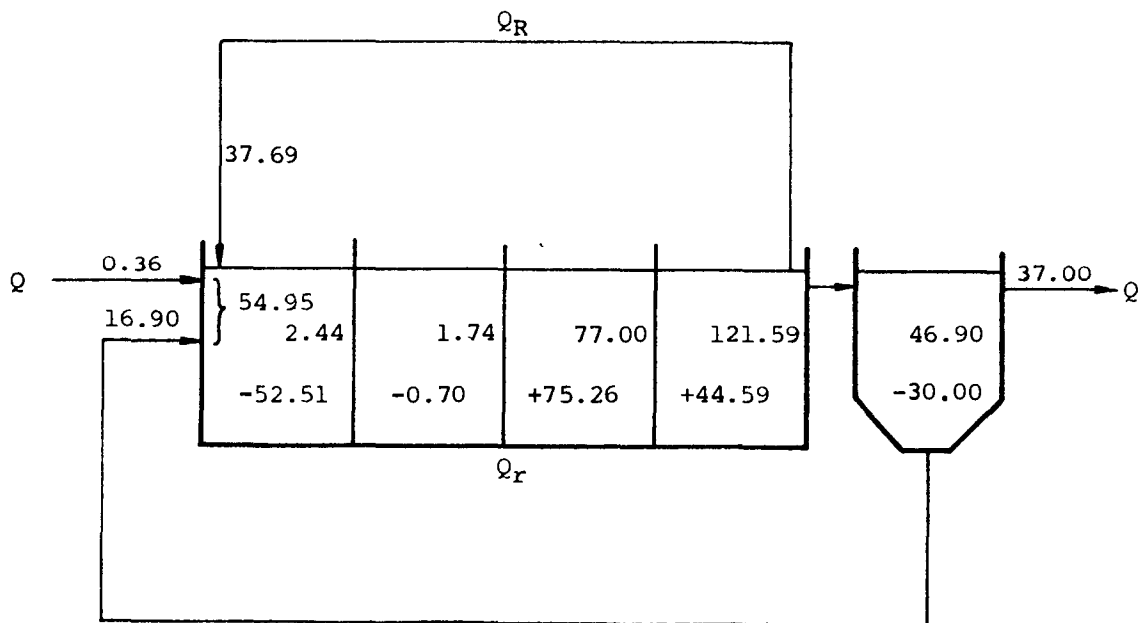
(kg/day)

$$Q = 5,840 \text{ m}^3/\text{day}$$

$$Q_r = 6,080 \text{ m}^3/\text{day}$$

$$Q_R = 0 \text{ m}^3/\text{day}$$

Run 3



(kg/day)

$$Q = 5,920 \text{ m}^3/\text{day}$$

$$Q_r = 6,100 \text{ m}^3/\text{day}$$

$$Q_R = 5,400 \text{ m}^3/\text{day}$$

Fig. 5-3 Mass balance of $(\text{NO}_2^- + \text{NO}_3^-)\text{-N}$

The values shown are determined by multiplying the $(\text{NO}_2^- + \text{NO}_3^-)\text{-N}$ concentration of composite sample by the flow rate, and are given in terms of kg/day. The plus sign before the figures shows an increase, and the minus sign a decrease of $(\text{NO}_2^- + \text{NO}_3^-)\text{-N}$. The increase corresponds to that part of $\text{NH}_4^+\text{-N}$ oxidized by nitrification, whereas the decrease corresponds to the amount of nitrogen taken into the sludge and denitrified into N_2 gas.

From Fig. 5-3, it is found that the final sedimentation tank has a large nitrogen removal capacity; namely, in RUN 2, the final sedimentation tank shouldered 63% of the total nitrogen removal efficiency, and in RUN 3, 36%. It is inferred that the denitrification in the final sedimentation tank is mainly undertaken by endogeneous respiration.

In RUN 2, the recirculation of the nitrified liquor was carried out from the final sedimentation tank alone, and the recycle ratio was about 1. In this run, little or no denitrification took place in the second denitrification compartment. RUN 3 was operated by driving the recycling pump to increase the recycle ratio to about 2 and hence to increase the concentration of $(\text{NO}_2^- + \text{NO}_3^-)\text{-N}$ at the inlet of the first denitrification compartment. In this case, the amount of nitrogen removed in the first denitrification compartment increased, and the amount of nitrogen removed in the final sedimentation tank decreased equally. Even in RUN 3, however, the denitrification in the first denitrification compartment was predominant, and the second denitrification compartment contributed little to the removal of nitrogen.

By comparing RUN 2 and RUN 3, it is found that nitrified liquor should preferably be recycled from the tail end of the nitrification tank. The reason is as follows.

- (a) The amount of $(\text{NO}_2^- + \text{NO}_3^-)\text{-N}$ running into the final sedimentation tank becomes small, reducing the amount of nitrogen to be removed in the final sedimentation tank. As a result, the volume of scum generated becomes small.
- (b) It is possible to reduce the surface loading of the final sedimentation tank to a minimum.
- (c) As the amount of $(\text{NO}_2^- + \text{NO}_3^-)\text{-N}$ running into the denitrification tank increases, the denitrification capacity of the denitrification tank can be used effectively.

Fig. 5-4 shows the relationship between the T-N removal efficiency (η) and the recycle ratio ($R = R_r + R_R$).

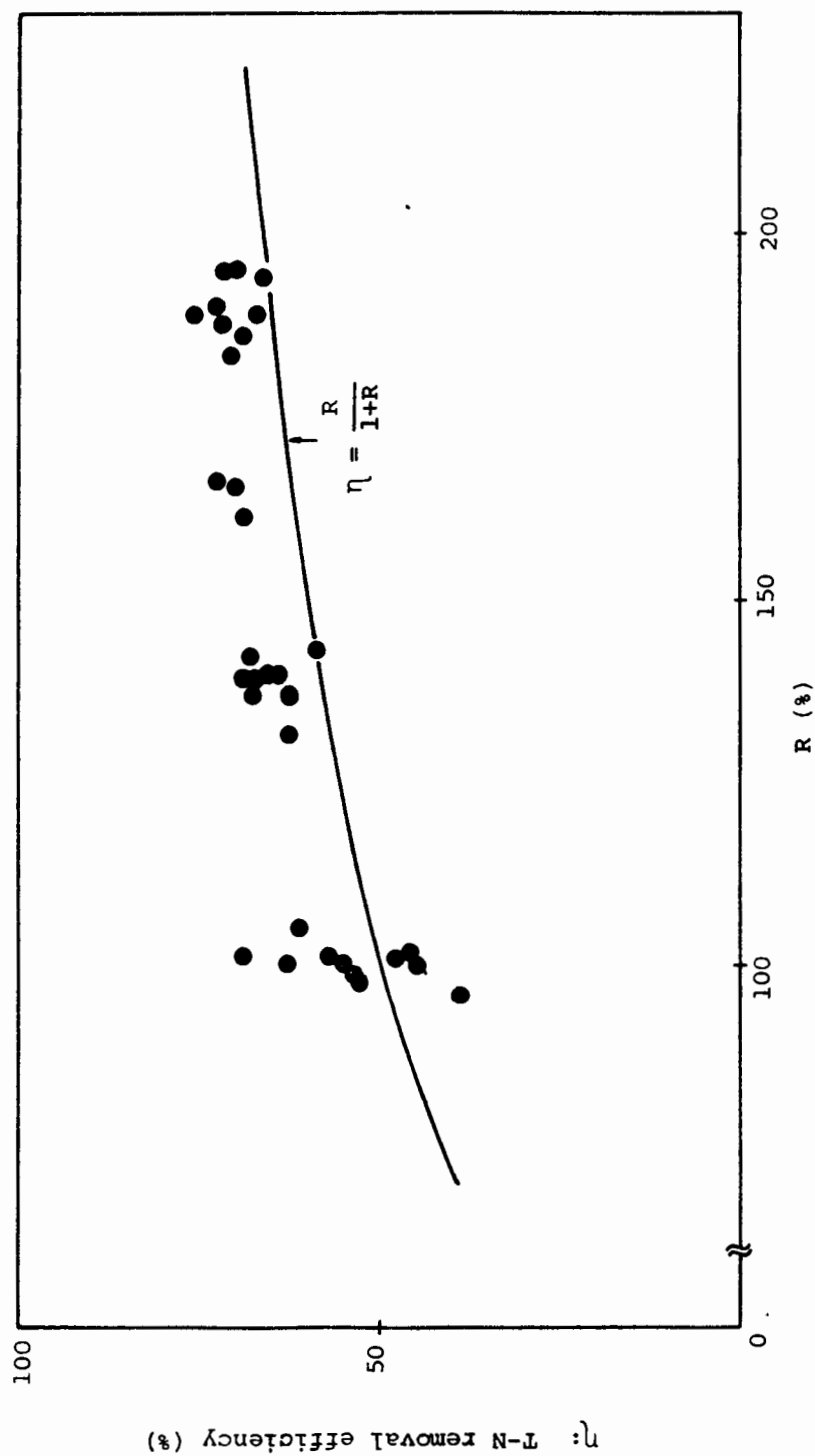


Fig. 5-4 T-N removal efficiency vs. recycle ratio

The solid line in Fig. 5-4 represents the theoretical maximum T-N removal efficiency ($\eta = \frac{R}{1+R}$). The measured values are 5 to 6% higher than the theoretical ones. The difference may be ascribable to the nitrogen component taken into the sludge.

As already stated, the demonstration plant is planned to be run for about one year. In the future, the influent flow rate will be increased as much as possible, and at the same time the detention time will be reduced. The problems concerning the operation and maintenance of the full-scale plant will be further investigated.

6. FEASIBILITY STUDY OF NUTRIENT REMOVAL PROGRAM

Originally, the Lake Biwa Regional Wastewater Treatment Plants were designed with the conventional activated sludge process, and the construction of the Konan Chubu Purification Center was started in October 1978 by the Japan Sewage Works Agency. In October 1979, the Regulations Concerning Eutrophication Control in Lake Biwa were promulgated. It was thus necessary to add nutrients removal processes to the conventional activated sludge process. On the basis of the effluent limitations on nitrogen and phosphorus stipulated in the regulations, the target effluent quality was set as shown in Table 6-1.

Table 6-1 Target effluent quality

	Design influent quality (mg/l)	Design effluent quality (mg/l)	Removal efficiency (%)
BOD	180	5	97
COD	70	20	71
SS	240	5	98
T-N	30	10	67
T-P	3.3	0.3	91

6.1 Alternative Combined Processes

According to the Japan Sewage Works Agency's Lake Biwa advanced wastewater treatment pilot plant survey results and also to the results of surveys conducted by the Public Works Research Institute of the Ministry of Construction, four alternative plans for the removal of nutrient salts at the Lake Biwa Regional Wastewater Treatment Plants were formulated.

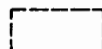
Table 6-2 lists the removal efficiencies of the four alternative combined processes and their constituent unit processes.

Table 6-2 Alternative processes for nutrients removal

Combined process	Flow diagram	Characteristics	Removal efficiency (%)								
			Unit process								
			h	a	b	c	e	f	d	g	Overall
I	<pre> graph LR A["Aeration tank (carbon oxidation + Nitrification + Denitrification)"] --> B["Final sedimen- tation tank"] B -- ① --> C["Alum precipi- tation"] B -- ② --> C B --> D["Rapid sand filter"] D -- ③ --> E["Activated carbon adsorption"] E -- ④ --> F[" "] </pre> <p>(c) (e) (f) (g)</p>	BOD				90	50	30		35	97.7
		COD				75	40	20		60	95.2
		SS				75	45	75		-	96.6
		T-N				60	10	10		-	67.6
		T-P				30	87.5	40		-	94.8
II	<pre> graph LR A["Aeration tank (Carbon oxidation + Nitrification)"] --> B["Final sedimen- tation tank"] B -- ① --> C["Alum precipi- tation"] B -- ② --> C B --> D["Packed bed denitrification"] D -- ③ --> E["Activated carbon adsorption"] E -- ④ --> F[" "] </pre> <p>(b) (e) (d) (g)</p>	BOD			90		50		30	35	97.7
		COD				75		40		20	95.2
		SS				75		45		85	97.9
		T-N				20		10		95	96.4
		T-P				30		87.5		50	95.6

Table 6-2 (Continued)

Combined process	Flow diagram	Characteristics	Removal efficiency (%)								
			Unit process								
			h	a	b	c	e	f	d	g	Overall
III	<pre>graph LR; A[Aeration tank
(coagulant addition)] --> B[Final sedimentation tank]; B -- 1 --> C[Rapid sand filter]; C -- 2 --> A; A -- d --> D[Activated carbon adsorption]; D -- 3 --> B; subgraph Labels; L1((1)) --- B; L2((2)) --- C; L3((3)) --- D; end</pre>	BOD		90				30		35	95.4
		COD		75				20		60	92.0
		SS		85				75		-	96.3
		T-N		20				10		-	28.0
		T-P		85				40		-	91.0
IV	<pre>graph LR; A[Aeration tank] --> B[Final sedimentation tank]; B -- 1 --> C[Alum precipitation]; C -- 2 --> A; A -- h --> D[Activated carbon adsorption]; B -- e --> E[Rapid sand filter]; E -- 3 --> D; D -- 4 --> C; subgraph Labels; L1((1)) --- B; L2((2)) --- C; L3((3)) --- E; L4((4)) --- D; end</pre>	BOD	85				50	30		35	96.6
		COD	70				40	20		60	94.2
		SS	80				45	75		-	97.3
		T-N	20				10	10		-	35.2
		T-P	30				87.5	40		-	94.8



Activated carbon adsorption process is planned in future

Process (I) removes nitrogen through RNDP into which a conventional activated sludge process is modified, removes phosphorus through the alum precipitation process, and then polishes the effluent through a rapid sand filter.

Process (II) uses an aeration tank for BOD removal and nitrification (CCONP), a alum precipitation process for phosphorus removal, and PBDP to denitrify and polish the effluent.

Process (III) removes BOD and phosphorus simultaneously by injecting alum into the aeration tank, and polishes the effluent through a rapid sand filter. It is not provided with a denitrification process.

Process (IV) uses a conventional activated sludge process to remove BOD, a alum precipitation process to remove phosphorus, and a rapid sand filter to polish the effluent. It is not provided with a denitrification process.

6.2 Comparison of Construction and Operation Costs

For the four alternative combined processes, rough design calculations were made with respect to plant capacities of 50,000 m³/day, 100,000 m³/day and 500,000 m³/day for the purpose of comparing the costs for construction, operation, and maintenance.

The conditions upon which the cost accounting was made are as follows.

- (1) The wastewater treatment facility, sludge treatment facility, administration office and electric facility were the subjects of construction cost calculation.
- (2) The sludge treatment facility was composed of thickening, dewatering and cake delivery, and was common to all the alternative processes.
- (3) The electric charges and chemical costs constituted the operation costs. The personnel costs were excluded.
- (4) The prices as of 1979 were used for calculation.

Table 6-3 shows the construction costs of the processes, and their cost functions.

Table 6-3 Construction costs and their functions

Unit: ¥10⁶

	Wastewater flow rate (Q) (10 ³ m ³ /day)			Cost function
	50	100	500	
Process I	6,786	11,316	42,950	$284Q^{0.806}$
Process II	6,523	10,836	39,149	$302Q^{0.782}$
Process III	4,703	7,829	26,665	$244Q^{0.755}$
Process IV	5,695	9,278	33,492	$270Q^{0.774}$

Table 6-4 shows the operation costs of the processes, and their cost functions.

Table 6-4 Operation costs and their functions

Unit: ¥/m³

	Wastewater flow rate (Q) (10 ³ m ³ /day)			Cost function
	50	100	500	
Process I	22.86	21.93	20.16	$28.2Q^{-0.054}$
Process II	25.67	24.39	22.87	$30.8Q^{-0.048}$
Process III	13.72	12.50	11.33	$18.4Q^{-0.079}$
Process IV	16.22	14.84	13.53	$21.4Q^{-0.075}$

Processes (I) and (II) are designed to remove not only BOD and SS, but also nitrogen and phosphorus. Compared with Process (II), Process (I) costs 4 to 10% more in terms of construction cost. As regards the operation cost per unit volume of wastewater, Process (II) costs 11 to 13% more than Process (I).

Processes (III) and (IV) are designed mainly for removal of BOD, SS, and phosphorus. It is found that Process (III) in which coagulant is injected into the aeration tank is 14% to 20% less in terms of construction cost than Process (IV) which has an independent precipitation process. As regards the operation cost, Process (III) costs 15% to 17% less than Process (IV).

6.3 Evaluation of Alternative Processes

For the purpose of determining a buildup plan for the Lake Biwa Regional Wastewater Treatment Plants, the four alternative processes were evaluated according to the following requirements.

- (1) To meet the target effluent quality specified in Table 6-1.
- (2) To minimize the plant modification as the construction or design of the plants is under way for a conventional activated sludge process.
- (3) To make the best use of the already installed facilities and to minimize the modification costs.
- (4) To minimize the increase in the operation and maintenance cost due to the addition of nitrogen and phosphorus removal processes to the secondary treatment facilities.
- (5) To make the entire system flexible for future enlargement of the plants.

According to these requirements, the process evaluations were made and Process (I) was finally chosen. Table 6-5 shows the estimated quality of influent of the Konan Chubu Purification Center and its design effluent quality based upon Process (I).

Table 6-5 Estimated Influent and Effluent Quality

	Influent	Removal Efficiency (%)				Effluent
	(mg/l)	Primary Sedimentation	RNDP	Alum Precipitation	Rapid sand Filter	(mg/l)
BOD	180	30 (126)	90 (12.6)	50 (6.3)	30 (4.4)	4.4
COD	70	25 (52.5)	75 (13.1)	40 (7.9)	20 (6.3)	6.3
SS	240	40 (144)	75 (36.0)	45 (19.8)	75 (5.0)	5.0
T-N	30	13 (26.1)	60 (10.4)	10 (9.4)	- -	9.4
T-P	3.3	13 (2.9)	30 (2.0)	85 (0.3)	40 (0.2)	0.2

Remark: Figures in parenthesis show effluent quality of each unit process.

Two wastewater treatment plants of Lake Biwa Regional Sewerage System are now under construction adopting Process (I). In 1982, the Konan Chubu Purification Center will start service. This marks the first practical step in the implementation of the nutrients removal process in Japan. And every effort will be made to ensure the success of the eutrophication control project in Lake Biwa.

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Eighth US/JAPAN Conference
on
Sewage Treatment Technology

EFFECT OF DETERGENTS AND SOAP ON MUNICIPAL WASTEWATER TREATMENT

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1. INTRODUCTION

In the field of sewage treatment, it was in the 1960's that increasing concern was first paid to synthetic detergents. This was because ABS, that was considered to be biologically hard, brought about a foaming reaction in aeration tanks or some other apparatus. In Japan, softening of synthetic detergents was started in 1965 and completed in 1973. Afterwards, there is almost no problem of the surfactants contained in synthetic detergents in sewage treatment.

Lately, however, a number of people have again been concerned with synthetic detergents. The two events described below have led to this concern.

- 1) For the purpose of water pollution control in Lake Biwa, Shiga prefectural authorities established a regulation against the sale and use of synthetic detergents containing phosphate. It is called the "Regulation on the Prevention of Eutrophication in Lake Biwa" and was put into force in July, 1980.
- 2) Mishima city authorities distributed synthetic detergent and washing powder to each home in a certain housing area in order to compare their influence upon the sewage treatment process. As a result, it was found that organic loads and PO_4 load of influent were reduced and the water quality of effluent was markedly improved by replacing synthetic detergent with powdered soap.

With the above two events as a turning-point, a considerable number of local public authorities have investigated the influence of phosphate-containing synthetic detergents, phosphate-free synthetic detergents and powdered soap upon the environment and the sewage treatment.

Fig. 1 shows the products of synthetic detergents and soaps in Japan. The amount of soaps used over the period from 1976 to 1980 was $1.36 \sim 1.73$ kg/head/year (average 1.48 kg/head/year), and the amount of synthetic detergents used was $5.88 \sim 9.14$ kg/head/year (average 7.75 kg/head/year). The total amount of detergents used was $8.24 \sim 10.70$ kg/head/year (average 9.23 kg/head/year). The term "detergents" used here is intended to include synthetic detergents and soaps.

Almost all the surfactants as detergent materials are made from petrochemicals except for the fatty acid salts and certain types of higher alcohol. Major surfactants used in Japan include linear alkyl benzene sulfonate (LAS), alkyl sulfate (AS), polyoxyethylene alkyl ether sulfate (AES), alpha-olefin sulfonate (AOS), fatty acid salt (FA), and polyoxyethylene alkyl ether (AE). AE is a nonionic surfactant and the others are all anionic surfactants.

The present report evaluates the influence of detergents on the sewerage system, based on the indoor experiments and on the field investigation conducted at sewage treatment plants. The purpose of this investigation was to clarify the following:

- 1) Discharge of surfactants from houses to a sewage treatment plant
- 2) Concentration of surfactants in sewage
- 3) Behavior of surfactants in sewage treatment process
- 4) Influence of detergents on activated sludge process

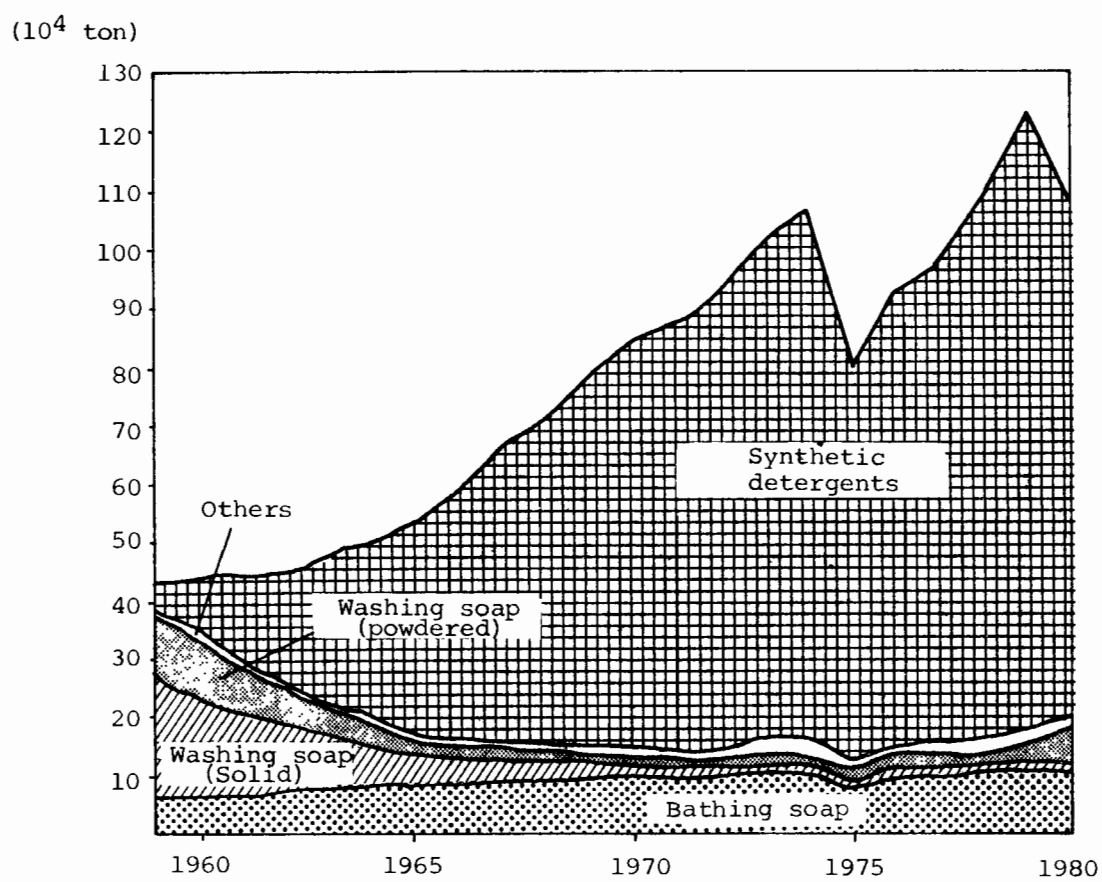


Fig. 1 Products of synthetic detergents and soaps in Japan

2. ANALYSIS OF SURFACTANTS

2.1 ANIONIC SURFACTANTS (EXCLUDING FATTY ACID SALTS)

The concentration of anionic surfactants in the water sample was determined by the colorimetric method of methylene blue active substances (MBAS).

The concentration of anionic surfactants in sludge was determined by measuring MBAS in the extract which had been obtained through the reflux of sludge with ethyl alcohol.

2.2 NONIONIC SURFACTANTS

The concentration of nonionic surfactants in the water sample was determined by the colorimetric method of cobalt thiocyanate active substances (CTAS).

The concentration of nonionic surfactants in sludge was determined by measuring CTAS in the extract which had been obtained through the reflux of sludge with ethyl alcohol. However, CTAS was only derived from the surfactant, but also detected from activated sludge. The concentration of CTAS from activated sludge was about 20 mg.CTAS.2000 mg.MLSS.

2.3 FATTY ACID SALTS

The fatty acid salts in the water sample were extracted with chloroform and acetylated with the addition of N,N-dimethylformamide dimethylacetal, and determined by gas chromatography (FID).

Fig. 2 shows one example of the gas chromatograms prepared for the measurement of the fatty acid salts.

The fatty acid salts in sludge were extracted with chloroform by use of soxhlet extraction and the extract was operated in the same manner as the water sample.

The fatty acid salts measured were palmitate, oleate and stearate. However, oleate was calculated in terms of stearate and the sum of palmitate and stearate was regarded as the value of fatty acid salt.

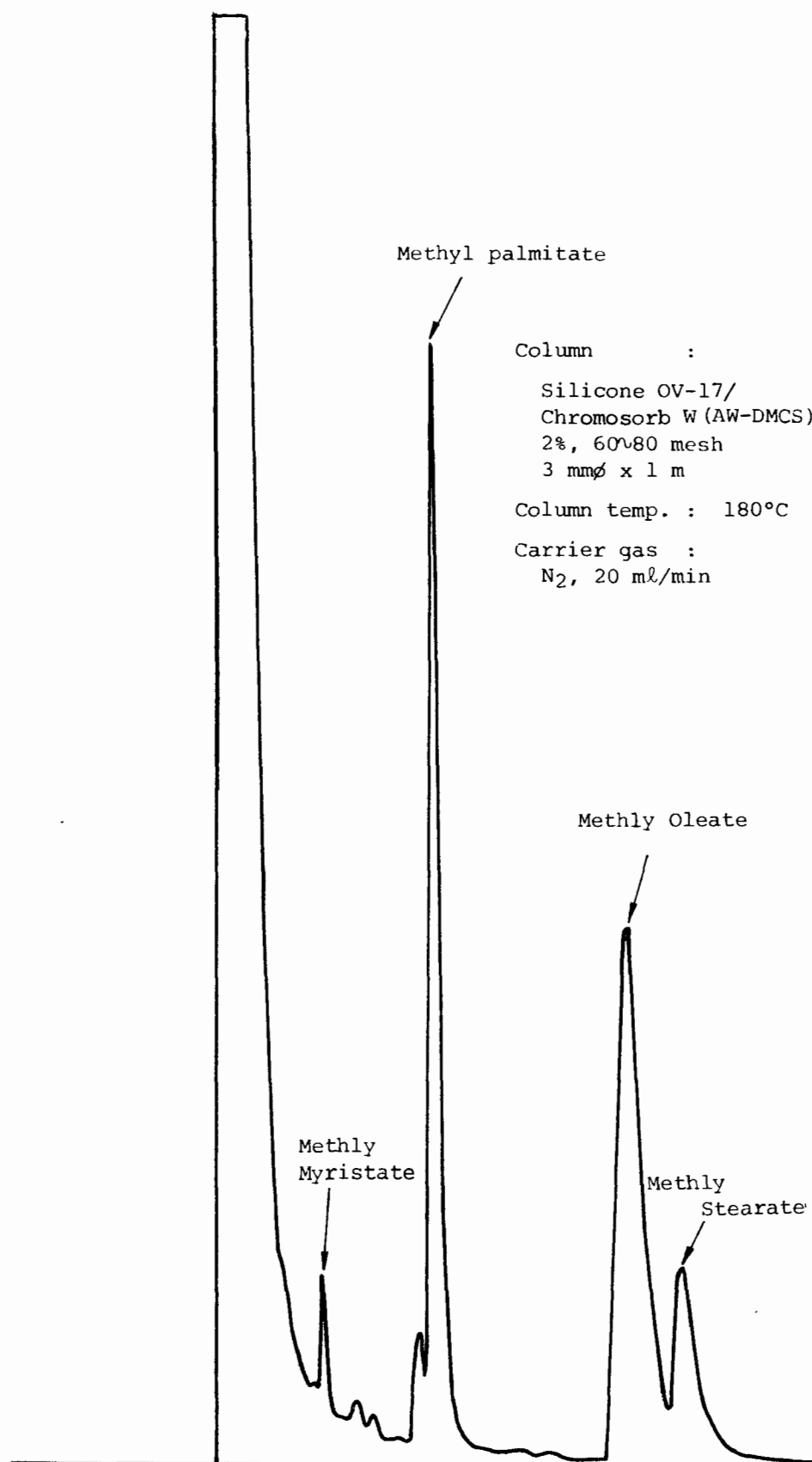


Fig. 2 Gaschromatogram of fatty acid methyl esters
390

3. LOSS OF SURFACTANTS IN SEWER SYSTEM

3.1 EXPERIMENT ON SOLIDIFICATION OF DETERGENTS

3.1.1 Method

This experiment was conducted as follows. Sample solutions of sewage were filtered with filter paper No. 5C and detergents were added. Then, the solutions were stirred for 1 hr with stirrers and SS, TS, VTS, COD, and TP in the suspensions were measured. Further, the suspensions were filtered with GF/B, and COD and TP in the filtrates were measured. The types of detergents used in this experiment are shown in Table 1. The amounts of detergents added were 500, 1000, 2000 and 3000 mg/l respectively.

Table 1 Detergents and their major components

Detergents		Major components
I	Powdered washing soap	Surfactant (70%)
II	Synthetic detergent	Surfactants (24%), Phosphate (10%) Sulfate (10%), Silicate (10%)
III	Phosphate free synthetic detergent	Surfactants (20%), Sulfate (10%), Silicate (10%), Almino Silicate (<10%)
IV	Phosphate free synthetic detergent	Surfactants (24%), Carbonate (10%), Sulfate (10%), Silicate (10%), Almino Silicate (<10%)

The standard amount of detergents used for washing at home is 40 g/30 l.

3.1.2 Results and discussion

The solidification rate of detergents was calculated in terms of $(SS/TS) \times 100$. As a result, the rates of detergents I, II, III and IV were 15.8 ~ 51.2% (average 30.4%), 4.6 ~ 8.1% (average 6.2%), 10.5 ~ 13.1% (average 12.0%) and 8.3 ~ 9.8% (average 8.9%) respectively. For detergent I, there was a tendency for the solidification rate to decrease as its concentration was increased. For the other detergents, the solidification rate was almost constant regardless of their concentration.

Assuming that the solidification rate of organic components in detergents can be expressed as the solidification rate of COD_{Mn} , that of organic components was calculated in terms of $(1 - D \cdot COD/COD) \times 100$. As a result, the rates of detergents I, II, III and IV were 36.2 ~ 73.3% (average 56.6%), 2.2 ~ 12.6% (average 7.3%), 0 ~ 12.6% (average 3.2%) and 0 ~ 5% (average 1.8%) respectively. The organic matter contents of the above detergents were calculated in terms of $(VS/TS) \times 100$, and they were 62.4%, 30.9%, 19.2% and 20.7% respectively. The product of the organic matter content in a detergent with its solidification rate of organic components represents the solidification rate of organic components for the detergent. The values in detergents I, II, III and IV were 35.3%, 2.3%, 0.6% and 0.3% respectively. Comparing these values with the solidification rate of detergents, the following can be considered.

- 1) For washing powder, 30 ~ 35% of the powder added to water is solidified, solidification rate of soap is higher than that of synthetic detergents. The solid matter is almost completely composed of organic substances. Therefore, it seems that the high solidification rate of powdered soap is due to the formation of metallic soap as a result of the reaction with Mg^{++} and Ca^{++} ions in the sewage.

- 2) The solidification rate of phosphate-containing synthetic detergents is about 6%, rather low in comparison with that of powdered soap and phosphate-free synthetic detergents. About 1/3 of the solid matter is composed of organic matter and the remainder is inorganic. It is confirmed that phosphate, one of the major inorganic components, completely dissolves in water. Further, it is hardly considered that sulfate becomes solidified. Accordingly, most of the solidified inorganic matter is presumed to be silicate.
- 3) The solidification rate of phosphate-free synthetic detergents is 9 ~ 12%, rather higher than that of phosphate-containing synthetic detergents. Inorganic matter occupies 95 ~ 97% of the solid matter. The solidified inorganic matter is presumed to be composed mainly of silicate and almino silicate.

3.2 SEDIMENTATION OF SURFACTANTS IN HOUSE INLET AND THEIR ADHESION TO SEWERS

3.2.1 Method of investigation

It has been said that when synthetic detergents for washing are replaced by powder soap, the load of organic pollutants, especially BOD load, reaching the sewage treatment plant will increase. In order to estimate such increase of pollutants, it must be taken into consideration that powdered soap precipitates in house inlets or adheres to the sewers. As a result, the rate of the pollutants reaching the treatment plant becomes constant, which means that equilibrium is established in the sedimentation, adhesion, desquamation, and biodegradation of powdered soap.

This investigation was conducted at the F sewage treatment plant in F City in March 1981. The F sewage treatment plant and its covering area are summarized below:

Inhabitants	5700
Inflow	1200 m ³ /day
Aeration time	13.7 hrs
Return sludge rate	100%

Surface loading of primary sedimentation tank	6 m ³ /m ² /day
Surface loading of final sedimentation tank	5.3 m ³ /m ² /day

The F sewage treatment plant is located in a sloping area of the city, so that time sewage takes to reach the plant is about 20 min.

Fig. 3 is a flow diagram of the F sewage treatment plant.

In order to investigate the influence upon the sewage treatment when syenthetic detergents for washing has been replaced by powdered soap, the F City authorities distributed washing powder free of charge to each household in this area over a four-month period from September to December 1980.

The results of the investigation during this period have not yet been compiled. In the households of this area, 78.6% of the total had been using synthetic detergents until September 1980, but during the period from September to December 1980, 98.9% of the total households used washing powder instead of synthetic detergents. In January 1981, a questionnaire survey was conducted concerning the use of powdered soap in each home of this area. According to the results of the survey, 70% of the total households answered that they would continue to use powdered soap. Accordingly, it is presumed that a considerable number of the households were using powdered soap during the period of this investigation, which was conducted in March 1981.

The investigation on the sedimentation of soap in house inlets was conducted at the house inlet for five households in a 5-story public apartment house in this area. First, all the sediment deposited in the house inlet was collected, and COD_{Mn}, TS, TP, KN, MBAS, CTAS, and fatty acid salts in the sediment were measured. Two days later, the sediment was again collected from the same house inlet, and the same items were measured. The investigation on the adhesion of soap to the sewer was conducted at two manholes in this area, collecting the adhesive from the bottom or thereabout of the Hume concrete pipe. Then, COD_{Mn}, TS, TP, KN, MBAS, CTAS, and fatty acid salts in the adhesive were measured. Two days later, the adhesive was again collected from the same pipes, and measurements were made for the same items.

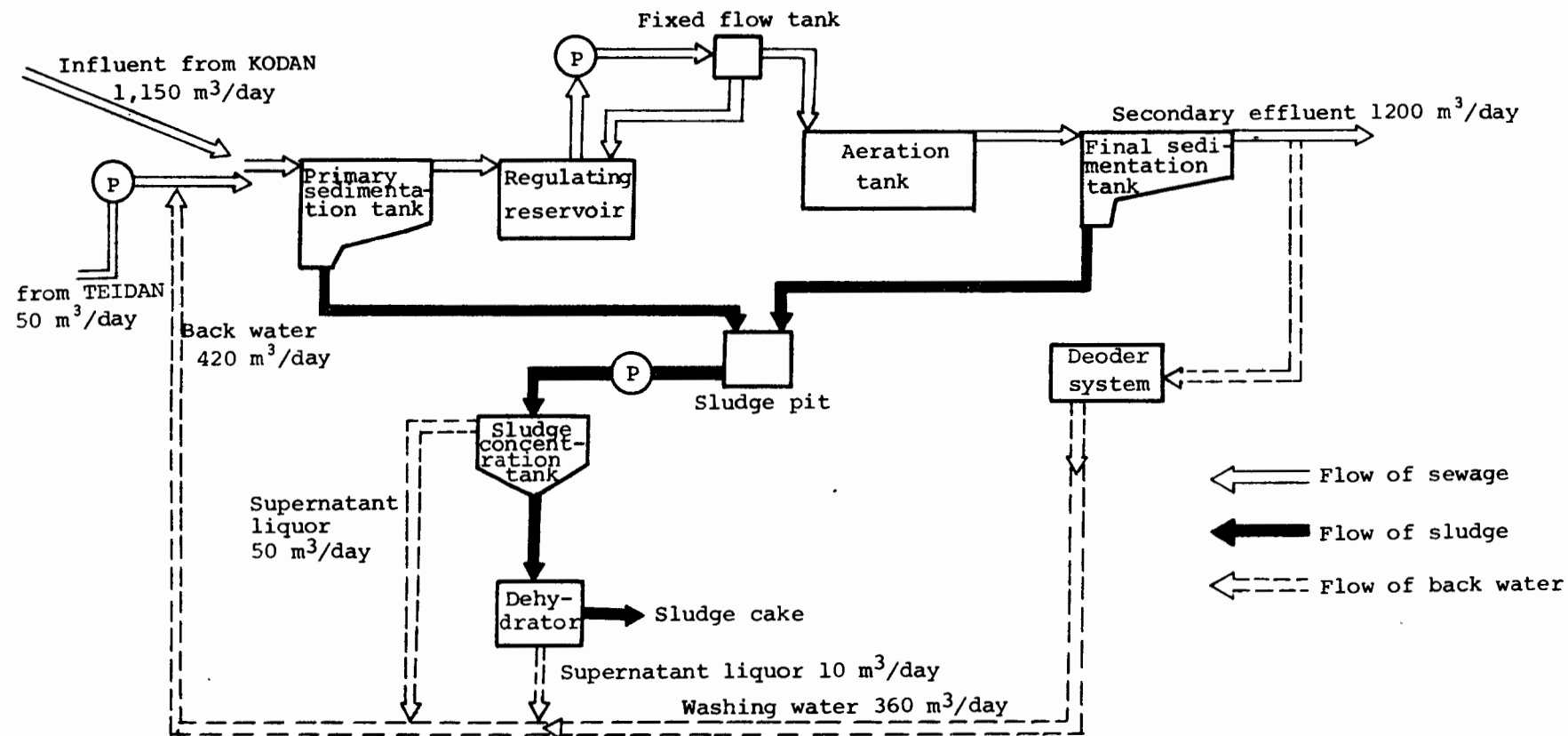


Fig. 3 Flow diagram of F sewage treatment plant

The term "adhesive" used herein indicates the solid matter adhering to the metal brush with which a fixed area of the Hume pipe was brushed.

3.2.2 Results and discussion

Table 2 shows the results of measurement for the sediments in a house inlet and the adhesive to Hume concrete pipes.

The drainage, not containing feces, from 5 households ran into the house inlet, of which the size was 45 cm x 45 cm x 60 cm deep. Assuming that the average number of families for each household is 3.5, the total number of persons for the house inlet is 17.5. Assuming that all the house inlets in this area have the function equal to the investigated inlet, the sediment per day in this area is estimated as follows:

Example: Fatty acid salts

Sediment for two days	22.4 g/5 families/2 days
Sediment per head per day	$(22.4 \div 17.5) \div 2 =$ 0.64 g/head/day
Total sediment in this area	$\frac{0.64 \times 5700}{1,000} = 3.65 \text{ kg/day}$

The value of 3.65 kg/day is considered to be maximum value per day in this area (maximum sediment). The results of calculation on the maximum sediment in the house inlet are shown in Table 2.

The estimation of the adhesive in the Hume concrete pipes is also shown in Table 2. The estimation of maximum adhesive was conducted as follows:

Example: Fatty acid salts

Adhesive for two days	0.1 g/m ² /2 days
Total length of sewer pipe	14,140 m
Pipe diameter	ø250 ~ 400 mm, assuming that the average is ø300 mm.
Total area of pipe wall	13,300 m ²
Total adhesion in this area	$\frac{0.1 \times 13300}{2 \times 1000} = 0.66 \text{ kg/day}$

The value of 0.66 kg/day is considered to be maximum value per day in this area (Maximum adhesive).

The actual adhesive on the Hume pipes is presumed to be a far smaller value than the maximum adhesive (Table 2). Because the adhesive on the Hume pipe does not adhere to the whole wall inside the pipe in a uniform manner and much more adhesive is considered to be on the portion closer to the bottom of the Hume pipe, while the adhesive in the pipe was estimated by the values of the portion close to the bottom. Further, the maximum adhesive obtained is the value for a new wall of the Hume pipe.

Table 3 shows the maximum sedimentation in the house inlet, the maximum adhesive in the Hume pipe and the discharge from house to sewage treatment plant. Taking into consideration that the maximum adhesive on Hume pipe has been overestimated, the discharge of TP, KN, MBAS and CTAS is presumed to be more than 95%. However, even considering that the maximum adhesive on Hume pipes has been overestimated, the discharge of fatty acid salts is considered to be about 70%.

Table 2 Measurement of sediments in house inlet and adhesive in Hume concrete pipe

		COD _{Mn}	TS	VTS	TP	KN	MBAS	CTAS	Fatty acid salts
House inlet	Equilibrium sediments (g/5 families)	41.5	1250	467	5.43	33.4	1.44	2.19	72.5
	Sediments for two days (g/5 families/two days)	-	280	187	1.32	5.98	0.73	2.06	22.4
Hume pipe	Equilibrium adhesive (g/m ²)	12.8	55.3	30.3	1.09	3.6	0.85	0.16	0.35
	Adhesive for two days (g/m ² /two days)	3.06	11.9	6.4	0.33	0.55	0.058	0.026	0.10

Table 3 LOSS of TP, KN and surfactants in sewer system

		TP	KN	MBAS	CTAS	Fatty acid salts
I	Maximum sediment in house inlet (kg/day)	0.2	1.0	0.12	0.34	3.65
II	Maximum adhesive in Hume concrete pipe (kg/day)	2.2	3.6	0.38	0.18	0.66
III	Influent loadings* (kg/day)	11.1	67.8	5.51	7.24	9.26
	Discharge from house to sewage treatment plant (%) (III/(I+II+III) x 100)	82.2	93.6	91.7	93.3	68.2

* Refer to Table 5, inhabitants 5700

4. BEHAVIOR OF SURFACTANTS IN SEWAGE TREATMENT PLANT

4.1 SURFACTANT CONCENTRATION IN SEWAGE AND SLUDGE

There are several reports on MBAS concentration in the influent. Sudo²⁾ conducted investigation at a municipal sewage treatment plant in 1979, and reported that MBAS in influent was 5.9 ~ 7.9 mg/l and the removal by the secondary treatment was 93 ~ 96%. Yamane et al.³⁾ report that MBAS in influent was 11.7 mg/l (weekly mean) at a housing area sewage treatment facilities for the inhabitants of 1000 ~ 1400.

In this investigation, in order to clarify the surfactants in sewage and sludge, MBAS, DTAS and fatty acid salts were fractionally determined.

4.1.1 Method of investigation

Investigation was conducted 7 times at 5 plants. For influent, primary effluent and secondary effluent, sampling was conducted at 1 or 2 hour intervals and a 24 hr composite sample was prepared by mixing in proportion to the flow for each day. The primary sludge and waste activated sludge were sampled once a day.

MBAS, CTAS and fatty acid salts in each sample were analyzed.

4.1.2 Results and discussion

The results of analysis for each type of surfactant is shown in Fig. 4. MBAS in the influent was 2.5 ~ 6.4 mg/l (average 4.7 mg/l), CTAS 1.9 ~ 4.8 mg/l (average 3.1 mg/l), and fatty acid salts 2.4 ~ 6.7 mg/l (average 4.1 mg/l). MBAS in the primary effluent was 2.5 ~ 15.0 mg/l (average 5.9 mg/l), CTAS was 1.8 ~ 23.0 mg/l (average 7.9) and fatty acid salts was 0.6 ~ 2.3 mg/l (average 1.6 mg/l). MBAS and CTAS in the primary effluent were high in comparison with those in the influent. It was because the values at one plant were abnormally high. If these abnormal values are excluded, MBAS and CTAS were 3.6 mg/l and 4.1 mg/l, respectively, on the average. MBAS in the secondary effluent was 0.3 ~ 0.7 mg/l (average 0.5 mg/l), CTAS was 0.1 ~ 1.1 mg/l (average 0.6 mg/l) and fatty acid salts were 0.4 ~ 4.3 mg/l (average 1.6 mg/l).

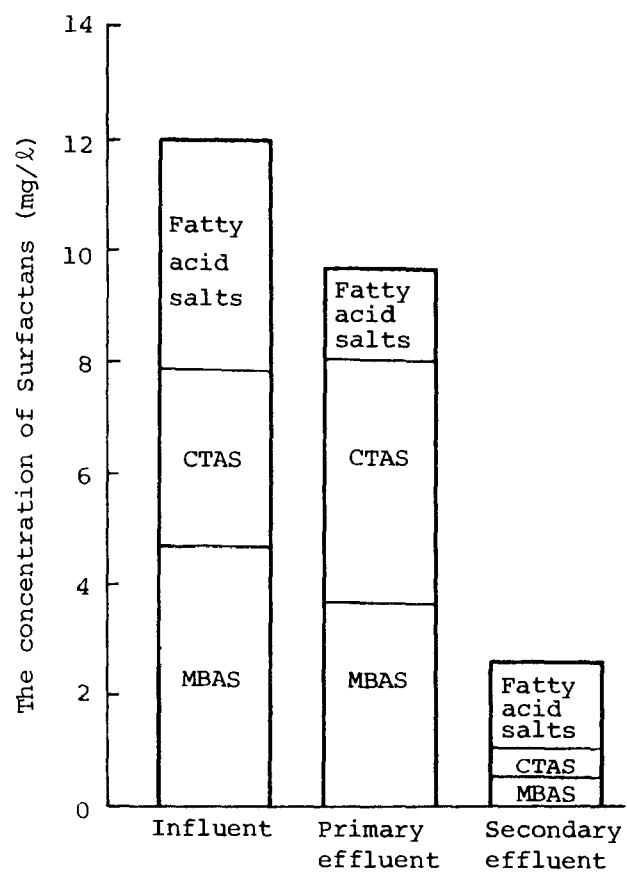


Fig. 4 Surfactants in sewage

As a result, MBAS removal was 23% in primary sedimentation. CTAS concentration was 32% higher in the primary effluent than in the influent. The cause is not apparent, but the influence of back water from the sludge treatment system can be considered. Because CTAS not derived from detergents was detected in activated sludge. The removal of fatty acid salts was 61% in primary sedimentation.

The removal by secondary treatment were 86% for MBAS, 85% for CTAS and 0% for fatty acid salts.

Therefore, it was found that MBAS and CTAS were removed mainly in the activated sludge treatment process and fatty acid salt was removed mainly in the primary sedimentation tank.

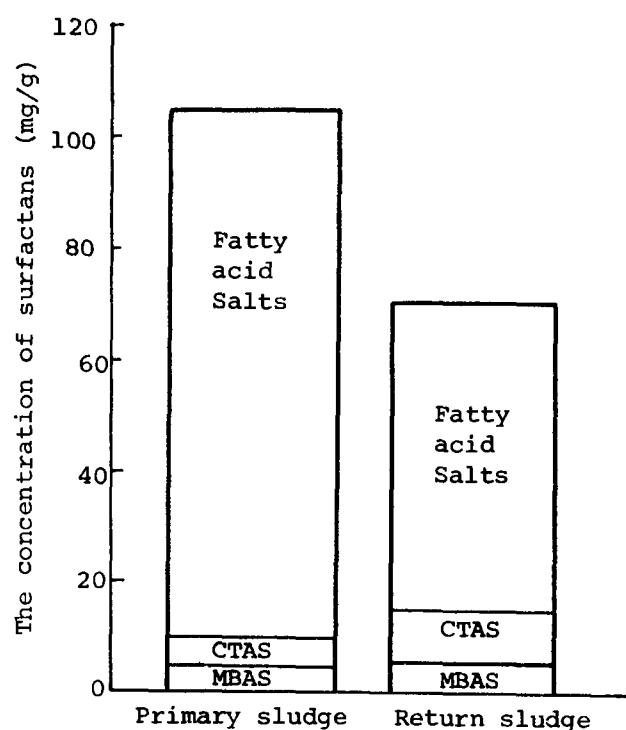


Fig. 5 Surfactants in sludge

The results of analysis for several types of surfactants contained in primary sludge and return sludge is shown in Fig. 5. In primary sludge, MBAS was average 3.8 mg/g, CTAS was average 3.0 mg/g and fatty acid salts were 96.9 mg/g. The concentration of fatty acid salts in primary sludge was high, which coincides with the tendency of fatty acid salts to be removed in primary sedimentation.

In return sludge, MBAS was 3.4 mg/g, CTAS was 19.7 mg/g and fatty acid salts were 47.2 mg/g.

As a result of analyzing the activated sludge that was cultivated with detergent-free synthetic sewage, MBAS was 1 ~ 2 mg/g, CTAS was 10 ~ 11 mg/g and fatty acid salts were about 5 mg/g. These results suggest that the high CTAS value found in return sludge was not due to the detergent itself and certain components in activated sludge were detected as CTAS.

As for fatty acid salts found in return sludge, the influence of food oil and others can be considered as one of the causes for its high concentration.

4.2 BEHAVIOR OF SURFACTANTS IN SEWAGE TREATMENT PROCESS

4.2.1 Method of investigation

Investigation was conducted at F sewage treatment plant in F City. Treatment facilities and operational conditions have already been outlined in 3.2.1.

As shown in Fig. 3, the influent to F treatment plant was composed of "influent from KODAN" coming from the major part of a housing area and "influent from TEIDAN" consisting of 50 m³/day of domestic sewage and 420 m³/day of back water. For the influent from TEIDAN, as the fluctuation of pollutants load was considered to be low in the course of a day, sampling was conducted 4 times a day, but for the influent from KODAN, as a rule, sampling was conducted every hour.

For the influent to the aeration tank and secondary effluent, samples were collected every hour, and 2-hour composite samples were prepared.

The measured items are as shown in Table 4.

Table 4 Measured items

Influent	Flow (overall width weir method)				
Effluent	Flow (flowmeter)				
		COD	D-TOC	MBAS	pH
Water temperature		D*-COD	K-N	D*-MBAS	BOD
Fatty acid salts		SS	D-K-N	CTAS	SS
D*-Fatty acid salts		TS	T-P	D*-CTAS	
		VTs	D*-T-P		

Note: * indicates the filtrate filtered with GF/B.

4.2.2 Results and discussion

The results of investigation are summarized in Table 5. For the average concentration of influent, BOD was 201 mg/l, COD 91.4 mg/l, SS 219 mg/l, KN 44 mg/l and TP 7.2 mg/l. These are average values for small-scale treatment plants for housing areas in Japan. In the plant, as the regulating reservoir was installed, the water quality was uniformized to a certain degree so that the minimum concentration of influent to the aeration tank was higher than that of the influent to the plant.

Concerning these general water quality items, their removal by treatment was favorably conducted and the operation of the plant was excellent during the investigation.

MBAS in influent was 3.6 mg/l, CTAS 4.7 mg/l, and fatty acid salts 6.0 mg/l. MBAS concentration was low in comparison with 11.7 mg/l³⁾.

The proportions of suspended type of these components in influent was almost 0% for MBAS, 20% for CTAS and 56% for fatty acid salts. MBAS is almost soluble, but its removal before the aeration process was nearly 30%. In this plant, the retention time in the primary sedimentation tank was 9.3 hrs and, if the retention time in the regulating reservoir is added to the above time, the average retention time becomes 13.7 hrs. A considerable biodegradation must be conducted through these processes. The removal of fatty acid salts was 77% before the aeration process. Therefore, it can be considered that biodegradation took part in the removal of the above fatty acid salts.

In the aeration tank and final sedimentation tank, MBAS removal was 86% and CTAS was 91%. The concentration of fatty acid salts in influent of the aeration tank was comparatively low. Its removal was only 67% by activated sludge process, but the total removal through the whole process reached 92%.

The total removal of MBAS and CTAS were 90% and 92% respectively, and the removal rates were also considerably high.

Table 5 Measurement of water quality

Type	Classification	Flow (m ³ /hr)	Temperature (°C)	pH	BOD (mg/l)	COD (mg/l)	Dis- solved COD (mg/l)	SS (mg/l)	K-N (mg/l)	Dis- solved K-N (mg/l)	T-P (mg/l)	Dis- solved T-P (mg/l)	MBAS (mg/l)	CTAS (mg/l)	Fatty acid salt (mg/l)
Influent	Average	64	-	-	201	91.4	45.8	219	44	31	7.2	4.5	3.6	4.7	6.0
	Maximum	197	16.9	8.96	387	262	101	468	124	107	13.7	10.0	10.9	12.4	44.8
	Minimum	27	13.1	7.22	46.6	37.3	16.6	49	19	10	2.9	2.1	1.2	1.6	0.7
Influent to aeration tank	Average	-	-	-	116	67.6	33.8	86	30	25	4.9	4.1	2.5	4.3	1.4
	Maximum	-	14.6	7.58	133	77.9	39.4	104	35	27	5.2	4.7	2.7	4.5	1.6
	Minimum	-	13.8	7.12	94.5	55.3	28.0	77	26	22	4.4	3.6	2.4	4.2	1.0
	Removal by primary sedimen- tation	-	-	-	42.3	26.0	26.2	61.5	32.2	17.8	32.3	11.5	29.6	8.5	77.3
Secondary effluent	Average	-	-	-	5.2	11.4	10.1	2.1	25	-	3.0	-	0.4	0.4	0.5
	Maximum	-	13.9	7.44	6.2	12.0	10.5	3.8	26	-	3.0	-	0.4	0.5	0.1
	Minimum	-	12.9	7.07	3.8	10.8	9.6	0.4	24	-	3.0	-	0.4	0.3	0.7
	Removal by aeration	-	-	-	95.5	83.0	70.1	97.6	16.8	-	38.9	-	85.7	90.7	66.7
Total removal		-	-	-	97.4	86.9	77.9	97.0	42.1	-	58.7	-	89.9	91.5	92.4

Table 6 shows "per capita loading daily" calculated from total influent loadings (g/day) and inhabitants. In the table, the number indicated in parentheses is the reference value for the basic unit based on the "Sewerage Facilities Design Manual and its Commentary" published by the Japan Sewage Works Agency.

The organic pollutants as BOD and COD in the table are not as high as the values in the "Design Manual". Even in the area where powdered soap as used at a high rate, a marked increase in "per capita loading daily" is not found. On the contrary, even if synthetic detergents were not used so much, there was no tendency for the TP value to decrease.

Table 6 Per capita loading daily

(Unit: g/capita/day)

BOD ₅	COD _{Mn}	SS	T-P	K-N	MBAS	CTAS	Fatty acid salts
54.3 (44~84)	24.5 (22~42)	59.2 (40~70)	1.95 (1.4~2.2)	11.9 (12~13)	0.97	1.27	1.6

5. INFLUENCE OF DETERGENTS ON ACTIVATED SLUDGE PROCESS

5.1 EFFECT OF DO IN AERATION TANK

DO level in the aeration tank is the important factor affecting the activated sludge treatment. The purpose of this experiment was to investigate the influence of surfactants upon the activated sludge treatment when the DO concentration in the aeration tank was lowered.

5.1.1 Method of investigation

In this experiment, activated sludge was placed in a container and stirred to prevent the sludge from precipitating. The control of DO was conducted by the adjustment of aeration. The cultivation was conducted by the till and draw method once a day. The DO level was controlled through conversion of the air rate through two-step adjustment.

The air rate for high DO level was set at about 600 ml/min so that, after the addition of substrate, the DO saturation could be reached in 1 ~ 3 hours. The air rate for low DO level was set at about 300 ml/min so that the DO saturation rate could reach 10 ~ 30% in 24 hours. In case of low DO, its control was difficult in this experiment. Dodecylbenzenesulfonic acid sodium salt (DBS) was added, at concentrations of 5 mg/l, 10 mg/l and 20 mg/l, to the synthetic wastewater having BOD of about 150 mg/l. If the concentrations of DBS are converted to MBAS concentrations, they are 7.2 mg/l, 14.5 mg/l and 29.0 mg/l respectively.

5.1.2 Results and discussion

The water quality of effluents is shown in Table 7, and DBS decay in the aeration tank is shown in Fig. 6. Table 7 shows the values obtained after 24 hours of reaction time had elapsed. Under the conditions of low DO, when the concentration of DBS was higher, DBS in the effluent tended to increase. In case 29 mg/l as MBAS was added, DBS remained by 27% in the effluent.

When the low DO conditions were compared with the high DO conditions, the addition of 7.2 mg/l as MBAS did not bring about the influence of DO, but on adding 14.5 mg/l as MBAS, a change in effluent quality concerning TOC and BOD was found. There was almost no change in COD and MBAS.

In the change with time during 24 hours, as shown in Fig. 6, the removal of TOC under the addition of 14.5 mg/l as MBAS was progressing favorably in low DO condition for 5 hours, but the high DO condition was preferable for 24 hours. In case 29 mg/l as MBAS was added, the similar change of TOC with time appeared for 5 hours, but the values obtained after 24 hours were better in high DO condition.

Table 7 Water quality of effluents

Analysis Sample		BOD (mg/l)		COD (mg/l)		TOC (mg/l)		MBAS (mg/l)		Dissolved COD (mg/l)	Dissolved TOC (mg/l)
		Measured	Average	Measured	Average	Measured	Average	Measured	Average		
Low DO concent- ration	DBS	5.1	4.6	6.6	6.8	8.6	7.6	0.34	0.32	6.1	1.7
	7.2	4.0		5.5		9.6		0.29			
	(mg/l)	4.6		8.4		4.7					
	DBS	9.5	11.7	9.7	11.1	11.3	17.7	0.43	0.39	6.7	10.5
	14.5	14.4		8.1		22.1		0.34			
	(mg/l)	11.1		15.5		19.3					
	DBS	6.4	16.7	11.9	19.3	18.0	32.6	6.23	7.75	16.6	23.9
	29.0	31.0		18.5		43.3		9.27			
	(mg/l)	12.3		27.4		36.1					
	DBS	5.7	6.1	13.1	9.8	4.5	8.9	0.30	0.34	5.5	6.7
	7.2	5.6		8.1		3.1		0.41			
	(mg/l)	7.1		8.2		11.4					
High DO concent- ration	DBS	8.4	5.7	13.5	11.6	4.6	8.3	0.79	0.64	6.7	4.1
	14.5	1.9		10.0		9.6		0.49			
	(mg/l)	4.7		11.4		10.3					
	DBS	4.9	4.6	11.3	9.9	9.7	11.2	1.55	1.39	7.3	3.2
	29.0	4.0		7.5		14.3		1.23			
	(mg/l)	5.0		11.0		9.5					

5.1.3 Summary

When DBS concentration was 14.5 mg/l or less as MBAS, there was no influence on the water quality of the effluent. But when 29 mg/l of MBAS was added, MBAS was liable to remain in effluent under the conditions of low DO. The operational condition was stable when the DO concentration was high. In MBAS concentration flow into the plant, at present, no influence of DO level was found.

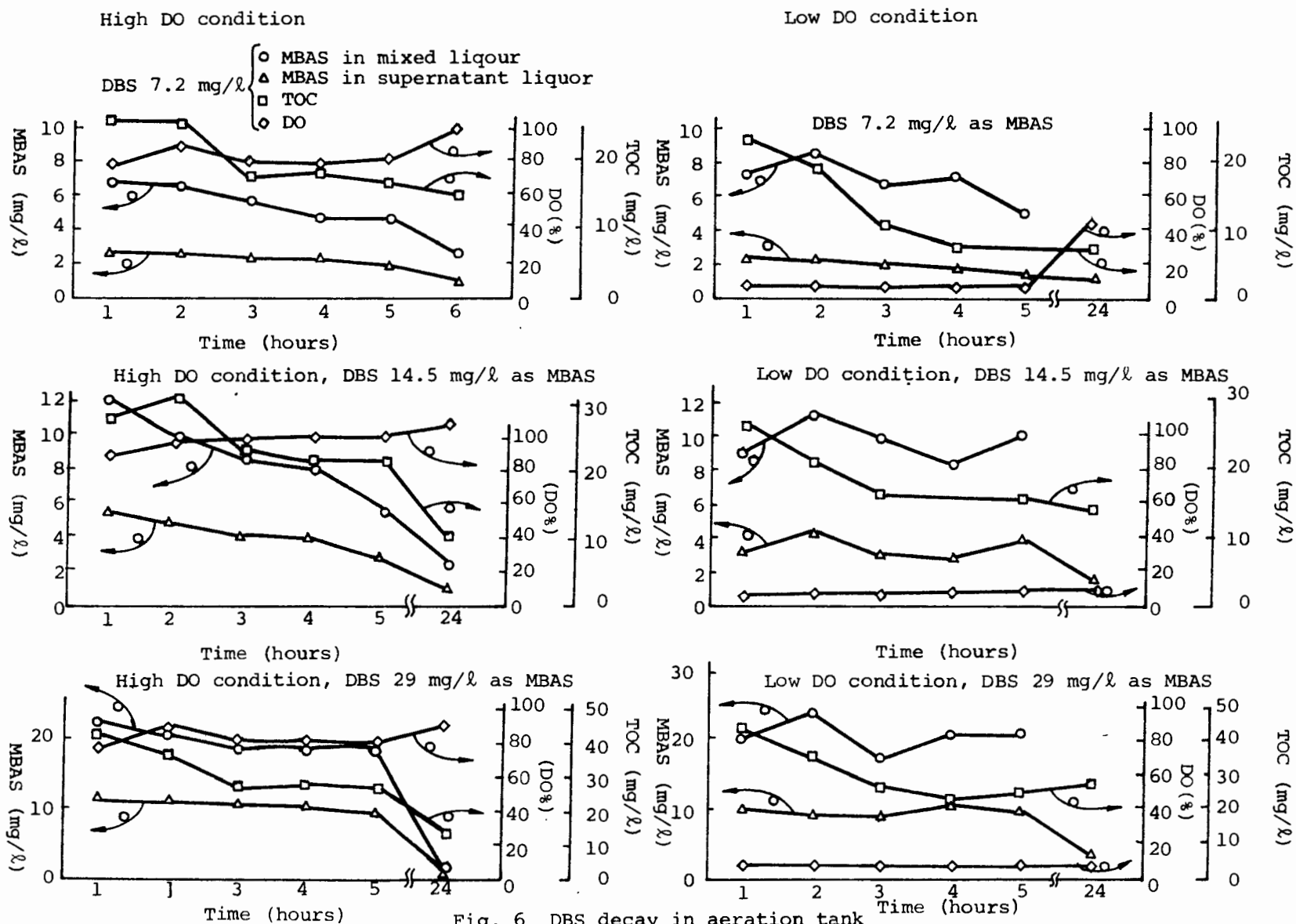


Fig. 6 DBS decay in aeration tank

5.2 EXPERIMENT FOR TREATMENT OF DETERGENTS ADDED TO SYNTHETIC SEWAGE

In order to clarify the influence of detergents on activated sludge treatment, this experiment was conducted under the conditions of the conventional activated sludge method. The influence of detergents was investigated by examining the water quality of the effluent, and the properties of sludge, biota and some other factors.

5.2.1 Method

The detergents used were powdered soap, nonionic synthetic detergents, anionic synthetic detergents, and Zeolite containing detergents. These detergents (Table 8) are all commercially available.

The main components of synthetic sewage are meat extract, yeast extract, peptone and some minerals. Seed sludge is the sludge in a sewage treatment plant which is operated under a comparatively low organic loadings.

The experimental apparatus is composed of a 100-ℓ aeration tank and a 50-ℓ sedimentation tank. The aeration tank is divided into ten sections with partitions.

The experiment was conducted in a constant-temperature room at 20°C with an aeration time of about 7 hrs for about 70 consecutive days.

The amount of detergents added is shown in Table 8. This concentration is somewhat higher than that of detergents detected in the influent of the sewage treatment plant..

However, the amount of Zeolite-containing detergents was reduced, because its foaming reaction was remarkable in the aeration tank.

Table 8 Detergents added to synthetic sewage

Unit	Detergents	Dose (mg/l)
A	Control	
B	Powderd washing soap	80 (48)*
C	Nanionic synthetic detergent	60(14.4)
D	Anionic synthetic detergent	60(14.4)
E	Zeolite containing detergent	40 (8)

Note: * The number of () means the concentration of surfactants

5.2.2 Results and discussion

The results of the experiment are shown in the table in terms of the average values during the period that was considered to be a steady state. These values were obtained about 1.5 months after the experiment had been started.

The water quality of the influent and the properties of sludge are shown in Table 9, and the water quality of the effluent and removal are shown in Table 10. When detergents were added to the synthetic sewage, powdered soap and Zeolite-containing detergents produced turbidity, and the formation of SS was observed.

Although the DO concentration in the aeration tank was comparatively high during the experimental period, it can be thought that the operation was completed under the conditions of the conventional activated sludge method.

In the case of powdered soap, SVI was high, and the most sludge was produced. The conversion rate of BOD to sludge was about 0.6 for powdered soap, and 0.3 ~ 0.4 for other detergents.

As shown in Table 10, the water quality of effluent processed by each apparatus showed a favorable value; the removal of organic matter was high; and no adverse influence due to the detergents was found. Further, nitrification took place.

The removal of Nitrogen and Phosphorus was rather low, compared with that of the sewage treatment plant (Nitrogen removal: 32%, Phosphorus removal: 49.9%). However, their values were reasonable, judging from the quantity of removal.

when no detergents were added, various types of surfactants were detected in the effluent and activated sludge. Especially, in the activated sludge, CTAS and fatty acid salts were detected at a high concentration.

Fig. 7 shows the change with time in TOC and surfactants present in the aeration tank.

In the aeration tank, both are similar to the removal pattern of substrates that are liable to be comparatively utilized as activated sludge.

For the biota in the activated sludge of this experiment, microscopic examination could be conducted for 12 genera - mainly *Epistylis*, *Vorticella*, *Litonotus*, *Acanthamoeba* and *Rotifera*.

The fauna in the steady state of each experimental apparatus is as shown in Table 12. *Rhizopoda* comparatively often appeared in any condition. *Aspidisca* appeared when no detergents were added or powdered soap and nonionic synthetic detergents were added. But they were not seen when anionic synthetic detergents and Zeolite detergents were added. This tendency was seen for *Opecularia*. *Epistylis*, *Vorticella* appear in any condition. Although the reason might be SRT of 10 days or more, *Rotifera* and *Nematoda* appeared comparatively often.

The population was 1,700 ~ 3,600 N/ml when no detergents were added, and was almost the same degree except for the case of powdered soap. On adding powdered soap, the population was 3,700 ~ 4,300 N/ml and very large with other cases.

As for filamentous bacteria, its population was smallest of these five conditions when anionic synthetic detergents were added. In the other cases, they appeared to such a degree that they occupied 30 ~ 60% in floc of activated sludge.

Table 9 Operation conditions of experimental units

Unit		A	B	C	D	E
Detergents		Control	Powdered washing soap	Nonionic synthetic detergent	Anionic synthetic detergent	Zeolite containing detergent
Influent (mg/l)	BOD	108.6	156.9	141.2	136.2	124.1
	COD	53.3	65.7	68.6	62.0	60.4
	TOC	58.5	75.9	70.3	63.6	59.5
	K-N	19.24	19.22	19.48	20.1	19.69
	T-P	5.20	5.43	5.08	6.56	4.99
Aeration tank	pH*	6.7~7.4	6.7~7.4	6.3~7.2	6.6~7.4	6.7~7.4
	DO* (mg/l)	7.7	7.1	7.6	7.4	7.4
	SVI**	73	134	73	106	110
	MLSS (mg/l)	1,645	2,120	1,960	1,841	1,885
	$\frac{MLVSS}{MLSS}$	0.87	0.88	0.86	0.85	0.85
	ΔS^{***} (g/day)	14.4	28.4	19.2	13.5	15.7
	SRT (day)	11.4	7.46	10.2	13.6	12.0
	BOD-SS loadings (g/g.day)	0.22	0.25	0.24	0.25	0.22
	COD-SS loadings (g/g.day)	0.11	0.10	0.12	0.11	0.11

Note: * Measured in the middle of aeration tank

** Measured at the end of aeration tank

*** Generated activated sludge

Table 10 Water quality of effluents

Unit			A	B	C	D	E
Detergents			Control	Powdered washing soap	Nonionic synthetic detergent	Anionic synthetic detergent	Zeolite containing detergent
Water quality of effluents	Turb. (cm)		45	74	40	35	50
	SS (mg/l)		7.9	5.8	7.7	10.3	6.8
	BOD (mg/l)	T*1	2.3	3.5	3.2	3.7	4.0
		S*2	2.3	1.3	0.3	1.4	1.0
	COD (mg/l)	T	9.6	8.9	10.3	14.0	9.6
		S	7.9	7.3	7.2	10.0	6.9
	TOC (mg/l)	T	5.0	5.8	5.0	5.9	4.4
		S	4.5	5.2	4.7	5.5	4.0
	K-N (mg/l)	T	1.24	1.15	0.69	1.10	0.60
	T-P (mg/l)	T	4.37	3.92	4.30	5.40	4.21
	NH ₄ ⁺ -N (mg/l)		0.68	0.31	0.41	0.76	0.51
Removal (%)	NO _T -N*3 (mg/l)		13.88	12.45	14.66	13.51	15.55
	O-P (mg/l)		4.18	3.51	4.03	4.96	3.90
	BOD	T	97.8	97.8	97.7	97.2	96.3
	COD	T	82.1	86.3	84.9	77.4	34.1
	TOC	T	91.4	92.4	92.9	90.7	92.6
	T-N	T	21.4	29.2	21.2	27.3	13.0
	T-P	T	16.0	28.5	15.4	17.7	15.6

NOTE: *1 Total

*2 Soluble (GFB)

*3 NO_T-N: (NO₂+NO₃)-N

Table 11 Surfactant concentration in water samples and mixed liquor

Unit		A	B	C	D	E
Detersents		Control	Powdered washing soap	Nonionic synthetic detergent	Anionic synthetic detergent	Zeolite containing detergent
Influent (mg/l)	MBAS	0.03			10.3	6.0
	CTAS	0.03		13.5		
	Fatty acid salts	0.10	60.6			
Effluent (mg/l)	MBAS	0.27			0.35	0.79
	CTAS	0.17		0.47		
	Fatty acid salts	0.54	3.25			
Mixed liquor (mg/l)	MBAS	1.96			6.67	6.42
	CTAS	23.9		29.2		
	Fatty acid salts	24.1	59.5			

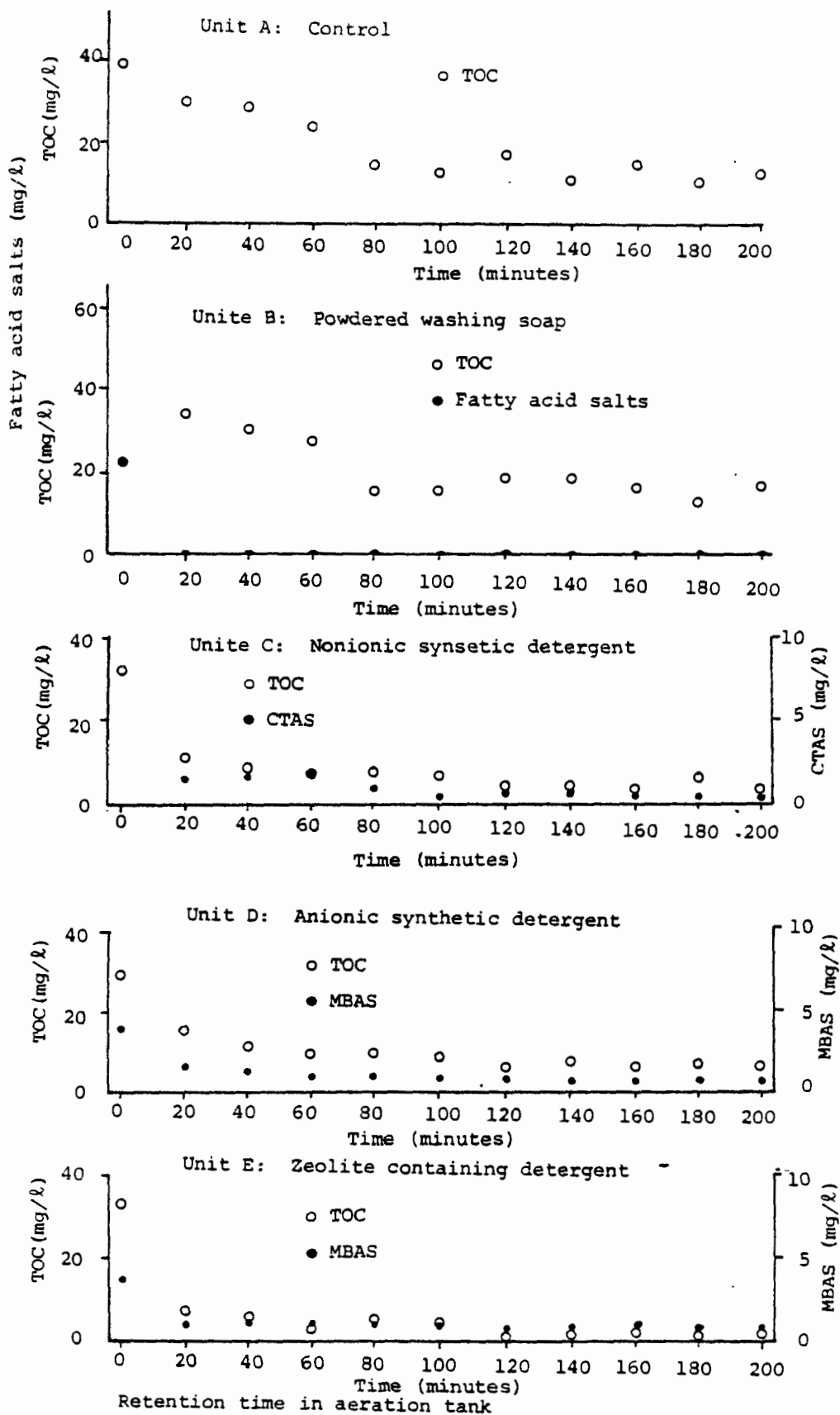


Fig. 7 The decay of dissolved components in aeration tank

These results revealed that, in the biota of activated sludge, there was almost no difference between the cases when detergents were added and not added, and the population was almost the same except for powdered soap. Filamentous bacteria had a poor appearance rate when anionic synthetic detergents were used, but in other cases they appeared to a considerable extent. In case of powdered soap, the population was larger than that of other conditions.

Table 12 Microorganisms in activated sludge

Fauna \ Unit	A	B	C	D	E
<i>Amoeba</i>	50~ 300	100~2,225	50~ 750	125~1,125	125~1,425
<i>Arcella</i>	25~ 425	25~ 325	75~ 225	425~ 650	300~2,575
<i>Euglypha</i>	25~ 175	25~ 75	25~ 350	25~ 100	25~ 275
<i>Chilodonella</i>		50	25		
<i>Litonotus</i>	200	25~ 400	25~ 175	125~ 650	50~ 325
<i>Aspidisca</i>	25~ 375	50~ 400	225		
<i>Euplotes</i>	25~ 50			25	25
<i>Epistylis</i>	50~2,300	25~ 975	25~1,150	25~ 325	25~ 600
<i>Vorticella</i>	75~ 725	150~1,925	275~3,850	175~ 800	70~ 650
<i>Opercularia</i>	25~ 50	325~ 625	100~ 175		50
<i>Carchesium</i>					25
<i>Zoothamnium</i>	150	775			
<i>Vaginicola</i>		25			25
<i>Acinata</i>	50	25~ 50			25~ 50
<i>Tokophrya</i>			25		25
<i>Lecane</i>	145~ 500	50~ 400	125~ 450	325~1,150	100~ 650
<i>Rotaria</i>	25~ 50			25	
<i>Lepadella</i>	25~ 100		25~ 100	300	
<i>Trichocerca</i>				75	25
<i>Diplogaster</i>	125~ 425	25~ 150	25~ 200	25~ 250	25~ 625
<i>Nais</i>	75				25
Unknown	25~ 250	25~ 125	50~ 250	75	50~ 75
Population (N/ml)	1,700~3,600	3,700~4,300	1,800~3,200	1,900~3,900	1,400~3,200

5.2.3 Summary

Under the operational conditions of the conventional activated sludge process, the experiment was conducted by adding 48 mg/l of powdered soap, 14 mg/l of nonionic synthetic detergent, 14 mg/l of anionicanionic synthetic detergent and 8 mg/l of Zeolite-containing detergent to synthetic sewage, and the following results were obtained:

- 1) From the viewpoint of the operation of the experimental apparatus, it was noted that powdered soap and Zeolite-containing detergent markedly solidified in the sewage. Accordingly, it was suggested that in actual facilities those detergents were liable to precipitate into the sewer system or run into the plant as SS, so that their removal in a primary sedimentation tank would be high.
- 2) Zeolite-containing detergent tested brought about a remarkable forming reaction in the aeration tank. We had some difficulties in the maintenance and control of the apparatus. Such problems were caused by the kinds of surfactants containing in detergents.
- 3) Judging from the properties of activated sludge and the water quality of effluents, no adverse influence of detergents upon the activated sludge treatment was found. Further, there was no tendency for surfactants to accumulate in the activated sludge. The concentration of surfactants in effluents was low, so that the surfactants were apparently biodegraded by the activated sludge.
- 4) The fauna in the activated sludge was similar on these five conditions. In the case of powdered soap, the population became larger than in other cases. Filamentous bacteria had poor appearance when anionic synthetic detergents were used.

5.3 EXPERIMENT FOR TREATMENT OF DETERGENTS ADDED TO MUNICIPAL SEWAGE

This experiment was conducted to investigate the influence of detergents upon the activated sludge treatment. The detergents added to municipal sewage were phosphate-containing anionic synthetic detergents, phosphate-free anionic synthetic detergents (containing Zeolite), phosphate-free nonionic synthetic detergents and powdered soap.

5.3.1 Method

(1) Apparatus and operation

In this experiment, as shown in Fig. 8, 5 units for activated sludge treatment were used. Samples were a municipal sewage (control) and municipal sewages in which detergents were added. As municipal sewage, the primary effluent was used in the treatment plant of the river basin sewerage system, and the return sludge in the same treatment plant was used as seed sludge.

The sewage storage tank was supplied with sewage every two days (about 11 p.m.) and at the same time detergents were added. The supply of sewage from the sewage storage tank to the aeration and sedimentation tank was conducted four times a day, and the amount of sewage supplied was 10 l per one time. The supernatant liquor in the aeration and sedimentation tank was automatically drained into the effluent storage tank after the completion of sedimentation. Aeration and sedimentation were conducted four times a day, one aeration period being four hours and one sedimentation period two hours.

The BOD-SS loadings in the aeration and sedimentation tank was set at 0.2 ~ 0.4 kg/MLSS.kg.day.

The analysis of water quality was conducted for the sewage in each sewage storage tank, the effluent in each effluent storage tank and MLSS in each aeration and sedimentation tank.

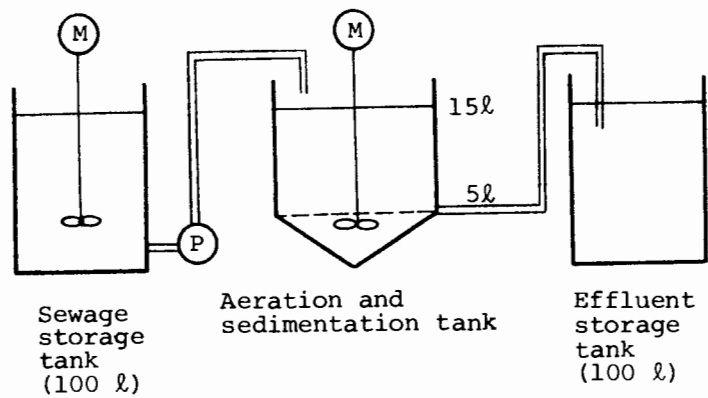


Fig. 8 Activated sludge unit

(2) Detergents used in this experiment

The detergents used in this experiment are as follows:

- Detergent A : Phosphate-containing anionic synthetic detergent, MBAS content is 38% per solid matter
- Detergent B : Phosphate-free anionic synthetic detergent containing synthetic Zeolite, MBAS content is 32% per solid matter
- Detergent C : Phosphate-free nonionic synthetic detergent, CTAS contents is 24%, MBAS is 2.2%
- Powdered soap D: Phosphate-free fatty acid type surfactant, fatty acid salts is 70%

(3) Dose of detergents

It is said that the average concentration of MBAS (per day) flowing into a Municipal treatment plant is 5 ~ 10 mg/l, and the average concentration of MBAS in influent in a treatment plant for a housing area is 10 ~ 20 mg/l.

Therefore, the MBAS concentration in sewage for the experiment was set at 10 mg/l and 20 mg/l, so as to correspond to the actual values. The value in a high concentration of MBAS was set at 50 mg/l.

Municipal sewage originally contains 4 ~ 6 mg/l of MBAS, so appropriate amounts of detergents containing 5 mg/l, 15 mg/l and 45 mg/l of MBAS respectively were added to make their respective concentrations to 10 mg/l, 20 mg/l and 50 mg/l. The amount of nonionic surfactants added was set at almost the same level as that of MBAS. In the case of powdered soap, amounts of soap corresponding to 15 mg/l, 45 mg/l, 105 mg/l of fatty acid salts respectively were added.

(4) Samples

Samples prepared are as follows

Sewage No.1: Primary sedimentation effluent (control)

Sewage No.2: Sewage No.1 + Detergent A

Sewage No.3: Sewage No.1 + Detergent B

Sewage No.4: Sewage No.1 + Detergent C

Sewage No.5: Sewage No.1 + Powdered soap D

Effluent Nos.1 ~ 5 correspond to Sewage Nos.1 ~ 5 respectively

(5) Experimental period

The experimental period was determined to be about 5 months. According to the concentration of surfactant, this period was divided as follows: Table 13 shows the results of analysis for test detergents.

i) Run I (October 2 ~ November 25)

As the surfactant in detergents, 0.5 mg/l of synthetic detergent and 0.8 mg/l of powdered soap were added, and the amounts added were gradually increased to 5 mg/l and 15 mg/l respectively.

During the period from 5 November to 25 November activated sludge acclimated the detergents to 5 mg/l and 15 mg/l.

ii) Run II (November 26 ~ January 10)

As the surfactant in the detergents, 7 mg/l of synthetic detergents and 2 mg/l of powdered soap were added. Then, the amount added was gradually increased to 15 mg/l and 45 mg/l respectively. It was for the period from November 26 to January 10 that activated sludge acclimated the detergents to 15 mg/l and 45 mg/l respectively.

iii) Run III (January 12 ~ March 19)

As the surfactants in the detergents, 20 mg/l of synthetic detergents and 55 mg/l of powdered soap were added. Then, the amount added was gradually increased. As the surfactant in the synthetic detergents, 45 mg/l of synthetic detergent was added. Then, due to foaming, it became impossible to increase the concentration any more, so the amount added was limited to 45 mg/l. In the case of powdered soap, the limit was set at 105 mg/l. During the period from February 23 to March 19 activated sludge acclimated the detergents to 45 mg/l and 105 mg/l.

Table 13 Analysis of detergents

Item Detergents	Solid (w/w %)	BOD ₅ * (mg/l)	COD _{Mn} * (mg/l)	TP (%)	TOD (%)	MBAS (%)
A	88.6	147	80	7.36	69.8	38
B	94.6	176	79	0.02	58.4	32
C	-	186	78	0	67.9	22
D	93.9	1300	240	0	115	0

Note: * Measured 0.1% water solution of detergents

5.3.2 Results and discussion

The results for the period when activated sludge acclimated the detergents under the conditions of Run I, II and III have been collated.

(1) BOD

With the increase in the amount of detergents added, BOD of sewage increased and average BOD of sewage No.5 to which powdered soap had been added, was higher than that of other sewage. Especially, average BOD of sewage No.5 in Run III, III, as the powdered soap corresponding to 105 mg/l as surfactant was added, was 313 mg/l and a little over 2.7 times the 114 mg/l of average BOD of sewage No.1 to which no detergents had been added.

Table 14 shows BOD of sewage and effluents. The average of activated sludge effluent in Run III was 5.4 mg/l for No.2, No.2, 6.0 mg/l for No.3, 7.1 mg/l for No.4 and 9.1 mg/l for No.5. If they are compared with 5.5 mg/l of average BOD of Effluent No.1, there is no marked difference.

Accordingly, by adding detergents to sewage, the BOD in sewage was increased, but there was no tendency for effluent BOD to rise.

Table 14 BOD measurement of influents and effluents

(Unit: mg/l)

Sample	Run I			Run II			Run III		
	Influ-ent	Efflu-ent	Re-moval	Influ-ent	Efflu-ent	Re-moval	Influ-ent	Efflu-ent	Re-moval
No.1	81	3.3	96	96	4.2	96	114	5.6	95
No.2	91	3.5	96	101	4.9	95	154	5.4	96
No.3	81	4.4	95	103	7.2	93	153	6.0	96
No.4	82	4.0	95	101	5.8	94	162	7.1	96
No.5	104	3.8	96	169	5.4	97	313	9.1	97

(2) COD

When the amount of detergents added is small, there is no marked difference between the COD of sewage No.1 and the COD of Nos. 2 ~ 4. However, when the amount became large as Run III, the COD of sewage Nos. 2 ~ 5 was a little higher than that of sewage No.1. (See Table 15.)

Table 15 COD measurement of influents and effluents

(Unit: mg/l)

Sample	Run I			Run II			Run III		
	Influ-ent	Efflu-ent	Re-moval	Influ-ent	Efflu-ent	Re-moval	Influ-ent	Efflu-ent	Re-moval
No.1	41	9.5	77	46	11.2	76	66	11.4	83
No.2	42	11.7	72	48	12.4	74	76	18.8	75
No.3	41	10.4	75	48	12.1	75	76	17.9	76
No.4	41	11.8	71	54	12.3	77	100	18.8	81
No.5	42	10.6	75	55	12.0	78	93	16.1	83

The average COD in effluent Nos. 2 ~ 5 is not remarkably different from that of Effluent No.1 in Run I and II.

The average COD of effluent in Run III are 18.8 mg/l for No.2, 17.9 mg/l for No.3, 18.8 for No.4 and 16.1 mg/l for No.5, and they were a little higher than 11.4 mg/l of average COD of effluent No.1.

Accordingly, when the amount of detergent added was increased, the COD of sewage increased and the COD of effluent became somewhat higher than that of effluent to which detergents had not been added.

(3) TOC

As the amount of detergents increased, the TOC in sewage Nos. 2 ~ 5 increased. (Run III)

The average values of TOC in effluent Nos 2 ~ 5 were not much different from the value of effluent No.1 in Run I and II in which a small amount of detergents were added. The average values of TOC of each effluent in Run III were 15.2 mg/l for No.2, 15.7 mg/l for No.3, 15.9 mg/l for No.4 and 13.9 mg/l for No.5. They were a little higher than 7.8 mg/l of the average TOC of effluent No.1.

Accordingly, when the amount of detergents added was increased, TOC in sewage increased, and TOC of effluents became a little higher, compared with that of effluent to which detergents had not been added. (Run III)

Table 16 TOC measurement of influents and effluents

(Unit: mg/l)

Sample	Run I			Run II			Run III		
	Influ-ent	Efflu-ent	Re-moval	Influ-ent	Efflu-ent	Re-moval	Influ-ent	Efflu-ent	Re-moval
No.1	42.5	5.7	87	53.4	11.7	78	66.1	7.8	88
No.2	44.2	6.8	85	62.4	13.9	78	84.7	15.2	82
No.3	44.5	7.2	84	66.2	14.4	78	86.6	15.7	82
No.4	45.0	7.6	83	70.1	15.1	78	104.0	15.9	85
No.5	49.2	7.3	85	77.9	14.5	81	126.2	13.9	89

(4) Suspended solids

The sewages in which the SS concentration increased due to the addition of detergents were sewage No.3 (Run III) and No.5. The cause for No.3 is considered to be due to the synthetic Zeolite contained in detergents. The cause for No.5 is considered to be due to the fatty acid salts contained in powdered soap D and to the so-called soap scum affected by the sewage. (See Table 17.)

The average concentration of effluents Nos. 2 ~ 5 was not much different from that of effluent No.1. The average concentrations of SS of effluents in Run III were 19 mg/l for No.1, 3.9 mg/l for No.2, 6.4 mg/l for No.3, 5.0 mg/l for No.4 and 5.4 mg/l for No.5.

Therefore, when synthetic Zeolite-containing detergents and powdered soap were added to the sewage, the SS concentration increased, but the addition of other detegents did not bring about the increase of suspended solids. The SS concentration of effluents, composed of the above sewage processed through activated sludge treatment, did not increase.

Table 17 SS measurement of influents nad effluents

(Unit: mg/ℓ)

Sample	Run I			Run II			Run III		
	Influ-ent	Efflu-ent	Re-moval	Influ-ent	Efflu-ent	Re-moval	Influ-ent	Efflu-ent	Re-moval
No.1	53	2.6	95	53	3.7	93	82	1.9	98
No.2	48	6.2	87	47	3.7	92	80	3.8	95
No.3	49	4.1	92	48	3.9	93	117	6.4	95
No.4	47	5.3	89	52	4.9	91	80	5.0	94
No.5	56	3.7	93	73	3.3	95	157	5.4	97

(5) MBAS

The sewages in which MBAS was increased by the addition of detergents were sewage No.2 ~ No.4 (Run. III). Especially, MBAS of sewage Nos. 2 and 3 was 50.1 mg , which was about ten times that of sewage No.1 (5.1 mg/ℓ). The reason why MBAS was so high was that the detergents containing anionic surfactants were added. (See Table 18.)

The average value of MBAS in effluent Nos. 2 ~ 5 was not much different from that of effluent No.1. The average MBAS values of effluents Run III were 0.09 mg/ℓ for No.1 0.54 mg/ℓ for No.2, 0.42 mg/ℓ for No.3, 0.05 mg/ℓ for No.4 and 0.10 mg/ℓ for No.5.

Table 18 MBAS measurement of influents and effluents

(Unit: mg/l)

Sample	Run I			Run II			Run III		
	Influ- ent	Efflu- ent	Re- moval	Influ- ent	Efflu- ent	Re- moval	Influ- ent	Efflu- ent	Re- moval
No.1	5.5	0.08	99	6.2	0.10	98	5.1	0.09	98
No.2	10.5	0.12	99	21.2	0.21	99	50.1	0.54	99
No.3	10.5	0.13	99	21.2	0.28	99	50.1	0.42	99
No.4	6.0	0.07	99	7.5	0.29	96	9.3	0.05	99
No.5	5.5	0.09	98	6.2	0.14	98	5.1	0.10	98

(6) Total phosphorus

The sewage of which the total phosphorus was increased by the addition of detergents was sewage No.2. Especially in Run III, its concentration was 12.8 mg , which was about 3.2 times that of sewage No.1. The phosphorus concentration of other sewages to which phosphate-free detergents were added, was almost the same with that of sewage No.1. (See Table 19.)

The average concentration of the total phosphorus in effluents was not much different from that of effluent No.1, except for No.2. The average concentrations of the total phosphorus of effluents in Run III were 1.40 mg/l for No.1, 9.02 mg/l for No.2, 0.83 mg/l for No.3, 0.64 mg/l for No.4, 0.48 mg/l for No.5.

Table 19 TP measurement of influents and effluents

(Unit: mg·P/ℓ)

Sample	Run I			Run II			Run III		
	Influ-ent	Efflu-ent	Re-moval	Influ-ent	Efflu-ent	Re-moval	Influ-ent	Efflu-ent	Re-moval
No.1	3.37	1.54	54	3.17	1.37	57	4.03	1.40	65
No.2	4.24	2.36	44	5.81	3.63	38	12.8	9.02	30
No.3	3.23	1.48	54	3.15	1.34	57	3.98	0.83	79
No.4	3.26	1.63	50	3.13	0.70	78	3.97	0.64	84
No.5	3.25	1.40	57	3.15	0.45	86	3.80	0.48	87

(7) Total nitrogen

The addition of detergents did not bring about an increase in the total nitrogen in the sewages. Their concentration was almost the same as that of sewage No.1. (See Table 20.)

The average concentration of the total nitrogen in effluents, except for effluent No.5 in Run III, was not much different from effluent No.1.No.1.

In order to keep activated sludge in a favorable condition, it is said that a nutrient ratio such as BOD : N : P = 100 : 5 : 1 is required. When the removal rate of BOD is increased, the removal rates of nitrogen and phosphorus increase according to this proportion. The BOD removal of No.5 in Run III was 304 mg/ℓ, which was higher than that of other sewages (No.1: 108 mg/ℓ , No.4: 155 mg/ℓ, etc.) For this reason, the nitrogen in Run III is considered to have decreased.

Accordingly, the addition of detergents to the sewage did not bring about a change in the total nitrogen in the sewage. The nitrogen contents of effluents were almost the same when the BOD removal was not high.

Table 20 TN measurement of influents and effluents

(Unit: mg/l)

Sample	Run I			Run II			Run III		
	Influ- ence	Efflu- ence	Re- moval	Influ- ence	Efflu- ence	Re- moval	Influ- ence	Efflu- ence	Re- moval
No.1	21.4	17.6	18	23.3	16.4	30	30.6	22.2	27
No.2	21.3	18.0	15	23.7	16.9	29	31.0	22.9	26
No.3	23.7	18.5	22	22.5	16.9	25	31.9	23.5	26
No.4	22.3	18.2	18	22.6	15.6	31	31.6	20.8	34
No.5	22.4	16.2	28	21.6	13.7	37	30.4	13.7	35

(8) Properties of activated sludge

i) Population of Protozoa

The number of microorganisms in activated sludge repeatedly increased and decreased during the experiment, but the addition of detergents did not cause the microorganisms to become extinct or unbalanced in distribution. Among the microorganisms, the dominant species are *Peritricha* such as *Vorticella* *Epistylis*. The population in Run III is shown in Tables 21 and 22.

Accordingly, when detergents are used within the normal range it is considered that they do not have an adverse influence on the microorganisms conducting biological treatment.

Table 21 Population of microorganisms in activated sludge

(Unit: N/ml)

Date	March 10				
Samples	No.1	No.2	No.3	No.4	No.5
<i>Peritricha</i>	6440	2000	2160	2380	19700
<i>Aspidisca</i>	2510	40	20	200	360
<i>Litonotus</i>	420	530	690	420	980
<i>Suctoria</i>	40	-	-	-	-
<i>Rotifera</i>	130	20	-	40	160
<i>Nematoda</i>	70	20	-	20	130
<i>Ciliata</i> (swimming type)	20	-	-	200	-

Table 22 Population of microorganisms in activated sludge

(Unit: N/ml)

Date	March 16				
Sample	No.1	No.2	No.3	No.4	No.5
<i>Peritricha</i>	5250	10100	3950	7600	26700
<i>Aspidisca</i>	5950	150	-	-	2180
<i>Litonotus</i>	500	1750	200	4160	18000
<i>Suctoria</i>	-	-	-	-	-
<i>Rotifera</i>	250	150	-	40	250
<i>Nematoda</i>	50	150	50	160	470
<i>Ciliate</i> (swimming type)	-	-	-	960	180

ii) MLVSS/MLSS

The average values of MLVSS/MLSS and SVI of activated sludge, measured during the experiment, are shown in Table 23. MLVSS/MLSS of Sample No.1, to which detergents had not been added, was 66 ~ 74%. MLVSS/MLSS of sample No.3, to which detergent B containing Zeolite was added, in Run III fell to 60%. In Sample No.5, to which powdered soap had been added, MLVSS/MLSS rose to 80% in Run II. MLVSS/MLSS of No.2 and No.4 were 70 ~ 80% and 74 ~ 77%, respectively. They were not much different from Sample 1.

Accordingly, when synthetic detergents are used within the normal range, the proportion of MLVSS/MLSS in the activated sludge cannot be considered to alter remarkably. However, if synthetic detergents for washing are converted to powdered soap, the proportion of MLVSS/MLSS will increase to a certain degree, even when the soap is used within the normal range.

iii) SVI

Due to the characteristics of sewage used in this experiment, SVI of Sample No.1, to which detergents had not been added, was lower than SVI of the activated sludge in an ordinary treatment plant. SVI of the activated sludge in this experiment was also low. (See Table 23.)

SVI of sewage No.1 was somewhat low compared with that of Sample 1 when the amount added was increased. In case of nonionic synthetic detergents and powdered soap, when the amount added was increased, there was a tendency for SVI to increase.

Table 22 MLVSS/MLSS and SVI measurement of activated sludge

Sample	Run Item			
		I	II	III
No.1	MLVSS/MLSS	66%	74%	73%
	SVI	65	51	67
No.2	MLVSS/MLSS	71%	80%	70%
	SVI	57	56	48
No.3	MLVSS/MLSS	68%	69%	60%
	SVI	69	58	47
No.4	MLVSS/MLSS	75%	74%	77%
	SVI	57	56	101
No.5	MLVSS/MLSS	68%	81%	87%
	SVI	71	101	86

5.3.3 Summary

The experiment for the activated sludge process was conducted by adding detergents to municipal sewage.

The results were as follows:

- (1) When the detergent corresponding to 15 mg/l of surfactant (45 mg of powderd soap) was added to sewage (containing 4 ~ 7 mg/l of MBAS), the concentrations of BOD, TOC, suspended solids (powdered soap added), MBAS (anionic synthetic detergent added), and the total phosphorus (phosphate-containing synthetic detergent added) of the sewage somewhat increased, compared with those of non-added sewage. However, no influence was found upon the water quality of effluents.
- (2) When the detergent corresponding to 45 mg/l of surfactant (105 mg of powdered soap) was added to sewage, the concentrations of BOD, COD, TOC, suspended solids, and MBAS of the sewage increased.

COD, TOC, and total phosphorus (phosphate-containing synthetic detergent added) of the effluent tended to somewhat increase in comparison with those of the effluent from sewage No.1.

- (3) Microorganisms in activated sludge, even though detergents were added to the sewage, did not become extinct or unbalanced in distribution. MLVSS/MLSS of the sludge to which a detergent containing Zeolite had been added somewhat fell to 60%. MLVSS/MLSS of the sludge to which powdered soap had been added rose to 80% as the amount added was increased. The MLVSS/MLSS of the activated sludge to which other detergents had been added was 70 ~ 80%. The values were not much different from that of the sludge to which detergents were not added.

SVI of activated sludge tended to be lowered when the amount of anionic synthetic detergents added was increased. SIV of the samples to which nonionic synthetic detergents or powdered soap had been added, tended to rise.

From the above results, it can be said that there is no serious influence of detergents upon the activated sludge and the water quality of effluents when they are used within the normal range, but there is a possibility that the water quality will deteriorate when a large quantity of detergent is used.

6. CONCLUSIONS

To clarify the influence of detergents upon the sewerage system, a field investigation at sewage treatment plants and laboratory experiments have been conducted. The results are as follows:

- (1) For the concentration of surfactants in influent, MBAS was 2.5 ~ 6.4 mg/l (average 4.7 mg/l), CTAS 1.9 ~ 4.7 mg/l (average 3.1 mg/l), and fatty acid salts 2.4 ~ 6.7 mg/l (average 4.1 mg/l). This MBAS concentration is almost an average value in municipal sewage.

For the concentration of surfactants in primary sludge, MBAS was 3.8 mg/g on an average, CTAS 3.0 mg/l on an average and fatty acid salts 96.9 mg/g on an average.

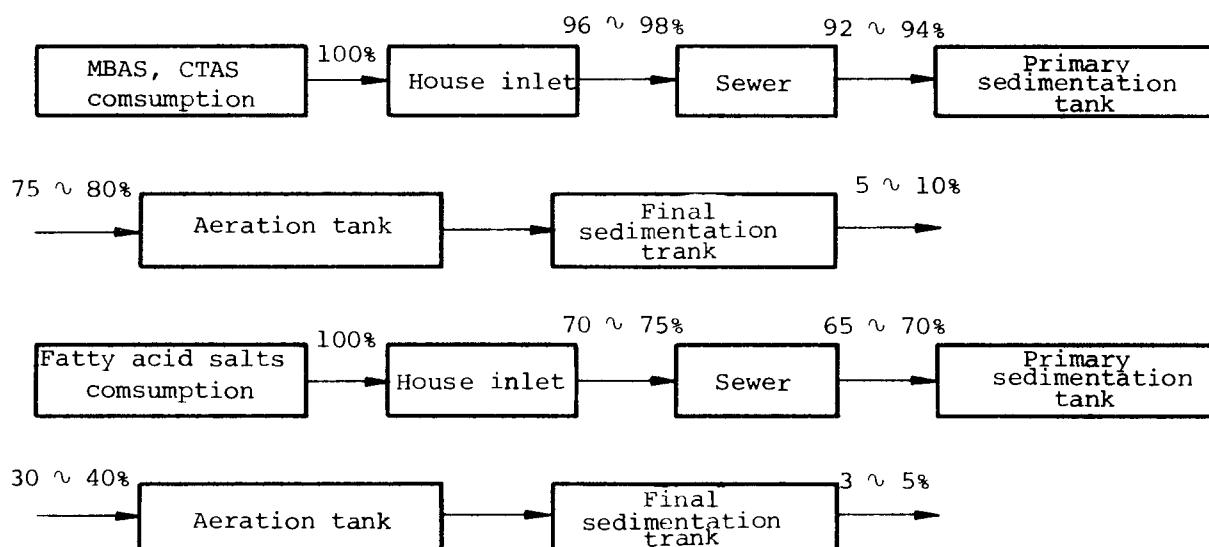
For the concentration of surfactants in waste activated sludge, MBAS was 3.4 mg/g on an average, CTAS 19.7 mg/g and fatty acid salts 47.2 mg/g. It was considered that the high concentration of CTAS and fatty acid salts in activated sludge was due to the influence of factors other than detergents.

- (2) The biodegradation of DBS as a type of LAS is affected by DO level in aeration tanks in case of high concentration of DBS. When DBS of 14.5 mg/l or less as MBAS is added, no adverse effect on the water the water quality of effluent is found in either high DO condition or low DO condition. However, when the DBS of 29 mg/l in terms of MBAS is added, MBAS remains in the effluent under low DO condition.
- (3) Using the samples prepared by adding detergents to synthetic sewage or municipal sewage, an activated sludge treatment experiment was conducted, and the following results were obtained:
 - o Even if the concentrations of MBAS, CTAS, and fatty acid salts in test sewage were set to 20 mg/l, 20 mg/l and 50 mg/l respectively, water quality was almost the same as in the control system. However, when the concentrations of MBAS, CTAS, and fatty acid salts were set to 50 mg/l, 50 mg/l, and 110 mg/l respectively, the water quality of effluents was somewhat deteriorated.

- o The fauna in activated sludge to which detergents had been added was the same as that of the control system. However, the population of microorganisms was larger in the powder soap-added system than in other systems.
- o MLVSS/MLSS was around 65 ~ 77%. As the concentration of Zeolite-containing detergent increased, MLVSS/MLSS fell to 60% or less.

Further, when the concentration of powdered soap increased, MLVSS/MLSS rose to 80%.

- (4) It was concluded that MBAS, CTAS, and fatty acid salts behave as follows:



MBAS and CTAS are removed mainly by aeration tank. On the contrary, fatty acid salts are considerably removed by house inlets and primary sedimentation tanks, so that its loadings to the activated sludge process is 30 ~ 40% of the consumption.

- (5) Even if consumers convert from synthetic detergents to powdered soap, almost no influence on sewage treatment is anticipated, as long as the amount of detergents currently used is maintained. Further, it is concluded that detergents have no adverse effect on sewage treatment, if the detergent borne surfactant concentration in sewage remains as low as it now is.

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Eighth US/Japan Conference
on
Sewage Treatment Technology

**Fifth Five-Year Sewerage System Development Program and
Long-Range Prospects for Development of Sewerage Systems**

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1. PREFACE

With the enactment of the Sewerage Act in 1900 as impetus, the construction of modern sewerage systems began in Japan. Under the Sewerage Act, the responsibility for construction and control of sewerage systems rests with municipalities. It is stipulated in the act that when a municipality is to construct a sewerage system, approval from the competent minister (central government) is necessary. Though the Sewerage Act had been enacted, sewerage systems had been developed only in a small number of cities before 1945 when World War II came to an end. At that time, only Tokyo, Osaka, Kyoto, Nagoya and two other cities had sewage treatment plants.

After 1945 and in the 1950s, there was little progress in the implementation of sewerage projects in Japan. The level at which sewerage systems were diffused was very low. The reason may be ascribed to the following factors.

- (1) Each household was equipped with a privy. Night soil was carried off to farmland as manure.
- (2) The necessity of sewerage systems was not well recognized.
- (3) For any municipality, which was the mainstay entity for sewerage projects, it was difficult to secure a financial source for the construction of sewerage systems.
- (4) National policy was intended primarily to strengthen the basis of industry with measures for infrastructure such as sewerage to be made afterward.

From the latter half of the 1950s to the 1960s when the Japanese economy began to develop, however, the environment was progressively aggravated, and the deterioration of river water quality on the outskirts of urban areas was conspicuous.

In contrast to advanced Western nations, Japan featured a strikingly fast speed at which population became concentrated in urban areas (this concentration is still in progress), rapid economic growth -- particularly, a high growth rate for industry -- and a resultant conspicuous concentration of production and consumption in specific regions. The consequence was that water pollution markedly accelerated in the 1960s.

With an area of about 370,000 km², Japan is nearly as big as the State of California. The greater part of the national land is mountainous. Less than 25% of the total area is inhabitable but 113 million people, equivalent to half of the total population of the United States, live there. For the reasons that have been introduced earlier, the fact that practically no accumulation was made in terms of facilities brought about a number of difficulties in the development of sewerage systems. As examples, it may be said that trunk, lateral, branch sewers had to be buried in completely built-up areas with high population densities in almost all the cities; that for environmental considerations on the periphery, the costly shield construction method, instead of the less costly open cut construction method had to be frequently used in the construction of trunk sewers; that it was difficult to acquire tracts of land for treatment plants; and that since the water quality deteriorated at a faster pace than the development of sewerage systems, it was necessary to have secondary treatment while the construction of sewers was in progress. The cost-effectiveness and performance of sewerage systems were consequently affected to a considerable degree by these facts.

The sophisticated development of sewerage systems began in the latter half of the 1960s when pollution of urban rivers was taken up as a social issue. That period coincided with the time when the confusion which had marked the immediate postwar years more or less dissipated, driving the people to take an interest in accumulating social overhead capital. In Japan, it was decided to carry out the development of sewerage systems under a five-year program, as is the case with other public works projects. Before the present or fifth five-year program started in fiscal 1981, there have been four five-year programs. Under the past four programs, ¥15,120 billion (\$68.7 billion, with one dollar = 220 Yen) has been invested in terms of prices prevailing in 1980. When the first five-year program started, the population covered by sewerage systems was 7,090 thousand (with the coverage rate at about 7%), but by the end of fiscal 1980, sewerage systems had been developed for 34,500 thousand (with the coverage rate at 30%). As to whether this figure is worth that amount of investment, there might arise varied arguments according to the

American standard of judgment. If you feel the value is high, it should be pointed out that the reason is attributable to specified circumstances in which Japan is placed. Be the matter what it may, it is a fact that there have been sharp rises in the population for which sewerage systems have been developed and in the amount of investment. In 1971, or 10 years ago, when the U.S.-Japan Conference on Sewage Treatment Technology met for the first time, the population for which sewerage systems had been developed stood at 17.5 million with ¥37.3 billion (\$170 million) invested per year, whereas the population for which sewerage systems had been developed has increased to 34.5 million and the annual amount of investment to ¥1,800 billion (\$8.18 billion) by 1981.

In Japan, there are needs for the nationwide development of sewerage systems both in urban and rural areas as facilities indispensable for the assurance of the national minimum for the people. The Fifth Five-Year Sewerage System Development Program started in fiscal 1981 with a total investment of ¥11,800 billion (\$53.6 billion), targeting to develop sewerage systems for 54 million (with a coverage rate of 44%). As Japan is likely to sustain a setback as in the case of other economies of the world, optimism cannot necessarily be warranted as to whether this total amount of investment may be sustained as scheduled. But the fact is that investment in sewerage systems is strongly supported by the people and high hopes are pinned on a systematic development of sewerage systems.

2. THE PRESENT SITUATION OF SEWERAGE SYSTEMS

Sewerage projects are carried out by local autonomous bodies as the project entities in Japan. Basically, there are two methods in which sewerage projects are executed. One is the method in which public sewerage systems are developed by a municipality for itself. In the other method, treatment plants and trunk sewers are constructed and maintained by a prefectural government to collect the sewerage from a number of municipalities. The latter is known as a regional sewerage system. It is adopted in areas where the built-up areas of cities are linked together and for which the development of sewerage systems will be of considerable effect in terms of cost-effectiveness

and conservation of the water quality of rivers, etc. In either case, a project is carried out with a construction grant from the national government. One feature is that construction grants from the national government for regional sewerage systems are higher than those for public sewerage systems. In addition, the improvement of urban drainways is carried out as part of a sewerage system for preventing the flood of urban areas, as these drainways are known as urban sewer conduits.

In fiscal 1980, sewerage systems were constructed, with a total outlay of about ¥1,800 billion (\$8.18 billion). In regional sewerage systems, about ¥310 billion (\$1.4 billion) was invested.

Public sewerage projects were carried out in 695 municipalities in fiscal 1980. The relations between the population scale of municipalities and the municipalities where a public sewerage project was performed are shown in Fig. 1. Public sewerage projects are being conducted in about one-fifth of all municipalities in the nation. Municipalities where a public sewerage project has yet to be carried out have a population of less than 50,000. This suggests that the development of sewerage systems must be stepped up positively for smaller municipalities.

In areas where the development of sewerage systems is not under way, wastewater and night soil are treated by either on-site septic tanks or night soil purification plants. Regional sewerage projects were performed at 69 places throughout the nation in fiscal 1980. These projects are performed in 38 prefectures, encompassing as many as 600 municipalities. Regional sewerage systems are developed in major cities and their peripheries. Plans call for the coverage by regional sewerage systems of one-third of the total planned population. The regional sewerage system is identical in some respects with the system used by the County Sanitation District of Los Angeles in California but differs from the Los Angeles system in the sense that construction and operation are directly conducted by prefectural governments, not by a board of municipality.

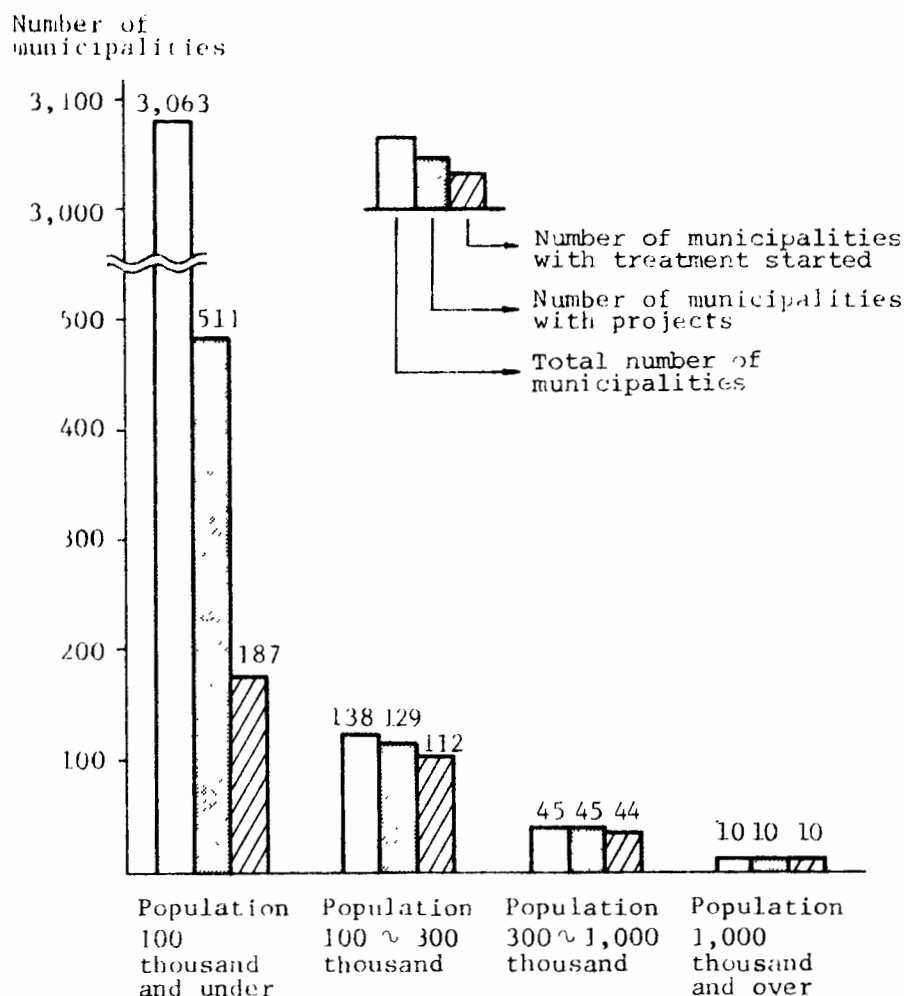


Fig. 1 Execution of Sewerage Projects
(as of the end of FY1980)

The population covered by sewerage systems in Japan at present was about 34.5 million as of the end of fiscal 1980 (March 1981), and the coverage rate to the total population was about 30%. Half of those covered, or about 17 million people, are provided with sewerage systems in Tokyo, Osaka, Nagoya and eight other major cities. In other words, 72% of the approximately 27 million people living in the 11 major cities of Japan are using public sewerage systems. The coverage rate for other areas with a total population of about 90 million is low, and only about one-fifth (19.2%) are using public sewerage systems (Fig. 2). As has been mentioned earlier, this suggests that it is an urgent task to develop sewerage systems for smaller cities in future.

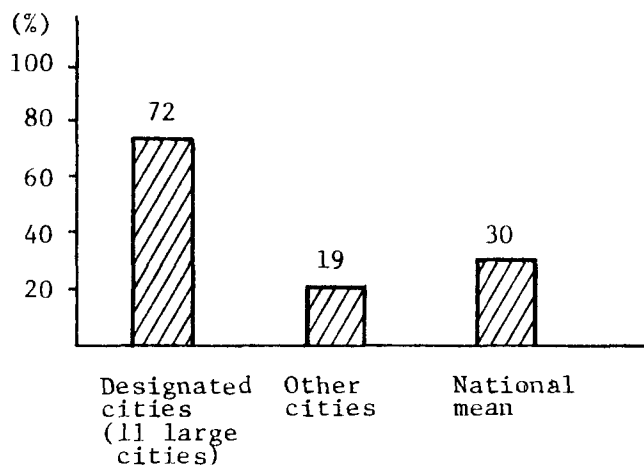


Fig. 2 Coverage Rate of Sewerage Systems to Population
(as of the end of FY 1980)

Up to now, a total of about 81,000 km of sewers have been constructed, including about 43,000 km (54%) of combined sewers, about 11,000 km (13%) of sanitary sewers and about 27,000 km (33%) of storm sewers. Combined sewers are adopted by major cities and cities which started construction earlier while the cities which started in the last 10 years have almost unexceptionally adopted separate sewer systems. The values above include those for sewers to be connected with plants under construction. It is estimated that about 80% of the 81,000 km is actually in use.

As of the end of fiscal 1980, 429 sewage treatment plants were in operation and an additional 400 plants were under construction. The total capacity of the sewage treatment plants in operation is about 25 million m³ per day (6,250 mgd), and the daily maximum dry weather flow is about 23 million m³ per day (5,750 mgd). The rate of industrial wastewater to the total flow is estimated at about 20%. Almost all the plants in operation are equipped with facilities for secondary treatment by biological processes. About 90% of them use the activated sludge process (Table 1). There have recently appeared increasing signs of the use of the fixed growth type process such as with rotating biological filters. The effluent standard of the secondary sewage treatment process is less than 20 mg/l in terms of BOD, but the quality of effluent is better than this at most plants.

Table 1 Wastewater Treatment Plants by Process

Class	Pri- mary	Middle Class			High Class							
Planned Daily Maximum Dry Weather Flow (thousand m ³ /d)	Sedi- menta- tion	High- rate Trick- ling Filter	Modi- fied Aera- tion	High- rate Area- tion and Sedi- menta- tion	Conven- tional Acti- vated Sludge	Step Aera- tion	Extend- ed Aera- tion	Con- tact Stabi- liza- tion	Pure Oxy- gen	Oxida- tion Ditch	RBC	1980 Total
Less than 5	4	7	1	5	43	5	17	3	2	4	1	92
5-10		3		4	40	8			1			56
10-50		8		11	105	29	1				1	155
50-100	1	1		2	44	19						67
100-500				2	26	24						53
More than 500	1				1	5						6
Total	6	19	1	24	259	90	18	3	3	4	2	429

NOTE: The breakdown of 429 wastewater treatment plans is as follows:
 public sewerage, 402 plants; regional sewerage, 25 plants; public sewerage for industrial
 wastewater 2 plants.

It is a thorn in the side of Japan, the national land of which is limited, to dispose of sludge -- and particularly, to acquire places for sludge disposal. The total quantity of sludge generated is estimated at about 2.4 million m^3 per year. Of this amount, 45.0% is land filled, 34.0% is used for coastal reclamation, 7.0% for dumping in the sea and 14.0% for use on farmlands. Liquid sludge disposal, as is done in the United States, is not adopted in Japan at all. All sludge is mechanically dewatered and carried off to places of disposal in the form of sludge cake or ash. Incineration is performed in many places, particularly major cities, to reduce volume. About 40% of the sludge generated throughout the nation is incinerated. From the standpoint of energy-savings, incineration presents a problem, and it is to be hoped that some substitute methods will appear. In light of the insular nature of Japan, it is argued in some quarters that ocean dumping should be put into greater consideration while also ascertaining its safety. Fig.3 illustrates the present situation of sludge disposal. When the coverage rate reaches about 44%, sludge production is estimated to exceed about 4.2 million m^3 per year.

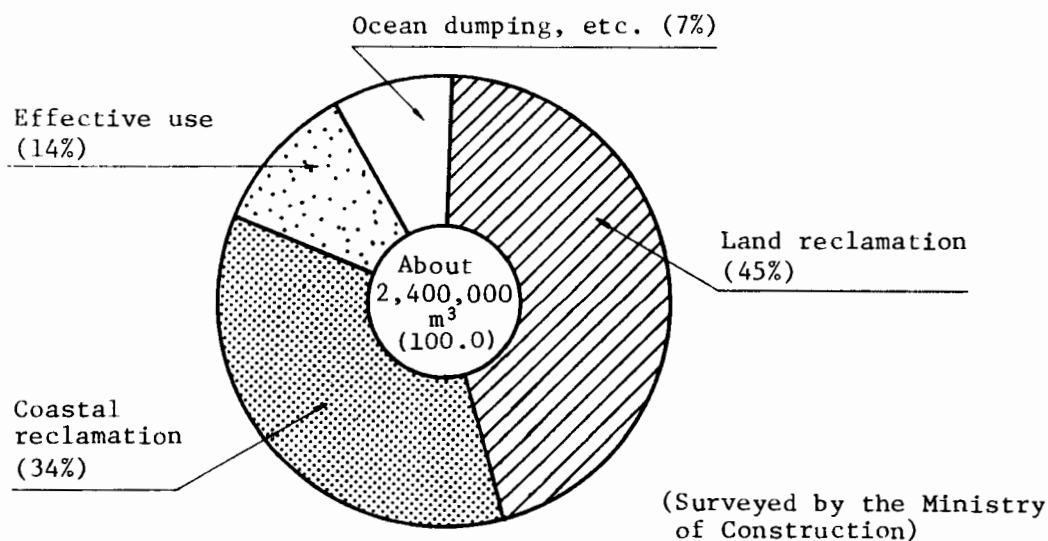


Fig. 3 Breakdown of Final Disposal of Generated Sludge (1980)

3. FIVE-YEAR SEWERAGE SYSTEM DEVELOPMENT PROGRAMS IN THE PAST

The systematic development of sewerage systems from the 1960s have evolved under five-year programs, the first one of which started in 1963. As has been introduced earlier, the year 1981 was the first year for the Fifth Five-Year Sewerage Program. In Japan, most public works are carried out under a five-year program. Five-year programs are formulated with the concurrence of Government ministries and agencies in line with the national economic program. It is a practice to work out a national economic program every five years, and in most cases to set up targets for the development of the national economy in a given period of five years. The magnitude of investment in public works is determined by the project, and investment is made in a well coordinated manner under this program. It is believed that Japan's economic development is an outcome of the successful implementation of such five-year economic programs.

The five five-year sewerage system development programs, of which the first started in 1963, have been implemented on the basis of such an economic program. In the meantime, the emphasis in Japan's public investment has shifted from infrastructure for industry to that for living. In this connection, investment in sewerage systems has increased at a remarkably fast pace. Fig 4 indicates changes in the rate of investment in sewerage projects, the rate of investment to Gross National Production (GNP) and the rate of investment to the formation by the Government of fixed capital since 1958. As is discernible from these figures, there has been a sharp rise in the outlay for sewerage since the latter half of the 1960s. Particularly in the last 10 years, the outlay has increased about tenfold. When it is taken into account that GNP has increased about 3.9 times and the formation by the Government of fixed capital 4.7 times, it is evident that the rate of investment in sewerage systems is extremely high. The year 1974 witnessed an economic setback due to the oil crisis, but there was no significant drop in the rate of investment in sewerage systems.

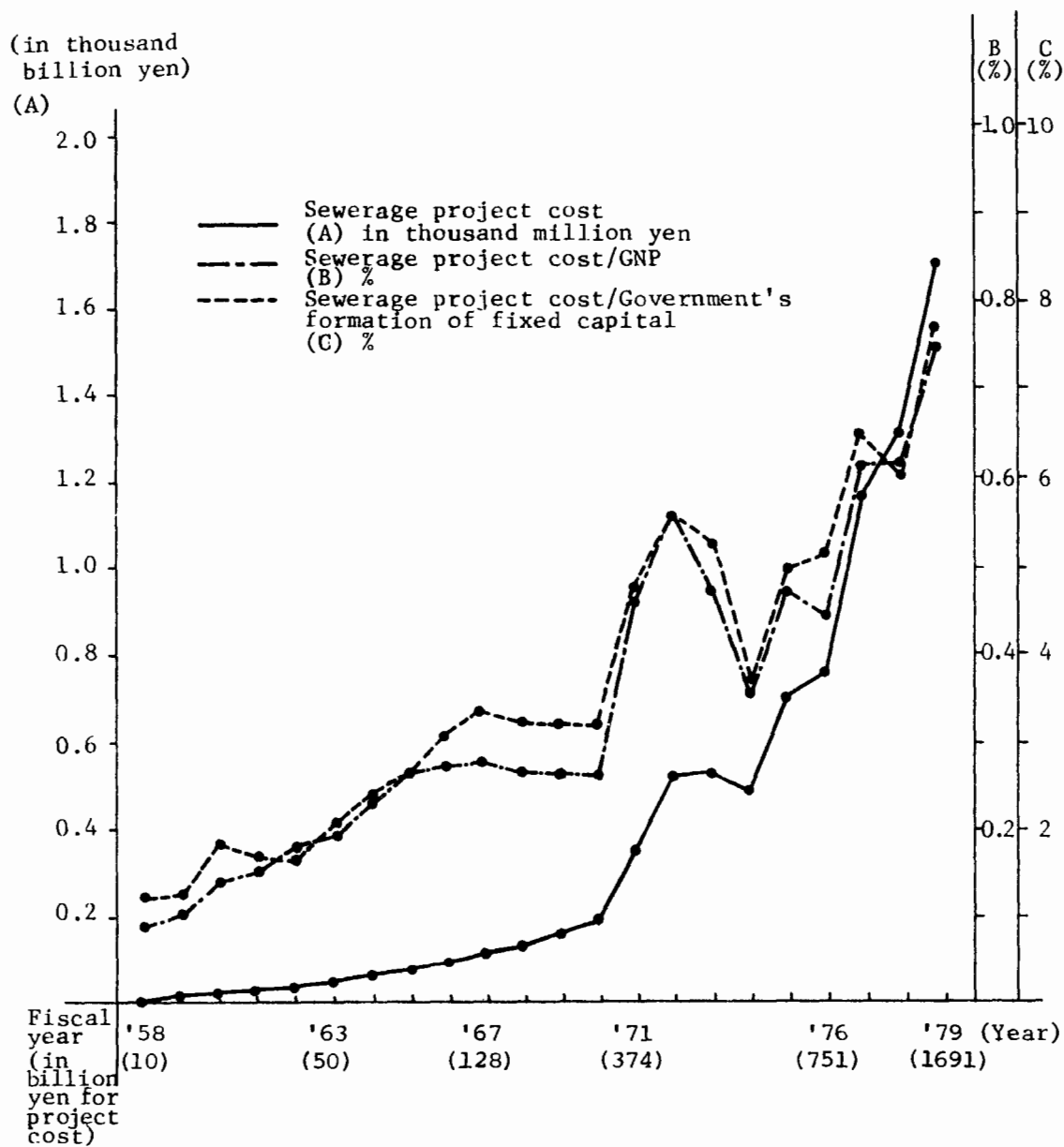


Fig. 4 Shares in Sewerage Project Cost, GNP and Government's Formation of Fixed Capital

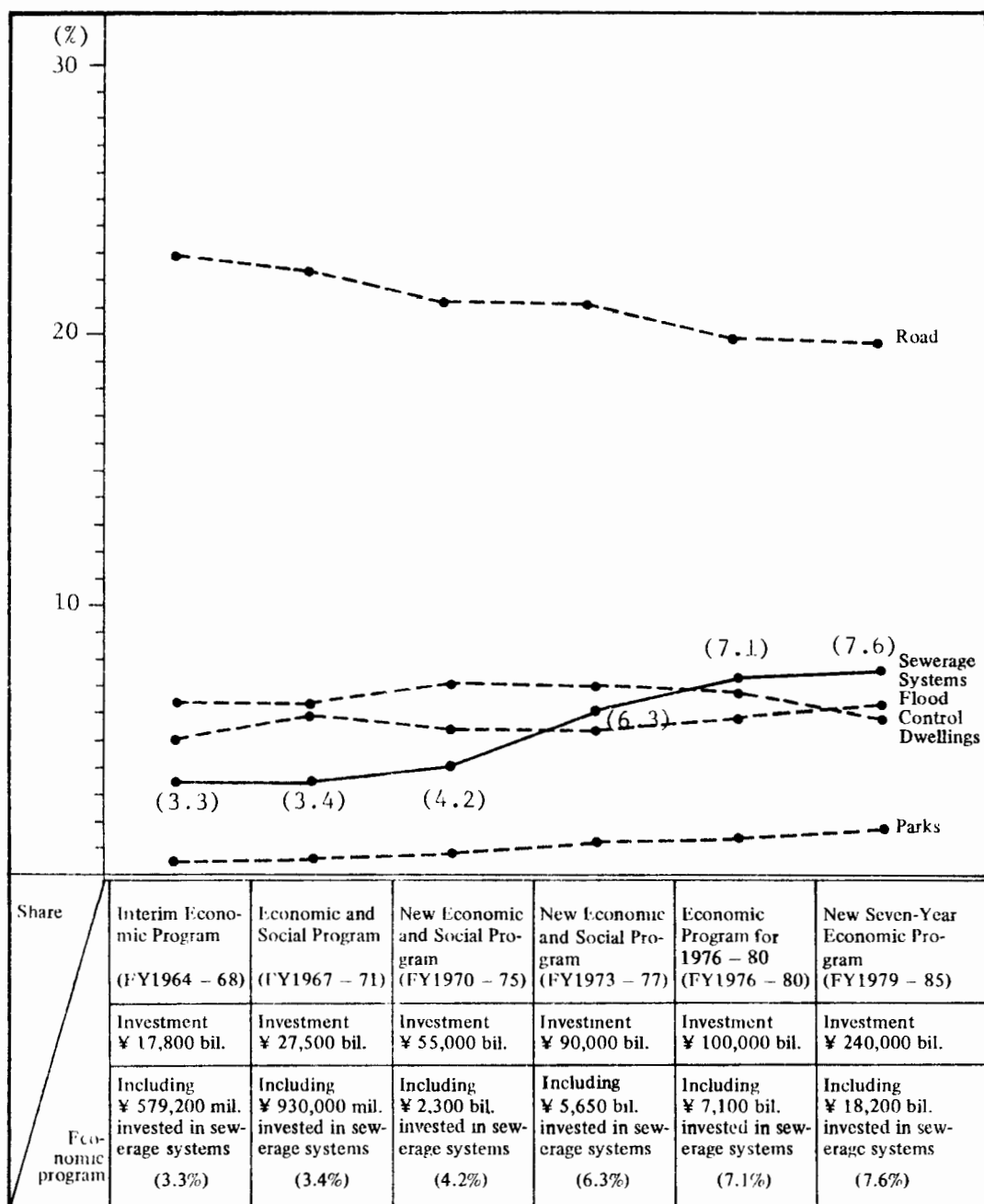


Fig. 5 Share for Sewerage Systems under Economic Program

Fig. 5 indicates the relations between the national economic program and investment in sewerage systems. Six national economic programs have been formulated since 1964. The share of sewerage systems in investment increased each time a new economic program was worked out. Of the total amount of ¥17,800 billion (\$80.9 billion), invested under the interim economic program for 1964-68, sewerage projects shared 3.3% or ¥579 billion (\$2.6 billion). Under the existing seven-year New Economic and Social Program (FY1979-85), which was formulated in 1979, a total amount of ¥240,000 billion (\$1,090 billion) is scheduled to be invested. Of this amount, sewerage systems share 7.6% or ¥18,200 billion (\$82.7 billion), indicating that the share has increased upwards of 2.3 times. In Fig. 5, the rate of investment in sewerage systems is present in relation to investment in other projects under the new economic program, including roads, flood control, dwellings and parks. Aside from the amount of investment in the construction of roads, which has a share of more than 20%, the figures show that the amount of investment in sewerage systems is higher than that of investment in other public works. Investment in sewerage systems has increased, because there is a rising awareness among the people that sewerage is a basic facility indispensable for the development of the living environment and the prevention of water pollution and because such investment is strongly supported by the people.

For the development of sewerage systems, there have already been four five-year programs -- the first (FY1963-66), second (FY1967-70), third (FY1971-75) and fourth (FY1976-80).

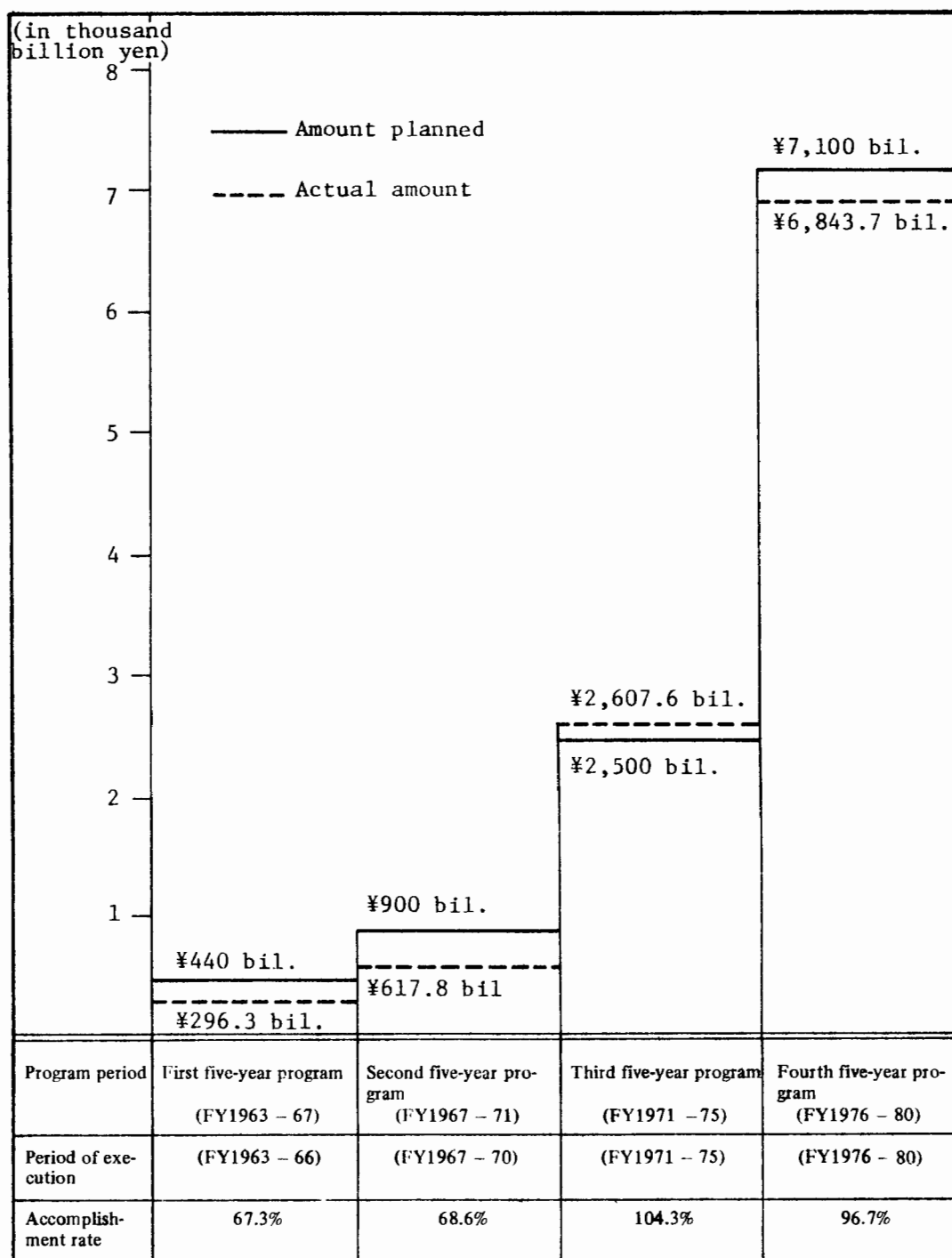


Fig. 6 Five-Year Sewerage Development Programs in the Past and Actual Records (excluding reserve funds)

Fig. 6 indicates the investment amounts (planned and actual) under the four five-year programs. As has been stated earlier, the five-year programs are formulated in accordance with the national economic program. The investment amount was roughly tripled each time a five-year program was replaced by a new one. Fig. 7 illustrates the annual changes in the population for whom sewerage systems are made available and in the coverage rate. As of the end of 1980, the population for whom sewerage systems were made available stood at 34.5 million and the coverage rate reached about 30%. In spite of such systematic investment in sewerage systems, the targets have not been attained as scheduled because of the reasons which have been introduced earlier. Under the fourth five-year program, for example, the coverage rate was scheduled to reach 40% and the population with sewerage systems was expected to be about 45 million by the end of fiscal 1980.

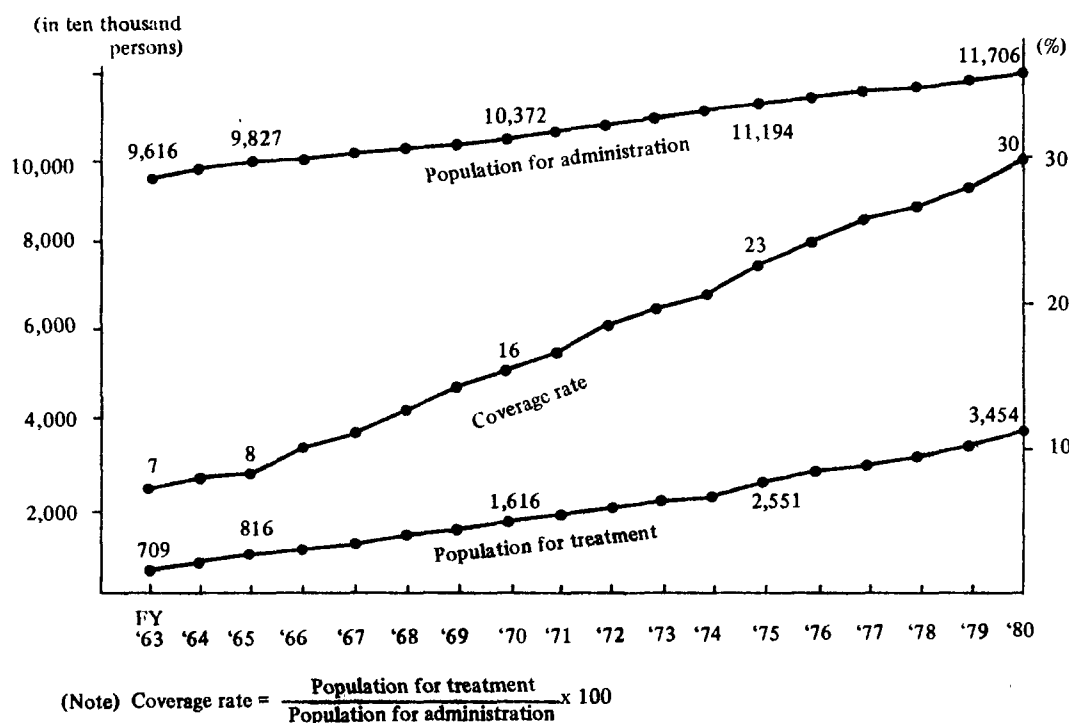


Fig. 7 Changes in Coverage Rate of Public Sewerage Systems

1st Program (FY1963-66)

The first program, which started in 1963, was formulated in accordance with an interim economic program which was worked out to correct the strains which had emerged from the policy to develop the Japanese economy at a fast pace. The high economic growth which had remained at upwards of 10% a year boosted private investment and served to develop the production sector at a strikingly fast pace, thereby casting light on the marked delay in public investment -- particularly, in dwellings and other livelihood facilities. Here, sewerage systems were assigned to serve as the core in the development of the livelihood infrastructure. On the other hand, river pollution began at a rapid pace in this period. But emphasis was put on raising the coverage rate (about 7% in those years), which had remained far lower than that of advanced foreign countries. One reason is that there was not a full awareness of the role of sewerage systems in the prevention of water pollution, and another reason is that the administration for sewerage was dualistic. In other words, the construction and management of sewage treatment plants were placed under the responsibility of the Ministry of Health and Welfare, whereas the Ministry of Construction took charge of sewers. In those years, sewage treatment plants were primarily positioned as facilities only for the improvement of the livelihood infrastructure from the standpoint of public health and hygiene. For this reason, investment was made by each municipality, and the mutual relations among municipalities from the point of water pollution control were not taken into serious account.

Second Program (1967-70)

The scale of investment was enlarged for the second program, and greater emphasis was put on the role of sewerage systems in the prevention of water pollution than in the first program. For this purpose, the second program was defined as "working for an improvement of the urban environment and contributing to the healthy development of cities and public hygiene and the conservation of the water quality of water bodies." Under this program, a regional sewerage system, effective

for comprehensive water pollution control, was adopted for the first time.

In those years, measures against water pollution were formulated mostly with an across-the-board control over individual sources of pollutants or by the use of an effluent limitation standard. But there emerged a so-called stream limitation standard, according to which sewerage systems would be formulated with consideration given to the allowable load of pollutants on the basis of the capacity of the water bodies. Sewerage systems under the second program, however, represented none other than the first step for the prevention of pollution. The Sewerage Act in those years was outdated in nature, as it emphasized the development of the living environment. The Act was not positioned for the prevention of pollution in sewerage. Under the second program, the total investment amount was conspicuously insufficient, as some sewage plants could not be put into operation on schedule. One thing worthy of special mention for Japan's sewerage administration was the fact that a revision of the laws made it possible for the Ministry of Construction to take charge of the construction, maintenance and control of total sewerage systems.

Third Program (FY1971-75)

In 1970 or the year before the start of the third program, the Sewerage Act was revised, and this revision was epoch-making for the history of sewerage systems in Japan. At the Diet session when the law was revised, known as the "Diet session against pollution," the deliberation on a number of bills concerning anti-pollution measures was conducted, and the bill for a revision of the Sewerage Act was submitted as one of the important bills to implement measures against water pollution. In the revision of the Sewerage Act, the conservation of public water quality was incorporated as one of its purposes. Against the pollution of public waters, the Government formulated environmental standards for the water quality and environment in respect to the major public water bodies, and it was decided to attain these targets within about five years.

A note-worthy concept was introduced in those years. By this concept, sewerage systems were considered as part of a comprehensive water quality conservation project, a compatible program was formulated for each one of the waters bodies for which environmental standards had been worked out, and investment would be made in a systematic manner according to the program. This concept was known as a stream limitation standard, according to which it was decided to develop sewerage systems in Japan. Partly due to the establishment of one strict standard after another, it was in those years that many sewage treatment plants began to go as far as to incorporate in their future programs treatment at levels more advanced than secondary treatment.

The third program called for an outlay of ¥2,600 billion (\$11.8 billion), as against an initially scheduled investment of ¥2,500 billion (\$11.4 billion). However, only about 50% of the projects incorporated in the program were accomplished, due to the outbreak of the oil crisis in 1973-74. There was rising dissatisfaction among the people at the delay in construction of sewerage systems. The fact that the accomplishment rate was so low was attributable to the following factors:

- 1) Rises in the unit cost of construction as brought about by inflation;
- 2) Rises in the cost necessary for safety measures for the construction of sewers;
- 3) Rises in the cost for an upgrading of plants so that their effluent may be made to meet standards;
- 4) Rises in the costs of additional facilities, such as for covers and anti-air pollution equipment, which were required as environmental or aesthetic measures for sewage treatment plants.

Fourth Program (FY1976-80)

The fourth program was formulated in the midst of circumstances where the role of sewerage systems was of increasingly greater importance. In the meantime, environmental water quality standards were worked out for all major water bodies in Japan. Under the Pollution Prevention Program which was formulated for areas urgently in need of the accomplishment of environmental standards, the areas encompassed in this program increased in number.

On the other hand, the people's calls for the construction of sewerage systems came out in the form of "national minimums" for their living environment. Not only in the built-up areas where the development of sewerage systems had been carried out, but also in mainstay rural communities, there were strong calls for the development of sewerage systems.

In other words, sewerage projects were made broader in coverage from major cities so that they also include local cities. The fourth program envisioned a raising of the rate of diffusion in the population from 22.8% at the end of fiscal 1975 to 40% by the end of 1980 with a total investment amount of ¥7,100 billion (\$32.3 billion).

As in the case of the third program, many of the targets could not be attained, though funds had been made available for practically every project. For one thing, this was because of a delay in the implementation of the third program. Another reason is that as an attempt had been made to develop sewers and plants at the same time, the rate at which they were developed was thrown out of balance.

As a result of systematic investment, however, it should be noted that sewerage systems have accumulated and the pace of development will become faster from now on.

Progress of the development from 1963 may be summarized as follows:

- 1) Phase in which priority was given to the development of the national economy and the development of sewerage systems was not conducted to a full extent;
- 2) Phase in which the national economy developed at a constant pace and the development of sewerage systems was done from the standpoint of public health and hygiene;
- 3) Phase in which water pollution was in progress and the development of sewerage systems was taken into account for water pollution control;
- 4) Phase in which the development of sewerage systems was conducted with due consideration given to the prevention of pollution of major and other cities and their peripheries;

- 5) Phase in which the development of sewerage systems progressed so far as to encompass local cities and rural communities.

4. FIFTH FIVE-YEAR SEWERAGE SYSTEM DEVELOPMENT PROGRAM

4.1 Background for the Program

As has been mentioned earlier, the construction of sewerage systems has been carried out at a level lower than what the people have hoped for even with the four five-year programs in the past. As there are strong calls for sewerage systems, their development is an urgent task. On the other hand, measures for water pollution control have been strengthened year by year. The mass control of water quality was put into effect in 1979, and effluent regulation in the three major metropolitan areas of Tokyo, Osaka and Nagoya was strengthened to a great extent. As there was a delay in the construction of public sewerage systems, criticisms were made that only private enterprises were assigned with measures to prevent water pollution. As there was progress in the eutrophication of stagnant waters, such as lakes and bays, calls emerged for the implementation of measures against sewage treatment plants which discharged wastewater into these waters. To social calls for the saving of energy and resources, the field of sewerage could not remain indifferent, and there have been increasingly strong calls for the construction of sewerage systems for which the cost and the consumption of energy are low. Now that there do not necessarily seem to be bright prospects for the future of the Japanese economy, arguments have evolved as to whether enormous amounts of money could be invested continuously in sewerage systems as scheduled.

4.2 New Seven-Year Economic Program

As has been explained earlier, investment has been systematically made in sewerage systems in accordance with the national economic program. The fifth five-year program is based on the new economic program formulated in 1979 for the period of fiscal 1979-85.

The basic factors for the background of this program consist of the necessity of harmonizing structural changes in the global economy and the increasingly unstable supply and demand of resources, energy and

food with an international economic community which will deal with these elements; the necessity of a reasonable economic growth adaptable to changes in the structure of the Japanese economy, such as the transformation of economic growth; and the necessity of building a new welfare society which will respond to a rapid increase in the number of elderly in the population, dispersion of the population and industry and the increased awareness of the people on the need to shift from a quantitative expansion of their lives toward qualitative improvement.

For the management of the Japanese economy in future, basic policy under this program include ① correcting disparities in each economic sector, ② working for a conversion of the industrial structure and overcome limits on the availability of energy, and ③ striving to realize a new welfare society unique to Japan. For the economic management of Japan, the targets are to ① accomplish full employment and stabilize prices, ② stabilize and replenish the national life, ③ collaborate in and contribute to the development of the international economic community, ④ assure economic security and foster the infrastructure for economic development, ⑤ reconstruct finance and come up with new financial responses. For the accomplishment of these targets, it has been decided to carry out a variety of measures in a harmonious way.

For the stabilization and replenishment of national life, measures will be implemented for the development of social security, replenishment of education, promotion of sciences and culture, replenishment of consumers' lives, redevelopment of dwellings, conservation and development of the environment, assurance of safety and replenishment of social capital in order to assure a stable and serene national life, an affluent and worthy-to-live national life and a comfortable and tasteful national life. Particularly in respect to the replenishment of social capital, the basic policy is outlined as follows:

- (1) The level at which social capital is developed has been gradually improved in recent years but is considerably lower than that of advanced nations. Social capital service is relatively retarded, compared to the desire for the replenishment of social consumption, which is growing in conjunction with improvements in the economic activity of the private sector

and in the private consumption level. It is necessary to further replenish social capital service.

Public investment plays an important function as part of a policy for the control over total demand. The development of social capital with public investment is something which ought to be stepped up in a systematic and planned manner from a long-range standpoint. Though the financial situation is rigid, roughly ¥240,000 billion (\$109 billion) of public investment (in terms of prices prevailing in fiscal 1978 and including compensations for land acquisition) will be made during the program period in an attempt to improve the relative balance for the economic activity of the private sector and replenish social capital service which will be required for the national life. The stock of social capital (fixed public net assets) is expected to increase almost twice by fiscal 1985.

- (2) In stepping up the development of social capital, there is a need to select and give priority to sectors of investment in order to contribute to a balanced development of the national land, while responding to the sophistication and diversification of the people's needs.

During the term of this program, priority will be given to investment in sectors which are directly tied in with the national life in line with the concept of residence, etc. and the development of social capital will be stepped up in the following manner. In this case, however, keeping the balance between types of social capital and between social capital and economic activity will be taken into account.

First, in order to improve the environment for everyday life and to contribute to a qualitative improvement of the national life, an attempt will be made to work for a replenishment of the facilities associated with the living environment. The quality of dwellings will be improved and the development of facilities for which development has been delayed, such as

sewerage systems, solid waste treatment facilities, urban parks, welfare facilities and cultural and educational facilities, will be stepped up. In other sectors, moreover, priority will be given to the development of facilities closely tied in with national life, such as roads, urban rapid transit systems, remote places, sea ports and airports of offshore islets, traffic safety facilities, urban rivers and rural environment facilities.

Moreover, in order to check the concentration of population and industry in major cities and work toward balancing the use of the national land and promoting the countryside, the basic transport and communication facilities, national land conservation facilities and other facilities which will become the infrastructure for the above purposes will be streamlined. In order to reinforce the production infrastructure in rural communities, facilities for agriculture, forestry and fisheries will be streamlined.

The program contains concrete targets for the promotion of social capital development in this direction. It is scheduled to invest ¥18,200 billion (\$82.7 billion) in seven years from fiscal 1979 to fiscal 1985 with a view to bringing to about 55% the rate of coverage for sewerage systems to the total population

Even in the national economic program, therefore, the development of sewerage systems is considered insufficient and it has been agreed to put greater emphasis on investment in sewerage systems. The share of sewerage projects in the total outlay has been increased by 0.5% from the 7.1% appropriated under the previous program (Table 2). This is proof that the importance of sewerage facilities and the urgency of their development are recognized and high expectations are pinned on their development in future.

Table 2 New Seven-Year Economic Program

(in hundred million yen)

Economic program Public works	Economic Program for 1976-80		New Seven-Year Economic Program (1979-85)	
	Investment	Share(%)	Investment	Share(%)
Roads	195,000	19.5	460,000	19.2
Dwellings	65,000	6.5	135,000	5.6
Sewerage systems	71,000	7.1	182,000	7.6
Urban parks	15,400	1.5	45,000	1.9
River conservation	55,000	5.5	142,000	5.9
Total	401,400	40.1	964,000	40.2
Grand amount	1,000,000	100(%)	2,400,000	100(%)

4.3 Substance of the Program

The fifth program which encompasses the five-year period of April 1981 to March 1985 contains the following targets.

- ① The development of sewerage systems in local cities will be encouraged to promote the concept of residence;
- ② The development of sewerage systems will be encouraged to prevent the flood of built-up areas and enhance the safety of the urban life;
- ③ To prevent the eutrophication of stagnant waters such as lakes, the installation of advanced wastewater treatment facilities will be promoted;
- ④ The development of sewerage systems for rural communities will be encouraged;
- ⑤ Regional sludge disposal projects will be encouraged as well as projects of effluent reuse and resource recovery from sludge.

a) Scale of Investment and Targets of Development

In the economic program above, the scale of investment was set at ¥18,200 billion (\$82.7 billion). With the less bright prospects for the future economic situation taken into account, it was decided to extend this economic program for an additional 1.5 years or so and to set the investment scale under the five-year program at ¥11,200 billion (\$50.9 billion). The major targets include the construction of about 12,000 km of sewers and that of sewage treatment plants and pumping stations enough for another 11 million people. In the sector of regional sewerage systems, the major targets include, among others, the development of 1,200 km of trunk sewers and that of sewage treatment plants enough for about another 8 million people. If the program is accomplished as scheduled, about 54 million people out of a total population of 123 million will be able to use public sewerage systems and the coverage rate will be raised to 44% by the end of fiscal 1985. The investment plans are shown in Table 3.

Table 3 Project Cost for Fifth Five-Year Sewerage Development Program

(million yen)

Classification	New program (FY 1981-85)
Public sewerage systems	8,391,000
Subsidized projects	5,161,500
Local independent projects	3,229,500
Regional sewerage systems	2,230,000
Subsidized projects	2,073,900
Local independent projects	156,100
Urban sewerage systems	460,000
Specified public sewerage systems for industries	27,000
Subsidized projects	18,100
Local independent projects	8,900
Specified public sewerage systems for conservation of the environment	102,000
Subsidized projects	76,500
Local independent projects	25,500
Coordination outlay	590,000
Reserve fund	-
Total	11,800,000
Total for subsidized projects	7,790,000
Total for local independent projects	3,420,000

Incidentally, the unit costs for the construction of sewerage under this program are ¥550 thousand (\$2,500) per person on the national average. Broken down, the cost includes ¥157 thousand (\$713) per person for the construction of treatment plants, ¥393 thousand (\$1790) per person for sewers, and ¥27 million (\$122 thousand) per hectare for branch sewers.

b) Priority Matters

Priority matters under the five-year program are as follows:

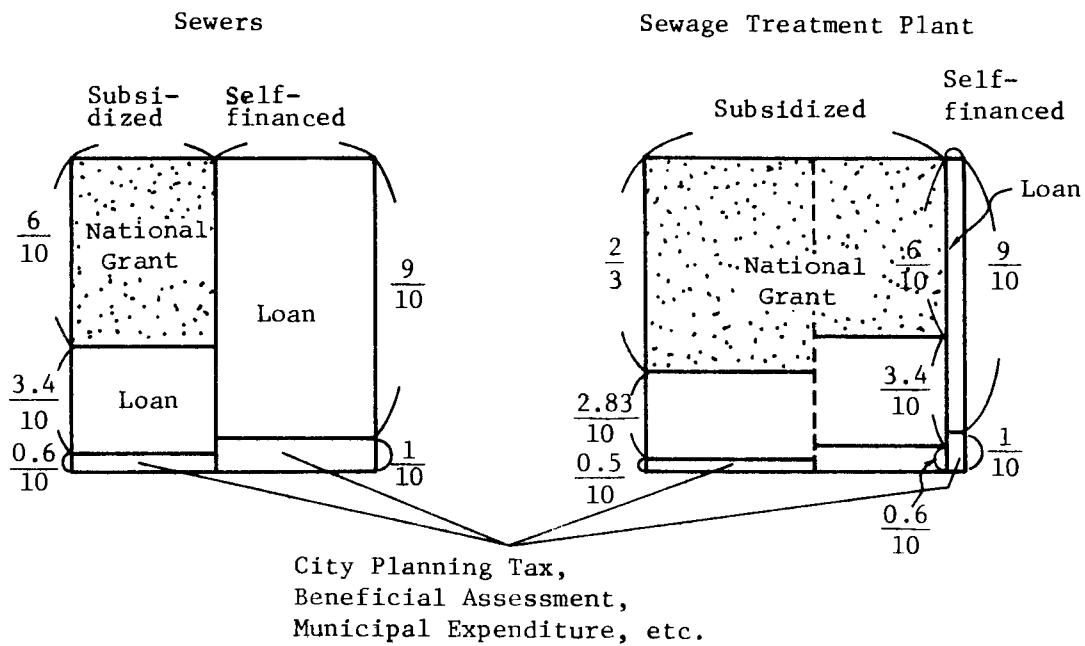
- a. To promote the concept of residence incorporated in the Third Comprehensive National Development Program, the development of sewerage systems in local cities will be positively stepped up;
- b. To prevent the flood of built-up areas, etc., and work for an improvement of the living environment, the development of sewerage systems will be promoted;
- c. As part of housing and housing site policy, the development of sewerage systems in new urban areas will be stepped up on a priority basis;
- d. To conserve water quality of lakes and other natural environments and improve the living environment of rural communities, the development of sewerage systems will be stepped up;
- e. To work for the accomplishment and maintenance of water quality and environmental standards, advanced treatment will be stepped up;
- f. To establish stable disposal sites for sludge in metropolitan areas, regional disposal of sludge will be promoted.
- g. Measures for energy saving, such as energy recovery from sludge, will be encouraged.

c) Financial Plan

The sewerage systems in Japan may roughly be classified into two types -- public sewerage systems and regional sewerage systems.

The financial sources for the construction of sewerage systems consist of, among others, national construction grants, local loans, municipal funds, city planning taxes, prefectural grants and beneficial assessments. The composition of these financial sources is shown in Fig. 8 in respect to public and regional sewerage systems.

(1) Public Sewerage



(2) Regional Sewerage

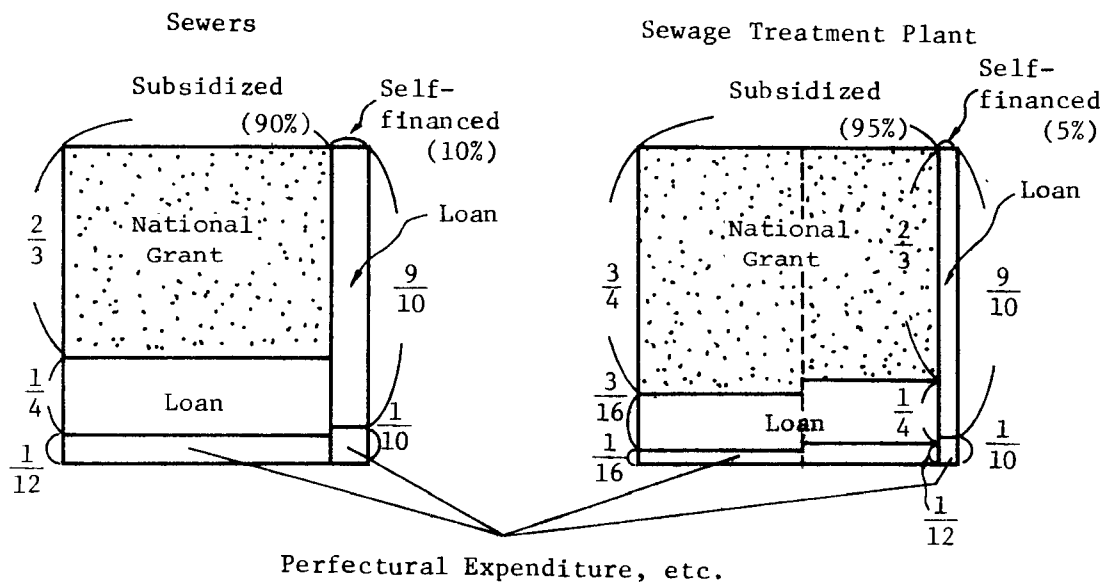


Fig. 8 Composition of Financial Sources for Construction Cost in Sewage Works

a. National Grants

Now that sewerage systems must be developed as part of the policy of water pollution control from a national point of view, the rate of national subsidization of the cost of the construction of sewerage systems has been raised gradually. Since 1974, the rate has been set at 6/10 for public sewerage systems (2/3 for sewage treatment facilities) and 2/3 for regional sewerage systems (3/4 for sewage treatment facilities).

Of a sewerage system construction project, the objects of national grants are confined to the main sewer, pumping facilities and treatment plants, and they are specified in a notice of the Ministry of Construction. The rate of the construction cost of the facilities to be subsidized by the national government to the total construction cost is 60% (45% for designated cities and 75% for general cities) for public sewerage systems, and in the case of regional sewerage, this rate is 93% under the 4th Five-Year Program.

b. Local Loans

For a certain percentage of the construction cost, the floatation of loans or bonds is authorized, which constitutes a principal financial source for any construction project.

By type of fund, sewerage project loans consists of government funds, financing corporation funds and private funds (publicly subscribed funds on the market and bank-related funds). Of the amount planned for fiscal 1978, government funds accounted for 30%, public financing corporation funds 23% and private funds 47%, indicating there are signs that the rate of advantageous government funding is on the decline and that of costly private funds is rising.

c. Beneficial Assessments

Beneficial assessments for sewage works are imposed on the owners of land in the drainage area of public sewerage systems in accordance with a municipal ordinances and the City Planning Law.

Of the 700 cities for which public sewerage systems are approved, 348 cities adopted this system in fiscal 1980. The assessment per unit was ¥150-350/m³ in most cases.

The beneficial assessments collected in fiscal 1980 accounted for about 1.0% of the cost for the construction throughout the nation and about 2.2% for cities which had collected assessments from beneficiaries. Now that the financial situation of local autonomous entities is aggravated, there are calls for a re-inforcement of the beneficial assessment system and a raise in the rate of appropriation to the financial source for the construction.

d. City Planning Taxes

City planning taxes are levied by municipalities on the owners of tracts of land and buildings in city planning areas as object taxes to make up for the cost required for city planning projects. Therefore, the city planning tax revenue is appropriated for city planning projects other than the public sewerage system project. In fiscal 1979, the rate of city planning taxes in the cost of the construction of public sewerage systems throughout the nation was about 1.3%.

e. Prefectural Grants

Subsidies were extended by 12 prefectures (out of 47 prefectures throughout the nation) in fiscal 1979 to the municipalities engaged in a public sewerage project. The share of prefectural government funds in the cost for the construction of public sewerage systems stood at 0.5%, and there had been signs of a drop in this share since 1974.

As for the cost for the construction of regional sewerage systems, it is made a fundamental rule for the prefectural government and the related municipalities to go fifty-fifty on the amount gained by deducting national grants from the total cost. The apportionment of the cost share among the related municipalities is normally undertaken according to the planned waste-

water quantity ratio or the planned drainage area ration for each municipality.

In respect to the shares of the national and local autonomous entities in the financial source for the construction of sewerage systems in fiscal 1981, a national government share of 39% of the total project cost of about ¥1,800 billion (\$8.18 billion) has been decided with the local autonomous entities' share set at 61%. Plans call for the appropriation of about 93% of the local autonomous entities' share with local loans. The share for national grants was raised each time a new five-year program was worked out, rising from 22% in 1967 to 39% at present. There have continuously been strong calls from local autonomous entities to raise the national government's share. In light of the public nature of sewerage systems and the urgency of conserving water quality, the share of the national grants has been on the upturn. Now that the nation's financial situation has been aggravated, it is presumably quite difficult to raise the rate further under the fifth program. Therefore, a financial plan will probably be formulated with the present rate left as it is. It has been suggested that the allocation of individuals' shares should be reviewed in a financial program for sewerage projects to say the least for the reinforcement of the beneficiaries' share.

5. LONG-RANGE PROSPECTS FOR DEVELOPMENT OF SEWERAGE SYSTEMS

The ultimate purpose of the development of sewerage systems is to enable all of the people to use public sewerage systems, prevent built-up areas from being flooded, prevent the pollution of rivers, lakes and maritime regions and maintain water quality at levels which meet environmental standards and, assure the availability of safe water for use and dispose of generated sludge in a stable manner. For this purpose, it is necessary to construct operate and maintain sewerage systems in an efficient and economical way. The approaches to achieve the above goals in a long-range perspective have been under discussion.

The long-range prospects for the development of sewerage systems in Japan in terms of 20 years in the future, as proposed by a national study committee, are as follows:

① Improvement of Sewerage Coverage Rate

Sewerage systems will be completely developed in built-up areas, whether they be in major or local cities, the development of the living environment for major hamlets in rural communities in areas other than the built-up districts will be stepped up, and the development of sewerage systems for the conservation of the water quality of lakes, etc., the natural environment of which is superior, will be promoted so that the ratio of the covered population to the total population may brought in live with the level of advanced Western nations by the year 2000.

There are a number of views concerning the level of the coverage rate which should be attained in Japan over the long run. With the concentration of population in urban areas now in progress, it is estimated that urban population will account for 71.5% (96.5 million) of the total population by the year 2000. Nevertheless, it is a fact that sizable population groups still continue living in rural communities. No convincing conclusion is drawn as to whether a wastewater treatment and disposal system as is available at present could be of much effect in areas other than for cities. At present, the target is to make sewerage systems available for 100% of the urban population and about 90% of the total population. Incidentally, water supply, including that which is supplied under a rural water supply system, is supplied to about 90% of the total population, and sewerage systems should be developed for hamlets with a population of up to 200. In the United States, the coverage rate stands at about 72% at present, and it is understood that the question of the development of an on-site or collection type for adoption by the remaining portion of the population is under discussion. In the case of Japan, the rate of development of the collection type will presumably rise in light of the high population density. Be the matter what it may, it remains an urgent task come out with a method in which sewerage systems may be developed in the countryside -- particularly, in areas with a significantly low population density.

② Upgrading Wastewater Treatment

To satisfy the water quality and environmental standards and prevent a eutrophication of the stagnant waters, an attempt will be made to reinforce measures for the minimization of pollutants at their generation sources. In sewerage projects, too, advanced waste treatment will be carried out in necessary areas to eliminate greater amounts of organic matter and nutrients.

In Japan, almost all rivers and other public waters are subjected to strict environmental water quality standards. It is the target and wish of the people to restore all waters throughout the nation to their beautiful condition and maintain them as such, and to make it possible to use safe water. When it is taken into account, however, that the flow of rivers near cities is decreasing due to an increase in the use of water and that high self-purification capacity cannot be expected for rivers in Japan as their slopes are steep and they flow down quickly, it is estimated that it will be necessary to have a considerable number of sewage treatment plants for which some forms of advanced treatment will be required.

The extremely high concentration of industry and population also is a reason for necessity of advanced waste treatment. It is necessary to make a full study of the degree to which the people's consensus may be gained on their share of the financial burden, which will increase in conjunction with an introduction of sophisticated treatment techniques, now that there are calls for an balancing of the outlay for sewerage systems for the whole economy and for energy saving and considering that the people, after all, have to bear the maintenance and control costs of sewerage systems as taxpayers. It is a fundamental rule at present for the users of sewerage systems to bear their operation and maintenance costs, but there is a need to study who will bear the additional costs incidental to an introduction of advanced waste treatment and how this will be done. At the same time, it is equally necessary to develop less costly, energy-saving treatment techniques.

③ Improvement of Existing Facilities

To maintain the functions of sewerage systems, the improvement of outdated facilities will be promoted and that for combined sewers will be carried out, depending on the characteristics of a given area.

At present, the design of conduits and storm sewers for the disposal of storm water is accomplished on the basis of the rainfall probability for 5-10 years, but the calculation of this probability is not considered sufficient by any means. In fact, the lowland sections of many cities have flooded almost every year, and there is a need to advance the probability year which is compatible with the capacity of rivers. It is also necessary to devise non-structural measures, such as for the underground permeation of stormwater.

It is also a major task for Tokyo, Osaka and other large cities to develop measures against overflows from combined sewers in wet weather. The findings of many research works indicate that the pollution load of overflows cannot be ignored in terms of water pollution, and it is desirable that appropriate measures be worked out. However, the cost for improvement will presumably be tremendous, and measures are likely to be formulated step by step, depending on the development of sewerage systems.

④ Execution of Regional Disposal System for Sewage Sludge

For Japan whose national land area is limited, the disposal of wastewater and sludge is a very difficult issue. To secure stable places for disposal on a long-range basis, elaborage surveys have been carried out for agricultural and other effective uses, and these have been realized in many districts. In major metropolitan regions, where sludge is generated in significantly large amounts, it is considered difficult to make use of the whole quantity and it has become an urgent task to devise a system of regional disposal. At present, surveys are under way to construct reclaimed places for regional disposal in some parts of the bays of Tokyo and Osaka.

(5) Reuse of Effluent

It is known that the precipitation amount for Japan is great (1,800 mm/year) and Japan is relatively rich in water. As a result of the concentration of population and industry in urban areas, it is estimated that the major metropolitan regions will become short of water in the near future. Since that hopes cannot depend on the development of dams, it is an urgent task to establish a system for effluent recycling.

6. SUMMARY

The present situation of sewerage system development, the situation of systematic investment in sewerage systems, the substance of the Fifth Five-Year Sewerage System Development Program and long-range prospects for the development of sewerage systems in Japan are summarized in this section. As there was a delay for Japan's embarkation upon the development of sewerage systems, many difficulties have been encountered, but the development might be described as having made rapid progress thanks to systematically planned investment. The focus for the development of sewerage systems also has been shifted from major cities to smaller cities and its emphasis from considerations of public health and hygiene to the prevention of water pollution. There is apprehension about the possibility of accomplishing the targets contained in the fifth five-year program due to the aggravation of national finances, but there are many sewerage projects that have yet to be carried out. Several other five-year programs will be formulated in future in an attempt to attain the long-range development targets.

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CURRENT ISSUES IN WATER POLLUTION CONTROL ADMINISTRATION IN JAPAN

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1. CURRENT STATE OF POLLUTION IN JAPANESE WATERS

Although still very far from a satisfactory state, an overall improvement of water quality may be noted in Japanese waters in recent years. Let me start by examining the pollutants for which environmental water quality standards have been established under the provisions of the Basic Law for Environmental Pollution Control. As regards toxic or harmful substances relating to protection of human health such as cadmium, mercury and PCBs, the ratio of samples exceeding the respective standards to the total number of samples taken has continued to decline over the years. (See Table 1.)

Table 1. Ratio of Samples Exceeding Water Quality Standards Relating to Protection of Human Health To The Total Number of Samples Taken

Substances	Water Quality Standards	1970 (%)	1975 (%)	1979 (%)
Cadmium	0.01 ppm	2.8	0.31	0.13
Cyanide	N.D.	1.5	0.02	0.01
Organic P	N.D.	0.2	0	0
Lead	0.1 ppm	2.7	0.32	0.00
Cromium (VI)	0.05 ppm	0.8	0.02	0.01
Arsenic	0.05 ppm	1.0	0.24	0.16
Total Mercury	0.0005 ppm*	1.0	0**	0**
Alkylmercury	N.D.	0	0	0
PCBs	N.D.	-	0.38	0.05

(* Annual averages; **In number of sampling sites)

Among a number of substances and items for which there exist water quality standards relating to the preservation of our living environment, taking BOD levels in rivers for example (See Table 2), a general trend toward improvement may again be noted here. The non-compliance rate, however, is high among small rivers and streams flowing through large urban districts. The rate of achievement of environmental standards in 1979 was: 69% for water bodies belonging to Category AA, 75% for Category A, 60% for Category B, 46% for Category C, 56% for Category D, and 41% for Category E, with an overall achievement rate of 65%.

Table 2. Ratio of Samples Exceeding Water Quality Standard for BOD in Rivers

Water Quality Categories	Water Quality Standard for BOD	1971	1975	1979	(Number of Water Bodies 1979)
AA	1 ppm	36.7	31.4	23.9	(287)
A	2 "	30.9	24.4	21.5	(1002)
B	3 "	35.6	27.4	28.1	(497)
C	5 "	39.9	42.6	43.4	(227)
D	8 "	52.8	37.8	36.5	(81)
E	10 "	70.2	49.7	43.5	(142)

Taking for another example the number of coliform groups, one would get a non-compliance rate of 79% for all water bodies in Category AA (the standard is less than 50 MPN/100 ml), 73% for Category A (ditto, less than 1,000 MPN/100 ml), and 61% for Category B (ditto, less than 5,000 MPN/100 ml). No improvement can be seen in this respect.

Turning next to COD levels in coastal waters, a general trend for improvement may again be recognized, but the rate of achievement in 1979 was: 64% of water bodies in Category A, 82% in Category B, 99% in Category C, with an overall achievement rate of 78%. The rate was particularly low in the "enclosed" or "semi-enclosed" bodies of water. (See Table 3)

Table 3. Ratio of Samples Exceeding Water Quality Standard for COD in Coastal Waters to the Total Number of Samples Taken

Water Quality Categories	Water Quality Standard for COD	1971	1975	1979	(Number of Water Bodies 1979)
A	2 ppm	38.5	19.6	17.4	(222)
B	3 "	30.5	18.3	17.1	(193)
C	8 "	15.2	7.4	4.7	(117)

Looking at COD levels in lakes and reservoirs, here again one may note an overall trend for improvement, but the actual rate of achievement of water quality standard is extremely low, as illustrated by the achievement rates of 24% of water bodies in Category AA, 56% in Category A, only 7% in Category B, with an overall average rate of 42%. (See Table 4)

Table 4. Rate of Achievement of the COD Standard for Lakes and Reservoirs

Water Quality Categories	Water Quality Standard for COD	1971	1975	1979	(Number of Water Bodies 1979)
AA	1 ppm	13.9	62.0	73.5	(21)
A	3 "	79.4	69.4	56.8	(62)
B	5 "	91.8	84.6	74.2	(14)
C	8 "	-	40.7	70.8	(1)

2. WATER POLLUTION IN SEMI-ENCLOSED COASTAL WATERS AND AN AREAWIDE POLLUTION LOAD CONTROL SYSTEM

In the three metropolitan regions of Tokyo Bay, Ise Bay and Seto Inland Sea lives 53% of the total population of Japan, and 65% of manufactured goods are produced by these regions. As naturally to be expected, a large amount of pollutants is discharged into these so-called "enclosed" water areas. The flushing rate, or turnover of water with the outer fringes of the ocean, is very small, and organic substances are easily retained and accumulate in these water bodies. Due to the great influx of nutrients, moreover, the process of eutrophication is in rapid progress. As a result, the water pollution problem for these waters is getting worse and more complex every year.

In order to cope with this worsening progress of water pollution, the 84th session of the National Diet in 1978 enacted a Special Measures Law for Seto Inland Sea Environmental Preservation and amendments to the Water Pollution Control Law, both of which went into effect in June, 1979. With the objective of achieving and maintaining environmental water quality standards, the new Law and the amendments of 1978 introduced a system of measures aimed at an "areawide control of water pollution load", which sought to cut down on the amount of total pollutant loadings into the three water areas in an effective and integrated manner, from all sources including domestic households and hydraulic load of rivers in upstream inland areas.

Targeted reduction in COD load, according to the type of sources and to each prefecture, goals for raising the percentage of sewered population, basic guidelines on setting new effluent control standards, etc. were to be specified in a comprehensive Pollutant Reduction Plan. The Plan was approved by the Prime Minister in March, 1980, which set a deadline of 1984 for reducing the COD loadings to the levels shown in Table 5.

Table 5. General Features of Tokyo Bay, Ise Bay and Seto Inland Sea regions and the Goals for Reduction of COD Load

	Tokyo Bay	Ise Bay	Seto Inland Sea
Surface Area of Water Body (Km ²)	1,400	2,300	23,000
Volume of Water (X100 million m ³)	540	460	7,330
Population (1976) (X1,000)	22,200	9,090	28,130
Manufactured Goods (1976) (trillion yen)	32.2	18.4	44.5
COD Loadings in 1984 (target) (tons/day)	660	426	1,283
Domestic Sources	386	179	517
Industrial Sources	180	208	666
Others	94	39	100

In this connection it is important to note that expanding sewerage networks is an essential requirement for the achievement of these goals, as indicated by the still very low rate of sewered population in these regions: in 1977, it was 39% in the Tokyo Bay region, 27% in the Ise Bay region, and 31% in the Seto Inland Sea region.

Along with reduction of COD loadings, inflow of nutrients must be controlled so as to reduce production of organic materials by phyto-planktons utilizing those nutrients. In 1979, for example, 25% (700,000 tons) of all fishery products in Japanese coastal waters was raised in the Seto Inland Sea. The number of "red tides" observed in the Sea, which was 48 in 1967, jumped from 79 in 1970 to 320 in 1975 and 218 in 1980. The red tides along with them brought heavy damages to fishery production in the Seto Inland Sea, and became an important issue to be urgently tackled with.

In the face of this event, measures were taken under the Provisional Law for Seto Inland Sea Environmental Preservation to reduce by 1976 the COD loadings from industrial sources by more than one half of 1972 levels. Thus, the average COD concentration in the whole Sea area in 1979 was brought to 1.3 ppm, while its transparency (Secchi depth) was 6.4 meters, Total N 0.26 ppm, Total P 0.026 ppm. The overall rate of achievement of water quality standards was 76% (49% for Category A, 80% for Category B, and 98% for Category C). Compared with national averages, the achievement rate of water bodies in Category A, which make up a major part of the Inland Sea, is very low.

Furthermore, daily inflow of phosphorous into the Seto Inland Sea reaches as high as 81 tons per day. (34 tons from domestic sources, 33 tons from industrial sources, and 14 tons from other sources).

In order to raise the water quality of the Seto Inland Sea, the Special Measures Law for Seto Inland Sea Environmental Preservation provides for measures to reduce the input of nutrients into the Sea area. On the basis of this Law, Director-General of the Environment Agency in July, 1979 directed the governors of relevant prefectures to draw up plans, including specific measures, to reduce or maintain the present level of P loadings into the Sea by 1984, depending on the prevailing state of pollution by the nutrient. Further studies will be conducted regarding possibilities for reduction of P concentrations in Tokyo and Ise Bays.

3. EUTROPHICATION OF LAKES AND RESERVOIRS

Of the 98 lakes and reservoirs for which water quality data are available, 54 show a pH of 8.5 or above, and a DO of more than 10 ppm. In these lakes, pH rises as CO₂ is taken up by the multiplying population of algae, and DO increases to the extent it becomes super-saturated. Many of the lakes and reservoirs have thus become enriched with nutrients. Lake Suwa, for example, had a nitrogen concentration of 0.26 ppm and phosphorous concentration of 0.02 ppm in 1931, whereas the corresponding figures rose to 1.59 ppm and 0.63 ppm respectively in 1979.

Table 6. COD Levels in The Major Lakes of Japan
(in 1979)

Name of Lake	Water Quality Category	Maximum Concentrations Observed	Daily Means at Standard Sampling Sites (75% Values)	75% V/W.Q.S. Values
Tega-numa	B (5 ppm)	78 ppm	34 ppm	6.8
Mikata-goko	B (5 ppm)	82 "	21 "	4.2
L. Kitagata	B (5 ppm)	11 "	10 "	2.0
S. Suwa	A (3 ppm)	42 "	7.2 "	2.4
Kasumiga-ura	A (3 ppm)	36 "	14 "	4.7
Hachiro-gata	A (3 ppm)	15 "	10 "	3.3
L. Biwa	AA (1 ppm)	7.6 "	4.4 "	4.4

It should also be noted that Japanese lakes, many of which are still in an oligotrophic state and have long been associated with one of the greatest transparencies in the world, are rapidly deteriorating in terms of their clarity/transparency. Secchi depth of Lake Shikotsu, for example, decreased from 25 meters in 1925 to 18.7 meters in 1978, while in Lake Towada it was

reduced from 20.5 meters in 1930 to a mere 9.8 meters in 1978. The phenomenon of lake eutrophication is not restricted to natural lakes, either. It now extends to many of the man-made reservoirs constructed for drinking water purposes. Of a total of 155 reservoirs surveyed in 1979, 55 is reported to have suffered cases of fungus-like odor and taste in the water. In other cases, filters were clogged by mass populations of algae and filtering efficiency lowered.

4. ENVIRONMENTAL PRESERVATION OF LAKES AND RESERVOIRS

Water tends to stagnate long in closed water bodies such as lakes and reservoirs making pollutants prone to accumulate. Therefore, lakes and reservoirs are apt to be polluted, in which it is very difficult to achieve environmental quality standards compared with rivers and the sea. Moreover, the progress of eutrophication often causes the excessive growth of aquatic life, such as algae (fresh-water red tide, water bloom, etc.), giving rise to serious obstacles to the water use, such as obstruction of filtering for water supply, offensive odor and taste in drinking water, and damage to fishery. Meanwhile, water polluting factors for lakes and reservoirs vary largely, depending on the natural and social conditions of drainage basins. Therefore, some of them cannot be fully coped with through the existing measures based on the Water Pollution Control Law. This being the case, it is essential to carry out new, special measures for the protection of the water quality of lakes and reservoirs comprehensively and systematically while making full use of the existing system for water pollution control. It is also believed imperative to preserve the abundant waterside environment of lakes and reservoirs.

From the said point of view, the Director-General of the Environment Agency asked the Central Council for Environmental Pollution Control on Oct. 15, 1980 to submit a Recommendation on a Desirable System for the Environmental Protection of Lakes and Reservoirs. On Jan. 27, 1981, the council submitted the recommendations based on a study by its Subcommittee on Water Quality.

Salient points of the recommendation are as follows:

(1) The Government should indicate a basic idea for the environmental protection of lakes and reservoirs, and guidelines for protective measures.

(2) The prefectural governors should map out environmental protection plans regarding lakes and reservoirs which are considered to require the comprehensive and systematic measures of protection. The national and local public bodies should carry out the plans steadily and vigorously.

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

(4) The environment of lakes and reservoirs represents a close combination of water quality and the natural environment of the neighborhood. Therefore, positive measures should be carried out to preserve the natural environment of the areas in the vicinity and at the same time, to ensure the cleanliness and beauty of the water surface and waterside, areas, and maintenance of open spaces.

(5) To protect the natural environment of lakes and reservoirs, various existing systems should be fully utilized. At the same time, attention should be paid to the functions of preserving water quality and serving as water source, which are seen in the natural environment surrounding lakes and reservoirs. In this context, study should be made, as necessary, for the purpose of establishing a new system of designating areas in which certain types of conduct would be controlled.

(6) The State should give maximum possible financial aid to local public bodies, while both the State and local public bodies should strive to provide workshop operators, etc. with assistance regarding financing and taxation.

(7) The Government should seek to establish a legal system for the environmental protection of lakes and reservoirs in line with this recommendation as soon as possible.

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Regional Sludge Management Program in the Tokyo Bay Basin

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1. INTRODUCTION

Tokyo Bay is a typical stagnant water body along with the Inland Sea of Seto and Ise Bay from the view point of water pollution countermeasures. (see Fig. 1). It is long north to south with a length of about 80 km. Its maximum width, east to west, is 30 km, the average width being 16 km. The area of the water surface of the bay is about 1,400 km² and the bay holds 54 km³ of water. Its northern part is mainly sedimentary and the water there is less than 40 m in depth but the southern part of the bay contains a large undersea valley of several hundred meters in depth. The narrow-part of the entrance to the bay is only 6 km wide and this strait prevents the exchange and movement of sea water, thus keeping a stagnant state in the northern part of the bay. The basin of Tokyo Bay extends in the NW-SE direction across the bay and has an area of about 7,500 km². The four prefectures (Saitama, Chiba, Tokyo and Kanagawa) that form the basin comprise 149 municipalities (86 cities, 45 towns and 18 villages). Thus, this is a densely-populated area. According to 1975 statistics, the basin, with its 7,511 km², represented only 2% of the nation's total area of 377,535 km² but had 22.1 million people (19.7% of the national population of 112 million). Its industrial output amounted to ¥28 trillion (or about US\$ 127 billion, at 1US\$ = ¥ 220), or 22% of the national total of ¥127.4 trillion (or about US\$ 579 billion). This means that pollution sources are extremely concentrated there. Under these circumstances, to meet environmental water quality standards for Tokyo Bay sewage treatment for the entire bay basin requires AWT, and sludge treatment and disposal must be suitably planned. The Ministry of Construction has, since 1979, been studying creation of the most economical regional sewage management program for the Tokyo Bay basin. In fact, a presentation on this regional sewage management program was made in the Seventh U.S.-Japanese Conference on Sewage Treatment Technology. This paper mainly discusses a sludge management program for the Tokyo Bay Basin.

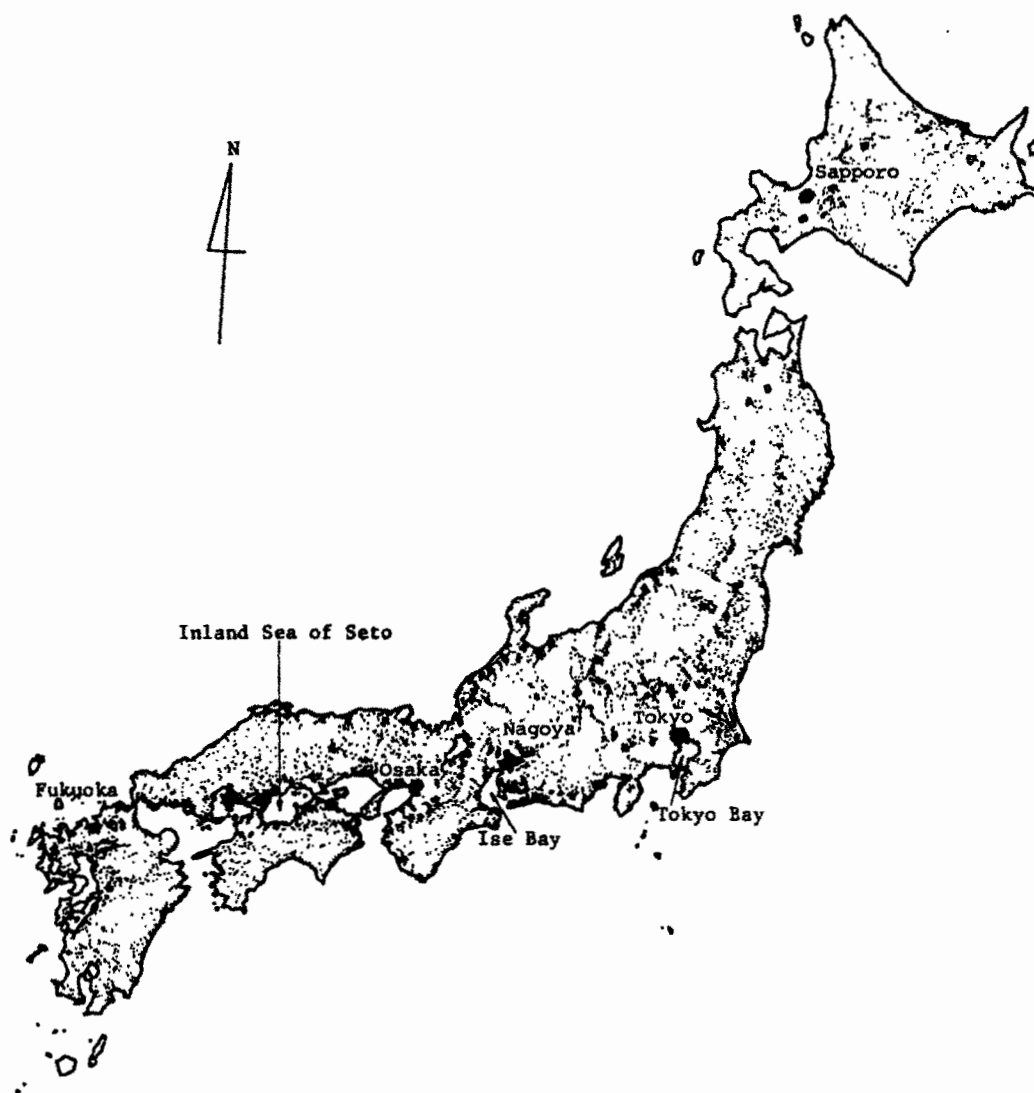


Fig. 1 Location of Tokyo Bay

2. ENVIRONMENTAL WATER QUALITY STANDARDS FOR TOKYO BAY AND ITS PRESENT STATE OF WATER QUALITY

Environmental water quality standards have been set in accordance with Article 9 of the Basic Law for Environmental Pollution Control and the environmental water quality standards for coastal waters are categories A, B and C (see Table 1) by the Environment Agency Notification. The powers to designate specific waters and their categories rest with the National Government as far as important waters are concerned. Similar powers for other waters are delegated to prefectural governors. The national government designation for Tokyo Bay is shown in Fig. 2. This bay is divided into 18 areas (two Category A areas, eight Category B areas and eight Category C areas) with 48 datum points where water quality is measured at least once a month. Fig. 3 chronologically shows the extent to which the environmental water quality standards for typical stagnant water bodies have been attained. In Tokyo Bay, this rate considerably improved from 44% in 1974 to 61% in 1979. Table 2 shows the attainment rates in the different areas of Tokyo Bay. In 1975, environmental water quality standards were met in all Category C areas but not in the areas of Categories A and B. Whereas, in 1979 they were met also in three of the eight Category B areas. In the Category A areas, the standards were not met but actually the COD_{MN} values improved.

In Tokyo Bay, which is in a stagnant water body, eutrophication occurs due to the poor exchange of water and rich nutrients such as phosphorus and nitrogen which are present. Thus, government guidance on phosphorus control for this bay is being proposed. Yet, the conduct of phosphorus control will be a big problem, because phosphorus is not included in the water quality items of the current environmental water quality standards.

Table 1. Environmental Water Quality Standard Values for Coastal Waters

Item Category	Standard values					
	Purpose of utilization	(pH)	Chemical oxygen demand (COD _{MN})	Dissolved oxygen (DO)	Number of coliform groups	N-hexane extracts
A	Fishery class 1, bathing and uses listed in B-C.	7.8 - 8.3	2 ppm or less	7.5 ppm or more	1,000 MPN/100 ml or less	Not detectable
B	Fishery class 2, industrial water and uses listed in C.	7.8 - 8.3	3 ppm or less	5 ppm or more		Not detectable
C	Conservation of Environment	7.0 - 8.3	8 ppm or less	2 ppm or more		

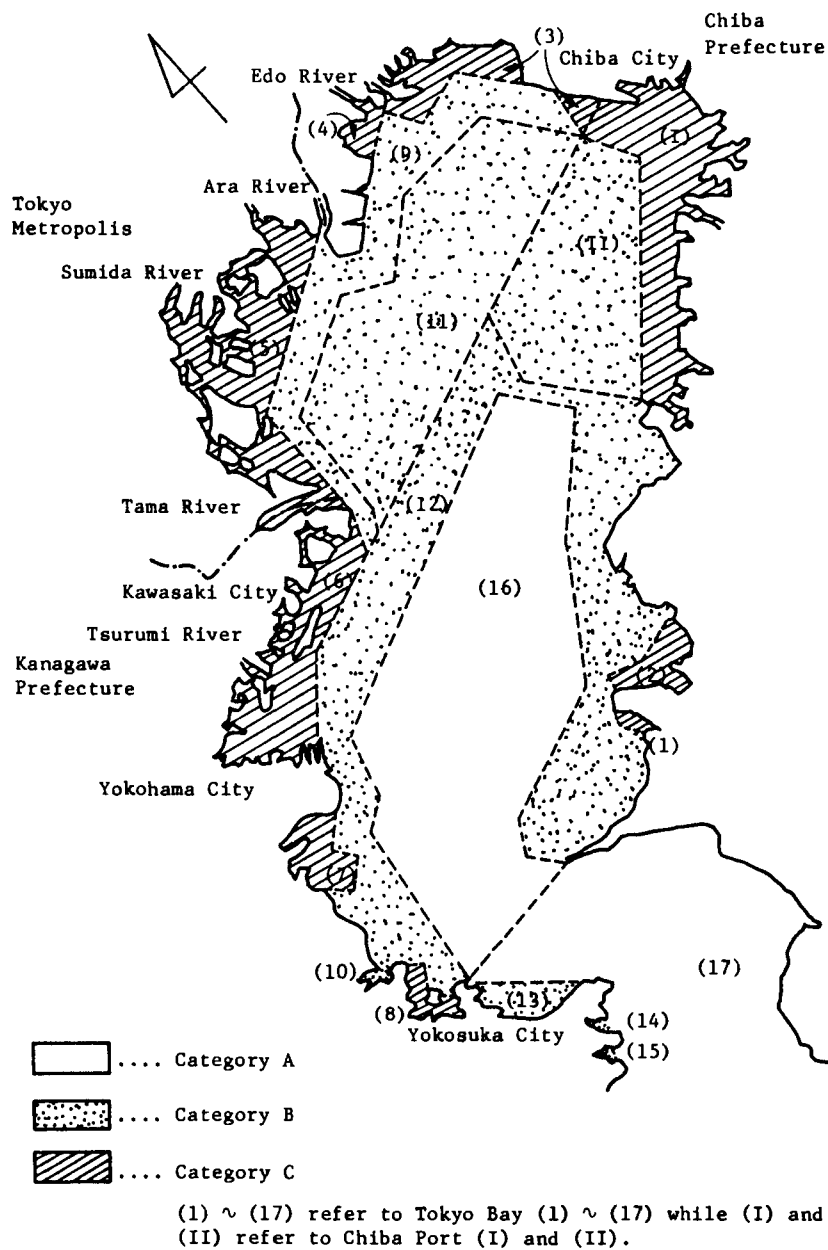
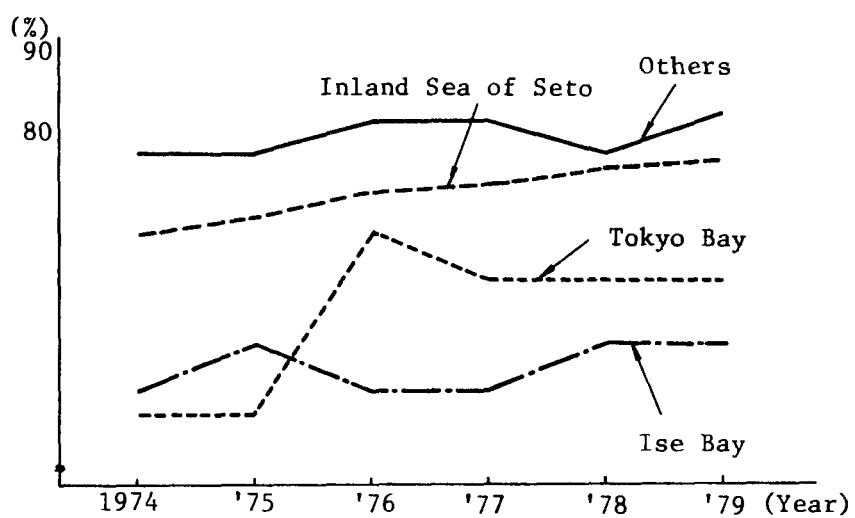


Fig. 2 Environmental Water Quality Datum Points, Tokyo Bay



(Remarks)

1. Source: Environment Agency
2. Rates of attainment were determined as follows:

$$\frac{\text{Number of areas meeting environmental water quality standard}}{\text{Number of areas covered by environmental water quality standards}} \times 100 (\%)$$

Fig. 3 Chronological Change in Rates of Attainment of Environmental Water Quality Standards

Table 2. Table on State of Attainment of Environmental
Water Quality Standards in Tokyo Bay 1974 - 1979
(In terms of COD_{MN})

Name of Water Body	Category	1974	1975	1976	1977	1978	1979
Chiba Port (I)	C	O	O	O	O	O	O
" (II)	B	X	X	X	X	X	X
Tokyo Bay 1	C	O	O	O	O	O	O
" 2	C	O	O	O	O	O	O
" 4	C	O	O	O	O	O	O
" 5	C	O	O	O	O	O	O
" 6	C	X	O	O	O	O	O
" 7	C	O	O	O	O	O	O
" 8	C	O	O	O	O	O	O
" 9	B	X	X	X	X	X	X
" 10	B	X	X	O	X	X	X
" 11	B	X	X	X	X	X	X
" 12	B	X	X	X	X	X	X
" 13	B	X	X	O	O	O	O
" 14	B	O	X	O	O	O	O
" 15	B	X	X	O	O	O	O
" 16	A	X	X	X	X	X	X
" 17	A	X	X	X	X	X	X
Rate of attainment of environmental water quality standard		44%	44%	67%	61%	61%	61%

O indicates environmental water quality standards that were met and
X indicates those that were not met.

3. CURRENT SEWAGE TREATMENT IN THE TOKYO BAY BASIN

The Ministry of Construction has conducted a need survey on local governments concerned to determine the current state of sewage treatment and its prospects. According to the results of this need survey, the current status of sewage treatment in the Tokyo Bay basin are shown in Table 3, Fig. 4 and Table 4.

As can be seen from Table 4, the 63 sewage treatment plants located in the Tokyo Bay basin can be divided by treatment processes into 33 plants using conventional activated sludge, 21 plants using stepped aeration, three plants using extended aeration, two plants using pure oxygen activated sludge, two plants using high-rate aeration and two plants using the high-rate trickling filter process most of these make secondary treatment. Some sewage treatment plants in Tokyo use the AWT process, employing coagulation and sand filtration.

Effluent standards of sewage treatment plants are set uniformly for the entire nation by the Water Pollution Control Law. But for waters where the environmental water quality standards are difficult to attain by employing the national uniform standards, prefectural governors may set more stringent effluent standards than the national uniform standards. In the case of Tokyo Bay, the prefectural governors concerned have fixed more stringent standards which require BOD and SS to be 20 ~ 50 mg/l and 30 ~ 70 mg/l, respectively. Effluents from three sewage treatment plants exceeds the effluent standard in terms of BOD value. This is probably because one of these plants is overloaded even during dry weather flow by 50%, while the other two are overloaded through combined treatment with night soil. As for SS, the standards in terms of SS value are exceeded at one sewage treatment plant.

The rate of industrial wastewater in the volume of sewage discharge into sewage treatment plants differs from plant to plant, ranging from 0 to 50% of the treated sewage.

Under the Sewerage Law, sewers must receive both domestic and industrial wastewater. Thus, it is mandatory for public sewers to receive sanitary sewage from household and also industrial wastewater from factories in the corresponding drainage area. However, used water not requiring treatment and other water unlikely to be improved by

sewage treatment such as cooling water and other non-polluted sewage can be directly discharged into public water bodies even if these are located in the drainage area.

Further, the Sewerage Law controls industrial effluents by providing pretreatment plants for more or less the same level of standards for toxic substances contained in industrial effluents as are set by the Water Pollution Control Law (see Table 5). For toxic substances contained in sludge, there are quality standards set by a Prime Minister's Office ordinance in accordance with the Waste Disposal and Public Cleansing Law (see Table 6).

To guarantee the appropriate disposal of effluents, the Sewerage Law provides not only for direct punishment, improvement order and supervisory action against the violation of effluent standards listed in Table 5 but also for reporting, inspection, etc. as preliminary checks. The sewage works managers conduct monitoring in accordance with these provisions.

As stated already in the section dealing with the attainment of environmental water quality standards, the attainment of environmental water quality standards in Tokyo Bay is unsatisfactory. And in order to improve water quality in large-scale stagnant water bodies such as Tokyo Bay, an effluent mass control system became necessary; thus, the Water Pollution Control Law was amended and the amended law was enacted, effective in June 12, 1979.

The effluent mass control system is designed to steadily improve water quality by uniformly and effectively reducing the total pollution load for heavily polluted large-scale stagnant water bodies. Tokyo Bay, Ise Bay and the Inland Sea of Seto were designated as three large scale stagnant water bodies. With 1984 as the target year, reduction targets by sources and by prefectures in terms of pollution loads (COD_{MN}) were set for waste water discharged into public waters in these designated areas. Table 7 shows COD_{MN} loads to be reduced in Tokyo Bay by the effluent mass control system.

Table 3. Current Status of Sewage Treatment in the Tokyo Bay Basin

Name of prefecture	Number of municipalities	Prefectural population (in 1,000 persons)	Area sewerred (ha)	Pupulation sewerred (in 1,000 persons)	Rate (%)	Amount of sewage treated (in 1,000 m ³ /day)	Sludge production (in terms of de-watered cakes moisture content 75%: 1,000 t/year)	Number of treatment plants
Saitama	38 cities, 30 towns and 16 villages (84)	5,060	8,312	865	17	401	100	14
Chiba	17 cities, 10 towns and 1 village (28)	3,519	6,735	622	18	257	59	16
Tokyo	*27 cities, 5 towns and 1 village (33)	11,357	48,826	7,090	62	4,501	1,084	20
Kanagawa	4 cities (4)	4,224	11,707	1,477	35	933	141	13
Total	86 cities, 45 towns and 18 villages (149)	24,160	75,580	10,054	42	6,092	1,384	63

* The ward section of Tokyo was counted as one city.

75% was uniformly used for conversion as the moisture content for all dehydrated cakes.

Table 4. Current Status of Sewage Treatment Plant Operation in the Tokyo Bay Basin

(1) Saitama Prefecture

Name of local municipality	Name of sewage treatment plant	Process of treatment	Current capacity (m ³ /day)	Dry weather flow 1979 (m ³ /day)	Rate of industrial effluent (%)	Quality of crude sewage		Quality of effluent		Effluent standard (daily average value)	Remarks
						BOD (mg/l)	SS (mg/l)	BOD (mg/l)	SS (mg/l)		
Saitama Prefecture	Arakawa	Conventional activated sludge	211,300	187,100	15.3	103	115	11	9	BOD20 mg/l SS70 mg/l	Refer Fig.4 14
"	Furutonegawa	"	50,400	19,900	22.0	119	184	5	4	"	5
Kawagoe City	Takinoshita	Stepped aeration	45,000	39,700	10.8	158	104	16	13	"	11
"	Kasumigaseki No.1	"	8,900	5,100	0	138	116	9	6	"	8
"	Kasumigaseki No. 2	Extended aeration	1,100	900	0	136	127	12	9	"	9
Kawaguchi City	Ryoke	High-rate aeration	37,500	33,500	14.9	121	131	17	16	"	18
Omiya City	Omiya Nambu	"	20,700	24,000	3.4	212	84	40	26	"	15
Chichibu City	Chuo	High-rate trickling filter	17,400	18,400	11.5	90	49	48	28	"	1
Gyoda City	Gyoda	"	10,900	7,400	17.2	89	96	17	20	"	3
Tokorozawa City	Tokorozawa	Stepped aeration	72,800	34,200	5.3	206	157	12	8	"	13
Hanno City	Hanno	Conventional activated sludge	8,000	5,700	29.3	183	173	10	24	"	12
Sayama City	Sayamadai	"	5,500	4,800	0	248	176	13	8	"	10
Higashimatsuyama City	Ichinogawa	"	11,500	8,400	49.3	157	124	6	3	"	7
Sakado Tsurugashima	Kitasakado	"	11,900	12,000	0	151	153	15	12	"	6
Total			512,900	401,100							

(2) Chiba Prefecture

Name of local municipality	Name of sewage treatment plant	Process of treatment	Current capacity (m ³ /day)	Dry weather flow 1979 (m ³ /day)	Rate of industrial effluent (%)	Quality of crude sewage		Quality of effluent		Effluent standard (daily average value)	Remarks
						BOD (mg/l)	SS (mg/l)	BOD (mg/l)	SS (mg/l)		
Chiba Prefecture	Hanamigawa	Stepped aeration	102,500	70,400	1.2	161	113	6	3	BOD20 mg/l COD20 mg/l SS70 mg/l	30
Chiba City	Chuo	Conventional activated sludge	130,900	102,800	11.3	126	84	13	11	"	31
"	Ogura	"	2,100	2,500	0	203	133	15	9	"	32
"	Sakatsuki	"	10,000	7,700	0	181	107	12	5	"	33
"	Omiya Hokubu	Extended aeration	260	200	0	200	117	11	10	"	34
"	Omiya	Conventional activated sludge	1,300	1,300	0	214	142	14	8	"	35
Ichikawa City	Sugano	Stepped aeration	14,500	13,000	0	154	150	9	13	"	26
Funabashi City	Nishiura	Conventional activated sludge	16,700	11,000	0	122	155	11	4	"	28
Matsudo City	Kanegasaku	Stepped aeration	12,700	11,000	0	181	135	9	12	"	24
"	Kitakogane	Conventional activated sludge	9,600	7,600	0	226	209	9	14	"	22
"	Shinmatsudo	"	5,000	4,600	0	135	110	8	6	"	23
Kashiwa City	Toyoni	"	5,500	4,800	0	190	130	3	8	"	21
Yachiyo City	Katsutadai	Stepped aeration	5,400	5,200	0	214	140	14	18	"	27
Narashino City	Tsudanuma No. 2	Conventional activated sludge	4,500	3,900	0	219	157	7	8	"	29
Ichihara City	Kikuma	"	19,000	9,200	0	116	143	3	4	"	36
"	Aobadai	Pure Oxygen activated sludge	5,000	1,600	0	200	171	7	11	"	37
Total			344,960	256,800							

(3) Tokyo Metropolis

Name of local municipality	Name of sewage treatment plant	Process of treatment	Current capacity (m ³ /day)	Dry weather flow 1979 (m ³ /day)	Rate of industrial effluent (%)	Quality of crude sewage		Quality of effluent		Effluent standard (daily average value)	Remarks
						BOD (mg/L)	SS (mg/L)	BOD (mg/L)	SS (mg/L)		
Tokyo Metropolis	Kitatama No. 1	Stepped aeration	136,000	70,600	19.7	142	119	4	3	BOD20 mg/L COD35 mg/L SS30 mg/L	58
"	Tamaqawa Joryu	"	75,000	15,300	8.4	101	72	5	4	"	42
"	Ninamitama	Conventional activated sludge	52,800	16,100	3.7	243	182	9	4	"	56 Includes AMT (coagulation and sand filtration).
Tokyo Metropolis (ward section)	Shibaura	Stepped aeration	1,130,000	861,000	9.1	165	137	11	8	"	50
"	Morigasaki	"	1,410,000	993,000	3.9	129	132	5	2	"	51 Includes sand filtration facility.
"	Shingashi	"	705,000	290,000	23.5	121	101	10	6	"	44
"	Ochiai	"	450,000	580,000	2.4	162	172	12	12	"	45
"	Odai	"	358,000	290,000	20.4	113	97	8	6	"	46
"	Kosuge	"	150,000	73,000	5.5	107	63	5	4	"	49
"	Mikawashima	"	700,000	647,000	6.0	148	122	15	8	"	47
"	Sunamachi	"	680,000	494,000	12.3	189	139	11	8	"	48
Machida City	Tsurukawa	Conventional activated sludge	6,300	5,000	0	190	240	8	9	"	61
"	Machida	"	20,700	11,300	0	150	145	8	6	"	63
Mitaka City	Mitaka Tobu	Stepped aeration	21,000	32,500	9.4	186	210	28	38	"	59
Hachioji City	Kitano	"	82,000	56,000	0	200	220	14	26	"	53
"	Mejirodai	"	2,600	4,100	0	200	200	14	26	"	52
Tachikawa City	Nishikicho	"	77,500	42,000	4.4	253	127	17	16	"	57
Hino City	Tamadaira	Conventional activated sludge	5,800	4,500	0	213	252	18	11	"	54
Hagashikurume	Shitaya	"	15,100	13,000	0	175	159	13	5	"	51
Tama City	Sakuragaoka	Extended aeration	2,600	2,400	0	111	156	5	7	"	55
Total			6,080,400	4,500,800							

(4) Kanagawa Prefecture

Name of local municipality	Name of sewage treatment plant	Process of treatment	Current capacity (m ³ /day)	Dry weather flow 1979 (m ³ /day)	Rate of industrial effluent (%)	Quality of crude sewage		Quality of effluent		Effluent standard (daily average value)	Remarks
						BOD (mg/l)	SS (mg/l)	BOD (mg/l)	SS (mg/l)		
Kawasaki City	Iriezaki	Conventional activated sludge	300,000	211,700	48.7	108	90	10	8	BOD20 mg/l COD50 mg/l SS70 mg/l	70
"	Kase	"	122,400	63,000	31.1	60	57	8	5	BOD20 mg/l COD20 mg/l SS40 mg/l	65
"	Ikuta	Pure oxygen activated sludge	3,300	2,800	0	229	135	12	15	BOD20 mg/l COD50 mg/l SS70 mg/l	62
Yokohama City	Hokubu No. 1	Conventional activated sludge	195,600	158,000	10.1	140	130	10	4	"	69
"	Kohoku	"	69,700	28,000	3.7	78	74	9	4	BOD20 mg/l COD20 mg/l SS40 mg/l	67
"	Midori	"	73,800	7,300	0	360	450	6	3	"	66
"	Kanagawa	"	143,000	74,100	3.4	95	120	6	4	"	68
"	Chubu	"	64,800	68,700	3.9	140	100	11	6	BOD20 mg/l COD50 mg/l SS70 mg/l	73
"	Nambu	"	234,000	222,000	4.1	160	130	13	9	"	72
"	Kanazawa	"	56,000	5,800	20.5	160	160	4	4	BOD20 mg/l COD20 mg/l SS40 mg/l	74
"	Totsuka No. 2	"	66,000	38,600	15.8	220	190	11	5	"	71
Yokosuka City	Uemachi	"	35,200	19,400	2.2	136	113	14	17	BOD20 mg/l COD50 mg/l SS70 mg/l	75
"	Shitamachi	"	74,800	33,600	5.5	107	115	14	17	"	76
Total			1,438,600	933,000							

Table 5. Pretreatment Standards (Sewerage Law)

Toxic substances	Permissible limits	Remarks
Cadmium and its compounds	0.1 mg/l or less	} More stringent values may be imposed by local ordinances.
Cyanide compounds	1 mg/l or less	
Organic phosphorus compounds	1 mg/l or less	
Lead and its compounds	1 mg/l or less	
Hexavalent chromium compounds	0.5 mg/l or less	
Arsenic and its compounds	0.5 mg/l or less	
Total mercury	0.005 mg/l or less	
Alkyl mercury compounds	Not detectable	
P C B	0.003 mg/l or less	
Phenols	5 mg/l or less	
Copper	3 mg/l or less	
Zinc	5 mg/l or less	
Disolved iron	10 mg/l or less	
Disolved manganese	10 mg/l or less	
Chromium	2 mg/l or less	
Fluorine	15 mg/l or less	
P H	5 ~ 9	} Values out of this range are set by local ordinances.
B O D	600 mg/l or less	
S S	600 mg/l or less	
Normal hexane extracts		
Mineral oil	5 mg/l or less	
Animal and vegetable fats	30 mg/l or less	

Table 6. Prime Minister's Office Ordinance under
Waste Disposal and Public Cleansing Law

Toxic substances	Permissible limits	Remarks
Arsenic	1.5 mg/l	Elution test
Cadmium	0.3 mg/l	
Mercury	0.005 mg/l	
Lead	3 mg/l	
Organic phosphorus	1 mg/l	
Hexavalent chromium	1.5 mg/l	
Cyanide	1 mg/l	
Alkyl mercury	Not detectable	
P C B	0.003 mg/l	

Table 7 Total COD_{MN} Load Reduction Targets for Tokyo Bay by the Effluent Mass Control System Amount Regulation and Estimated Allowable COD_{MN} Loads Necessary to Meet Environmental Water Quality Standards (t/day)

Name of prefecture	1979	COD _{MN} reduction target for Tokyo Bay 1984	Estimated allowable COD _{MN} load necessary to meet environ- mental water quality standards
Saitama	155	141	46
Chiba	117	112	29
Tokyo	307	280	83
Kanagawa	143	127	40
Total for Tokyo Bay basin	722	660	198

Note: The above estimated algal production amounts allowable loads are based on the assumption that are 1/4 of their present level.

The recently determined total load reduction values are at levels to which pollution loads can be practically reduced and seem unlikely to allow environmental water quality standards for the entire Tokyo Bay to be attained and maintained. To attain the environmental water quality standards, it will be necessary to raise the level of sewage treatment further and, at the same time, carry out more stringent standards in the future.

4. PRESENT AND FUTURE OF SLUDGE MANAGEMENT IN THE TOKYO BAY BASIN

4.1 Sludge Treatment

At present, sludge resulting from sewage treatment is generally disposed of by the process shown in Fig. 5. As shown in Table 8, 23 out of the 63 sewage treatment plants in the Tokyo Bay basin practise sludge digestion and 20 practise sludge incineration; together these plants incinerated 700,000 tons (51%) out of the 1,380,000 tons (in terms of sludge cakes with a moisture content of 75%) which were produced in 1979.

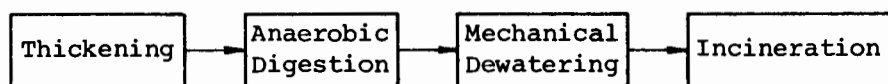


Fig. 5 Process of Sludge Treatment

Table 8. Number of Treatment Plants Provided with Digestion and Incineration and their Sludge Production in the Tokyo Bay Basin

Name of prefecture	Division	Digestion			Incineration		
		Yes	No	Total	Yes	No	Total
Saitama	Number of treatment plants	8	6	14	5	9	14
	Sludge production	26	74	100	81	19	100
Chiba	Number of treatment plants	6	10	16	2	14	16
	Sludge production	22	37	59	16	43	59
Tokyo	Number of treatment plants	6	14	20	9	11	20
	Sludge production	598	486	1,084	522	562	1,084
Kanagawa	Number of treatment plants	3	10	13	4	9	13
	Sludge production	62	79	141	84	57	141
Total	Number of treatment plants	23	40	63	20	43	63
	Sludge production	708	676	1,384	703	681	1,384

(Note) Sludge production is in terms of dewatered cakes (moisture content: 75%) in 1,000 ton/year.

4.2 Sludge Disposal

As shown in Table 9, most of the sludge produced at the sewage treatment plants located in the Tokyo Bay basin is disposed of by land-fill and reclamation from the sea.; 533,000 m³/year, or 71% of 751,000 m³/year which is the total amount of sludge disposed of by land-fill and land reclamation was from downtown Tokyo and used for reclamation from the seashore at the Central Breakwater.

Sludge effectively used as a resource in its ultimate disposal by recycling for greens and farmlands is, in many cases, land application for parks and farmlands as compost. In most land reclamation, dewatered cakes and incineration ash produced at sewage treatment plants are used as they are.

In the case of Tokyo, where reclamation from the sea constitutes most of reclamation, dewatered sludge cakes and incineration ash produced at its sewage treatment plants are used for reclamation from the sea. In so doing, alumina cement is added at a rate of about 10% to the dewatered cakes and incineration ash for the convenience of bulldozer operation and is used for reclamation work and with adjustments so that the compressive strength of soil after reclamation will be more than 0.5 kg/cm². Photo 1 shows a general view of the Chubo (Central Breakwater) Mixing Plant of Tokyo. Photo 2 shows sludge being shipped after cement mixing and curing.

Table 9 Methods of Sludge Disposal and Amounts
Disposed in the Tokyo Bay Basin (1979)

Method of sludge disposal	Amount disposed (1,000 ton/year)	Remarks
Land reclamation	128	The moisture content varies with disposal method.
Reclamation from the sea	623	
Land application to greens and farmlands	10	
Others	21	
Total	782	



Photo 1 General View of Chubo (Central Breakwater)
Mixing Plant, Tokyo



Photo 2 Shipment of Cement-Hardened and Cured Sludge

4.3 Heavy Metal Content in Sludge

Table 10 shows the heavy metal content in thickened sludge and digested sludge for major sewage treatment plants in the Tokyo Bay basin. In the standard values for special fertilizers in accordance with a Ministry of Agriculture and Forestry Notification. As, Cd and mercury must be less than 50 mg/kg, 5 mg/kg and 2 mg/kg, respectively. At sewage treatment plants where the rate of industrial effluents in the crude sewage is high, the permissible Cd limit is sometimes exceeded and in some sewage treatment plants sludge cannot be used as a special fertilizer though this occurs only in apparently rare cases. The heavy metal content in sludge in Japan, as far as can be seen from the scant data available, is smaller than in Europe and America. However, farmers are deeply concerned over the problem of heavy metals and countermeasures against heavy metals handling is an important step toward the solution of the sludge utilization problem.

4.4 Effective Utilization of Sludge as a Resource

As stated above, the amount of sludge used for land application to greens and farmlands in the Tokyo Bay basin is comparatively small. This is probably because some farmers are very careful of the effect of heavy metals in sewage sludge on greens and farmlands, because the effects of heavy metals on soil are not certain and also compost facilities are still at an experimental stage. Moreover, sewage treatment plants in the Tokyo Bay basin are far from areas where sludge can be applied as compost for greens and farmlands. Transportation costs are high.

Compost facilities have recently begun to be installed at an increasing number of sewage treatment plants. At the Minamitama Sewage Treatment Plant in Tokyo which treats sewage from a residential area, a fully-automatic compost facility including four horizontal fermenting tanks with a daily treating capacity of 10 m³ was installed in March 1980. This sewage treatment plant produces a good quality of compost with a moisture content of less than 40% by adding returned compost by way of moisture adjustment to sludge having been dewatered to a moisture content of 70% by a filter press and causing aerobic fermentation to this mixture. Characteristically, this plant does not use any other additional

Table 10. Heavy Metal Content in Sludge (in Dry Sludge mg/kg) for Major Treatment Works

Sample Heavy metals	A-1	B-1	C-1	D-1	E-2	F-1	G-1	G-2	H-1	I-1	J-1
Fe	25,020	5,810	27,033	64,195	11,900	26,251	10,000	Un- measured	Un- measured	Un- measured	Un- measured
Cu	639	157	607	260	468	717	210	200	507	812	149
Zn	1,676	1,100	1,280	1,335	3,144	2,304	1,000	1,100	2,935	1,800	1,065
T-Cr	158	28	168	49	92	380	37	43	139	148	20
Hg	1.2	1.5	1.4	1.4	1.9	1.9	1.0	1.4	3.2	1.8	1.5
Cd	1.6	ND	5.0	3.0	3.8	21.7	2.1	3.0	4.0	11.1	1.4
Pb	67	90	70	93	167	98	80	64	168	212	55
As	5.4	1.5	4.2	15.4	5.6	4.9	Un- measured	5.0	8.0	6.5	4.2

(Note) A, B, C ... for samples indicate different treatment plants and 1 and 2 are crude and digested sludge, respectively. Treatment plants A, F and I treat relatively large amounts of industrial effluents. B, C, D, E, G, H, and J treat mainly domestic sewage.

material or drying equipment for composting. Compost produced is sold for ¥250 (US\$ 1.13) a package (20 kg) through the Tokyo Federation of Agricultural Cooperative Associations. Photo 3 shows the compost facility of the Minamitama Sewage Treatment Plant.

Also, the sludge utilization as construction materials (e.g., concrete aggregate, roadbed material, brick, etc.) is being actively studied, though this application has not yet become practical.

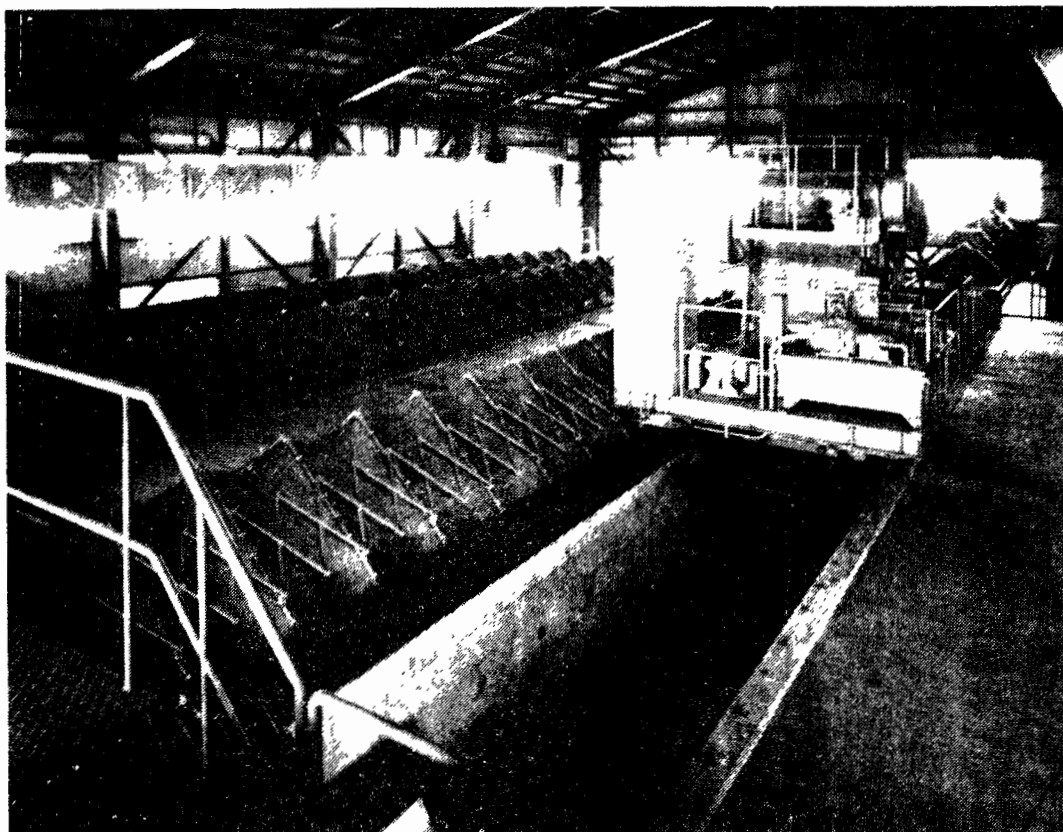


Photo 3 Compost Facility at Minamitama Treatment Plant, Tokyo

4.5 Forecasting of Sewage Sludge Production

The production of sludge is increasing with the growing volume of sewage to be treated. Table 11 shows the results of need surveys on sludge production by years and by prefectures.

It is predicted that the rate of increase of sludge production will be higher than the rate of increase of water consumption as the result of sludge production increasing due to employment of the AWT process.

Table 11 Sludge Production

(Unit: 1,000 t/year)

Name of prefecture \ Year		1979	1985	1990	1995	2000
In terms of amount of disposal	Saitama	34	93	309	719	1,384
	Chiba	46	73	117	180	269
	Tokyo	631	1,020	971	1,462	1,596
	Kanagawa	72	68	135	210	290
	Total for the Tokyo Bay basin	783	1,254	1,532	2,571	3,539
In terms of dewatered cakes	Saitama	100	321	602	1,047	1,505
	Chiba	59	140	316	552	757
	Tokyo	1,084	2,060	2,627	4,040	4,550
	Kanagawa	141	271	427	572	708
	Total for the Tokyo Bay basin	1,384	2,792	3,972	6,211	7,520

(Note) In terms of disposal amount, the moisture content varies by disposal methods. In terms of dewatered cakes, the moisture content is 75%.

4.6 Available Life of Current Sludge Disposal Sites for Land-fill and Land Reclamation

In view of the present state of sludge management, the amount of sludge that can be disposed of by current disposal sites is about 7,400,000 m³ -- mostly by land-fill, land reclamation and reclamation from the sea (see Table 12). The available life of current sludge disposal sites for land-fill and reclamation from the sea is less five years in many cases. This accounts for 69% of the total number of local municipalities and for 96% of the amount of sludge that cannot be disposed of in five years.

The present rate of effective use of sludge in the Metropolitan Tokyo area from viewpoint of sludge disposal is only about 1% and must be increased by future efforts.

Table 13 shows further details of periods relating to the remaining five years. In 15 local municipalities, the period is less than two years. Thus, it is extremely necessary to obtain more exact sludge disposal sites as soon as possible.

Table 12 Available Life of Current Sludge Disposal Sites for Landfill and Reclamation from the Sea and The Possible Amount of Sludge in The Tokyo Bay Basin

Available life Item	less than 5 years	5~10	10~15	15~20	More than 20 years	Unknown	Total
Number of local municipalities	22	5	1	0	0	4	32
Amount of disposal (1,000 m ³)	7,040	300	20	0	0	-	7,360

Table 13 Available Life of Current Sludge Disposal Sites where Disposal is Possible for Less than 5 Years

Available life Item	Less than 1 year	1 ~ 2	2 ~ 3	3 ~ 4	4 ~ 5	Total
Number of local municipalities	3	12	2	1	4	22

4.7 Problems Raised by Local Municipalities

At present, sludge is disposed of mostly by land-fill, land reclamation and reclamation from the sea. Local municipalities, finding it difficult to obtain disposal sites in their own jurisdictions, now often entrust sludge disposal to private enterprisers, in which case it is mostly disposed of within or outside of their own jurisdictions. Further, as can be seen from Table 14, about 45% of all sewage treatment plants in the Tokyo Bay basin have their sludge disposed of outside their own jurisdictions. Though the amount of sludge disposed represents as much as 91% of the total volume of sludge disposed of by land-fill and land reclamation from the seashore. It is mostly disposed of by reclamation. More than a half of the amount of sludge disposed of by land reclamation is disposed of outside its own jurisdiction.

The distance from each sewage treatment plant to its disposal site is indicated in Fig. 6. With nine of the 49 treatment plants whose distance to the disposal site is known, sludge must be transported for more than 100 km (the maximum being about 250 km) for disposal. This tendency is expected to increase in the future.

One factor making sludge disposal difficult is the problem of the content of such toxic substances as heavy metals. So, to ensure exact disposal, it is necessary to install efficient pretreatment facilities for industrial effluents and strengthen the monitoring system.

Table 14 Location of Disposal Sites and Disposal Amounts
(1979)

Disposal Item		Within own jurisdiction	Outside own jurisdiction or boundary	Total	Remarks
Number of treatment plants	Landfill and land reclamation	19	22	41	Amounts used for agriculture, and ocean disposal are excluded. The moisture content of sludge varies by disposal methods.
	Reclamation from the sea	8	0	8	
	Total	27	22	49	
Disposal amount (1,000 m ³ /year)	Landfill and land reclamation	63	65	128	
	Reclamation from the sea	623	0	623	
	Total	686	65	751	

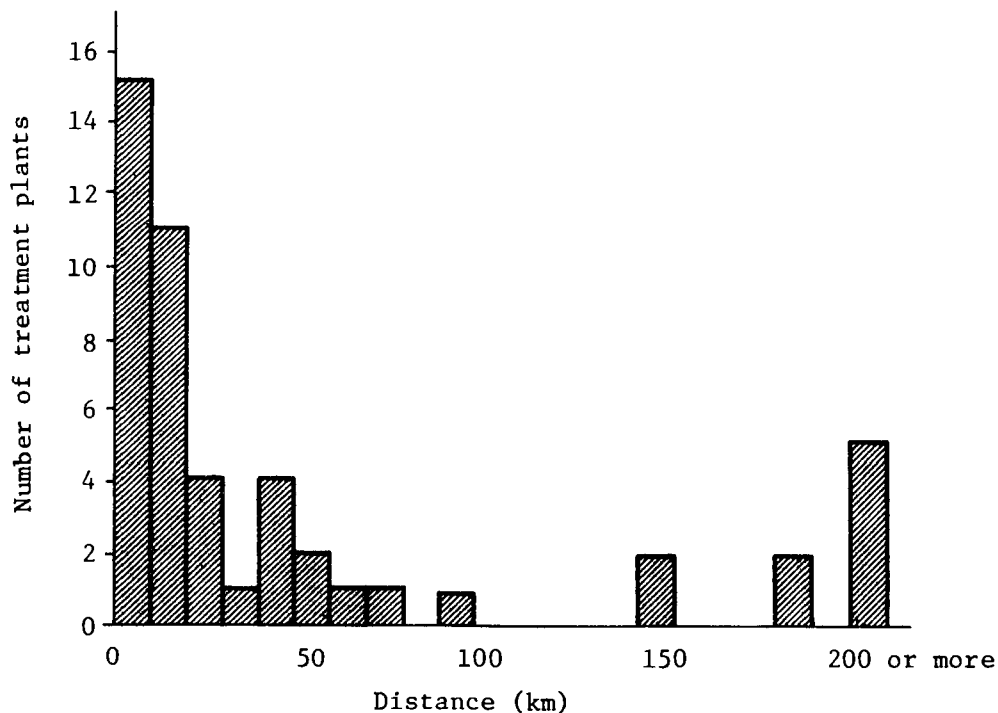


Fig. 6 Distance from Sewage Treatment Plant to Sludge Disposal Sites in the Tokyo Bay Basin

5. NECESSITY OF THE REGIONAL SLUDGE MANAGEMENT PROGRAM

5.1 Increase of Sludge Volume and Difficulty of Conducting Land-fill and Land Reclamation

As stated above, the available life of current sludge disposal sites for land-fill in the Tokyo Bay basin is less than five years in 22 (67%) of the 32 local municipalities, in the Tokyo Bay basin (including 15 local municipalities with less than two years) and sludge is now being disposed of mostly by reclamation from the seashore. As for land reclamation, sludge from some sewage treatment plants has to be transported to disposal sites more than 100 km away for disposal. Thus, securing sludge disposal sites is now extremely difficult.

Particularly, most land within 50 km of downtown Tokyo is used for commerce, residence and industry at such a density that it is considered impossible to locate the sites for sludge disposal within this 50-km sphere in the future (see Fig. 7). It is, therefore, basically necessary to reduce sludge production and promoting sludge utilization as a resource for agricultural and other purposes. If there is no choice but to dispose of sludge by land reclamation, sludge within the 50-km sphere cannot but be disposed of by, say, reclamation from the sea rather than by land-fill within this sphere because its disposal by land-fill and land reclamation within its own jurisdiction is simply impossible.

For areas beyond the 50-km sphere, study must be made to plan the joint disposal of sludge from two or three sewage treatment plants.

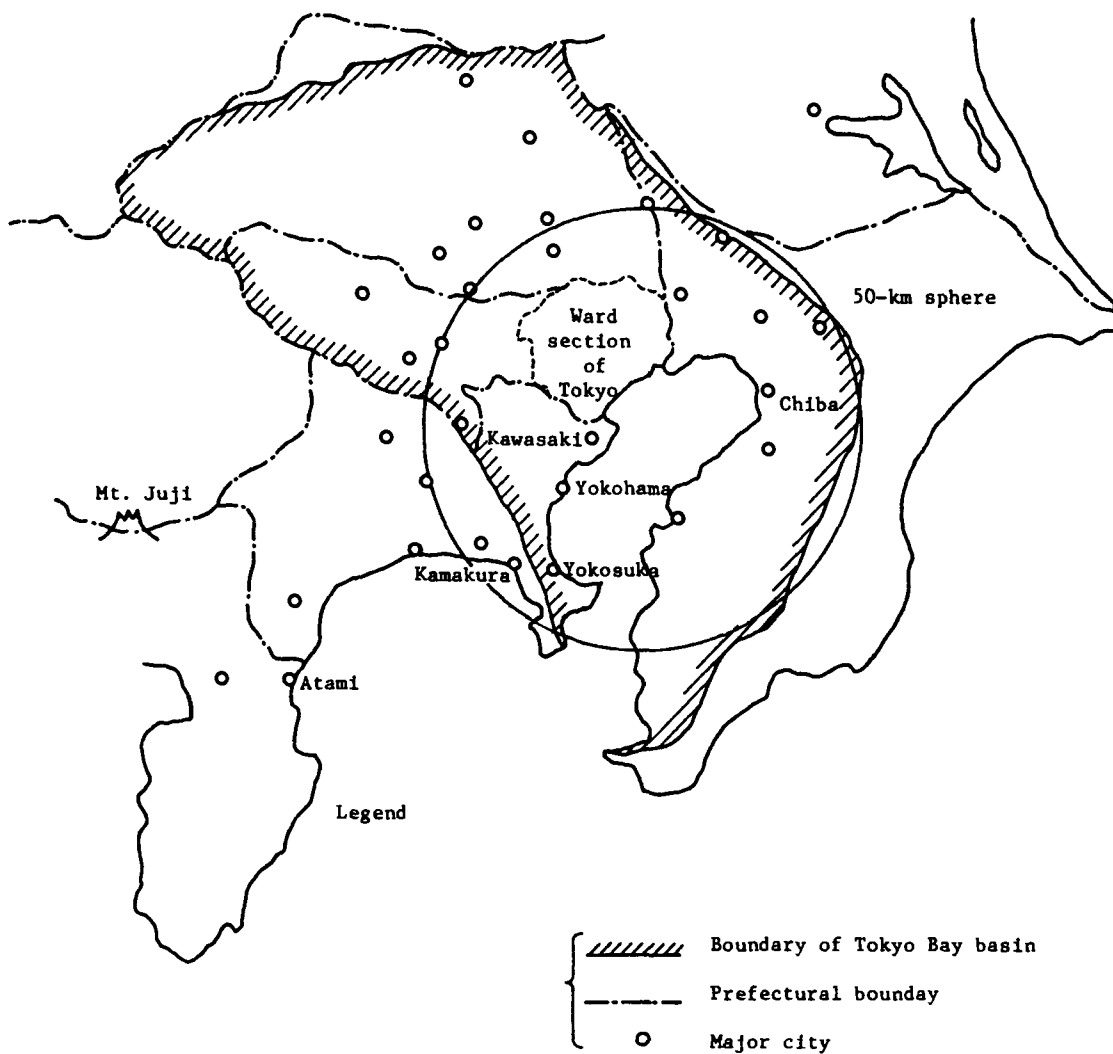


Fig. 7 50-km Sphere of Tokyo

5.2 Possibilities of Utilization of Sludge as a Resource and Reduction of Sludge Volume

Local municipalities, which are administrators of sewage works should do their best in the management of sludge. They should not be merely concerned with sludge disposal by land-fill and land reclamation but they should, from a long-range point of view, seek also to actively develop the possibility of effective utilization of sludge in the future for purposes that include its application to greens and farmlands. Regarding the conversion of sludge into a resource, study is being made for its practical and effective use (see Table 15). In fact, sludge is already used as compost for greens and farmlands, but the amount of sludge thus used is only a fraction of sludge production. To increase its use for this purpose, its quality control must be complete; particularly, it is important to try to eliminate the chance of toxic substances contained in industrial effluents from discharging into public sewers.

As for the use of sludge as a construction material, there are such potential applications as the use of sludge for roadbed, as a fill material and the use of sludge as aggregates, broken stones, bricks, and blocks. Related techniques have been developed considerably but further study must be made particularly from the viewpoints of cost, quantitative limitations and distribution that remain to be solved.

In any case, sludge has every possibility of being used as a resource but, because of its cost which is still high, the prospect is that large amounts of sludge will not be used as a resource in the near future and can be made available in only limited quantities.

As for the reduction of sludge volume, many sewage treatment plants are trying to reduce the job of ultimate disposal by such methods as incineration because of the difficulty of securing the necessary disposal sites. Since, however, incineration will require additional amounts of oil and other energy in view of the prospective increase of sludge production. It is hoped that a sludge incineration of an energy-saving type will be developed in the future. Further, sludge incineration is uneconomical as it requires additional equipment for deodorization and air pollution control.

Table 15 Possible Applications as Resources

Application	Character
For greens and farmlands	Compost, dry or dewatered cakes
For reclamation and land fill	Dewatered cakes, fermented stable
Roadbed and pavement material	Incineration ash, fermented stable, melted
Aggregates	Fired (incineration ash + shale, etc.)
Bricks, etc.	Fired (incineration ash + clay or silica sand)

5.3 Necessity of the Regional Sludge Management Program

Regional measures are necessary because of the increase of sludge production and the growing difficulty of treating and disposing of sludge in its own jurisdiction by local municipalities. This is charted in Fig. 8.

5.4 Necessity of Reclamation from the Sea

Sewage is collected in lowland by natural flow through sewers in its drainage area, and sewage thus purified is returned to the natural water circulation system. In this connection, sewage treatment plants are generally concentrated along rivers and bays. The Tokyo Bay basin is no exception. In Chiba, Tokyo and Kanagawa Prefectures, which are on Tokyo Bay, large sewage treatment plants are located in areas along the Bay. The 1995 sludge production in these bay-adjointing areas is estimated in Table 16 from the results of a need survey as part of the year's sludge production in the Tokyo Bay basin. It represents about 2/3 of the amount of sludge requiring disposal.

In the Tokyo Bay basin, the land space is already used at such densities for commerce, residence, industry and agriculture that it is practically impossible to secure sites suitable for sludge disposal. Further, dependence on land reclamation from the seashore in Tokyo Bay for the disposal of sludge is now inevitable for the following reasons:

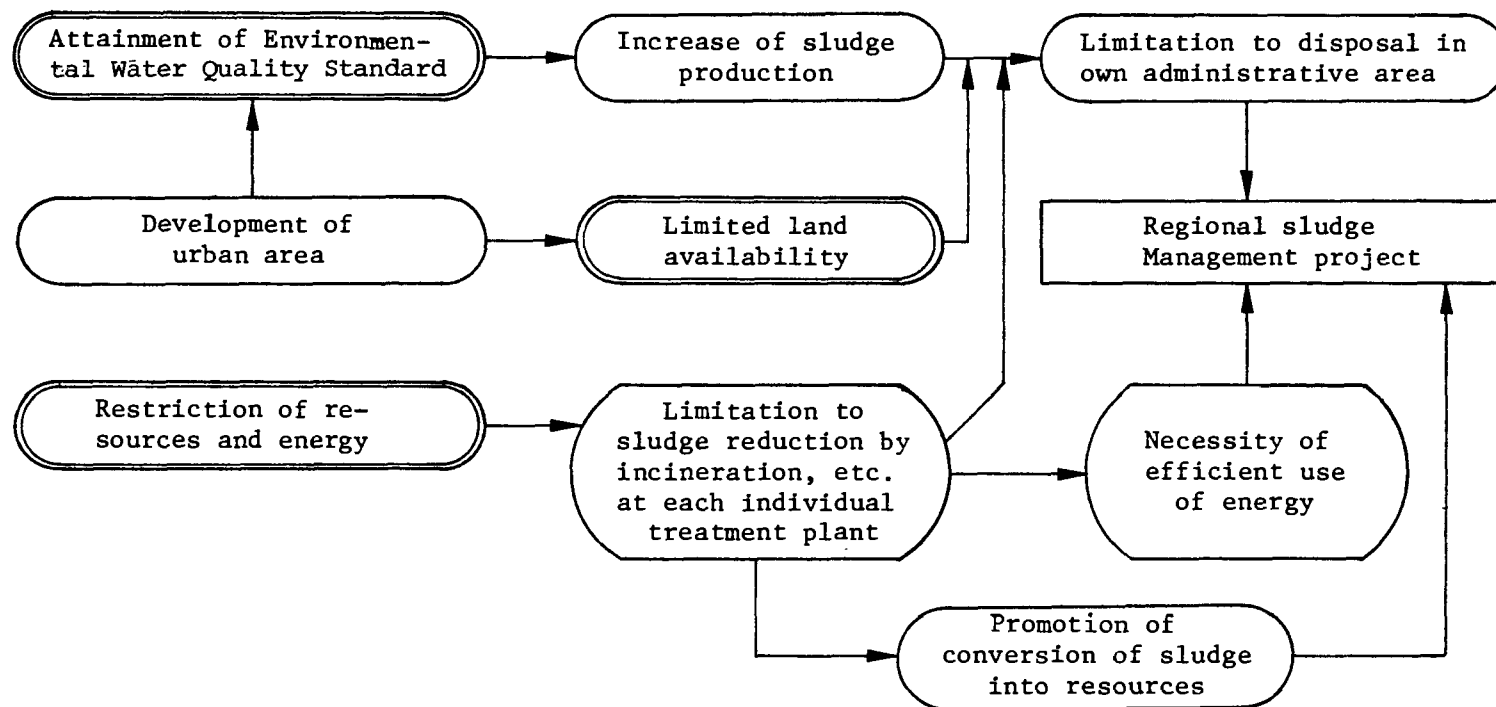


Fig. 8 Necessity of Regional Sludge Management Program in the Tokyo Bay Basin

- ① Disposal sites can only be acquired at increasingly great distances, from the sewage treatment plants which means many future problems including those of transport cost and transportation system.
- ② The bay-adjointing areas account for as much as 2/3 of the total sludge production in the Tokyo Bay basin. This rate may reach 90% if inland sludge can be transported easily and economically by using mass transportation.

But at sea, in Tokyo Bay too, the places left for land reclamation are limited. It is, therefore, important that local municipalities, while trying to reduce sludge production on the one hand, should cooperate in using these disposal sites effectively and economically.

Table 16 Sludge Production in Bay-Adjoining Areas in 1995
(in Terms of Disposal Amount) in Tokyo Bay

Item Form	Sludge production (1,000 m ³ /year)		Rate of bay- adjoining area (%)
	Total amount for the Tokyo Bay basin	Total amount for bay-ad- joining areas	
Dewatered cakes	1,943	1,260	65
Incineration ash	628	440	70
Total	2,571	1,700	66

Note: The moisture content varies by form of disposal.

6. RESEARCH AND TECHNOLOGY DEVELOPMENT FOR THE REGIONAL SLUDGE MANAGEMENT PROGRAM

6.1 Subjects of Study

To realize regional sludge management in the Tokyo Bay basin, study must be made on the following subjects:

① Coordination with Various Other Programs Concerning Tokyo Bay

In materializing this program, it must be thoroughly coordinated in the future with other programs including the reclamation plan for Tokyo Bay and the municipal solid waste disposal program for Tokyo Bay.

② Conduct of Environmental Assessment

The regional sludge management program involves the possibilities of causing secondary pollution, such as the effect of reclamation from the sea on the marine ecosystem and the problem of odors affecting local inhabitants, as well as the effect on ship navigation. Careful scientific investigation, research and studies must be initiated in preparation for an assessment aimed to protect these environments.

③ Development of Energy-Saving Technology

A tremendous amount of resources and energy is being consumed in sludge treatment and disposal. In the future, it will be necessary to try to develop resource- and energy-saving technologies including the reduction of chemical consumption, sludge incineration of the energy-saving type and the recovery of heat from digestion gas.

④ Long-range Outlook for Sludge Management

Land-fill and land reclamation have certain limits in Tokyo Bay basin. Efforts must be made to realize unlimited disposal of sludge so that sludge, a product of human activity, may be incorporated as much as possible into the circulation of natural resources. In the future, scientific investigation, research and studies into the possibilities of such unlimited disposal as the application to greens, farmlands and the ocean must be initiated, taking local conditions into con-

sideration. Application to greens and farmlands is already in practice but no comprehensive research or studies have been conducted on application to the ocean. This is because application of sludge to the ocean is likely to be opposed in this country, a fishery nation where people are in the habit of eating considerable amount of fish daily.

However, there is every likelihood that the application of sludge to the ocean, like its application to the land, can be incorporated into the circulation of natural resources by controlling its quality and the ocean environment, thereby contributing to the increase of marine resources. Comprehensive research and studies, including the problem of food chains, are necessary in this connection.

⑤ Study of Quantitative Reduction at Sources and Monitoring System for Quality Control of Sludge

The per-capita amount of sewage and the resultant amount of sludge have increased yearly. These must be reduced at their sources from the viewpoint of resource and energy conservation. It is also necessary to study an efficient monitoring system for the quality control of sludge.

6.2 Items of Technology Development

Hereunder is a description of important considerations among items of technical development and research including the program of joint treatment and disposal of sludge in the Tokyo Bay basin:

6.2.1 Transportation System of Sludge

Study will be made to see which means of transportation-pipeline, truck, tank lorry or barge- is most suitable to transport dewatered cakes, incineration ash yielded from the sewage treatment plant and raw sludge taken from the settling tank to the sludge treatment and disposal plant in each case.

Specifically, the following items will be studied:

① Study on transportation of raw sludge

- ② Study of the optimum moisture content of sludge transported by pipeline.

- ⑥ Study of the possible transportation distance by pumping.
 - ⑦ Study on pump blockages and countermeasures.
 - ⑧ Study to prevent sludge from becoming anaerobic through long-distance transportation and to cope with the variation in solid concentrations.
 - ⑨ Study of the design and installation of buried submarine pipes.
- ② Study transportation of dewatered cakes and incineration ash
- ① Study of sludge properties (including moisture content) in truck and barge transportation.
 - ② Study of different types of barges.
- 3 Establishment of optimum and cost-effective sludge transportation

6.2.2 Large-Scale Sludge Treatment and Disposal

The flow diagram shown in the following chart can be conceived as a regional sludge management program.

Study will be made of the following items concerning the sludge management program:

- ① Study on treatment of sludge yielded from the AWT process

Unlike sludge yielded from secondary treatment, sludge resulting from AWT is not easy for thickening and dewatering. So, it is necessary to select an optimum combination of thickening (pressure flotation, centrifuge) and dewatering method (belt press, filter press, centrifuge) for sludge from AWT.

- ② Study of large-scale sludge digestion

In this work, digestion is included as a process of the treatment of sludge by secondary treatment. Since this large-scale plant will treat considerably more bulky sludge than a conventional sewage treatment plant, it will be provided with a large digestion tank to make efficient treatment possible. Since few Japanese sewage treatment plants have large digestion tanks at present, study will be made of the structure and function of a large-scale digestion tank suitable for the mass treatment of sludge.

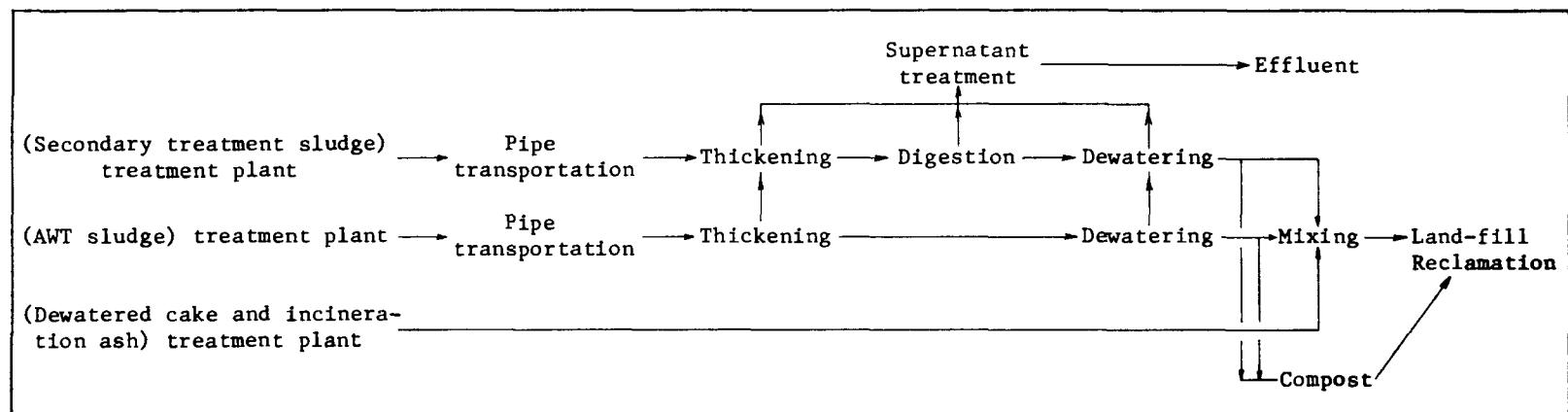


Fig. 9 Flow Chart of Regional Sludge Management Program

③ Study of cement mixing

Though Tokyo has experience in the mixing of cement necessary for this purpose, study will be made of the proportion of cement used for this purpose and the determination of the mixing method most suitable for this work in strength and cost.

④ Study of volume reduction by field composting

Since the dewatered cakes transported to disposal sites or those produced at sewage treatment plants must be reduced as much as possible, their volume may be reduced economically through field composting by aerobic fermentation. In Japan, the conversion of sludge into compost by means of high-speed fermenting equipment has been studied and made practical because of the locations of sewage treatment plants and the size of their sites, but no field composting has yet been used. So, study which includes demonstration tests will be made of field composting.

⑤ Energy-saving incineration

Since incineration is most effective in reducing the volume of sludge, study will be made of an energy-saving incinerator of the self-sustaining combustion type that can incinerate sludge with low fuel consumption.

6.2.3 Large-Scale Land Reclamation

① reclaiming methods

Study will be made to see how best to use incineration ash, mixed sludge, compost and the remaining soil from construction works for reclamation. The optimum reclaiming method will be studied particularly by testing soil conditions, deformation characteristics and coefficients of permeability, etc. by selecting separately the types of reclaiming materials, by testing the mix proportions of reclaiming materials and by studying the suitability of machines to be used.

② Treatment of leachate

When land-fill is used for sludge disposal, highly polluted leachate may exude from the reclaiming materials. The properties of the leachate exuding from sewage sludge, will be predicted and, at the same time, study will be made of an appropriate treatment for this leachate.

6.2.4 Supernatant from the Sludge Treatment Process

The supernatant from the digestion tank, the thickner and the dewatering facility are generally returned to the sewage treatment plant as return water to be treated. Study will be made of a treatment plant that can operate to purify upto the level of effluent standards as a separate treatment of supernatant.

6.2.5 Electricity Generating System by Digestor Gas

This is a plan to carry out gas electricity generation, using digestion gas, with the use of this electricity as the necessary energy for treatment and disposal purposes. Digestion gas electricity generation has been seldom practised in the past, though some cities were equipped for this purpose in Japan. At present, practical tests are being conducted to realize the plan. Study of a digestion gas electricity generating system best suited to this project will be made by referring the results of these tests.

6.2.6 Analysis of Cost-Effectiveness

Comprehensive study will be made of the cost-effectiveness of each individual project of sludge management and also for the regional sludge management project.

7. CONCLUSION

The above is a brief description of the necessity of the regional approach for a sludge management program in view of the present quality of water in Tokyo Bay, the present conditions of sludge treatment and disposal and prospects for the future in the Tokyo Bay basin. Further, it outlines the numerous problems that must be studied to carry out the regional sludge management program for the Tokyo Bay basin.

Eighth US/JAPAN Conference
on
Sewage Treatment Technology

STATUS AND OUTLOOK OF SLUDGE TREATMENT AND DISPOSAL IN KYOTO CITY

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1. PREFACE

Kyoto City is an international cultural and sightseeing city located in the middle course of the Yodo River with a population of 1,460,000. The water of the Yodo River, into which municipal wastewater is discharged, is utilized as valuable water resources for approximately 11 million people at its lower reaches. Consequently, the sewage works of Kyoto City is absolutely important not only for the improvement of the living environment but also for the preservation of water quality in the Yodo River.

The construction of sewer systems in this city was started in 1930. It was suspended for some years during World War II, but it was resumed and is progressing. At present, the sewer network covers 48.8% of the urbanization areas and 60.2% of its total residents as shown in Table 1 and Fig. 1. Together with the progress of sewer construction and more advanced treatment process, the treatment and disposal of sewage sludge has grown a big problem.

In this report, the history of sewage sludge treatment and disposal in Kyoto is described and current problems and its countermeasures are also discussed.

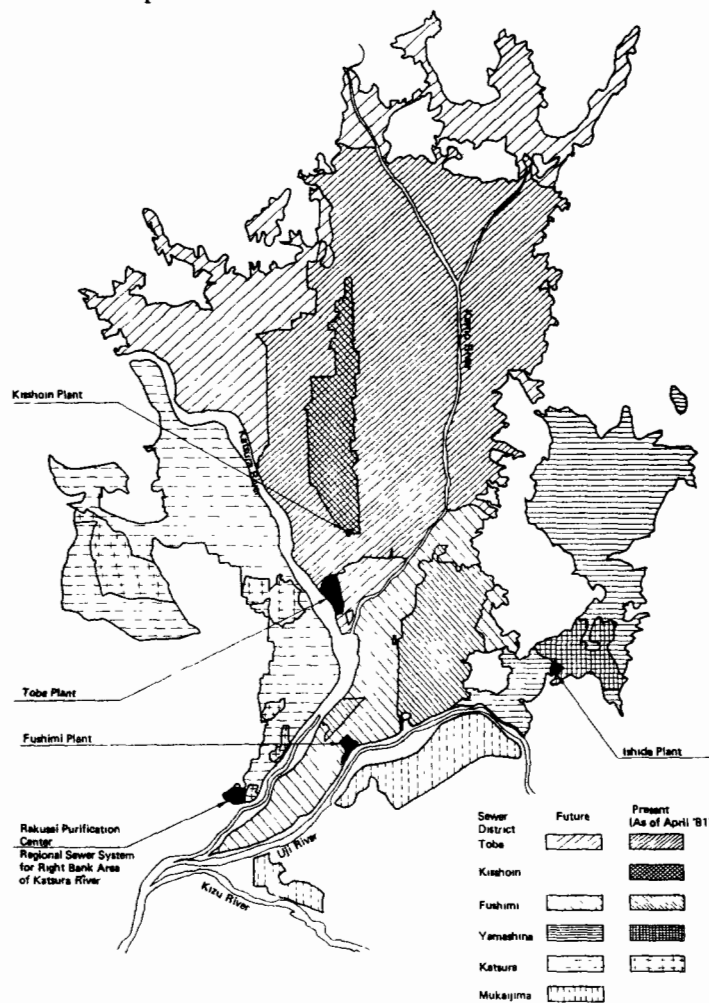


Fig. 1 Sewer District of Kyoto City

Table 1 Present Condition in Kyoto City
(As of April, 1981)

Area of Kyoto City	61,061 ha
Urbanization Area	14,906 ha
Population of Kyoto City	1,465,677
Sewer Service Area	7,275 ha
Coverage of Urbanization Area	48.8 %
Sewer Served Population	881,860
Coverage of Population	60.2 %
Total Length of Pipelines	2,051,210 m
Treatment Capacity	1,005,000 m ³ /day

2. HISTORY OF SLUDGE TREATMENT AND DISPOSAL

In consideration of the social and natural conditions in the city, it is necessary to select more effective process for sludge treatment and disposal. Kyoto is an inland city with an urbanization area of 14,906 ha and is surrounded by hills except for the south where the Yodo River is flowing. In the recent years, the urbanization has spread to the suburbs on the foot of hills. As a result, it has become difficult to obtain the site available for sludge disposal.

Kyoto City has four wastewater treatment plants, Toba, Kisshoin, Fushimi and Ishida now in operation as shown in Fig. 2. The history of sludge treatment and disposal is represented by that of the Toba plant. The Toba plant had a capacity of 78,000 m³/day at the beginning of operation in 1939. At present, its capacity has increased to 750,000 m³/day. This plant is planned to treat 1,225,000 m³/day in future as the biggest plant of the city.

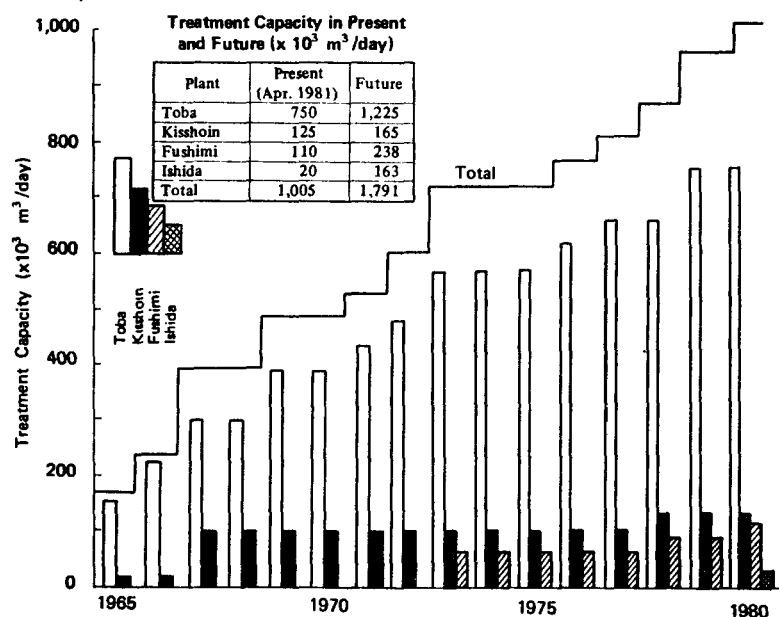


Fig. 2 Increase of Wastewater Treatment

The Toba plant is located at the site where the Kamo and Katsura rivers are joined in the south of the city. About 40 years ago when the operation was started, the plant was in the rural district. With the conditions of location and the economics of maintenance and operation taken into consideration, “anaerobic digestion—sun drying—landfill” was first adopted as the process for sludge treatment and disposal. It was evaluated as the best process from the results of operational studies at the Kisshoin plant with a 7,800 m³/day capacity and the pilot tests of the mechanical dewatering such as vacuum filtration.

But the process of sludge treatment and disposal has been replaced with new ones with the increasing quantities of influent and sludge produced that resulted from the expansion of sewer network and treatment capacity of the Toba plant as shown in Fig.3. First of all, it became impossible to treat sludge with sun drying, and rotary-drum vacuum filter (filtration area: 47 m²/unit × 4 units) was installed in 1965. Later, as the number of digestion tank and vacuum filter has increased to deal with the increasing quantity of sludge produced, the sludge dewatering has become possible. While the quantity of dewatered cake was rather small, the landfill

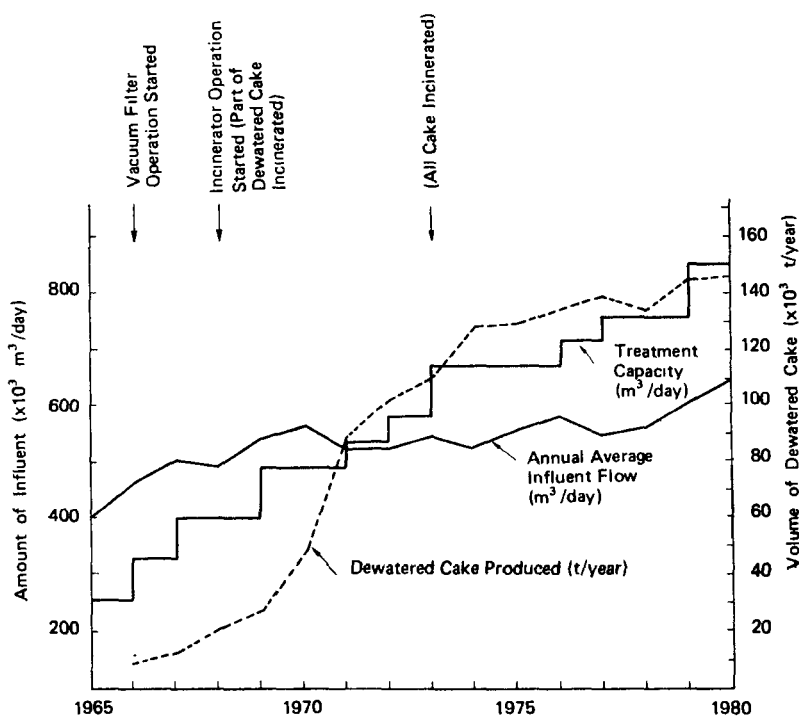


Fig. 3 Treatment at The Toba Plant

disposal was carried out in the yards of the Toba plant and others. In view of the difficult situation to obtain the landfill site to cope with the growing quantity of dewatered cake, it has become an urgent problem to reduce the quantity of sludge to be disposed of as much as possible.

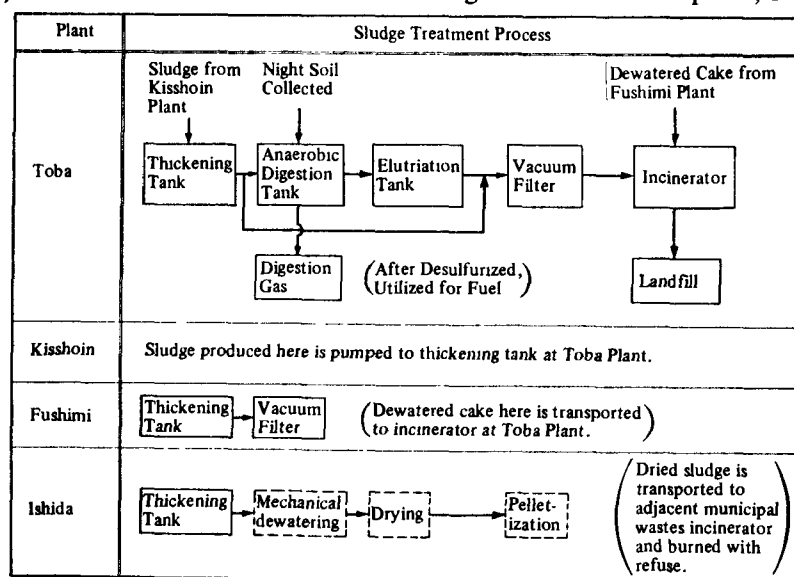
Consequently in 1968, the incineration of dewatered cake was started by multiple hearth furnace (incineration capacity: 60 t/day unit × 3 units). Later, the number of incinerator has increased, and since 1973, the whole dewatered cake has been incinerated. The raw sludge dewatering has been adopted without digestion process for the expanded facilities after the start of incineration although the sludge treatment process was "anaerobic digestion—mechanical dewatering" for the existing facilities.

In fiscal 1980, the whole dewatered cake of about 450 t/day (moisture content; about 75%) including cakes from other treatment plants was incinerated. Through the incineration, the amount of ash to be disposed of has been reduced to about one-seventh of dewatered cake weight, but the disposal quantity still has reached 63 t/day. Furthermore, the sludge quantity is expected to increase more and more with the expansion of the sewage service area in the future. In the urban environment where the urbanization is ongoing even in the suburbs, it is impossible to secure the landfill site for the vast quantity of sludge produced. The saving of resources and energy has become the worldwide problem. The urbanization of districts near the treatment plant is progressing year by year. From these changing situation, we should evaluate again the process of sludge treatment and disposal and replace with more effective facilities without pollution.

3. CURRENT STATUS OF SLUDGE TREATMENT

3.1 SLUDGE TREATMENT PROCESS AT EACH TREATMENT PLANT

The outlines of the sludge treatment process at four treatment plants of Toba, Kisshoin, Fushimi and Ishida are shown in Fig. 4. At the Toba plant, sludges pro-



Note: Processes in dot line are in the midst of planning.

Fig. 4 Outline of Sludge Treatment Process

duced here and pumped from the Kisshoin plant, about 1.5 km away, are fed into the thickening tanks. A part of night soil collected from unsewered area is poured into the digestion tank. There are two different sludge treatment processes. One is "thickening—anaerobic digestion—dewatering" and the other "thickening—dewatering." The dewatered cake is incinerated, and ash is disposed of on land.

Sludge produced at the Fushimi plant is treated in the "thickening—dewatering" process, and the dewatered cake is transported by truck to the Toba plant for incineration.

The Ishida plant was put into operation in January 1981. As the amount of influent is small at present, sludge is thickened and transported to the Toba plant for subsequent treatment. But a plan is under way to incinerate sludge produced with municipal refuse at a solid waste incineration plant adjacent to the Ishida plant following the treatment of the "dewatering—drying—pelletization."

3.2 STATUS OF SLUDGE TREATMENT

The present status of sludge treatment at the Toba plant is briefly described. The outlines of the main treatment facilities are shown in Table 2. The daily average solids balance of the sludge treatment process in 1980 is shown in Fig. 5. The figure shows that the quantities of primary sludge and waste sludge are 41 t/day and 40 t/day respectively. It also indicates that the total quantity of solids produced is 81 t/day. Moreover, some solids in side stream from the sludge treatment process

Table 2 Outline of Sludge Treatment Facilities at Toba Plant

(1) Thickening Tanks

	I	II	III	IV
Type	Circular Tank	Square Tank	Circular Tank	
Diameter	20.0m	17.0m×17.0m	20.0m	
Water Depth at Wall	3.0m	3.6m	3.0m	
Number of Tanks	2	2	4	
Capacity	966 m ³	1,160 m ³	942 m ³	

(2) Digestion Tanks (Two Stage Digestion)

	I	II	III	IV
Type	Cylindrical tank with cone bottom and fixed cover			
Diameter	25.0m	25.0m		25.0m
Water Depth at Wall	5.2m	8.2m		8.2m
Water Depth at Center	9.2m	11.1m		11.1m
Heating System	Heat Exchanger			Steam Blowing
Stirring System	Gas Stirring			
Number of Primary Digestion Tanks	1	2	3	3
Number of Secondary Digestion Tanks	1	2	1	1
Capacity	3,600 m ³ 3,090 m ³	4,400 m ³		
Temperature	30°C			
Detention Period	30 Days			22.5 Days

(3) Vacuum Filters

	I	II
Type	Belt Filter Type	
Diameter of Drum	3.5 m	
Length of Drum	4.2 m	
Surface Area of Filter	47 m ²	
Filtration Rate	17.5 kg dry solid/m ² day	
Number Installed	8	12

(4) Incinerators

Type	Multiple Hearth Furnace		
Outside Diameter	4.35 m	5.10 m	6.78 m
Height	10.04 m	11.34 m	12.35 m
Burning Temperature	800°C		
Number of Hearths	8		
Capacity	60t Sludge Cake/day	150t Sludge Cake/day	
Number Installed	2	1	3

are returned to the head of the wastewater treatment plant. Including these solids, solids of 132 t/day are fed to the thickening tanks. The quantity of sludge solids pumped from the Kisshoin plant and fed into the thickening tanks is 9 t/day; the quantity of night soil thrown into the digestion tank is 7 t/day; and the quantity of dewatered cake solids transported from the Fushimi plant for incineration is 13 t/day. All these solids are treated at the Toba plant.

In the balance of sludge solids like this, 401 t/day cake is produced at Toba. A total of 453 t/day of dewatered cake (moisture content: about 75%) including 52 t/day of dewatered cake from the Fushimi plant is incinerated into ash of 63 t/day. Part of ash is used as specific fertilizers and materials for the test of sludge reclamation. All other ash is transported by truck to the site for landfill.

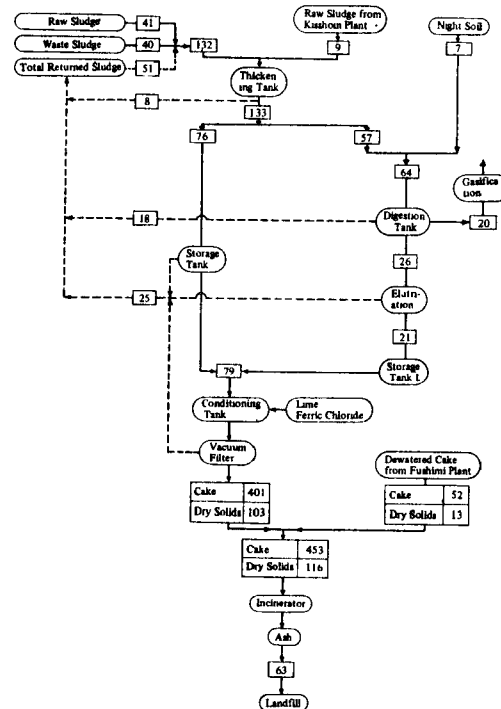


Fig. 5 Sludge Flow at Toba Plant

(Fiscal 1980, Dry Solids t/d)

4. PROBLEMS ON SLUDGE TREATMENT AND DISPOSAL AND ITS COUNTERMEASURES

4.1 PROBLEMS ON SLUDGE THICKENING

4.1.1 EFFECTS ON TREATMENT EFFICIENCY

The gravitational thickening has been conducted so far for combined sludge—raw primary sludge (from primary sedimentation tank) and waste sludge. But recently, combined sludge has tended to be thickened poorly. As shown in Fig. 6, the solids content in thickened sludge was 4~6% in the past, but has decreased to 2~3% in the recent years. The deterioration of thickening properties has the following adverse effects on the sludge treatment.

In case that the solids load is kept at the original design level of the digestion process, the quantity of sludge fed increases, causing several troubles such as the increase of heat required, the shortage of digestion period and the aggravation of quality of supernatant. From the above reason, the quantity of sludge fed to the digestion tank is controlled, so the operation efficiency of digestion tank is not so good. Even if sludge is digested for about 30 days in such operational condition, the solid content of digested sludge is only 3%, and some troubles occur in the following dewatering process.

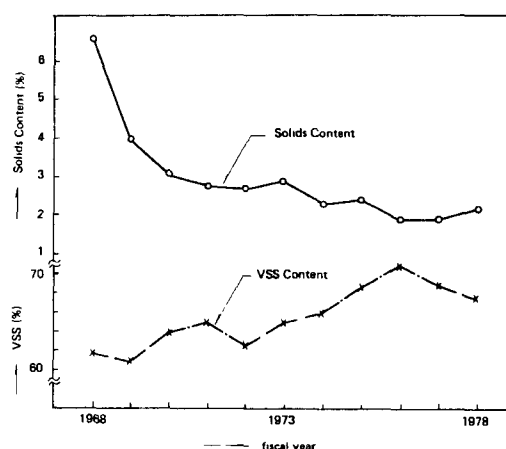


Fig. 6 Yearly Change of Solids Content and VSS Content in Thickened Sludge

Sludge is first conditioned with ferric chloride and slaked lime, and then dewatered with the rotary-drum vacuum filter. Figs. 7 and 8 show that the chemical dosage increases with the decrease of solids content and that the filter yield sharply decreases. Especially, as the digested sludge of low solids content produces hair cracks on sludge cake which reduce the vacuum pressure, it is difficult to dewater such digested sludge. And the poor filter yield needs the extended operation time which will cause the increasing cost of electric power consumption and operation personnel, and the expansion of facilities.

The increase of moisture and inorganic substances like slaked lime in dewatered cake causes the increase of the fuel consumption for incineration and the quantity of ash, and the cost of incinerator operation and ultimate disposal are raised resultantly.

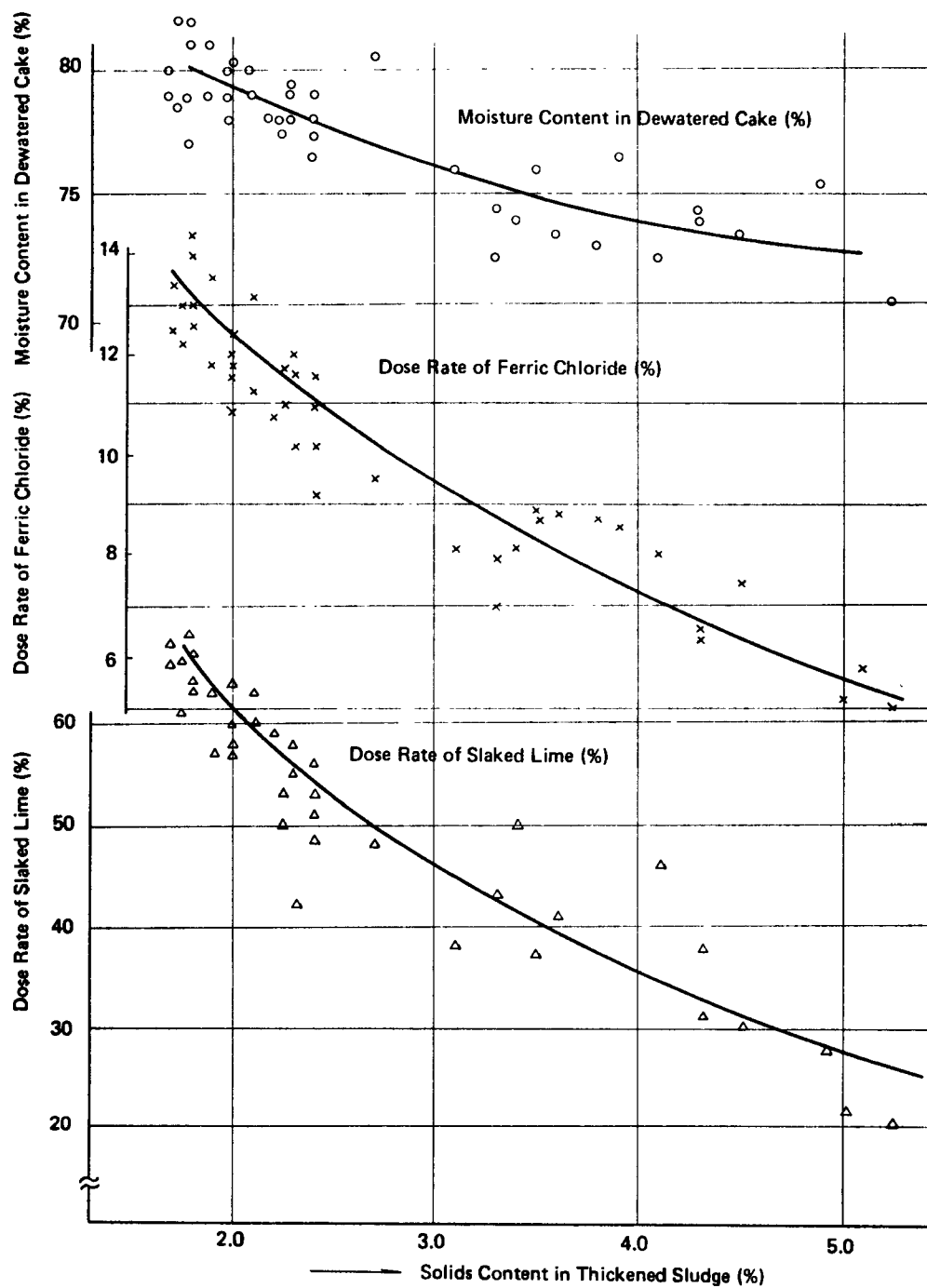
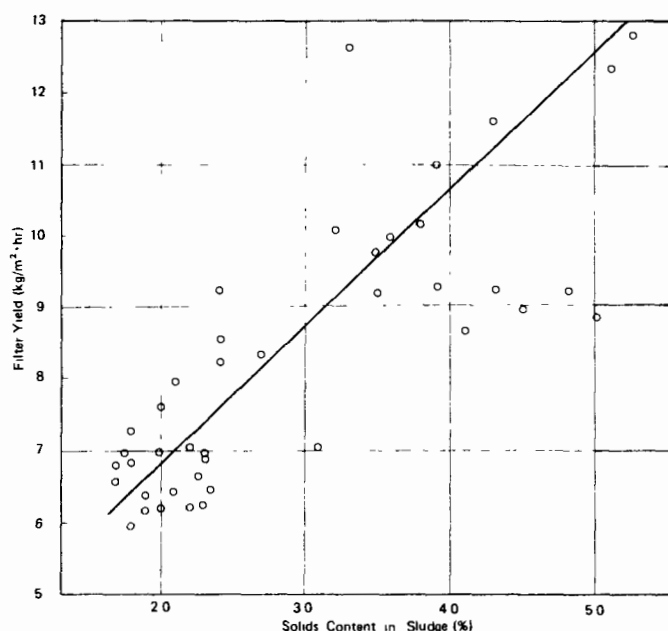


Fig. 7 Relation of Solids Content in Thickened Sludge with Chemical Dose Rate or Moisture Content in Dewatered Cake



Sludge Treatment Process for Expansion Plan at Toba Plant

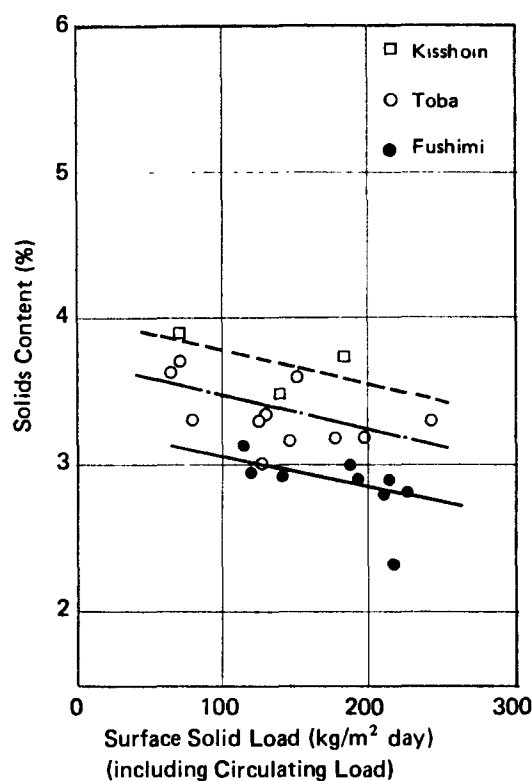
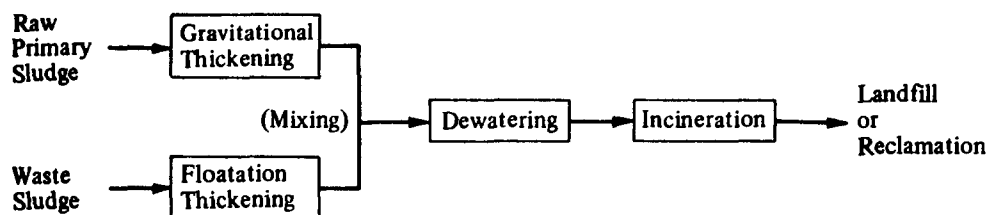


Fig. 9 Thickening of Waste Sludge by Dissolved Air Floation

4.2 ENERGY CONSUMPTION IN SLUDGE TREATMENT

4.2.1 BREAKDOWN OF ENERGY CONSUMPTION

At the sewage treatment plant, the vast amount of energy such as electric power and fuel is consumed. The energy consumption at all the treatment plants in Kyoto City by types of energy sources and by treatment processes are shown in Table 3 and Fig. 10. These data show that nearly 150 billion kcal of energy including electric power, heavy oil and digestion gas is consumed to treat about 260 million m³ of wastewater annually at all the treatment plants of this city.

As to the breakdown of consumed energy, electric power accounts for 45.3% and fuel for 54.7%. With regard to fuel, heavy oil covers about 67% and digestion gas about 33%.

Table 3 Energy Consumption at Wastewater Treatment Plants

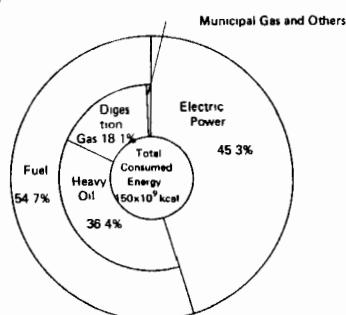
(Fiscal 1980)

Item \ Treatment Process			Pumping	Wastewater Treatment	Sludge Treatment	Total
Electric Power	Used Power	($\times 10^3$ kwh)	10,163	60,008	8,956	79,127
	Calorie Equivalent	($\times 10^6$ kcal)	8,740	51,607	7,702	68,049
Heavy Oil	Used Volume	(kl)	229	0	5,858	6,087
	Calorie Equivalent	($\times 10^6$ kcal)	2,061	0	52,722	54,783
Digestion Gas	Used Volume	($\times 10^3$ m ³)	0	0	4,958	4,958
	Calorie Equivalent	($\times 10^6$ kcal)	0	0	27,269	27,269
Municipal Gas, etc.	Used Volume	(—)	—	—	—	—
	Calorie Equivalent	($\times 10^6$ kcal)	0	210	36	246
Total Calorie ($\times 10^6$ kcal)			10,801	51,817	87,729	150,347

(Note)

Conversion Factors: Electric Power 1kWh = 860 kcal Heavy Oil 1 l = 9,000 kcal
Digestion Gas 1 m³ = 5,500 kcal Municipal Gas 1 m³ = 11,000 kcal
Propane Gas 1 kg = 12,000 kcal Light Oil Kerosene 1l = 8,400 kcal

(1) By Energy Sources



(2) By Treatment Process

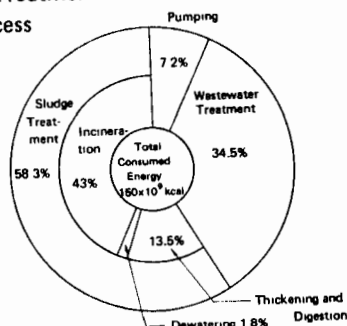


Fig. 10 Energy Consumption At Wastewater Treatment Plants (FY 1981)

The energy consumption by the treatment process shows 7.2% for pumping, 34.5% for wastewater treatment, and 58.3% for sludge treatment. More than half amount of consumed energy is spent for the sludge treatment. With respect to the energy consumption in the sludge treatment process, the thickening and digestion process accounts for 23%, the dewatering process for 3% and the incineration process for 74%. The energy for sludge treatment is supplied with electric power by 9% and with fuel by 91%. Fuel for heating the digestion tank is only digestion gas. The auxiliary fuels of incineration are heavy oil and digestion gas which accounts for 86% and 14% respectively.

Following the worldwide sharp rise of crude oil price, the purchase price of fuel and electric power has also risen remarkably from 1979 in particular, resulting in the annual increase of the maintenance cost, most of which is the energy expense. This trend has a great effect on the finance of sewage works.

4.2.2 COUNTERMEASURES FOR SAVING ENERGY

Countermeasures for saving energy have been conducted at each plant as much as possible. At the sludge treatment facilities, digestion gas is used to save heavy oil for incinerator. For six multiple hearth furnaces (incineration capacity: 630 t/day) at the Toba plant, heavy oil had been used formerly as auxiliary fuel. But, since the oil crisis in 1973, three of the six incinerators (60 t/day × 2 units; 150 t/day × 1 unit) have been remodeled to use both the digestion gas and heavy oil. The reduction of heavy oil consumption that has derived from the use of digestion gas is shown in Table 4. About 14% (950 kl) of fuel required for incinerator amounting to 6,800 kl turned on digestion gas (where digestion gas was converted into heavy oil equivalent), and the fuel cost was saved by 60 million yen in fiscal 1980.

Table 4 Saving of Heavy Oil Used for Sludge Incineration

Fiscal Year Item	1976	1977	1978	1979	1980
Cake Incinerated (t/year)	139,707	147,076	149,106	161,415	165,374
Heavy Oil Used (kl/year)	4,460	3,970	5,105	4,734	5,858
Digestion Gas Equiva- lent to Heavy Oil (kl/year)	694 (1,136)	920 (1,505)	991 (1,622)	984 (1,611)	948 (1,552)
Total of Fuel Used (as Heavy Oil) (kl/year)	5,154	4,890	6,096	5,718	6,806

(Note)

1. Conversion Factors Heavy Oil 1 ℓ = 9,000 kcal
Digestion Gas 1 m^3 = 5,500 kcal
2. Figures in parentheses show used volume of digestion gas ($\times 10^3 \text{ m}^3/\text{y}$)

4.3 ULTIMATE DISPOSAL OF SLUDGE

4.3.1 REGIONAL PLANNING FOR SLUDGE DISPOSAL

The procurement of ultimate disposal site for sewage sludge is in severe situation particularly because of the restrictions of inland city. Kyoto is unable to carry out the ocean dumping and the reclamation of sea surface as the ultimate disposal method. With the history of over 1,000 years, there are many cultural relics and scenic beauties to be preserved in this city. So the selection and procurement of site available for landfill is restricted. Accordingly, the incineration of the whole dewatered cake has been enforced both for the sanitary disposal and for reducing the ultimate disposal quantity as much as possible. In the future, the quantity of sludge produced is expected to keep increasing with the expansion of the sewer service areas. Fig. 11 presents the amount of dewatered cake is likely to reach about 800 t/day. In case of incinerating the whole cake, about 100 t/day of ash (on dry solid basis) are estimated to be disposed of. It is impossible to secure

in the municipal area the disposal site where such a large amount of wastes are landfilled.

Many other large cities in Japan have such a problem. The ultimate disposal of the huge amount of sludge accompanied with the municipal activities is one of the important administrative problems.

At present, the Japanese Government has announced a plan for the construction of regional ultimate disposal site. This plan is designed for local governments having difficulties in the appropriate disposal of wastes individually. The sphere of Tokyo Bay and that of Osaka Bay are tentatively listed as proposed areas. Kyoto is included in the proposed area of Osaka Bay sphere. The early enforcement of the plan for regional disposal site is expected to carry out the appropriate treatment and disposal of sewage sludge.

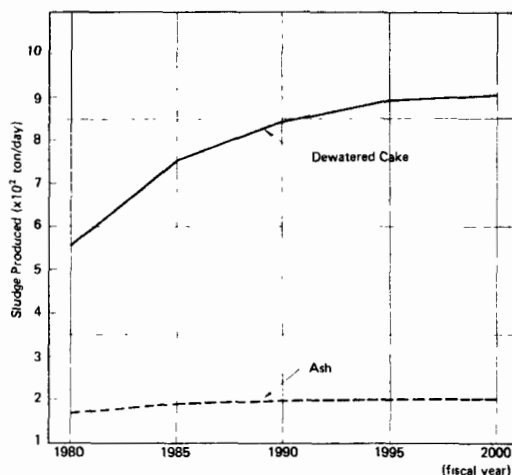


Fig. 11 Estimate of Sludge produced in the future

4.3.2 SLUDGE RECLAMATION

The ultimate disposal of sludge is the biggest problem for this inland city as described above. If further efforts should be made to utilize sludge as resources besides landfill disposal, it will be greatly desirable from the view of saving resources and energy. Therefore, a part of ash has been already utilized as soil conditioner for agricultural land and soft ground in this city. For more reuse of ash and dewatered cake, the following investigation is in progress.

For the utilization of ash as roadbed material, the on-the-site test has been conducted, and good result has been gained for the bearing strength of soil and like that. The investigation is also ongoing on the use of ash to alternative material for concrete product and mortar grout. The research for composting is under way at the Fushimi plant to apply dewatered cake to agricultural land as fertilizer. After about one-year fundamental experiments, the small-scale composting device with the horizontal gyle (cake fermenting capacity: 1 t/day) has been installed for the production of compost. With the cooperation of those concerned with agriculture, the survey has been conducted on the effect of its application to agricultural land on growth rate and soil condition. At the same time, the market of composted

product has been investigated. The composition of sludge compost is shown in Table 5. Further positive efforts should be made to utilize practically sewage sludge as resources.

Table 5 Composition of Sludge Compost

Item (Unit)		Average in Fiscal 1980	Regulatory Standards
Effective Element as Fertilizer	Nitrogen (%)	2.16	—
	Phosphate (%)	1.37	—
	Potassium (%)	0.21	—
Regulated Toxic Substances	Mercury (mg/kg)	0.79	2
	Cadmium (mg/kg)	2.25	5
	Arsenic (mg/kg)	2.5	50
Leaching Test of Toxic Substances	Total Mercury (mg/l)	0.0025	0.005
	Alkyl Mercury (mg/l)	ND	ND
	Cadmium (mg/l)	ND	0.3
	Lead (mg/l)	ND	3
	Organic Phosphorus (mg/l)	ND	1
	Hexavalent Chromium (mg/l)	ND	1.5
	Arsenic (mg/l)	ND	1.5
	Cyanide (mg/l)	ND	1
	Polychlorinated Biphenyl (mg/l)	ND	0.03

4.4 COUNTERMEASURES FOR POLLUTION CONTROL

On treating and disposing of sewage sludge, the necessary pollution control measures are taken for districts surrounding the treatment plants and disposal sites. Concentration of dust, sulfuric oxide, nitrogen oxide, and hydrogen chloride in the exhaust gas from incinerator are regulated by the Air Pollution Control Law and the Kyoto Prefectural Ordinance for Environmental Pollution Control. The cooling tower, sulfur removing and deodorizing tower, and electric precipitator are installed to treat exhaust gas from incinerator as shown in Fig. 12. Table 6 indicates that concentrations of pollutants in exhaust gas are well within the regulation standards. The site for landfill disposal of ash is subject to the regulations as one of treatment facilities for industrial wastes. To prevent the seepage from infiltrating underground, retaining-walls of reinforced concrete made are built around the disposal site with the bottom paved with asphalt. Then the disposal site meets with the structure standards of ultimate disposal facilities. Furthermore, the leaching test of toxic substances is regularly conducted for landfill of ash in accordance with the Waste Disposal and Public Cleansing Law. Table 7 shows the result of leaching test is far below the standards and there is no problem.

On the other hand, the Fertilizer Regulation Law is applied to the utilization of sewage sludge to green field as fertilizer. As indicated in Table 5, sludge compost at the Fushimi plant fully meets with the regulatory standards as specific fertilizer based on the law.

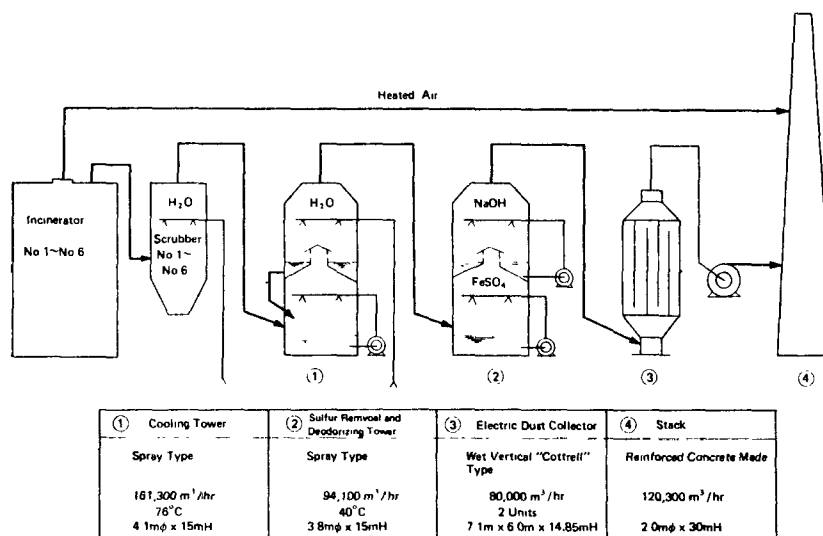


Fig. 12 Treatment Process of Exhaust Gas from Incinerators

Table 6 Analysis of Exhaust Gas from Incinerator

Item		Average in Fiscal 1980		Regulatory Standards
		Before Treatment	After Treatment	
Dust	g/Nm ³	0.182	0.0263	0.7
Sulfur Oxide	ppm	96.9	(k=0.228) 2.41	k=3.5
Nitrogen Oxide	ppm	65	62	300
Hydrogen Chloride	ppm	36	18.6	700

Table 7 Leaching Test of Toxic Substances in Incineration Ash

Item	(mg/ℓ)	
	Average in Fiscal 1980	Regulatory Standards
Total Mercury	ND	0.005
Alkyl Mercury	ND	ND
Cadmium	ND	0.3
Lead	ND	3
Organic Phosphorus	ND	1
Hexavalent Chromium	0.11	1.5
Arsenic	0.00	1.5
Cyanide	ND	1
Polychlorinated Biphenyl	ND	0.03

5. CONCLUSION

In Kyoto City, the sewerage construction is now progressing based on the new five-year plan (1981–1985) corresponding to the 5th five-year sewerage construction plan of the Japanese Government. It will be necessary to conduct the advanced wastewater treatment as a city located at the upper reaches of the Yodo River and the Seto Inland Sea. As a result, the treatment and disposal of ever-increasing sewage sludge is a crucial problem on sewage works in this city.

It is clarified that the conventional concept of disposing of ash on land is not enough to cope with ever-increasing sewage sludge. It is necessary to promote the utilization of sewage sludge as resources by means of positive researches and technology developments. Accordingly, the new proper system for treatment, disposal and utilization of sludge should be established in consideration of current situation of Kyoto and also from the standpoint of energy saving and pollution control. But, for the resolution of the problems on treatment and disposal of sewage sludge, it is necessary to gather not only the sewerage technology but also the extensive fields of knowledge. Furthermore, the cooperation of various fields and regional administrative policies are also indispensable.

COLLECTIVE PRETREATMENT OF INDUSTRIAL WASTEWATER FROM MINOR ENTERPRISES IN YOKOHAMA

October 1981

Washington D.C

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INTRODUCTION:

Yokohama has developed as an international trading city, an industrial city, and also a residential city. But, on the other hand, in the rapid urbanization, many problems have appeared such as the pollution of public waters, air pollution, and a marked reduction in green tracts of land, etc. For taking measures to cope with the situation, the demand of the citizens for strengthening of the regulations concerning industrial wastewater as well as for the planning and rapid construction of sewerage systems has grown.

Major enterprises which have large scale management resources and a high level of technological ability can implement measures to cope with industrial wastewater with relative ease. It is often, however, difficult for minor enterprises to take appropriate measures to cope with industrial wastewater because of their inferior economical and technological ability and their narrow sites. Therefore, when seeking an overall means to cope with industrial wastewater, the problem of minor enterprises is an important problem in need of a quick solution.

Yokohama city has planned to develop reclaimed land in coastal areas in order to remove factories scattered in the city to the land based on the plot of city redevelopment, aiming for improvement of the urban environment, and the recovery of urban functions, and further, to solve effectively the problems of environmental pollution such as industrial wastewater and noise, etc. as well as to advance the industrial modernization of the removed enterprises.

The collective pretreatment at Kanazawa of industrial wastewater in this report has been considered from the aspect of measures to cope with the industrial wastewater of minor enterprises.

In the following, the background and process of the reclamation undertaking at Kanazawa, and the planning, design and bearing of the expenses of the collective pretreatment of industrial wastewater will be stated.

1. Urban Renewal Strategy in Yokohama

Construction of the city of Yokohama originated in the conclusion of "the Treaty of Friendship between Japan and U.S.A." in 1858 by which Japan opened her gates to the world after the isolation of over three hundred years, and took her first step toward modernization. As the trade with other countries progressed after the opening of Yokohama harbour, commercial activities prospered in Yokohama, and it displayed all the atmosphere and color of a harbour city.

After that, Yokohama reclaimed tracts from the sea, and on the coastal reclaimed land, the iron industry, ship-building industry and further heavy and chemical industries centered on petrochemical industries have developed. Subsequently, a huge industrialized zone of Tokyo and Yokohama districts has been formed on a coastal strip of Tokyo Bay which connects Tokyo, Kawasaki, and Yokohama. Thus, Yokohama has had an additional character as an industrial city.

Further, the increased population in the Metropolis of Tokyo which resulted from Japan's high economical growth starting from the 1960s was faced by a rise in the price of land to move outside of Tokyo, so that there has occurred an extraordinary phenomenon of a marked population growth of Yokohama by about 100,000

per year mainly in its inland districts, which cannot be seen in other cities. Thus, Yokohama has had a character of a residential city as a dormitory town of Tokyo. Yokohama has now become the second city of Japan and has a population of about 2,800,000.

With such a background the urbanization of Yokohama has proceeded rapidly and disorderly. Consequently, various urban problems resulting from overcrowded cities have occurred. The environmental planning, and public utility investments for intensifying urban functions could not keep pace with the urban problems, this situation has resulted in the worst form of environment such as the mixing of residences and factories, a marked reduction in green tracts of land, flood damage, pollution of public waters, and air pollution, etc.

Measures to cope with such problems have two aspects, one of which is to direct and regulate rapid developments and industrial activities, and the other is to plan the arrangement of fundamental urban facilities. In Yokohama, rapid developments and industrial activities have been directed and regulated by the method of agreements with housing site developers and industrial enterprises, respectively. On the other hand, six fundamental projects for making the physique of the city by highly utilizing urban spaces, and the proper arrangement of various urban facilities have been determined. Through the execution of these measures, Yokohama is putting forward the construction of "Yokohama city where citizens can live a safe and comfortable life".

The six main projects, which are centered on the undertaking for strengthening the central part, and are connected with each other as shown in Fig. 1, will change the basic urban structure of Yokohama, and also play the role of tractor for undertakings of high citizen demand such as the planning of roads and sewerage systems, the construction of schools, residences, and parks, and defense against environmental pollution and disasters which are not included in the six main projects.

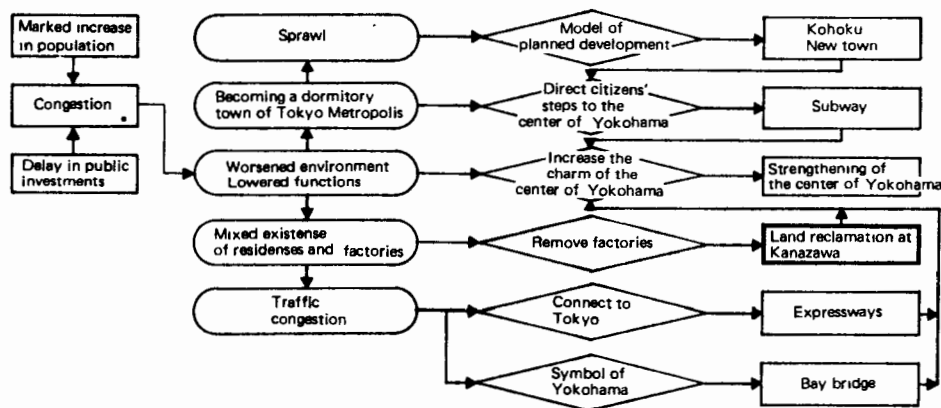


Fig. 1 Schematic Diagram of the Conception of the Six Main Projects

2. A Land Reclamation Project at Kanazawa

The land reclamation project at Kanazawa is being executed as one of the above six major projects.

2.1 The aim and outline of the project

Yokohama has developed by reclaiming tracts of land from the sea and constructing a harbour and factories. However, coast of Kanazawa was the final natural coastal line left to Yokohama as shown in Fig. 2. Therefore, the reclaimed land is to be considered to be utilized for all the citizens of Yokohama. The aim of the project is to provide factory sites having a business environment which is advantageous in location conditions, and to remove improperly located factories which are scattered in the residential areas as shown in Fig. 3 to suitable sites. Further the project also aims for the utilization of the old sites of removed factories for urban facilities such as road facilities, parks and green tracts of land, plazas for refuge, etc., shopping centers, and office streets, etc. to advance the redevelopment of the city, and, at the same time, the construction of an urban industrial development free from environmental pollution by directing the modernization and cooperation of removed enterprises (, in particular, minor enterprises). When executing the reclamation project, in addition to the reclamation and development undertaking, undertakings for removal of factories, etc., for utilization of the sites of removed factories, and for measures to cope with the change of fishermen's occupation for restoring the life of fishermen are needed as shown in Fig. 4, and these undertakings are complex ones which are connected with each other.

In addition, although reclamation projects so far have been financed by the money paid in advance by enterprises going into the reclaimed land, Yokohama city itself financed the reclamation project this time by ensuring a source of revenue with a West German loan, and municipal loan, etc. in order to carry out the work systematically and steadily for long periods based on the objective of developing sites for redevelopment of the city.

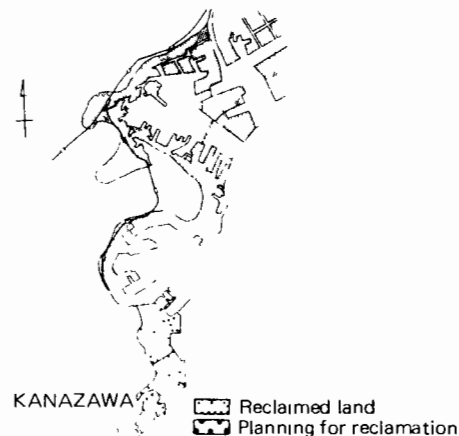


Fig. 2 Illustration of coastal lines

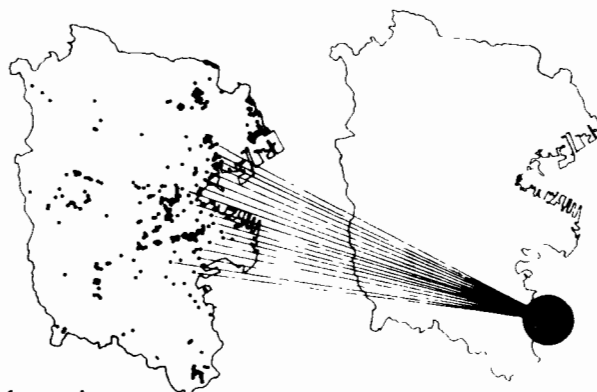


Fig. 3 Diagram illustrating the removal of factories located in the streets of the city

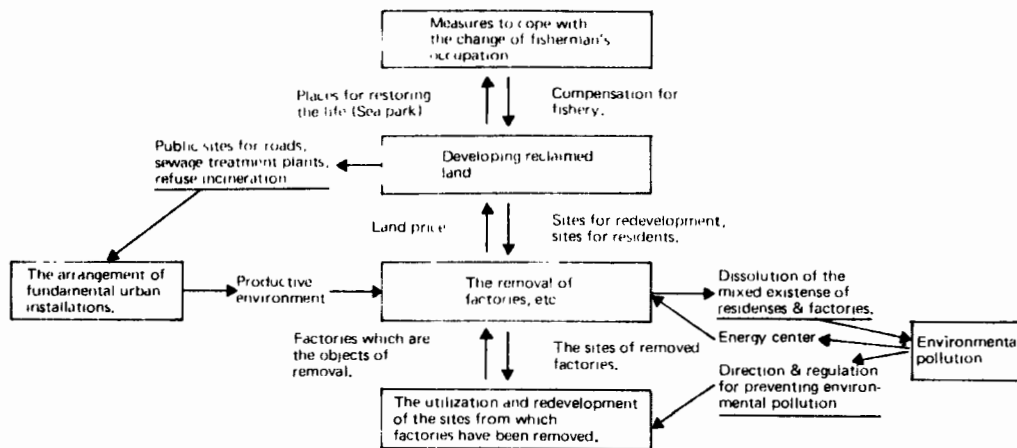


Fig. 4 Mutual relationship among undertaking accompanying the land reclamation project at Kanazawa.

2.2 Land use plan

The area of the reclaimed land is about 6,600,000 m², and its length is about 7,000 m.

The gist of the planning for utilizing the reclaimed land is as follows:

- The sites for rearranging sixty percent of the factories requiring removal of about 2,000 factories scattered in the object regions of redevelopment of the city will be assured, (the sites of removed enterprises will be effectively utilized as part of the project for redevelopment of the central part of the city).
- In the residential areas, 10,000 residences accommodating 30,000 residents will be built, eighty percent of which will be residences for employees related to the factories removed to the new sites, and the rest will be used for general citizens.
- Along the existing coastal line, sites for arranging parks and green tracts of land in various places will be assured. As the basic arrangement of the planning for utilizing the reclaimed land, the loop road of Tokyo Bay is built on the central part of the land, the sites for removal of factories are provided on the coastal side of the land, and residential areas on the mountain side of the land, and further, a sea park is located at the southern end of the land, its functions being distinctly scattered. To fulfil the objective of the project, public utility sites of high civil demand for construction of sewage treatment plants, refuse and waste incineration plants, parks, and roads, etc. amount to about 48 percent of the whole area of the reclaimed land. Divisions for utilizing the reclaimed land are shown in Fig. 5.

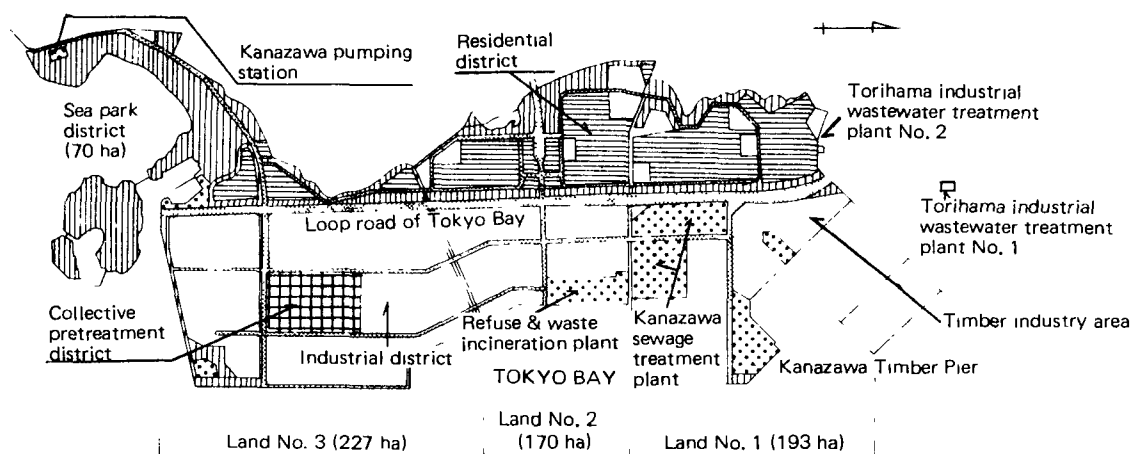


Fig. 5 A plan of land utilization for the reclaimed land at Kanazawa

2.3 Sewerage system plan

The reclaimed land belongs to Kanazawa treatment district of the public sewerage system based on separate sewers.

A plan of Kanazawa treatment district is shown in Fig. 6.

Collecting sewage from the existing town, the trunk sewer goes through the residential district of the reclaimed land to flow into the sewage treatment plant.

On the other hand, wastewater from the districts for removal of factories in the reclaimed land flows into the sewage treatment plant through another trunk sewer. The Kanazawa sewage treatment plant is located in the land No.1, and a part of the plant has been operating since 1979. The plant is characterized by the fact that treatment facilities are separated into two lines, one of them is for domestic wastewater from the existing town and the other is for industrial wastewater from the reclaimed industrial district.

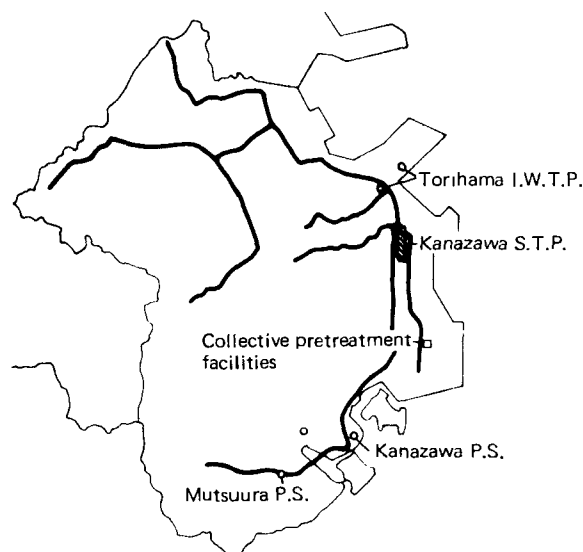


Fig. 6 A plan of sewerage system in Kanazawa treatment district

2.4 Basic plan for a factory development

2.4.1 Removed factories

Factories which are to be removed from existing city streets to the reclaimed land at Kanazawa are those which need to be removed to realize the four objectives of improvement urban environment, strengthening the central part of the city, modernization of urban industries, and recovery of green tracts of land in the city; these are, the purposes of the project, and the factories can be classified as follows:

- a. Factories which are obstructing the urban functions in areas that are to be re-developed with priority, and in areas where the mixed existence of residences

and factories is marked.

- b. Factories which are actually a source of environmental pollution.
- c. Factories which are located in sites where public undertakings are scheduled.
- d. Factories, of which grouping and cooperation can be effectively put forward, and which can contribute to the modernization of the urban industry of the city.

Of about 6,000 factories in the city, those which meet the above conditions are about 2,000 factories, about 60 percent of which need to be removed.

Moreover, the conditions of removal are as follows:

- a. The enterprise is to fall into one of the following industrial categories.
 - ① Manufacturing industry
 - ② Construction industry
 - ③ Service enterprise (servicing of cars, other repair work, lease of industrial machines and tools, laundry cleaning, photography, and the like).
 - ④ Wholesale business of reclaimed resources
 - ⑤ Packing enterprise
 - ⑥ Distribution and processing enterprise (enterprises where the removal was a necessity as confirmed by the City authorities, and the warehouses have already been resited).
- b. Of enterprises which have their works and are conducting business in the city; those which remove their works.
- c. Enterprises which have sufficient funds to effect the removal of their works and their management.
- d. Enterprises which have completely paid their municipal taxes, etc.

2.4.2 Factory arrangement on the reclaimed land

1,200 factories which are the object of removal have been arranged as follows so that organic economical activities can be carried out efficiently based on the plan for utilizing the reclaimed land.

Land No. 1: About 180 factories related to timber which will form a timber industry complex that is centered on Kanazawa timber pier (approximately 23 ha)

Land No.2: About 340 factories related to distribution, whose measures to cope with environmental pollution can be implemented without difficulties. (approximately 31 ha)

Land No.3: About 680 factories which are short of technology and funds necessary for implementing measures to cope with environmental pollution. (approximately 113 ha)

2.4.3 Measures to cope with industrial wastewater in the Land No. 3

Industrial wastewater discharged from factories into a sewage treatment district can be treated by the following three methods:

- a. Individual pretreatment
Each factory is furnished with pretreatment facilities by which the poisonous matter in the industrial wastewater is eliminated so that the wastewater is not

troublesome to sewage treatment. After that treatment, the wastewater is discharged into the public sewerage system, and then its organic pollutants are treated in a sewage treatment plant.

b. Collective pretreatment

After industrial wastewater containing poisonous matter has been gathered together, and treated in collective pretreatment facilities the wastewater is discharged into the public sewerage system, and then its organic pollutants are treated by a sewage treatment plant.

c. Direct discharge into the public sewerage system

Organic wastewater which can be treated by a sewage treatment plant is directly discharged into the public sewerage system.

To cope with industrial wastewater in existing urban area, factories have been required to furnish their own pretreatment facilities and the maintenance and operation of the installations have been assured by city authority inspectors. With this method, it is difficult for minor enterprises short of financial ability and technology to maintain and operate the pretreatment facilities in full accordance with the standards of industrial wastewater. Therefore, to make the factory development free from environmental pollution in the land No.3, the treatment of industrial wastewater by a collective pretreatment method has been adopted positively for the minor enterprises.

The advantage of collective pretreatment method resides in the fact that, on the enterprise side, the wastewater of a plural number of factories can be gathered together and treated economically, and, on the administration side, as close monitoring of industrial wastewater can be carried out, the efficient and precise instructions as a result of the monitoring, can be performed.

Examples of wastewater suitable for the collective pretreatment method are as follows:

- a. The pickling and plating processes, of which the treating process is complex, and the treating facilities are both expensive to install and difficult to maintain and operate.
- b. The wastewater of printing and dyeing, of which the treating is anticipated to become more difficult for the new wastewater standards which have been laid down concerning the regulation COD of the total amount and the color and so on.

The wastewater of chemical industries is not suitable for the collective pretreatment method, for its properties are not uniform and it is impossible to mix simply the wastewater from a number of different processes.

Based on the above consideration, the treating method of industrial wastewater in the land No.3 has been determined as in Table 1.

Table 1. Treating method of industrial wastewater

Treating method	Type of wastewater
COLLECTIVE PRETREATMENT	Plating wastewater Pickling wastewater Printing and dyeing wastewater
Individual pretreat- ment	Wastewater with oil content Wastewater of chemical industries, etc.
Direct discharge into the public sewerage system.	Wastewater of foodstuff processing, etc.

2.4.4 Arrangement of factories in the land No.3

When thinking of the arrangement of factories in the land No.3, the basic way of thinking is as follows.

- Factories of the same type of industry and of related types of industry are arranged in a mass.
- On the mountain side close to the housing developments, factories of light work which produce little vibration noise are sited, and on the sea side, heavy industries or factories which have much work outdoors are sited.
- Factories of the three types of industry of plating, pickling, and printing/dyeing which require the collective pretreatment of their wastewater are arranged in a mass. Further, the location is assured at the center of the land No.3 as the three types of industry are related to many other types of industry such as metallic, mechanical, and textile industries.

Based on the above conditions, the plan for the arrangement of factories has been determined as illustrated in Fig. 7.

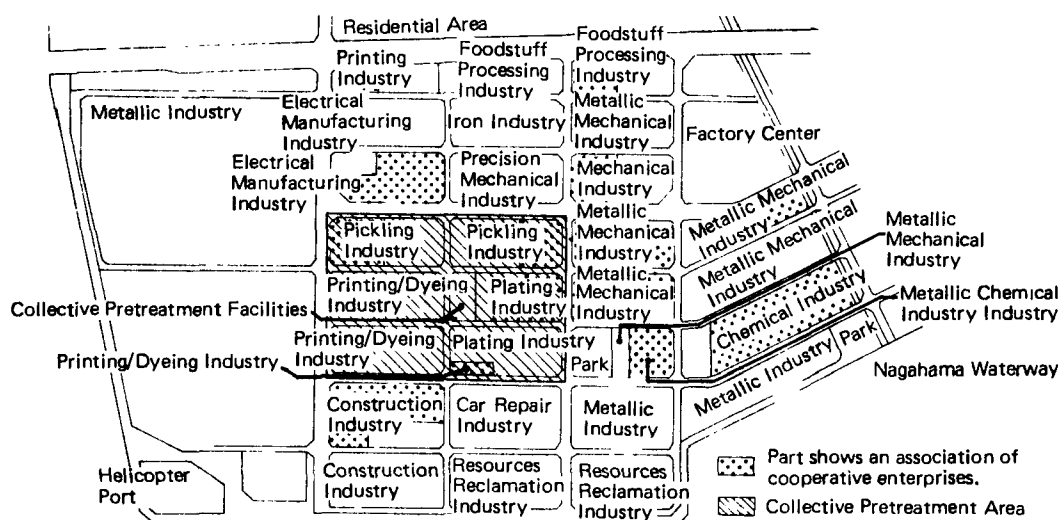


Fig. 7 Layout of enterprises in Kanazawa reclaimed land No. 3

3. Collective Pretreatment Facilities

3.1 Basic plan

3.1.1 Object factories

The object factories for the collective pretreatment are factories which meet the following two conditions.

- Minor enterprises (Capital: Under 100 million Yen. Number of employees: Under 300)
- Plating industry, pickling industry, and printing/dyeing industry

Based on the purposes of the reclamation project at Kanazawa, those factories which meet the above conditions have been selected out of 680 factories which should be relocated to the land No.3. The selection is shown in Fig.8, and its result in Table 2. Further Table 3 shows the result of classification of the scale of object enterprises by the number of employees. Table 3 indicates that minor enterprises where employees number under 20 account for 64%, and small enterprises where

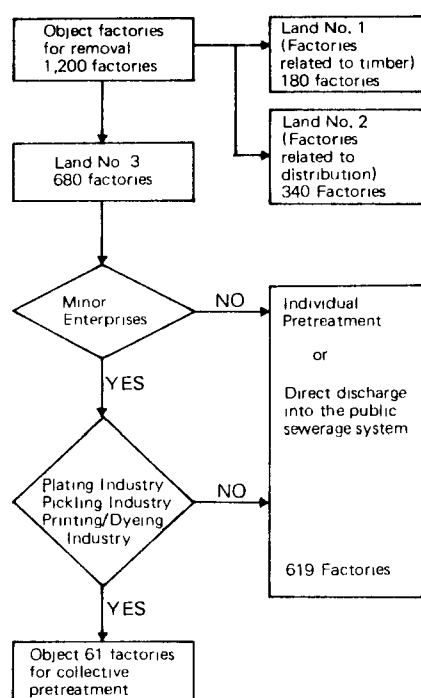


Fig. 8 Flow of the selection process of object factories

employees number under 100 account for 94%, from which one of the character of the reclamation project can be judged. Therefore, the collective pretreatment of industrial wastewater has great significance.

3.1.2 Site area of object factories

Although the collective pretreatment area is located nearly at the center of the Land No.3, when determining the site area of object factories, the desired area of 22 removal desiring factories was asked through a questionnaire. However, there was no means with which to investigate the desired site area of the future removal factories. The ratio of the increased site area to an existing site area when a factory is removed (That is, the desired site area after removal/the existing site area) was determined for the removal desiring factories by the types of industries. By multiplying the ratio by the existing site area of the future removal factories, the site area after removal of the future removal factories has been calculated. The result is shown in Table 4.

3.1.3 Wastewater collecting systems

The process wastewater of the plating industry is divided into three types by its nature: cyanide type containing cyan, chromium type containing hexavalent chromium, and acid/alkali type containing heavy metals. And can be divided into two

Table 2. Number of object factories for the collective pretreatment of industrial wastewater

Type of Industry	Removal Desiring factories	Future Removal Enterprises		Total
		*	**	
Plating Industry	9	2	16	27
Pickling Industry	7	4	5	16
Printing/Dyeing Industry	6	3	9	18
Total	22	9	30	61

*Enterprises which are located on the sites where public undertakings are scheduled

**Enterprises involving the occurrence of environmental pollution

Table 3. The scale of enterprises by the number of employees

Employees Type of Industry	Under 20 Persons	20-49 Persons	50-99 Persons	100-199 Persons	200-299 Persons	Total
Plating Industry	22	2	1	1	1	27
Pickling Industry	11	3	1	1	0	16
Printing/Dyeing Industry	6	7	4	1	0	18
Total	(64%) 39	(20%) 12	(10%) 6	(5%) 3	(1%) 1	(100%) 61

Table 4. Site area of object factories (m²)

Type of Industry	Plating Industry	Pickling Industry	Printing/Dyeing Industry	Total
Item				
Removal Desiring Factories	38,822	27,720	29,717	96,259
Future Removal Factories	6,548	18,682	42,898	68,128
Total	45,370	46,402	72,615	164,387

types by its concentration: the wastewater of low concentration that is discharged continuously, and that of high concentration that is intermittently discharged.

The process wastewater of the pickling industry is only a acid/alkali type containing heavy metals, and divided into two types by its concentration, one of them is low concentration type which is discharged continuously, and the other is high concentration type discharged intermittently. The process wastewater of printing/dyeing industry can not be divided by its nature and concentration.

As the wastewater of high concentration, which is small in quantity, has a large load amount because of its high concentration, it should be treated carefully. As the wastewater of high cyanic concentration and that of low cyanic concentration must be treated by different methods as described later, they are separately collected. For chromic and acid/alkali, the special treating process of the wastewater of high concentration has not yet been practised, so that the wastewater is treated by pouring it uniformly into the wastewater of low concentration. Therefore, the wastewater of high concentration and that of low concentration in chromium and acid/alkaline are mixed in factories, and then discharged.

Table 6 sets out the wastewater collecting systems by types of industry.

Table 5. Types of wastewater by the type of industry

Type of Industry	Nature of Wastewater	Concentration	
		Intermittent Wastewater	Continuous Wastewater
Plating Industry	Cyan Chromium Heavy Metals Acid/Alkali	High Concentration ditto ditto	Low Concentration ditto ditto
Pickling Industry	Acid/Alkali	ditto	ditto
Printing/Dyeing	BOD CDD Color	—	ditto

Table 6. Wastewater collecting systems by types of industry

Type of Industry	Wastewater Collecting System	The Number of Systems
Plating Industry	High Concentration Cyanide Line Low Concentration Cyanide Line Chromic Line Acid/Alkali Line	4
Pickling Industry	Acid/Alkali Line	1
Printing/Dyeing Industry	Printing/Dyeing Line	1

3.1.4 Planned water quantity

The wastewater quantity from the object factories of the collective pretreatment has been calculated by determining the unit daily mean wastewater quantity, and the ratios of the daily maximum wastewater quantity and the hourly maximum wastewater quantity to the daily mean wastewater quantity according to the types of industry, and the nature and concentration of the wastewater.

a. The quantity of the low concentration wastewater

① The unit daily mean wastewater quantity

The daily mean wastewater quantity per unit area of the site has been taken as $1.67 \text{ m}^3/\text{day} \cdot 100 \text{ m}^2$ for the plating industry, $1.07 \text{ m}^3/\text{day} \cdot 100 \text{ m}^2$ for the pickling industry, and $3.60 \text{ m}^3/\text{day} \cdot 100 \text{ m}^2$ for the printing/dyeing industry, from the data of industrial statistics, the result of water supply quantity, and the investigated value of the removal desiring factories, etc.

- ② The ratio of the daily maximum wastewater quantity to the daily mean wastewater quantity
Based on the result of water supply quantity of the removal desiring factories for the period of the past three years (1976 to 1978), the ratio of the daily maximum value to the daily mean value has been determined to be 1.5.
 - ③ The ratio of the hourly maximum wastewater quantity to the daily mean wastewater quantity
The ratio of the hourly maximum wastewater quantity to the daily mean wastewater quantity per unit time has been determined to be 2.5 for the plating and the pickling industries, and to be 2.0 for the printing/dyeing industry, referring to the result values of Torihama industrial wastewater treatment plant, and the examples of five plants in other cities.
 - ④ The ratio of wastewater quantities by types of wastewater in the plating industry
Although the wastewater of the plating industry is divided into three types, viz. cyanic, chromic, and acid/alkali, the ratio of the wastewater quantities of the three types has been taken as 23% for cyanic, 7% for chromic, and 70% for acid/alkali through the hearing investigation of the removal desiring factories.
- b. The ratio of the high concentration wastewater quantity to the low concentration wastewater quantity
For the calculation of the quantity of the high concentration wastewater which is discharged intermittently from the plating and pickling industries, there are the following methods.
- ① It is determined from the volume of bath tubs in factories and the replaced frequency by the type of wastewater.
High concentration wastewater quantity = volume of bath tubs × replaced frequency
 - ② Based on the low concentration wastewater quantity, it is calculated by determining the ratio of the high concentration wastewater quantity to the low concentration wastewater quantity.
High concentration wastewater quantity = low concentration wastewater quantity × the ratio
 - ③ It is investigated through direct hearings from the removal desiring factories.
This planning has adopted the method No.2. The ratio for the various types has been determined to be 2.2% for the cyanide, 0.4% for the chromic, 2.0% for the acid/alkali of the plating industry, and 0.5% for the acid/alkali of the pickling industry by the data of investigated values of the removal desiring factories, etc.
- From the above, Fig. 9 shows the calculating process of planned wastewater quantities, and Table 7 shows the quantities calculated.

Table 7. Planned wastewater quantities

(m³/day)

By types of Industry By type of wastewater		Low conc. wastewater q'ty			High conc. wastewater quantity daily max. (5)	Storm(6) water etc.	Planned wastewater q'ty	
		Daily Mean (3)	Daily Max (4)	Hourly Max			Daily Mean 3+5+6	Daily Max. 4+5+6
Plating Industry	High conc. Cyanic Wastewater	—	—	—	3	0	—	3
	Low conc. Cyanic Wastewater	85	130	215	—	0	85	130
	Chromic Wastewater	49	74	125	0.3	0	50	75
	Acid/Alkali Wastewater (1)	325	488	810	10	0	335	498
Pickling Industry	Acid/Alkali(2) Wastewater	495	743	1,237	4	74	573	821
Acid/Alkali Wastewater	(1)+(2) Total	820	1,231	2,047	14	74	910	1,320
Printing/Dyeing Ind.	Printing/Dyeing Wastewater	2,614	3,921	5,228	—	392	3,010	4,400

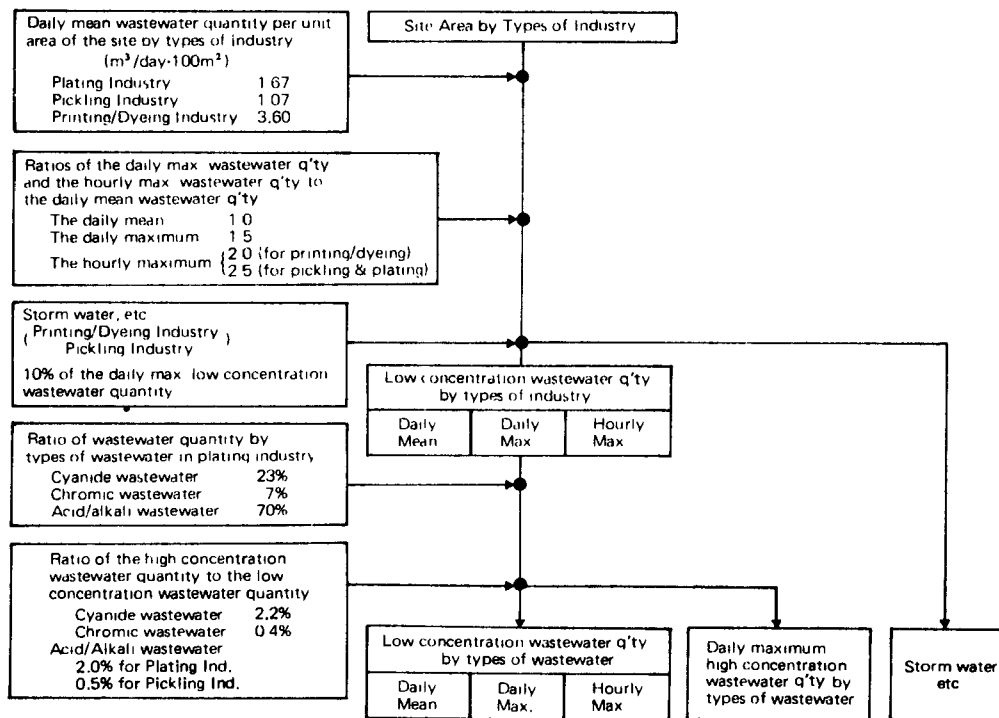


Fig. 9 Flow of the calculation of planned wastewater quantities

3.1.5 Planned water quality

a. The quality of the low concentration wastewater

Table 8 shows the quality of the low concentration wastewater due to various kinds of data of the investigation, etc. of factories in the city.

Table 8. The water quality of the low concentration wastewater

Type of Wastewater	Cyanic Wastewater (as cyan)		Chromic Wastewater (as hexavalent chromium)		Acid/Alkali Wastewater (as the total amount of heavy metals)		(mg/l)			
							Printing/Dyeing Wastewater		Mean	Max.
	Mean	Max.	Mean	Max.	Mean	Max.	BOD	COD		
Item										
Mean value of all data	104	340	113	430	158	514	354	—	509	—
The second highest value of data	170	400	104	580	227	683	540	—	820	—
Adopted value	170	400	110	580	230	690	360	540	510	820

For the reason that the mean and maximum concentrations of cyanic, chromic, and acid/alkali wastewater will tend to become higher hereafter as the factories will no doubt attempt to make a water saving, the facilities need to be given a certain degree of spare capacity, hence the second highest value of examined data has been adopted.

As there is little data in which the mean and maximum values of the concentration of printing/dyeing wastewater are separately described, the mean value of all data has been taken as the mean concentration, and the second highest value of the data has been taken as the maximum concentration.

b. The quality of the high concentration wastewater

As the major part of the high concentration wastewater is the replaced liquid of bath tubs, the concentration of plating baths and acid baths, etc. have been examined, and the mean value of all data has been adopted as shown in Table 9.

Table 9. Concentration of bath liquids

Cyanic Wastewater			Chromic Wastewater			Acid/Alkali Wastewater		
Type	Concentration		Type	Concentration		Type	Concentration	
Copper Plating	Cyan	60,000–80,000	Chromium plating	6 valent chro.	120,000	Nickel plating	Nickel	80,000
	pH	12.2 – 12.6		pH	2.4		Zinc	12,000
Zinc Plating	Cyan	20,000–40,000	Chromium plating (Barrel)	6 valent chro.	180,000	Picric acid waste liquid	Iron	70,000
				pH	2.0		pH	1.2
Zinc Plating (Low cyanide bath)	Cyan	10,000	Hard chromium plating	6 valent chro.	250,000	Alkaline fat	pH	13.2
				pH	1.8			
			Hard chromium plating	6 valent chro.	220,000	Electrolytic fat removing liquid	pH	13 – 14
				pH	2.0			
			Chromate bath	6 valent chro.	50,000 – 250,000			
Adopted value	Cyan	40,000	Adopted value	6 valent chro.	210,000	Adopted value	Total metals	70,000
	pH	13		pH	2		pH Acid	1
							Alkali	13

c. Planned water quality

As mentioned above, with both chromic and acid/alkali types, the high concentration wastewater and low concentration wastewater are mixed in factories, and then discharged into each sewage collection system. When mixed, the concentration is calculated by the following formula.

$$\begin{aligned}
 \text{Mean Concentration} &= \frac{\text{Mean of q'ty of the low concentration wastewater} \times \text{Mean concentration of the low concentration wastewater} + \text{High concentration wastewater q'ty} \times \text{Concentration of the high concentration wastewater}}{\text{Mean q'ty of the low concentration wastewater} + \text{High concentration wastewater q'ty}} \\
 \text{The maximum concentration} &= \frac{\text{Mean q'ty of the low concentration wastewater} \times \text{The max. concentration of the low concentration wastewater} + \text{High concentration wastewater q'ty} \times \text{Concentration of the high concentration wastewater}}{\text{Mean q'ty of the low concentration wastewater} + \text{High concentration wastewater q'ty}}
 \end{aligned}$$

The planned water quality determined in the above way is shown in Table 10.

3.1.6 Arrangement of factories

The site area of object factories of the collective pretreatment is approximately 172,500 m², corresponding to 6 blocks of divided sections of the land (1 block = 32,800 to 34,000 m²). The conditions to arrange the pretreatment facilities and removed factories effectively and economically in the six blocks can be arranged as follows.

Table 10. Planned water quality (mg/l)

Type of Wastewater	Item of Water Quality	Mean Value	Max. Value
High concentration cyanic wastewater	Cyan	40,000	—
Low concentration cyanic wastewater	Cyan	170	400
Chromic wastewater	6 valent chromium	1,400	2,000
Acid/Alkali wastewater	Total amount of heavy metals	1,500	2,000
Printing/Dyeing wastewater	BOD	400	550
	COD	550	850

- The collective pretreatment facilities are arranged in the center to minimize the total length of sewers. That is because the cost of construction and of maintenance and operation can be reduced, measures for accidents such as the breakage of the sewers and prevention of the inlets for storm water, etc. can be minimized.
- Factories are arranged in a mass by types of industry. That is because the economical activities of factories can be made smooth, and also sewage collection systems can be prevented from being congested.
- Plating factories are arranged close to the collective pretreatment facilities. That is because the construction cost of sewers can be reduced in plating factories which require four sewage collection systems.
- Sites for future removal factories are also arranged in a mass by types of industry for the purpose of making the division of factory sites more flexible because future removal factories have not decided individual sights and shapes yet.

Considering the above conditions, the location of the pretreatment facilities and the arrangement of factories have been determined as shown in Fig. 10. In addition, as a model example of cooperation, the example of the cooperative association of Yokohama plating factory development is shown in Fig. 11.

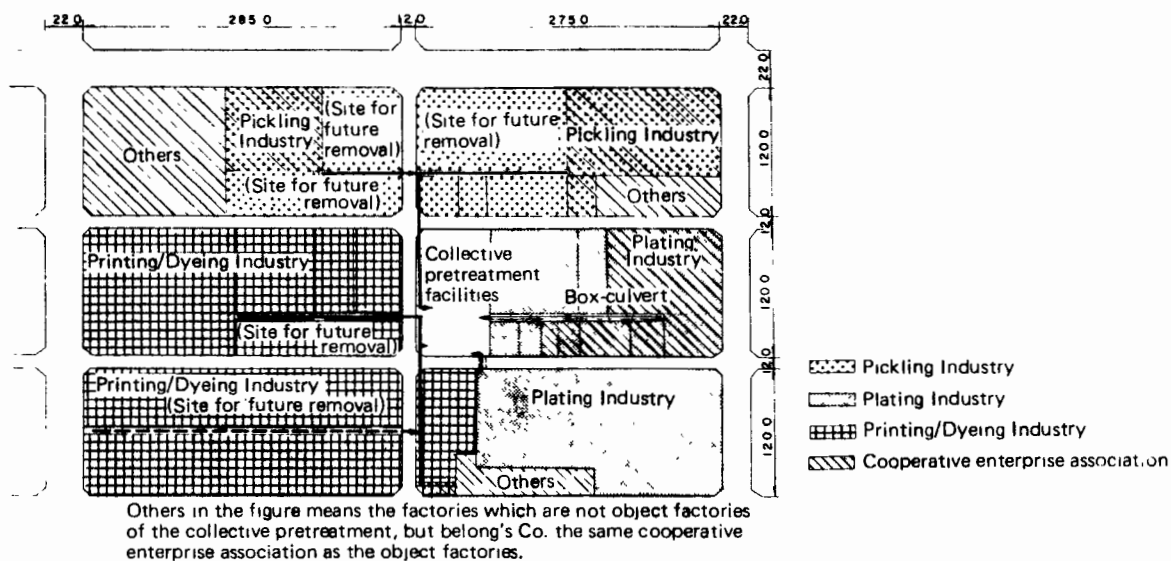


Fig. 10 Plan view of the planned arrangement of drainage pipework of factories and pretreatment facilities and sewer planning in the collective pretreatment area

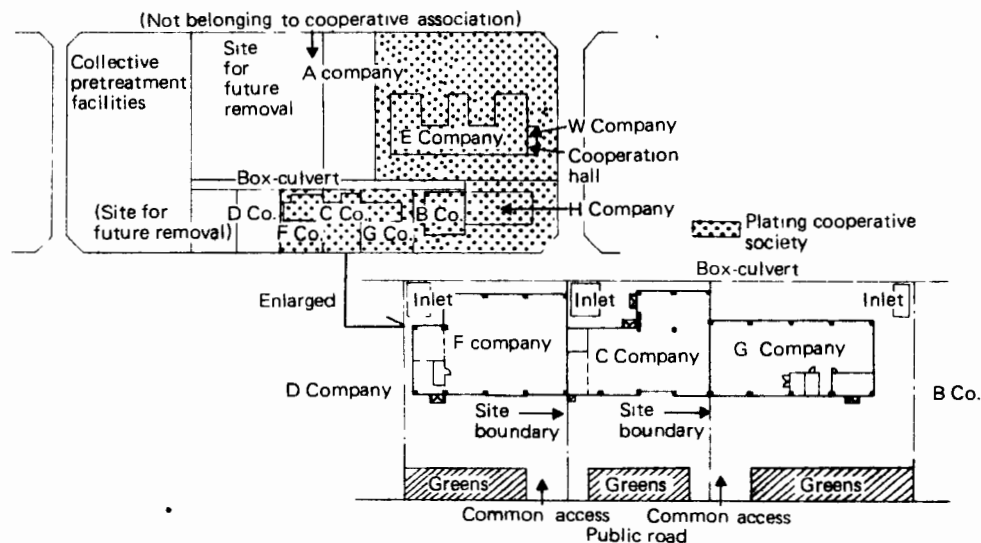


Fig. 11 Plan diagram of the cooperative association of Yokohama plating factory development.

3.2 Sewers

3.2.1 Sewers plan

As mentioned above, wastewater flows into the pretreatment facilities through one line for the type of industry in the printing/dyeing and pickling industries, and through four lines in the plating industry. As each enterprise is arranged in a mass by types of industry (refer to Fig. 10, ground plan), the exclusive sites for drain pipes have been assured along the site of each enterprise to minimize the total length of the sewers, that is advantageous for both the maintenance of the sewers and the construction of lateral sewers for the future removal factories. In the plating industry, for the following reasons, four systems of piping have been accommodated in a box-culvert as shown in Fig. 12.

- a. The whole aspect of the piping is accessible for inspection, and early detection of trouble is also possible. It is possible to rectify faults immediately, which will keep costs low.
- b. There is no possibility of incorrect installation, and the construction period of lateral sewers for future removal factories can also be reduced, and construction costs will be low.
- c. In the case of a leakage accident, poisonous wastewater will not penetrate into the earth, so that damage can be prevented.
- d. From both the view-points of maintenance and the construction cost, the box-culvert is more advantageous compared with open channels. Further, as piping material hard PVC pipes can be used.

3.2.2 Incidental equipment

a. Manholes

On manholes, two types of lining are provided divided into the invert part affected directly by wastewater and the wall body part. The invert part is made with a glass fiber reinforced plastics lining (FRP lining) which has excellent chemical resistance, and is also reliable in the respect of water tightness, the wall body part is made of two sheets of tar epoxy resin lining with a layer of glass wool between them because they are inferior in chemical resistance.

b. Inlet

When connecting the wastewater from factories to the sewers, an inlet is installed. The inlet is provided with the three functions of mixing (except with the cyanic wastewater, to mix sufficiently the two types of wastewater of low concentration and high concentration), metering, and monitoring to determine accurately the quantity and nature of the process wastewater and to make maintenance and operation easy.

Fig. 13 shows the structure of the inlet.

c. Flow meter

For determining accurately the process wastewater quantity of each enterprise in order to collect the appropriate charges, flow meters as shown in Table 11 are installed.

For the plating industry and pickling industry, turbine flow meters have been adopted for the following reasons.

- ① They are easy to maintain.
- ② If faults occur, they are easy to repair, inspect, and change.
- ③ For chemical resistant flow meters, they are relatively inexpensive.
- ④ They have been used for years as industrial flow meters.
- ⑤ It is possible for them to measure a small quantity (to the extent of $0.1 \text{ m}^3 / \text{h}$), too.

Further, for a high concentration cyanic wastewater, the measuring of flow quantity is carried out by a constant speed pump as the flow quantity is small. Moreover, in the wastewater of the printing/dyeing industry, foreign matter such as waste threads is commonly found, and fluctuation in flow quantity is large. Therefore, electromagnetic flow meters have been adopted which can measure the flow rate electrically.

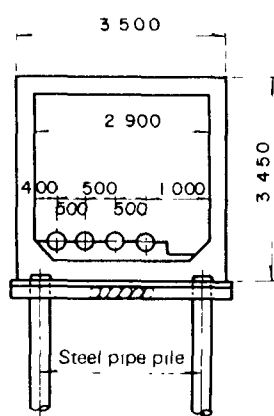


Fig. 12 Box-Culvert

Table 11. Flow meters by types of industry

Item		Flow meter	Material
Type of industry			
Plating industry	High concentration cyan	Constant speed pump	PVC
	Others	Turbine flow meter	SUS
Pickling industry		Turbine flow meter	SUS
Printing/dyeing industry		Electromagnetic flow meter	SUS

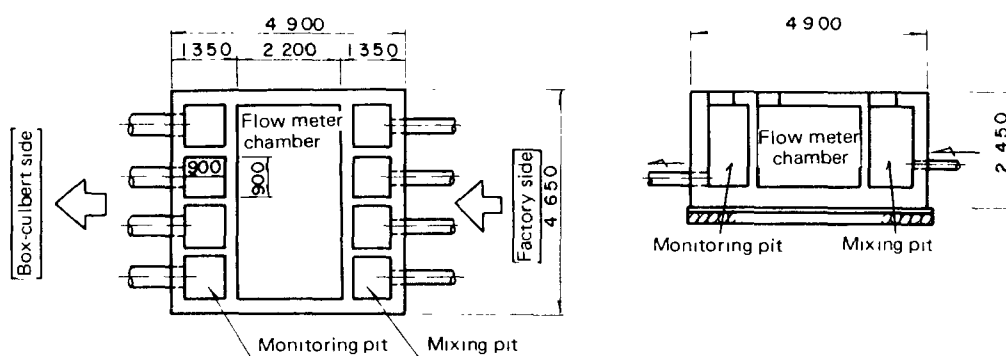


Fig. 13 Inlet

3.3 Pretreatment facilities

3.3.1 Design policy

When designing the treatment facilities, the following items have been considered as constituting the basic concept.

- The wastewater collected through five lines (high concentration cyan, low concentration cyan, chromium, acid/alkali, and printing/dyeing) is separately treated in each line for the following reasons.

① Water treatment

- Plating pickling wastewaters are different from printing/dyeing wastewater in nature and the chemical property of sludge, so they are separated.
- Low concentration cyanic wastewater is subjected to oxidation treatment, and chromic wastewater to reduction treatment which is the reverse of oxidation treatment, and is different from oxidation treatment in chemicals used, so the cyanic line is separated from chromic line.
- Ferric ions present in acid/alkali wastewater make it impossible to control the injection quantity of chemicals in the reduction reaction of chromium, so the chromic line is separated from acid/alkali line.
- As mentioned earlier, high concentration cyanic wastewater has the level of concentration reduced by electrolysis treatment in advance, and then

it is put into the low concentration cyanic wastewater line.

② Sludge treatment

- As printing/dyeing line sludge contains no poisonous materials, and can be treated in the same manner as domestic sewerage sludge, it is separated from other sludge containing metals, etc.
 - As aftermentioned chromic and acid/alkali line sludge can be successfully solidified with asphalt etc. The cyanic line sludge is not suitable for solidification, because cyan might be dissolved from solidified sludge.
 - So cyanic line sludge is separated from others.
 - As chromic line sludge can be recycled as valuable matter, it is separated from the sludge of other lines.
 - Therefore, acid/alkali sludge is separately treated too.
- b. The flowing time of wastewater from each enterprise into the treatment facilities is taken as being 10 hours, and the operating time of the facilities is taken as being 24 hours with continuous treatment so that the scale of the facilities is not made too large and uneconomical, by which the necessary capacity and ability of the treatment facilities are determined.
- c. The treated water is recycled in an attempt to make a saving on running costs.
- d. Automation is thoroughly carried out for the certainty of treatment and for minimizing the number of operators necessary.

3.3.2 Design conditions

Based on basic planned numerical values, design conditions have been established as follows:

a. Treated water quantity

The treatment ability of the facilities should be the planned daily maximum wastewater quantity as described in the basic plan (3.1).

b. Quality of water flowing into the treatment facilities

The treatment ability of the facilities should be the planned water quality determined in the basic plan (3.1).

c. Object items of treatment and the quality of treated water.

The quality of treated water should meet the numerical values which permit the treated water to be discharged into the public sewerage system based on the discharge standards of sewerage law, and the regulations of Kanagawa prefecture, etc. Table 12 shows the items of water quality, and Table 13 the quality of treated water.

Table 12. Treatment items

Wastewater	Water quality items to be treated
High concentration cyanide wastewater Low concentration cyanide wastewater	pH, cyan, copper, zinc, soluble iron, soluble manganese, nickel, lead.
Chromic wastewater	pH, hexavalent chromium, total chromium, copper, zinc, soluble iron, soluble manganese, nickel, lead.
Acid/alkali wastewater	pH, total chromium (except hexavalent chromium), copper, zinc, soluble iron, soluble manganese, nickel, lead.
Printing/dyeing wastewater	pH, appearance (coloration degree), extracted matter by n-hexane extracts, temperature, iodine consumption, BOD, COD, SS.

Table 13. Quality of treated water

Cyanide, chromic, and acid/alkali line	Water quality items	pH	Cyan	Hex-avalent chromium	Total chromium	Copper	Zinc	Soluble iron	Soluble manganese	Nickel	Lead
	Concentration	5-9	mg/l 1	mg/l 0.5	mg/l 2	mg/l 1	mg/l 1	mg/l 3	mg/l 1	mg/l 1	mg/l 1
Printing/dyeing line	Water quality items	pH	Appearance (Coloration degree)	n-hexane extracts		Temperature		Iodine consumption	COD	BOD	SS
	Concentration	5-9	200°	mg/l 30	mg/l 5	°C 45		mg/l 220	mg/l 200	mg/l 300	mg/l 300

3.3.3 Selection of treating methods

The most suitable treating methods of wastewater have been selected considering the certainty, economy, operability, and results, etc. of the treatment from among the methods now in practical use.

a. Plating and pickling wastewaters

① Cyanide

As the high concentration cyanic wastewater in the planning contains almost no iron and chromium, the electrolysis method is adopted.

As the electrolysis method becomes less efficient when the cyanic concentration is low, it is most suitable for the wastewater of high cyanic concentration, and also it tolerates fluctuation in water quality and is easy to maintain and operate. In addition heavy metals in the wastewater are precipitated on electrodes, the quantity of sludge occurrence being reduced.

For the treatment of low concentration cyan, an alkaline chlorination method has been adopted. The method can automatically control the treatment with a simple instrument which is easily to maintain and operate, and can ensure the stable quality of the treated water. However, it is characterized by the increased use quantity of chemicals when the cyanic concentration is high. Further, after high concentration cyan has been reduced to the cyanic concentration of about 1000 mg/l by electrolysis, it is mixed with the low cyanic concentration wastewater, and the mixture is eventually treated to give a cyanic concentration of under 1 mg/l.

② Hexavalent chromium

For the treatment of hexavalent chromium, a chemical reduction method employing sodium bisulfite has been adopted. The method can be automatically controlled with a simple instrument which is easy to maintain and operate, and is capable of ensuring the stable quality of treated water.

At the present time, although there is an ion exchange method for the recovery of hexavalent chromium, for the treatment of the chromium the chemical reduction method has been adopted because the recycling planning for reutilizing the chromium has not yet been determined in the factories of participating enterprises.

③ Heavy metals

Plating and pickling wastewater contains no special metals such as mercury, etc., and also in Torihama industrial wastewater treatment plant the

good quality of treated water is obtained by a coagulation precipitation method due to the formation of hydroxides. Therefore, for the treatment of heavy metals, the coagulation precipitation method due to the formation of hydroxides has been adopted. The method can be automatically controlled with a simple instrument, which is easy to maintain and operate, and is capable of ensuring the stable quality of treated water.

b. Printing/dyeing wastewater

For the decoloration treatment of printing/dyeing wastewater, the target water quality can be attained only by physicochemical treatment. However, for organic matter of COD, etc., the target water quality cannot be attained only by a physicochemical treatment, so that as an after treatment, a biological treatment which is effective for the elimination of organic matter is carried out. For the physicochemical method, a floatation method has been adopted which can tolerate fluctuation in water quality, is easy to maintain and operate, and requires only a small site area. For the biological treatment, a rotary disc method has been adopted which is easy to maintain and operate, and is economical to run.

c. Treatment of sludge product from each wastewater treating process Table 14 shows the estimated quantity of sludge produced.

Table 14. Quantity of sludge produced by various types of wastewater at each process

		Settled sludge		Thickened sludge		Sludge cake	
		Quantity (m ³ /day)	Moisture content(%)	Quantity (m ³ /day)	Moisture content(%)	Quantity (t/day)	Moisture content(%)
Cyanide wastewater		8.0	99.2	3.2	98	0.3	75
Chromium wastewater		26.0	99.2	10.4	98	0.83	75
Acid/alkali wastewater		446.6	99.2	178.6	98	14.3	75
Printing/dyeing wastewater	Floatation sludge	250.0	98	—	—	36.2	85
	Rotary disc sludge	53.2	99.2	21.3	98		

Note: The moisture content of settled sludge and thickened sludge is based on the results obtained by Torihama industrial wastewater treatment plant.

As cyanic sludge and chromic sludge are only produced in small quantities, two batchwise thickeners which can remain idle for a considerable time are installed, and are used alternately every other day and also fulfilling the role of a sludge storage tank.

As acid/alkali sludge is produced in large quantities, a continuous thickener which may have a small capacity is used for the sludge.

Further, as the rotary disc sludge of the printing/dyeing line, which is organic sludge, only occurs in small quantities a batchwise thickener is adopted for the rotary disc.

From the previous results, for the sludge of cyanic, chromic, and acid/alkali lines pressure filters have been adopted as dewatering facilities, and for the sludge of the printing/dyeing line centrifuges have been adopted as dewatering facilities.

3.3.4 Treating process and main equipment

Fig. 14 shows each treating process. Table 15 to 20 give a synopsis and list the equipment content of each treating process.

Table 15. Treating process of a high concentration cyanide line

Equipment Name	Synopsis	Retention time and capacity	Treating conditions	Chemical to be used	Specification
Pumping well which also serves as a storage tank.	As high concentration cyanic wastewater may be discharged instantaneously in a mass from each factory, the capacity of storage tank is increased to ensure uniformity and homogeneity in water quantity and water quality.	7 days 11m ³ x 2 tanks	—	—	An FRP tank is installed inside a concrete tank.
Electrolytic cell	Cyan is electrolyzed.	22 hours Batchwise treatment 1.5m ³ x 2 tanks	Voltage appr. 10V Current 6 to 10A	—	Made of steel plate, and the inside has a rubber lining.

Table 16. Treating process of a low concentration cyanic line

Equipment Name	Synopsis	Retention time and Capacity	Treating Conditions	Chemicals to be used	Specifications
Pumping Well	Wastewater which flows into a pumping well from sewers is pumped up into a storage tank.	15 minutes 8m ³	—	—	A concrete tank, of which the inside wall is coated with tar epoxy resin.
Storage Tank	Wastewater which flows into a pumping and homogeneity in water quantity and water quality to stabilize the operation of the treatment facilities.	1.6 days 115m ³ x 2 tanks	—	—	Ditto
Primary Oxidation Tank	The primary oxidation reaction of cyan is carried out in the tank.	1 hour 6.2m ³	pH over 10 ORP 300-350 mV	5% Caustic Soda, 12% Sodium Hypochlorite	A steel plate tank, of which the inside wall is coated with tar epoxy resin
Staying Tank	It enhances the reaction efficiency of the primary oxidation reaction, and also prevents the short circuit flow from the primary oxidation tank to the secondary oxidation tank.	1 hour 6.2m ³	—	—	Ditto
Secondary Oxidation Tank	The secondary oxidation reaction is carried out in the tank.	1 hour 6.2m ³	pH under 8 ORP over 650mV	5% Sulfuric Acid, 12% Sodium Hypochlorite	Ditto
pH Control Tank	pH is adjusted to form the hydroxide of heavy metals.	30 minutes 3.7m ³	pH 9-10	5% Caustic Soda	Ditto
Coagulation Tank	Floccules are coarsened to give the formed hydroxides of heavy metals a better property of sedimentation	10 minutes 1.0m ³	—	0.1% Polymer Coagulant	Ditto
Sedimentation Tank	It separates liquid and solid by the sedimentation of floccules of heavy metals.	3.5 hours 22m ³ overflow rate 20m ³ /m ² ·day	—	—	Ditto
Pumping Well	From here, the supernatant in the sedimentation tank is pumped up into a sand filter.	15 minutes 1.4m ³	—	—	Concrete tank
Sand Filter	Fine floccules in the supernatant of the sedimentation tank are removed by the sand filter.	φ1.2m x 2.4m 2 filters	—	—	A pressure type rapid filter, filtration rate 150m ³ /m ² ·day, filtration area 1.13m ² /unit
Sludge Thickener	Sludge separated in the sedimentation tank is thickened here.	1 day 9.7m ³ x 2 tanks	—	—	Batchwise, sludge for a day is put into a tank, a concrete tank.
Dewatering Filter	The thickened sludge is dewatered by the dewatering filters.	Operation for 6 hrs 1 unit of equip.	—	—	A pressure filter, Filtration rate 1.8kg/m ² ·hr Filter area 22m ² /unit

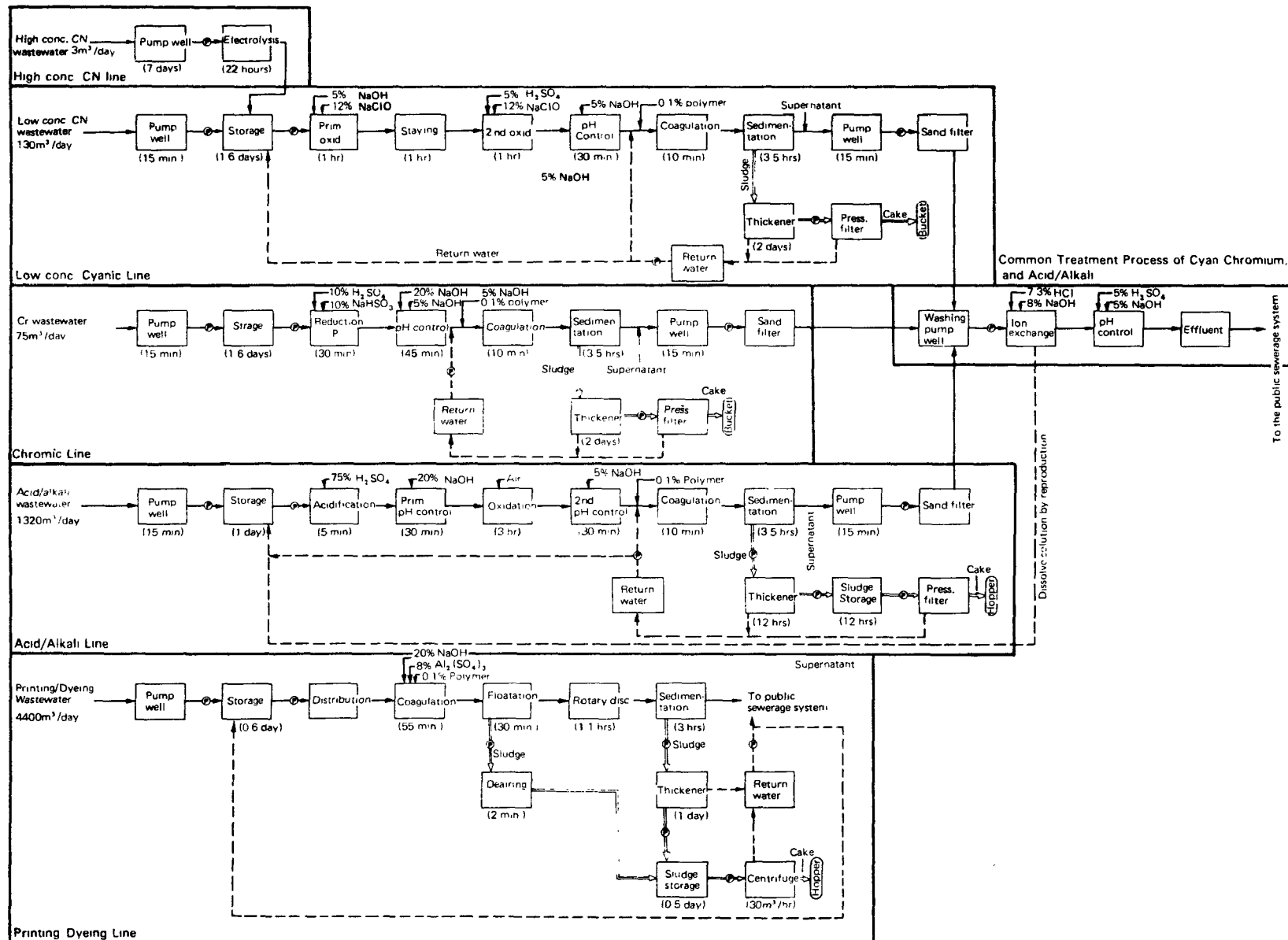


Fig. 14 Treatment flow sheet

Table 17. Treating process of a chromic line

Equipment Name	Synopsis	Retention time and Capacity	Treating Conditions	Chemicals to be used	Specifications
Pumping Well	Wastewater which flows into a pumping well from sewers is pumped up into a storage tank.	15 minutes 3.5m ³	————	————	A concrete tank with a FRP lining inside.
Storage Tank	The tank ensures uniformity and homogeneity in water quantity and water quality to stabilize the operation of the treatment facilities.	1.6 days 60m ³ x 2 tanks	————	————	Ditto
Reduction Tank	Hexavalent chromium is reduced to trivalent chromium.	30 minutes 1.7m ³	pH under 3 ORP under 250mV	10% Sulfuric Acid, 10% Sodium Bisulfite	A steel plate tank with a rubber lining
pH Control Tank	pH is adjusted to form the hydroxide of heavy metals. However, for the reason that the pH of the overflowing water from a reduction tank differs markedly from the pH setting, and the formed hydroxide of chromium is redissolved when alkali is injected in excess, the control process has been divided into two stages.	1st stage 15 minutes 2nd stage 30 minutes 1.7m ³	pH 7 pH 9	20% Caustic Soda 50% Caustic Soda	A steel plate tank with a rubber lining
Coagulation Tank	Floccules are coarsened in the tank to give the formed hydroxides of heavy metals a better property of sedimentation.	10 minutes 1m ³	————	0.1% Polymer Coagulant	A steel plate tank of which the inside wall is coated with tar epoxy resin
Sedimentation Tank	It separates liquid and solid by the sedimentation of floccules of heavy metals.	3.5 hours 16m ³ Overflow rate 20m ³ /m ² .day	————	————	A steel plate tank of which the inside wall is coated with tar epoxy resin
Pumping Well	From here, supernatant in the sedimentation tank is pumped up into a sand filter.	15 minutes 0.8m ³	————	————	A concrete tank
Sand Filter	Fine floccules in the supernatant of the sedimentation tank are removed by a sand filter.	φ0.9m x 2.4m 2 filters	————	————	Pressure type rapid filters, Filtration area 0.63m ² /filter, Filtration rate 150m ³ /m ² .day
Sludge Thickener	Sludge separated in the sedimentation tank is thickened here	1 day 27m ³ x 2 tanks	————	————	Batchwise. Sludge for a day is put into a tank. Two concrete tanks
Dewatering Filter	The dewatering filter dewater the thickened sludge.	Operation for 6 hrs 1 unit of equip.	————	————	A pressure filter, Filtration area 22m ² /unit Filtration rate 1.8kg/m ² .hr

Table 18. Treating process of a Acid/Alkali line

Equipment Name	Synopsis	Retention time and Capacity	Treating Conditions	Chemicals to be used	Specifications
Pumping Well	Wastewater which flows into a pumping well from sewers is pumped up into a storage tank.	15 minutes 54m ³	—	—	A concrete tank with an FRP lining inside.
Storage Tank	The tank ensures uniformity and homogeneity in water quantity and water quality to stabilize the operation of the treatment facilities.	1 day, 910m ³ x 2 tanks	—	—	Ditto
Acidification Tank	When the water flowing into the line has become strong alkaline, it is no longer possible to control the process. As a measure to cope with such an emergency, an acidification tank is installed, in which wastewater into the line is acidified.	5 minutes 6.9m ³	pH under 7	75% Sulphuric Acid	A steel plate tank with a rubber lining
Primary pH Control Tank	pH is controlled to form the hydroxide of heavy metals.	30 minutes 39.3m ³	pH 9-10	20% Caustic Soda	Ditto
Oxidation Tank	Bivalent iron is oxidized to trivalent iron by aeration.	3 hours 230.6m ³	—	—	A steel plate tank coated with tar epoxy resin inside, with a surface aerator.
Secondary pH Control Tank	The difference between the pH of the water flowing into the line and the pH accurately only by the primary pH control tank. For compensating for the primary pH control and for further adjusting the pH value which is changed by the aeration oxidation, the secondary pH control is carried out.	30 minutes 39.3m ³	pH 9-10	5% Caustic Soda	Ditto
Coagulation Tank	Floccules are coarsened to give the formed hydroxides of heavy metals a better property of sedimentation.	10 minutes 12.7m ³	—	0.1% Polymer Coagulant	Ditto
Sedimentation Tank	The tank separates liquid and solid by the sedimentation of floccules of heavy metals.	3.5 hours 285m ³ Overflow rate 20m ³ /m ² ·day	—	—	Circular radial flow type. Made of concrete.
Pumping Well	From here, the supernatant in the sedimentation tank is pumped up into a sand filter.	15 minutes 14.4m ³	—	—	Made of concrete
Sand Filter	Fine floccules in the supernatant of the sedimentation tank are removed by a sand filter.	φ3.5m x 2.4m 3 filters	—	—	Pressure type rapid filtration, Filtration area 9.8m ² /filter, Filtration rate 150m ³ /m ² ·day
Sludge Thickener	Sludge separated in the sedimentation tank is thickened here.	12 hours 235m ³	Solid matter load 60kg/m ² ·day	—	Continuous type, Made of concrete
Sludge Storage Tank	The thickened sludge is stored in the tank before dewatering.	For a half day 95m ³	—	—	Made of concrete
Dewatering Filter	The filter dewateres the thickened sludge.	Operation for 6 hours 2 units	—	—	Pressure filter, filter area 154m ² /unit, filtration rate 2.0 kg/m ² ·hour

Table 19. Common process of Cyanic, Chromic, and Acid/Alkali lines

Equipment Name	Synopsis	Retention time and Capacity	Treating Conditions	Chemicals to be used	Specifications
Pumping Well for Reverse Washing	The pumping well receives water flowing out of the sand filter of each line and stores it in readiness for reverse washing of each sand filter. It also serves as a pumping well for pumping water out into each ion exchange column.	20 minutes 95m ³	_____	_____	A concrete tank
Ion Exchange Column	It will adsorb heavy metals that cannot be removed by the coagulation precipitation method due to the formation of hydroxides of heavy metals.	1520m ³ /day 2 units SV20m ³ / m ³ ·h	_____	When reproduced 7.2% Hydrochloric Acid, 8% caustic Soda	A cylindrical vertical pressure tank made of steel plate, which has a rubber lining inside
Final pH Control Tank	As pH is 8 to 10 when the hydroxides of metals are formed, the pH is controlled to heavy before discharging the water.	30 minutes 36.6m ³	pH 7	5% Caustic Soda 5% Sulfuric Acid	A concrete tank
Discharge Water Tank	It is used to store created water which may be needed for the purpose of dissolving chemicals, etc.	80m ³ x 2 tanks	_____	_____	A concrete tank

Table 20. Treating process of a Printing/Dyeing line

Equipment Name	Synopsis	Retention time and Capacity	Treating Conditions	Chemicals to be used	Specifications
Pumping Well	Wastewater which flows into a pumping well from sewers is pumped out into a storage tank.	15 minutes 145.2m ³	_____	_____	A concrete tank
Storage Tank	It stores wastewater that flows into it from factories for a period of 10 hours; the water is then treated continuously for a period of 24 hours. Also, surface aeration is carried out for the homogeneity of water quality and for the prevention of putrefaction of wastewater.	For 0.6 day 2810m ³	_____	_____	A concrete tank Surface aerator (float type)
Coagulation Tank	In the tank, the pH of wastewater is adjusted, and flocculant is added to flocculate fine suspended matter.	30 minutes 50m ³ x 2 tanks	pH 7	20% Caustic Soda, 8% Alumearth 0.1% Polymer Coagulants	A steel plate tank, of which the inner wall is coated with tar epoxy
Floatation Tank	It mixes flocculated particles with pressure water to float and separate them.	*	_____	_____	A steel plate tank, of which the inner wall is coated with tar epoxy.
Rotary Disc Tank	In the tank, the organic matter in wastewater is decomposed.	1.3 hours, BOD disc surface loading 20g/m ² ·day	_____	_____	Tank is made of concrete, and disk is made of plastics
Sedimentation Tank	In this tank is precipitated and separated the excess sludge which occurs in the rotary disc tank. In addition, before the sludge flows into the sedimentation tank, polymer coagulants are added to the sludge to give it a better property of sedimentation.	2.1 hours Water area load 35m ³ /m ² ·day 191m ³ x 2 tanks	_____	_____	Circular radial flow type Made of concrete
Sludge Thickener	It thickens the sludge separated in the sedimentation tank.	1 day 53m ³ x 2 tanks	_____	_____	Batchwise, sludge for one day is put into a tank. Made of concrete
Sludge Storage Tank	It stores floatation sludge and thickened sludge before the dewatering process.	One day 147m ³	_____	_____	A concrete tank
Dewatering Equipment	It dewater sludge	Operation for 20 hours 2 units	_____	0.2% Polymer Coagulants	Centrifuge 10 m ³ / hour/unit

* 25 minutes
air/solid ratio 0.04kg air/kg-ss
pressure 4kg-F/cm²
Floatation rate 4m/hr
Water quantity
circulation rate 0.76
74m³ x 2 tanks

3.3.5 Planning for the arrangement of facilities

Conditions and the way of thinking for carrying out planning for the arrangement of facilities are as follows:

- a. Conditions for installing buildings on the reclaimed land are as follows:
 - ① Ratio of the area of green tracts of land – over 20 per cent (in the case of public facilities)
 - ② The building-to-land ratio – under 60 per cent
 - ③ Volume ratio – under 200 per cent
 - ④ Height limitation – under 31 meter
- b. The way of thinking for the arrangement of facilities are as follows:
 - ① As the treating processes are divided into many lines such as cyanic, chromic, acid/alkali, and printing/dyeing line, they are broadly divided into metallic lines and printing/dyeing line, and further, the metallic lines are arranged so that each treating process is distinguished distinctly.
 - ② Each treating process is arranged almost in a straight line from inflow to out-flow to make the flow of treatment as short as possible.
 - ③ Water flow between each unit of facilities should relay on gravity flow wherever possible to achieve energy saving.
 - ④ The printing/dyeing system is arranged to facilitate an additive installation.
 - ⑤ For ease of bringing in of equipment, and chemicals, and taking out of sludge, etc., a circumferential road is suggested for the site.

Fig. 15 shows the arrangement of facilities based on the above conditions and way of thinking.

As shown in Fig. 15, on the inflow side, storage facilities are installed, and in its upper part, each treating facility is installed.

On the outflow side, an electromechanical room is provided, in which sludge treating facilities and facilities for control and monitoring are arranged, and space related to a living room is ensured, too.

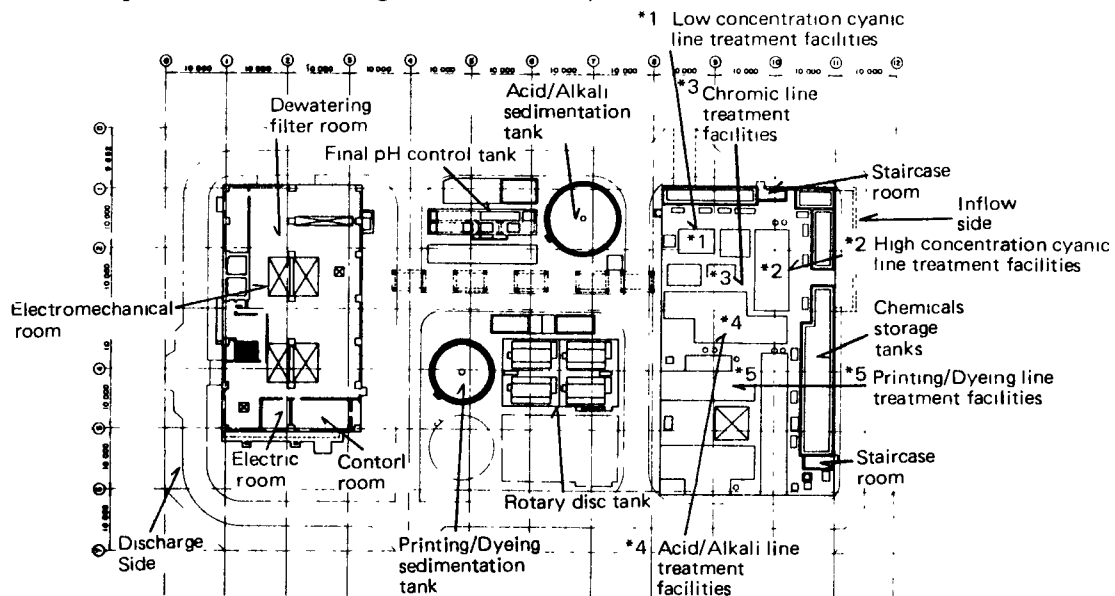


Fig. 15 Plan of facilities

3.3.6 Control

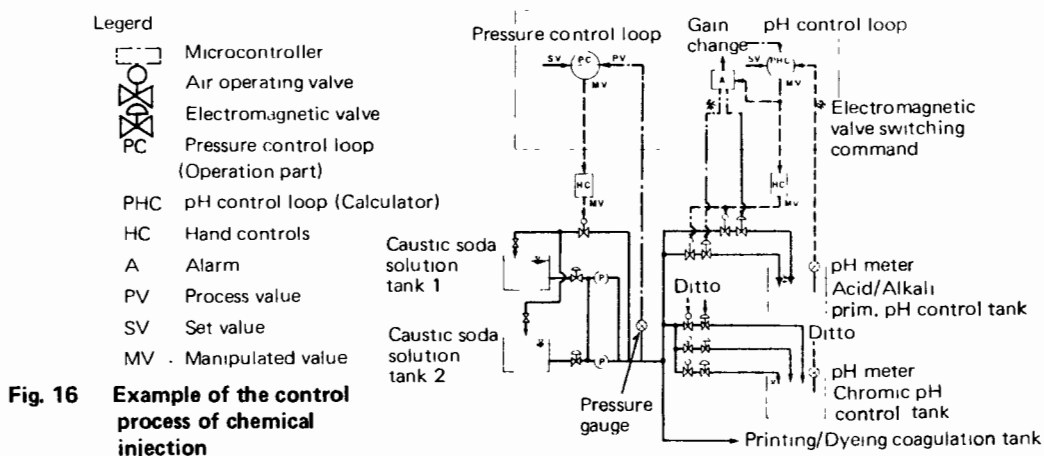
As the facilities treat industrial wastewater by five separate lines, many machines are incorporated such as small capacity machines of lift pumps and stirrers, etc., and filtering equipment and electromagnetic valves, etc. are also provided. It would require a great deal of manpower to operate manually many of these machines separately, therefore, a control circuit has been designed to operate each machine automatically by a water level control or timer, etc.

However, if necessary, each machine can be manually operated from a central control room, and when inspecting any machine, it can be operated from a local control panel close by.

The operation condition and failure condition of machines are automatically displayed individually in the central control room; a system which makes one man control possible is being established. When injecting chemicals into each treating tank, the necessary quantity of chemicals according to proportional calculation operation control which takes pH and ORP as targets is automatically injected by a digital controller.

In addition, all information of the facilities is transmitted to Kanagawa sewage treatment plant by remote monitoring equipment. Therefore, it is possible even for the operators of the sewage treatment plant to monitor the pretreatment facilities. Fig. 16 shows the control flow of the chemical injection system of 20 percent caustic soda as an example of automatic control systems.

The injection of chemicals by the conventional ON-OFF control system has not been adopted in this case, instead, a continuous control system has been adopted whereby chemical injection is carried out by the proportional calculation operation control of which the detective tip of pH or ORP for increased controllability. The injection points of chemicals are so many that a system for controlling the injection quantity by installing a variable speed motor at each injection point is questionable from the view point of construction cost and maintenance and control. Therefore, in the adopted injection system, two injection pumps have been installed for each chemical, and the control of the injection quantity has been carried out by air operating valves. Further, for such chemical injection of a large quantity range as caustic soda injection into a pH control tank of the chromic line, the necessary quantity of chemicals is injected through a plural number of installed injection pipes of different caliber. Moreover, the pressure control loop of Fig. 16 controls pressure in the supply pipes of a chemical at a constant level to stabilize the pH control loop.



3.3.7 Sludge disposal plan

As dewatered cake from the printing/dyeing line is organic sludge, it is possible to dispose of the sludge in the same manner as domestic sewage sludge. On the other hand, dewatered cake from each wastewater treatment line of cyanic, chromic, and acid/alkali lines contains a large quantity of cyan, chromium, copper, lead, and other heavy metals, so that its disposal should be carefully considered.

At the present time, in Japan laws and regulations prescribe judgement standards for the final disposal of industrial waste matter. Although it is considered to be possible to dispose of the dewatered cake containing metals, etc. by land disposal from the result of Torihama industrial wastewater treatment plant, the possibility of solidification disposal methods of higher safety is being examined from a long term prospect.

Table 21 shows the outline of solidification experiments with asphalt, cement, and plastics, and by melting, etc. using the dewatered cake of Torihama industrial wastewater plant.

When each solidification method is evaluated only by the result of dissolving out tests without taking into account economical factors, the solidification method by melting is the most safe for all kinds of sludge. But, for the sludge of chromic, and acid/alkali lines containing no cyan, the solidification method with asphalt or cement can be adopted. Although the adoption of the melting solidification method as a technologically permanent disposal method is the most desirable, its high cost is the question, for all the construction costs, etc., are, in general, to be borne by the associated enterprises. Therefore, in Kanazawa industrial wastewater collective pre-treatment facilities, the solidification with asphalt, etc. of the dewatered cake from chromic, and acid/alkali lines is executed, and the melt solidification method of the dewatered cake from the cyanic line is being examined.

Table 21. Technical comparison of various solidification methods

Basic conditions
1) Sludge cake containing heavy metals, etc., Treatment quantity, 1t/day
Ditto Moisture content 75%
Ditto Organic matter content, 20%
2) Working hours, 6 hours

Type		Micro Wave Melting			Solidification with Asphalt			Solidification with Cement			Solidification with Plastics		
Item													
Explanation of Methods		When irradiating micro-waves are directed onto an equal weight mixtures of sample sludge and the incinerated ash of sewage sludge, mainly the silica matter in the ash is fused and crystallized			Asphalt forms films on the surface of sample sludge, and when the sludge is molded by compression, space between particles within the solidified matter is reduced, the contact of heavy metals with environmental water being prevented			Hydration reaction between cement constituents and water, and sealants confine harmful matter within the sludge, and at the same time solidify the sludge. To increase the strength of the solidified matter, the addition of red clay or incinerated ash is needed			After sludge has been dried, and crushed, it is mixed with plastics, and then molded with an extruder.		
Mixing Ratio		Sludge cake 1 Incinerated ash 1 (Wet weight)			Sludge cake 1 Asphalt 0.37 (Dry weight)			Sludge cake 1 Cement 0.32 Sealant 0.022 (Wet, wt) Red clay 0.32			Sludge cake: Waste plastics 1 0.5 (Wet weight)		
Dissolving Out Test	Analysts Item	CN	C _r *6	T-Cr	CN	C _r *6	T-Cr	CN	C _r *6	T-Cr	CN	C _r *6	T-Cr
	Solidified Matter	○	○	○	×	○	○	×	○	○	×	○	○
	In water 6 months	○	○	○	×	○	○	×	○	○	×	○	○
	80°C, 20 days exposure	○	○	○	×	○	○	×	○	○	×	○	○
Operation & maintenance cost (¥ 1,000/t (wet cake))		140			67			50			91		
Construction (¥ 10 cost Mil)		43			24			17			24		

Note 1) Success (○), Failure (×) judgement
2) Success judgement standards . CN under 1mg/v, C_r*6 under 1.5mg/g, T-Cr under 1.5mg/v

4. Bearing of the Expenses

The industrial wastewater collective pretreatment facilities in this district are constructed, maintained, and operated by Yokohama city, and all the necessary expenses are borne by the associated enterprises in principle.

4.1 Construction costs and financial sources

The total construction cost of the facilities amounts to about 3.9 billion yen. The breakdown of the construction cost by types of construction is shown in Table 22.

Enterprises must deal responsibly with environmental pollution, etc., which occurs as a direct result of their activities for they are the source of the pollution. But, in particular, it is in many ways difficult for minor and mini-enterprises to solve the pollution problems effectively by themselves.

The Environmental Pollution Control Service Corporation is dealing with the allocation of funds for installing environmental pollution control facilities. Table 23 shows the financing system of the corporation. With the system, object enterprises for financing include local public bodies, and lending conditions are advantageous for minor enterprises and local public bodies. Therefore, Yokohama city has determined to make available the funds for the construction cost of the facilities.

Table 22. Enterprise expenses by types of construction

Unit: 1,000 yen

Type of Construction		Cost
Sewers		473,000
Treatment Facilities	Concrete Structure	646,000
	Building	443,000
	Machines	1,096,000
	Electric Equipment	570,000
Site Expenses		633,640
Office work Expenses		22,000
Total		3,883,640

Table 23 Financing system of the Environmental Pollution Control Service Corporation

Object facilities of financing	Objects of financing	Ratio of financing	Term of redemption (Including unredeemable term)		Unredeemable term		Rate of interest (Annual int. rate)			
			Machines or equipment	Others	Machines or equipment	Others	For three years after financing	After the fourth year after financing		
Collective environmental pollution control facilities	Minor enterprises and local public bodies	Under 80%	Within 10 years	Within 20 years	Within 1 year	Within 3 years	6.35%	6.85%		
	Major enterprises	Under 70%					7.65%	7.85%		
Individual environmental pollution control facilities	Minor enterprises and local public bodies	Under 80%	Within 10 years		Within 1 year		7.00%			
	Major enterprises	Under 50%					7.85%			

4.2 Calculation for sharing of costs

4.2.1 Distribution of shares in construction costs

The object enterprises of this project are of various types ranging from enterprises having only printing/dyeing wastewater or only acid/alkali wastewater to enterprises having two, three, or four kinds of acid/alkali, chromic, high concentration cyanic, and low concentration cyanic wastewater. Therefore, the sharing of construction costs has been calculated as follows.

- ① At first, out of the each cost of sewers, equipments and land, the portions which could be considered to be apparently depended upon the kinds of industries were calculated.

- (2) The cost concerned with metal plating industry was, further, subdivided into four portions by the properties of the wastewater.
- (3) Then, the cost of common facilities concerned with several industries and/or wastewater treatment processes was divided into several parts based on the factors such as wastewater quantity and quality load, electric load, sludge weight etc.
- (4) Finally, the total cost was allotted to each enterprise based on their discharging volume by the kinds of wastewater.

4.2.2 Calculation method of individual share in the costs

The individual share in the costs of each enterprise has been calculated from the following formulae.

$$\begin{aligned}
 \text{Printing/Dyeing Ind.} &= (\text{Share in the cost of Printing/Dyeing Industry}) \times \left(\frac{\text{The water quantity of each enterprises}}{\text{Planned water quantity (400 m}^3/\text{day)}} \right) \\
 \text{Pickling Industry} &= (\text{Share in the cost of Pickling Industry}) \times \left(\frac{\text{The water quantity of each enterprises}}{\text{Planned water quantity (747 m}^3/\text{day)}} \right) \\
 \text{Plating Industry} &= \left(\text{Share in the cost distribution of the high concentration cyanic line} \right) \times \left(\frac{\text{The water quantity of each enterprises}}{\text{Planned water quantity (3 m}^3/\text{day)}} \right) \\
 &\quad + \left(\text{Share in the cost distribution of the low concentration cyanic line} \right) \times \left(\frac{\text{The water quantity of each enterprises}}{\text{Planned water quantity (130 m}^3/\text{day)}} \right) \\
 &\quad + \left(\text{Share in the cost distribution of the chromic line} \right) \times \left(\frac{\text{The water quantity of each enterprises}}{\text{Planned water quantity (75 m}^3/\text{day)}} \right) \\
 &\quad + \left(\text{Share in the cost distribution of the acid/alkali line} \right) \times \left(\frac{\text{The water quantity of each enterprises}}{\text{Planned water quantity (498 m}^3/\text{day)}} \right)
 \end{aligned}$$

Table 24 shows the construction costs per one cubic meter of wastewater calculated from the above, which is a little less expensive than if each enterprise was to build separately its own pretreatment facilities.

Table 24. Construction cost (¥ 1000/m³)

Types of Industry	Plating Industry					Pickling Industry	Printing/Dyeing Industry
	High Concentration Cyanic Line	Low Concentration Cyanic Line	Chromic Line	Acid/Alkali Line	Average		
Construction Cost	41,300	1,550	3,230	1,070	1,560	1,270	420

4.2.3 Contract of bearing of the construction costs

Yokohama city has drawn up a contract for bearing of the construction costs and so on with each enterprise. The main points of the contract are as follows.

a. Bearing of the costs

The city draws up a contract of wastewater quantity by types of wastewater with each enterprise. Based on the contracted water quantity, the share in the costs of each enterprise is calculated as above. The city entered into a contract for the calculated share in the costs with each enterprise.

b. Jurisdiction of property

It is feared that the maintenance of the property of the collective pretreatment facilities by 61 associated enterprises in cooperation will cause dispute or other

troubles on management relating to the property in the future. Therefore, it has been decided that the property of the facilities belongs to Yokohama city and it is to be maintained by the city for the purpose of its smooth maintenance and operation.

c. Sanctions against nonfulfillment of the contract

Against delay in payment of shares in the construction costs by enterprises, payments not exceeding an annual rate of 14.6 percent shall be made.

d. Contract of bearing of the maintenance costs

Although the contract of bearing of the construction costs has been made on the premise that it is followed by the contract of the maintenance and operation of the facilities after the completion of construction, the latter contract will be drawn up separately.

Further, against enterprises that have no prospect of paying their share of the construction costs, it is possible to buy back their land based on the sales contract of land.

4.2.4 Maintenance and operation costs

All the maintenance and operation costs of the collective pretreatment facilities should be borne by the associated enterprises, and their allotment will be determined by total load amount taking account of water quality. Table 25 shows the value inferred by types of wastewater and by types of industry of the maintenance and operation costs per one cubic meter of treated water.

Table 25. Maintenance and operation cost

	By Types of Wastewater					By Types of Industry		
	High Concentration Cyanide Wastewater	Low Concentration Cyanide Wastewater	Chromic Wastewater	Acid/Alkali Wastewater	Printing/Dyeing Wastewater	Plating Industry	Pickling Industry	Printing/Dyeing Industry
Treatment Unit Price (Yen/ m ³)	50,000	900	3,000	700	300	1,200	700	300

5. Maintenance and Operation System

5.1 Limitation prohibition and obligations of associated enterprises

It has been decided that the collective pretreatment facilities shall belong to Yokohama city as mentioned above. Their property falls under the jurisdiction of the economic bureau, and the sewerage works bureau operates the facilities on a commission basis.

Between the city and removing enterprises, private contracts of the use of the collective pretreatment facilities are entered into. Although the effluent from the facilities is controlled by the sewerage law, wastewater from each enterprise is not directly controlled by the law. Therefore, it is feared that the responsibility of enterprises are not distinct, and at the same time the collective pretreatment facilities cannot be properly maintained and operated, so that clauses of limitation, prohibition and obligations of each enterprise are prescribed in the said contracts.

a. Limitations

- ① The limitation of wastewater quantity and its concentration
The daily maximum wastewater quantity and maximum concentration are limited so that the collective pretreatment facilities is not overloaded.
- ② The limitation of chemicals used in working processes
Of chemicals used in working processes, those which may obstruct the pretreatment (for example, EDTA, rochelle salt, etc.) are limited in use quantity.

b. Prohibition

- ① The prohibition of discharging wastewater that cannot be treated by the collective pretreatment; wastewater containing phenols, cadmium, mercury, and oils, etc. is not permitted to be discharged.
- ② Wastewater that is in danger of damaging the facilities; wastewater containing gasoline which will cause fire, and containing mud and sand which will cause the blockade of sewers, and hot wastewater.

c. Obligations

- ① The job of installing flowmeters
Integral flow meters are installed in the factory sites to check industrial wastewater volume discharged.
- ② The work of constructing monitoring pits
Monitoring pits for sampling of industrial wastewater are installed on the factory sites.
- ③ The task of maintaining flow meters and monitoring pits
- ④ The task of measuring the water quality
The water quality of industrial wastewater is periodically analyzed.

5.2 Monitoring by the city

Monitoring methods are as follows.

a. Monitoring of the quantity of wastewater

By flow meters installed by types of wastewater on the site of factories, wastewater is always monitored whether the quantity is under the contracted quantity or not.

b. Monitoring of the concentration of wastewater

On the site of factories are installed monitoring inlets, into which the wastewater of the city goes and from which the wastewater at all times is samples and analyzed to monitor whether its concentration is under the contracted concentration or not.

c. Monitoring of factories

For monitoring of factories as with the monitoring of general industrial premises in the city, enterprises are ordered to present a written application describing the content of work, chemicals to be used, registered facilities, and discharging methods, etc. for approval by the city. For the routine inspection of factories, it is a policy to monitor factories by placing emphasis on the type of factories requiring high frequency of monitoring.

6. Conclusion

This enterprising project is not designed on the same lines as established regarding previous measures to cope with industrial wastewater, but aims at progressive and permanent measures to cope with industrial wastewater while putting forward the "town building of Yokohama" toward the 21st century.

The characteristics of the project are as follows.

a. The aspect of preventive measures

- ① Although Kanazawa industrial wastewater collective pretreatment facilities have been planned as a measure to cope with wastewater of minor enterprises as in the case of Torihama industrial wastewater treatment plant, Torihama plant were planned with the object of enterprises that had arbitrarily gone into the reclaimed land, whereas Kanazawa facilities have been planned as a permanent measure to cope with industrial wastewater involving minor enterprises that are scattered in the city, and which are to be removed immediately or in the future.
- ② The city authorities decide on the locations of factories based on the nature of their wastewater.
- ③ In place of the conventional method that relies on money being collected in advance from object enterprises as the source of revenue, Kanazawa facilities are financed directly by the city authorities, through funds made available by the Environmental Pollution Control Service Corporation.
- ④ For bringing up minor enterprises, cooperation of enterprises going into the reclaimed land was recommended with the aim of advancing factory facilities and effective utilization of sites.

b. The aspect of facilities

- ① Kanazawa facilities have adopted a method to divide the cyanic into high and low concentration cyanic line, and the chromic wastewater into a chromic line and acid/alkali line including sludge treating process to treat them separately.
- ② The automatic control system has now reached the almost fully automatic state.

c. Other items

- ① Although sewers were laid under public roads in Torihama area, considering maintenance and inspection, coping with failures, and connection of lateral sewers, etc., sewers were laid in exclusive sites for wastewater pipes in Kanazawa facilities, and a box-culvert method was adopted in the plating industry which required four wastewater pipes.
- ② Although the wastewater quantity of each factory was determined based on the quantity of city water used at Torihama, the actual discharged wastewater quantity was confirmed by a flowmeter installed at the exit of wastewater for that purpose at Kanazawa. Monitoring inlets were also installed to make it possible to check the water quality at any time.

Acknowledgement

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AGRICULTURAL USE OF SEWAGE SLUDGE

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PART I BEHAVIOR OF HEAVY METALS IN SOIL AND
THEIR UPTAKE BY PLANTS

1. PURPOSE

In agricultural use of sewage sludge, one of the important areas of study is the behavior of heavy metals in the soil and their uptake by plants when sewage sludge has been utilized for agriculture. The plants growing in the soil where sewage sludge has been used, absorb soil- and sludge- born heavy metals into their leaves and stems.

Accordingly, in order to clarify which of these two types of heavy metals is absorbed by plants, experiments have been conducted using synthetic sludge including heavy metals tagged with radioactive isotopes ^{65}Zn and ^{109}Cd . The main purpose of the experiments is to investigate the following two subjects:

- (1) Uptake of added heavy metals by barley.
- (2) Relationship between the added heavy metals and their uptake by barley.

2. MATERIALS AND METHOD

2.1 Preparation of Synthetic Sludges

^{65}Zn - ^{109}Cd double tagged synthetic sludges were prepared in the same manner as in the previous report ⁽¹⁾, using a batch type activated sludge process. At the same time, non-tagged sludges having small amounts of heavy metals were also prepared. The culture medium used for preparing the synthetic sludges was a glutamine culture medium.

The glutamine culture medium was composed of 15 g of glutamic acid, 15 g of glucose, 5 g of ammonium chloride, 0.7 g of calcium chloride, 0.5 g of magnesium sulphate, 0.7 g of potassium chloride, 1.1 g of potassium dihydrogenphosphate, 2.9 g of sodium hydrogenphosphate (dodehydrate) and 0.1 g ferric chloride (hexahydrate). The above culture medium was regarded as one unit, and 1 - 1.5 units of the medium were added per day for cultivation.

For double tagged synthetic sludges, two types having low and high concentrations of heavy metals were prepared, and their concentrations and radioactivities are shown in Table 1-1.

Table 1-1 Heavy metal concentrations and radioactivities in tagged synthetic sludges

Type of sludge prepared	Zinc (per gram-Sludge)		Cadmium (per gram-Sludge)	
	μgZn	^{65}Zn cpm	μgCd	^{109}Cd cpm
Low-concentration sludge (L)	1252.4	$1,840 \times 10^5$	6.40	$1,408 \times 10^5$
High-concentration sludge (H)	2462.1	$1,572 \times 10^5$	10.65	$1,342 \times 10^5$

Further, a non-tagged low-concentration sludge was also prepared, and its heavy metal concentrations were $0.82\mu\text{gCd/g-Sludge}$ and $5.51\mu\text{gZn/g-Sludge}$ respectively.

2.2 Preparation of Tracer Heavy Metal Solutions

The solutions containing heavy metals as tracers were prepared by dissolving metallic chlorides having heavy metals equivalent to 4.25 times the concentration of heavy metals in the sludge shown in Table 1-1 and by adding 0.1mCi of ^{65}Zn and 0.1mCi of ^{109}Cd to the solution. 1 ml of this solution contains the same amount of heavy metals as are in the sludge when 0.5% of sludge based on 850 g of air-dried soil (=4.25 g of sludge) is added to the soil. Two types of tracer heavy metal solutions having high and low concentrations were prepared, and their heavy metal concentrations and radioactivities are shown in Table 1-2.

Table 1-2 Heavy metal concentrations and radioactivities in tracer heavy metal solutions

Type of solution prepared	Zinc (1 ml)		Cadmium (1 ml)	
	μgZn	^{65}Zn cpm	μgCd	^{109}Cd cpm
Low-concentration solution	5,322.7	$6,395 \times 10^5$	27.2	$4,474 \times 10^5$
High-concentration solution	10,464.0	$5,891 \times 10^5$	45.3	$4,259 \times 10^5$

2.3 Preparation Tracer Heavy Metal-absorbed Sludge

To 4.25 g of non-tagged sludge was added 1 ml of tracer-heavy metal solution prepared in 2.2, and it was mixed thoroughly, then, air-dried for two days.

2.4 Soil

The soil used for this investigation was a fine air-dried soil of Tanashi volcanic surface soil, which was of the same kind as in the previous report⁽¹⁾. The total cadmium and zinc contents were 0.39 $\mu\text{g/g-Soil}$ and 1.05 $\mu\text{g/g-Soil}$ respectively.

2.5 Method

850 g of the air-dried soil was sampled, and the tagged synthetic sludge, tracer heavy metal-absorbed sludge or tracer-heavy metal solution was added and thoroughly mixed. After that, the water content of the sampled soil was adjusted to make it up to about 60% of its maximum water content, and it was transferred to a pot for three months of incubation without adding any plants. The temperatures employed were 25°C during the day and 20°C during the night (day - 12 hrs; night - 12 hrs), and the water content of the soil was adjusted every 2-3 days. The method of adding tracer heavy metals to the soil is shown in Table 1-3.

In this soil, 20 grains of barley were sown and cultivated for 4 weeks under the same conditions as those of the incubation. After cultivation, the barley plants were cut down and their roots were removed from the soil, then, 50 g of air-dried soil was sampled.

Next, three months after the completion of the first cultivation test, exactly the same cultivation test for barley was conducted again.

Table 1-3 Method of adding tracer heavy metals

Added heavy metals or sludge		Method of adding tracer heavy metals			
Heavy metal concentration	Added amount of sludge (% of air-dried soil)	Sc	Si	I	Is
Low concentration (S _L)	0.5	○	○	○	○
	1.0	○	○	○	○
High concentration (S _H)	0.5	○	○		
	1.0	○	○		

Sc: Heavy metal-tagged synthetic sludge to be added to air-dried soil.

Si: Tracer heavy metal-absorbed sludge to be added to air-dried soil.

I : Tracer heavy metal to be directly added to air-dried soil.

Is: Non-tagged sludge to be added after tracer heavy metals have been directly added to air-dried soil.

2.6 Extraction of Tracer Heavy Metals from Soil

From the soil samples collected after the completion of each cultivation test, heavy metals were extracted by the following three methods, and the radioactivities of ^{65}Zn and ^{109}Cd in the extracts were measured.

(1) DTPA extraction

20 ml of mixed solution of 0.005M DTPA (Diethylenetriamine-pentaacetic acid), 0.01M CaCl_2 and 0.1M TEA (Tetraethyleneamine) (pH 7.3) was added to 10 g of air-dried soil, which was shaken for 2 hours.

(2) Extraction with 0.1 N hydrochloric acid

50 ml of 0.1 N HCl solution was added to 10 g of air-dried soil, which was shaken for 1 hour.

(3) Extraction with 1.0 N nitric acid

50 ml of 1.0 N HNO_3 solution was added to 5 g of air-dried soil, which was shaken for 2 hours.

2.7 Determination of Radioactivity and Heavy Metals in Leaves and Stems of Barley

To determine the radioactivity in the leaves and stems of barley, the sample was air-dried, put into a measuring tube as it was, and measured by the autowell gamma system. For the determination of heavy metals, the sample was dissolved by the wet method, then heavy metals extracted with 0.05 M acetic acid buffer (pH 4.5), and measured by a trace-metal analyzer equipped with dropping mercury electrode. The added heavy metals contents of the leaves and stems were determined by dividing the radioactivity in the leaves and stems by the specific radioactivity of the added heavy metals.

3. RESULTS AND DISCUSSION

3.1 Uptake of Added Heavy Metals by Barley

The cadmium concentration in the leaves and stems of barley is shown in Fig. 1-1. In the first experiment, the concentration of cadmium in the leaves and stems of barley was almost constant except that it was higher in the experiment sections which contained 0.5% of sludge (L) of Section I and also in the sections which contained 1.0% of sludge (L) of Section Is. In the second experiment, the cadmium concentration of the leaves and stems of barley was different in each section from the first experiment, and the cadmium concentration had a tendency to be somewhat higher in large quantity-added sections regardless of the addition method. This tendency was especially marked in Section I and Section Is.

Fig. 1-2 shows the relationship between the added cadmium concentration in the leaves and stems and the added amount of cadmium.

In both the first and second experiments, the added cadmium concentration in the leaves and stems increased almost proportionately to the concentration of added cadmium. No marked difference due to the addition method was found, but in Section Sc, there was a tendency for the concentration to become somewhat higher, compared with that of other sections.

Fig. 1-3 shows the proportion of added cadmium-concentration to the total cadmium in the leaves and stems. Except in the sections where 0.5% of sludge (H) of Section Si was added, the proportion of added cadmium concentration was high in Section Si, which agrees with the tendency found in the extraction of added cadmium.

Fig. 1-4 shows the zinc concentration in the leaves and stems of barley.

In both the first and second experiments, the zinc concentration in the leaves and stems was apt to increase slightly when the amount of added zinc was increased, but no marked difference was found in any of the experimental sections.

The uptake of added zinc (Fig. 1-5) increased almost in proportion to the amount of zinc added in the same manner as for cadmium, but a marked difference due to the addition method was found in this case. The added zinc concentration in the leaves and stems was highest in Section Sc, and it was 2-3 times that of Section Si where the

concentration was lowest. This tendency almost agreed with that of the extraction of added zinc. The main reason why the added zinc concentration in the leaves and stems was lowest in Section Si was considered to be the state of zinc present in the synthetic sludge.

The proportion of added zinc concentration in the leaves and stems is shown in Fig. 1-6, and the proportion was remarkably high in Section Sc. For example, if we compare Section Sc and Section Si in 1.0% Section S_H, their proportions were 54.0% and 21.7% respectively in the first experiment and 49.1% and 20.1% in the second experiment. In either case, the result in Section Sc was about 2.5 times that in Section Si.

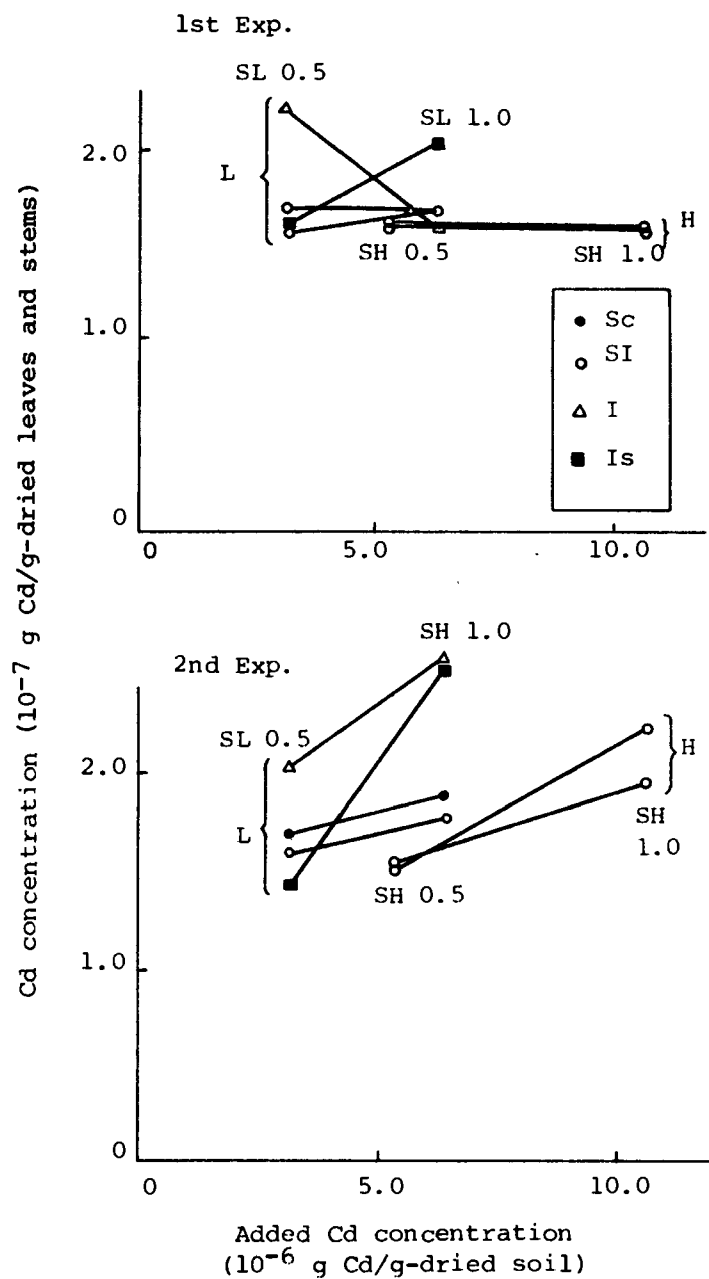


Fig. 1-1 Cadmium concentration in leaves and stems of barley

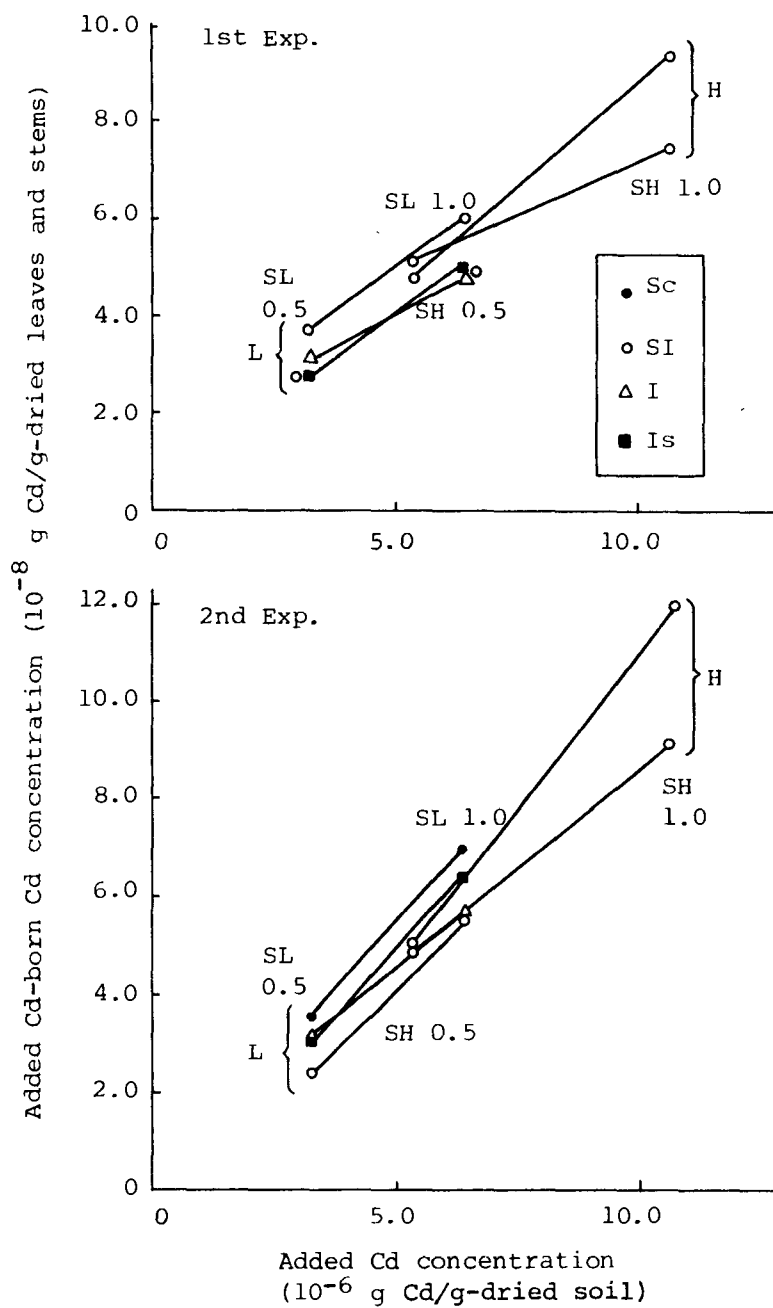


Fig. 1-2 Added cadmium concentration in leaves and stems of barley

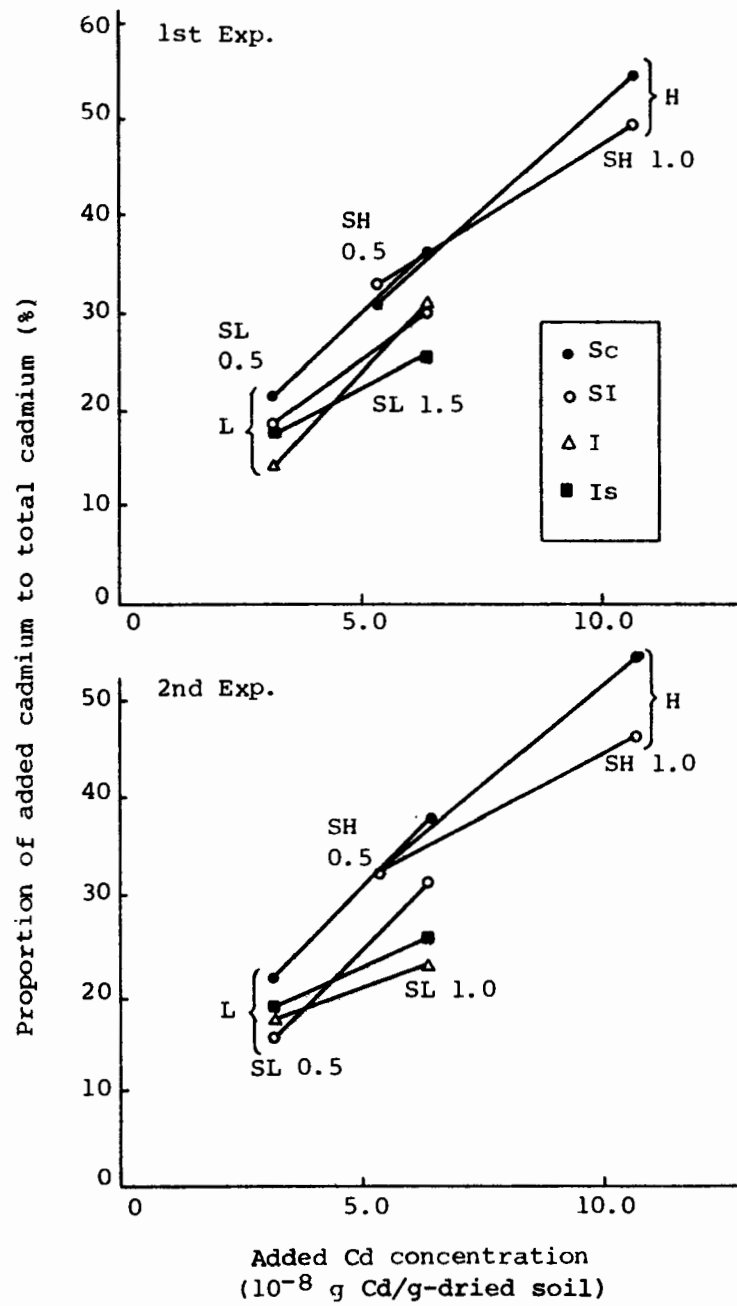


Fig. 1-3 Proportion of added cadmium in leaves and stems of barley

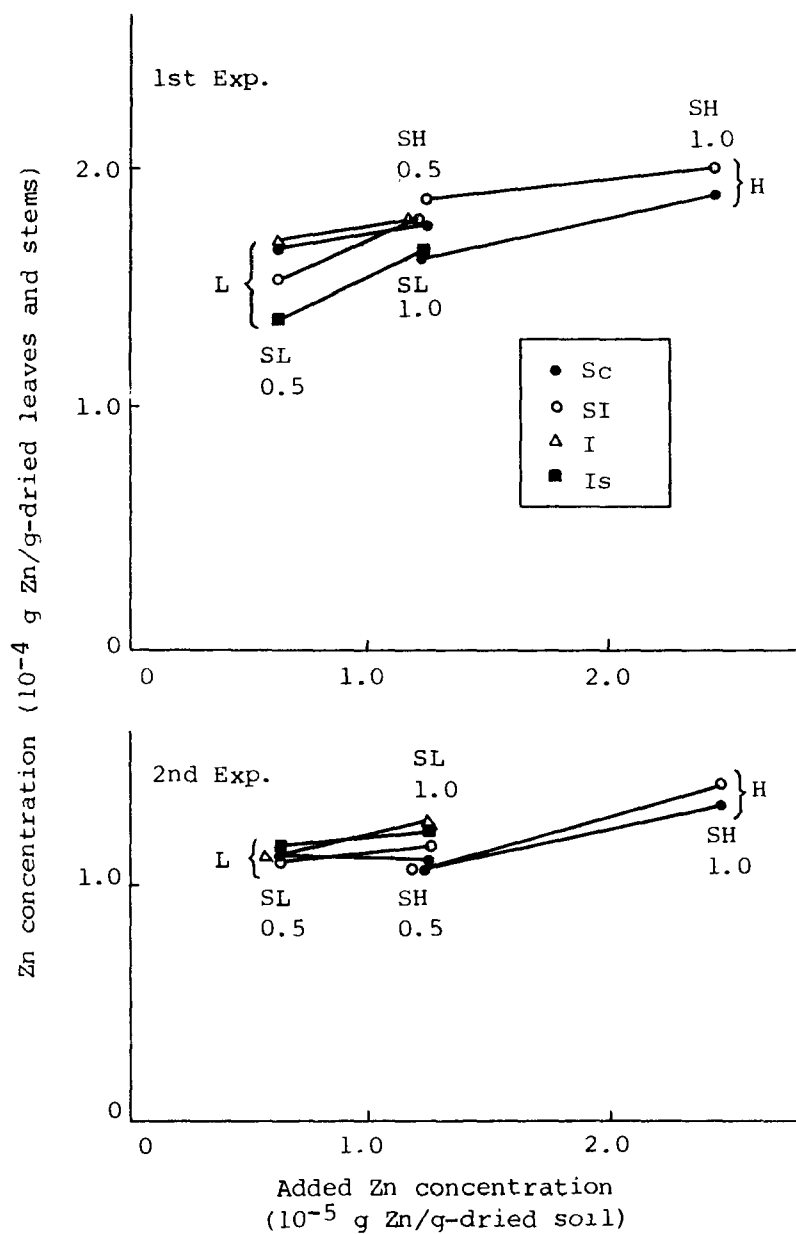


Fig. 1-4 Zinc concentration in leaves and stems of barley

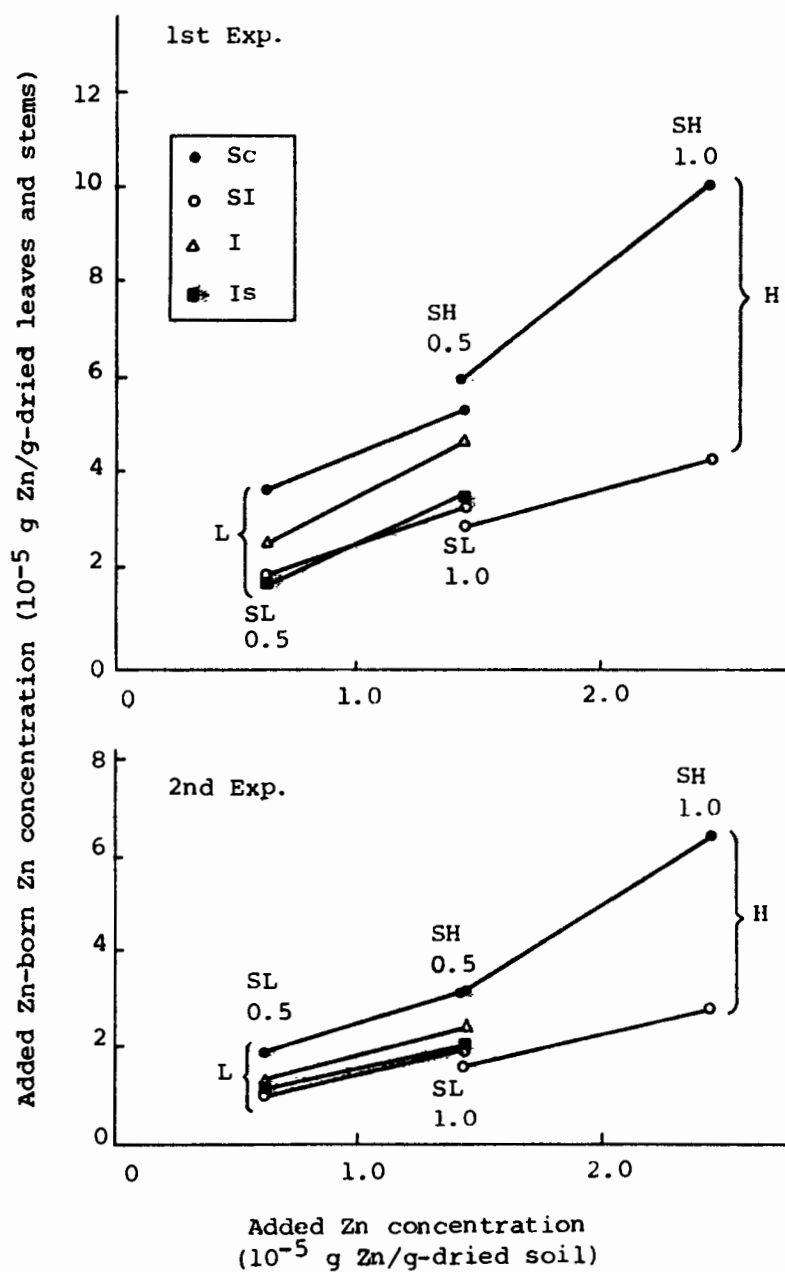


Fig. 1-5 Added zinc concentration in leaves and stems of barley

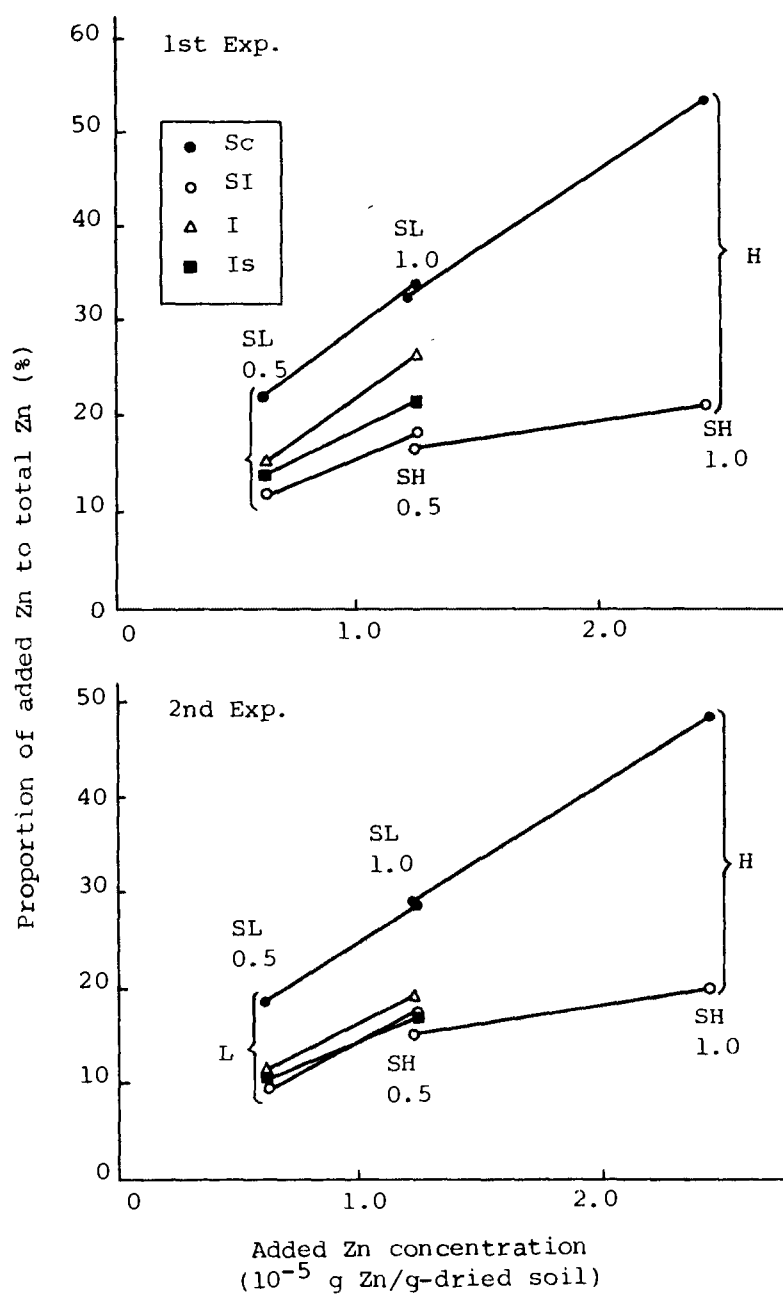


Fig. 1-6 Proportion of added zinc concentration in leaves and stems of barley

3.2 Relationship between Added Heavy Metals and their Uptake by Barley

Tables 1-4 (cadmium) and 1-5 (zinc) show the correlation between the extracted amounts of added heavy metals and their concentrations in the leaves and stems of barley. The correlation was remarkable regardless of addition or extraction methods, and even when Section Sc and Section Si were combined, a high correlation was found between these two factors.

Table 1-4 Correlation between added cadmium and its uptake by barley

Correlation coefficient

	Extraction method	Sc	Si	Sc + Si
1st Exp.	DTPO	0.995	0.965	0.959
	HCl	1.000	0.985	0.982
	HNO ₃	0.998	0.997	0.988
2nd Exp.	DTPO	0.995	0.993	0.983
	HCl	0.993	0.983	0.974
	HNO ₃	0.996	0.988	0.980

Table 1-5 Correlation between added zinc and its uptake by barley

Correlation coefficient

	Extraction method	Sc	Si	Sc + Si
1st Exp.	DTPO	0.967	0.954	0.960
	HCl	0.998	0.962	0.960
	HNO ₃	0.998	0.959	0.971
2nd Exp.	DTPO	0.998	0.978	0.992
	HCl	0.999	0.989	0.992
	HNO ₃	0.999	0.995	0.990

The marked correlation between the extracted amount of added heavy metals and their concentration in the leaves and stems suggests that added heavy metals reached an equilibrium with the soil-born heavy metals in the soil. In the case of their extraction with 1 N nitiric acid, the remarkable correlation also suggests that equilibrium was established to a certain degree even in the insoluble portion.

Further, it is assumed that these equilibria greatly depend upon the addition method or addition morphology of heavy metals, as well as the added amount of heavy metals. It seems that the heavy metals in sludge (Section Sc) established equilibria with soil-born heavy metals which were more soluble than those in the form of inorganic matter (Sections Si, I & Is). Therefore, the added heavy metals were more readily absorbed by barley in Section Sc than in other sections.

Furthermore, the results of this investigation suggest that the heavy metals in sludge were firmly combined with organic substances in the sludge even in the soil over a considerable period of time.

4. CONCLUSIONS

- (1) The heavy metals in sludge, compared with those added in the form of inorganic matter, were present in the soil in such a state that they were readily extracted by any extraction method. This tendency was especially remarkable for zinc.
- (2) The added cadmium concentration in the leaves and stems of barley was almost proportional to the concentration of added cadmium. In the case of zinc concentration, there was little difference in any of the experimental sections.
- (3) In this investigation, it has become apparent that the heavy metals in sludge present in the soil combine firmly with organic substances in sludge over a considerable period of time.

Reference

K. Tanaka, T. Mori and A. Narita, "Agricultural Use of Sewage Sludge" Progress Report (II), US/JAPAN JOINT RESEARCH PROJECT (Japanese Side), US/JAPAN Conference on Sewage Treatment Technology.

PART II COMPOSTING OF DEWATERED RAW CAKES PREPARED
BY USE OF ORGANIC COAGULANTS

1. RECENT STATUS OF COMPOSTING TECHNIQUE IN JAPAN

In Japan, a sludge composting technique has been rapidly developed since 1975 as a means of utilizing sludge for greens and farming lands. Composting is defined as the sludge treatment technology for decomposing or making inorganic the organic substances in sludge by using micro-organisms under aerobic conditions. Therefore the organic substances in sludge are stabilized, and become sanitary and easy to handle and store. No ill effect is given to the plants growing in the soil where composted sludge has been used.

As of January 31, 1981, the composting process has been adopted in three municipalities, Tendo City (5 ton/day), Yamagata City (15 ton/day) and Tokyo (8 ton/day). If the annual working efficiency of the process is 300 days, the compost output is estimated at about 2,500 ton. Further, the local municipalites scheduled to set up the composting facilities during fiscal year 1981 include Akita City (40 ton/day), Hitachi City (20 ton/day), Nozawaonsen-mura (8 ton/day), Chino City (6 ton/day), Tokorozawa City (20 ton/day) and Kagoshima City (150 ton/day, operation scheduled to be started during fiscal year 1981). Therefore, compost output of about 25,000 ton is estimated at the end of fiscal year 1982.

Recently, the composting technique is used as not only the sludge treatment process for agricultural use but also as the sludge drying or aerobic stabilization process.

2. PURPOSE

In the case of composting dewatered cakes containing inorganic coagulants such as slaked lime and ferric chloride, the recycling composting system in which a part of the composted product is sent back to the head of the fermentor as an adjusting material, has already been put into practice in Tokyo, Yamagata City, and Kagoshima City. The composting of dewatered cakes including organic coagulants (hereinafter referred to as "dewatered raw-cakes") is a composting system in which coarse organic substances such as paddy straw, chaff, sawdust, and bark are added to the cakes mainly to improve their physical properties (adjustment of water content, improvement in breathability). This system has been put into practice in Tendo City.

However, in order to adopt a composting technique in which coarse organic substances are added to the dewatered raw cakes, the facilities should be located in areas where the adding materials are abundant and in constant supply throughout the year. Such coarse organic substances were previously disposed of as waste, but due to the recent advancement in re-use technology, they are now being re-used as fodder, material for underdraining, fuel, and for the cultivation of mushrooms, and have therefore become more expensive.

The composting of dewatered raw cakes using polymer will meet serious future situations such as shortages in coarse organic substances and the consequent rise of composting cost. A method of composting dewatered raw cakes which does not require the addition of any coarse organic matters is essential, however.

This paper describes the feasibility of such a technique.

3. EXPERIMENTAL APPARATUS AND METHOD

3.1 Apparatus

Experiments have been conducted using horizontal scoop type experimental composting apparatus (fermentation capacity: 4 ton/day). The schematics of the primary fermenting apparatus and secondary fermenting house are shown in Fig. 2-1. The main instruments include a mixer, gyle, turner, hopper, vibrating sieve, meter, and control board. The specification of each instrument is shown in Table 2-1.

3.2 Method

3.2.1 Preparation of Return Compost

In order to compost dewatered raw cakes without addition of coarse organic matter, it is necessary to prepare return compost to be used for the adjustment of the water content of dewatered raw cakes. In this experiment, as the first step, dewatered raw cakes were mixed with dry digested cakes including polymer (hereinafter referred to as "dry digested cakes") to adjust the water content of the cakes, then they were aerobically composted.

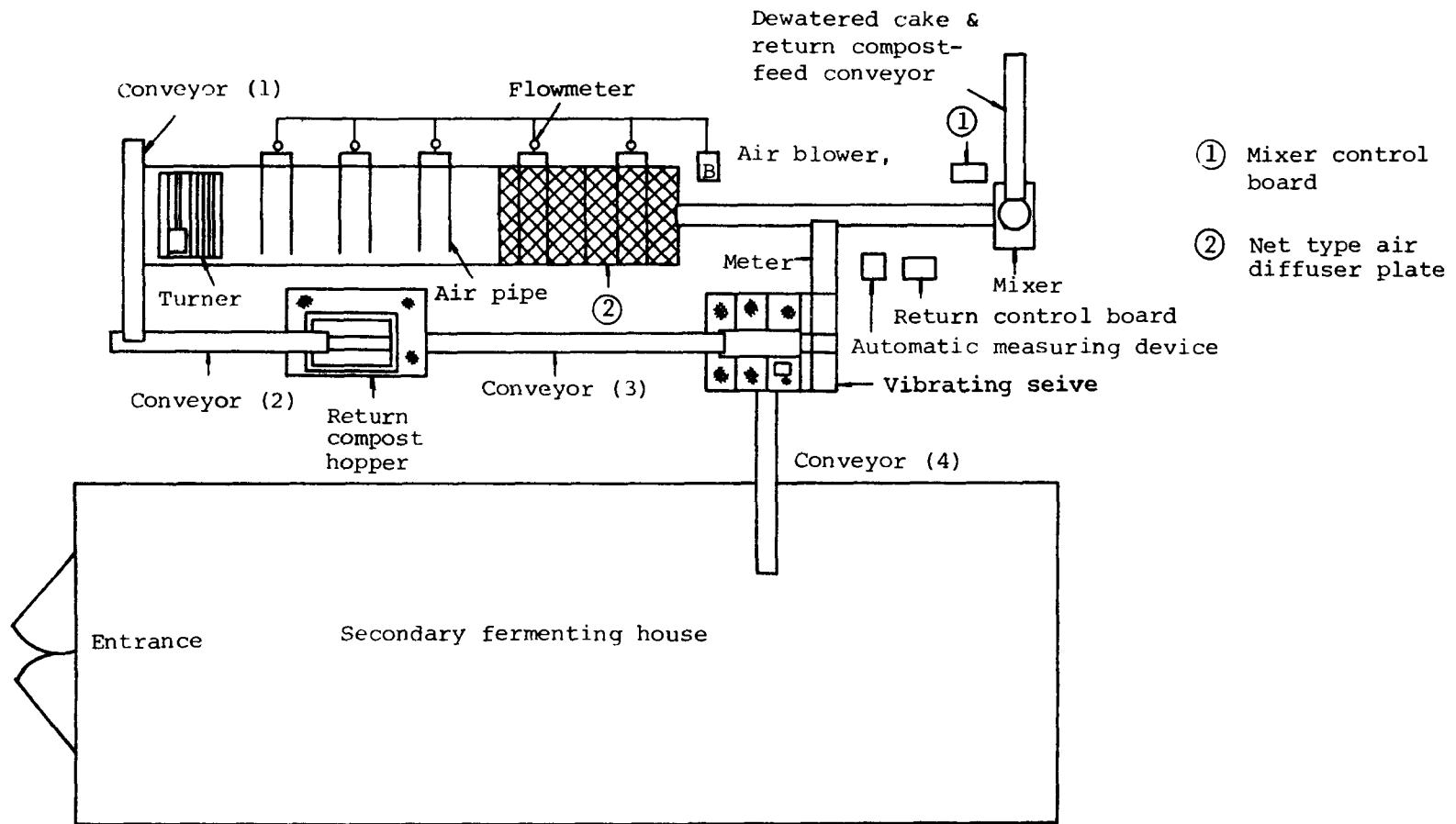


Fig. 2-1 Schematics of primary fermenting apparatus and secondary fermenting house

Table 2-1 Specification of major instruments

(1) Mixer	<p>Mixing batch capacity : 0.25 m³</p> <p>Mixture geared motor : 2.2 kW · 4P</p> <p>Discharge geared motor: 0.4 kW · 4P</p> <p>Discharge capacity : 100 kg/min</p> <p>Power supply : 200 V</p>
(2) Gyle	<p>Size : 10,900mm x 2,000mm x 1,600mm high</p> <p>Capacity : About 4 ton/day (wet) 1.5 ton/day (dry)</p> <p>Retention time : About 7 days</p> <p>Pile height : 1.0 - 1.2 m</p> <p>Turner : Number of turns 3/min, Travel 30cm/min</p>
(3) Aeration device	<p>Blower capacity : 4 m³/min, 1,000mmAq, Motor 1 kW</p> <p>Diffuser plate : 2,000mm x 2,000mm x 2,000mm high, 2 steel diffuser plates</p> <p>Diffuser pipe : ø 50 mm, VC x 3 pipes</p> <p>Flowmeter : 800 l/min 2 F-meters 400 l/min 3 F-meters Air volume adjustable by valve</p>
(4) Monitoring device	<p>Temperature : 6 thermostats, Recorder (6 pointer type)</p>
(5) Take-out and return device	<p>Conveyor : 0.75 kW</p> <p>Compost hopper : 2 m³</p> <p>Screw feeder : 3 varying speed 4, 11, 15 kg-wet/min</p> <p>Vibrating seive : Max. capacity 15 kg-wet/min Steel net plate (10, 20 & 30 mm)</p> <p>Automatic measuring device : Max. 600 kg/hr</p>
(6) Secondary fermenting house	<p>Size : Vinyl house, 20,000mm x 7,000mm x 2,000mm high, Entrance through which 2- ton vehicle is able to get in</p> <p>Pile height : 1 - 2 m</p>

As the second step, the compost prepared in the first step was mixed with dewatered raw cakes, then aerobically composted. This procedure was repeated several times to obtain return compost with a constant quality.

3.2.2 Properties of Test Materials

The properties of test materials, dewatered cakes, dry digested cakes, return compost and feed mixture, are shown in Table 2-2. Dewatered raw cakes were prepared by adding about 1% dried sludge of polymer coagulant to the mixture of primary sludge and excess activated sludge, and by dewatering it by centrifuge. Its properties are as follows: Moisture content 81 - 83%, pH 5.3 - 5.7, ignition loss 83 - 87%, and BOD 310 - 390 mg-O₂/g-DS. Thus, it has a high moisture content and, at the same time, has a large amount of easily-biodegradable organic matters.

As for dry digested cakes, the mixture of primary sludge and excess activated sludge was digested anaerobically at 37°C for about 30 days. After elutriation it was dewatered by a belt press filter using a polymer coagulant and heat-dried with an air dryer. Its properties are: Moisture content 15 - 30%, pH 7.0 - 7.3, ignition loss 42 - 46%, and BOD 47 - 63 mg-O₂/g-DS. Both its moisture content and the content of easily-biodegradable organic matters are low.

The properties of stable return compost are: Moisture content 32 - 37%, pH 7.9 - 8.2, and BOD 23 - 33 mg-O₂/g-DS. The BOD is markedly low, so that aerobical composting is considered to have been completed.

The properties of the feed mixture of the dewatered raw cakes and the return compost are: Moisture content 41 - 56%, pH 7.2 - 7.8, and BOD 46 - 81 mg-O₂/g-DS.

Table 2-2 Properties of test materials

Material	Item Unit	Moisture content	pH	Ignition loss	Calorific value	BOD ₅	C	N	C/N
		%		%	Kcal/kg-DS	mg O ₂ /g-DS	%	%	
Dewatered raw cake	Range	81-83	5.3-5.7	83-87	4,500-4,900	310-390	43-47	5.0-5.7	8.2-9.0
	Average	81.8	5.57	84.5	4,719	331	44.6	5.2	8.54
Dried digested cake	Range	15-30	7.0-7.3	42-16	2,300-2,500	47-62	22-23	2.5-2.9	7.7-9.2
	Average	21.2	7.15	55.0	2,436	54.7	22.8	2.7	8.5
Return compost	Range	32-27	7.9-8.2	42-44	2,360-2,390	23-33	22-33	2-3.5	6.4-8.5
	Average	34.8	8.05	43.3	2,384	29.0	22.7	2.9	8.1
Feed mixture	Range	41-56	7.2-7.8	45-51	2,400-2,700	46-81	23-30	3.1-4.3	6.7-8.7
	Average	48.7	7.50	48.3	2,596	67.3	25.5	3.60	7.3

4. RESULTS AND DISCUSSION

4.1 Pre-adjustment of Feed Mixture

In the pre-adjustment of the feed mixture, attention must be paid to the initial moisture content of the mixture and its mixing procedure. First of all, when the moisture content was 55% or more and the pile height of the mixture was about 1 m, lowering of its breathability was found due to compression by its own weight. Further, on turning, the particle diameters of the mixture were increased due to their own cohesion so that anaerobic fermentation was found to occur. However, when the initial moisture content of the feed mixture was 50% or less, such a phenomenon was not observed. Accordingly, when dewatered raw cakes including polymer alone are composted, it is necessary to lower the moisture content to 50% or less.

The dewatered raw cakes and the return compost were mixed conventionally inside the mixer, then discharged with a double screw at the bottom of the mixer. Therefore the mixture was liable to form into a mass, and its quality was lowered.

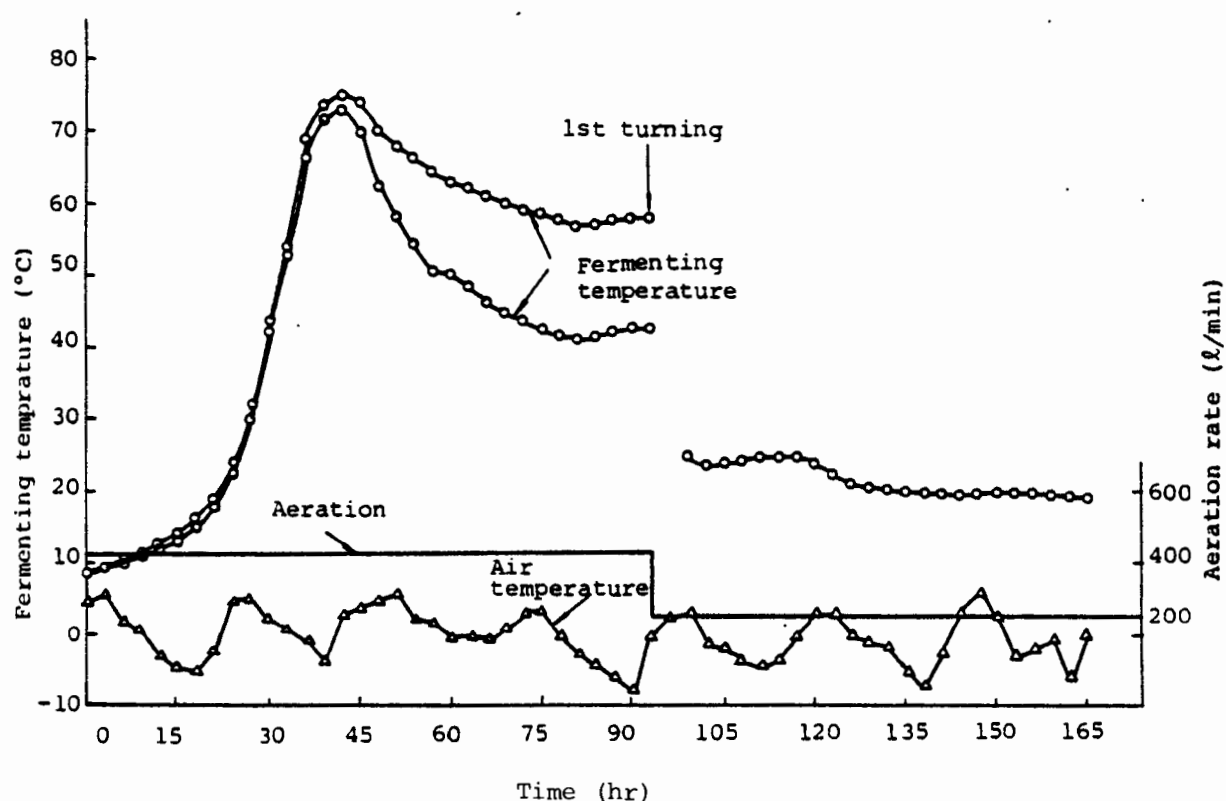
In this case, a paddle type mixer is considered to be preferable since its mixing time is short and the feed mixture is not kneaded in the mixer.

4.2 Change of Fermenting Temperature and Concentrations of Carbon Dioxide and Ammonia

4.2.1 Fermenting Temperature

The change of fermenting temperature with time is shown in the example of Fig. 2-2. The rising rate of the temperature at the initial stage of fermentation was 1.0 - 3.3°C/hr and the temperature was rapidly increased until the maximum fermenting temperature of 73 - 78°C was reached in about 40 hours. However, after turning, the fermenting temperature tended to decrease. It is considered that the BOD to be decomposed in the fed mixture was related to this phenomena. The BOD of the fed mixture of the dewatered raw cakes and the return compost in this experiment was $22 \times 10^3 - 46 \times 10^3$ g.O₂-VTS, which was considerably low, and about 1/3 - 1/8 of BOD (65×10^3 g.O₂-VTS) of the fed mixture of dewatered raw cakes

and dry digested cakes. In the case of the mixture of dewatered raw cakes and dry digested cakes a high temperature of 65°C or more was maintained even after turning. Therefore, the low BOD was considered to result in low total heat generation. Accordingly, if the extinction of germs, weed seeds, and some others are taken into consideration as the purpose of composting, the BOD of the feed mixture is significant and a fermenting temperature of 65°C or more should be maintained for at least 48 hours.



Experiment No. : K-8
 Date : 23 January - 3 February 1981
 Weight of feed mixture: 1,152kg(329kg of dewatered raw cakes + 823kg of return compost)
 Moisture content : 48.7%
 Ignition loss : 49.6%
 BOD : 78.8 mg-O₂/g-DS
 Aeration : 430 l/min 1.47 l/min/kg-VTS(fed mixture)
 8.22 l/min/kg-VTS(cake)

Fig. 2-2 Change of fermenting temperature over time
 (Horizontal scoop type fermenter)

4.2.2 Concentrations of Carbon Dioxide and Ammonia Gas

Fig. 2-3 shows the change in the concentrations of carbon dioxide and ammonia gas in the small size composting apparatus (35 l).

The concentration of carbon dioxide was 0.4% at the start of the experiment, but 12 hours later it began to increase rapidly; 24 hours later the maximum concentration of about 11% was reached, then decreased rapidly to 0.5% 48 hours after the start of the experiment. After that, turning was conducted, but almost no further carbon dioxide was generated. This tendency was also found in other experiments. As for the concentration of ammonia, the ammonia gas began to generate rapidly when the maximum concentration of carbon dioxide had been reached, and the maximum concentration of about 8,000 ppm was reached about 36 hours after the start of the experiment. After that, it decreased with the time and fell to 2,500 ppm 45 hours after the start of the experiment.

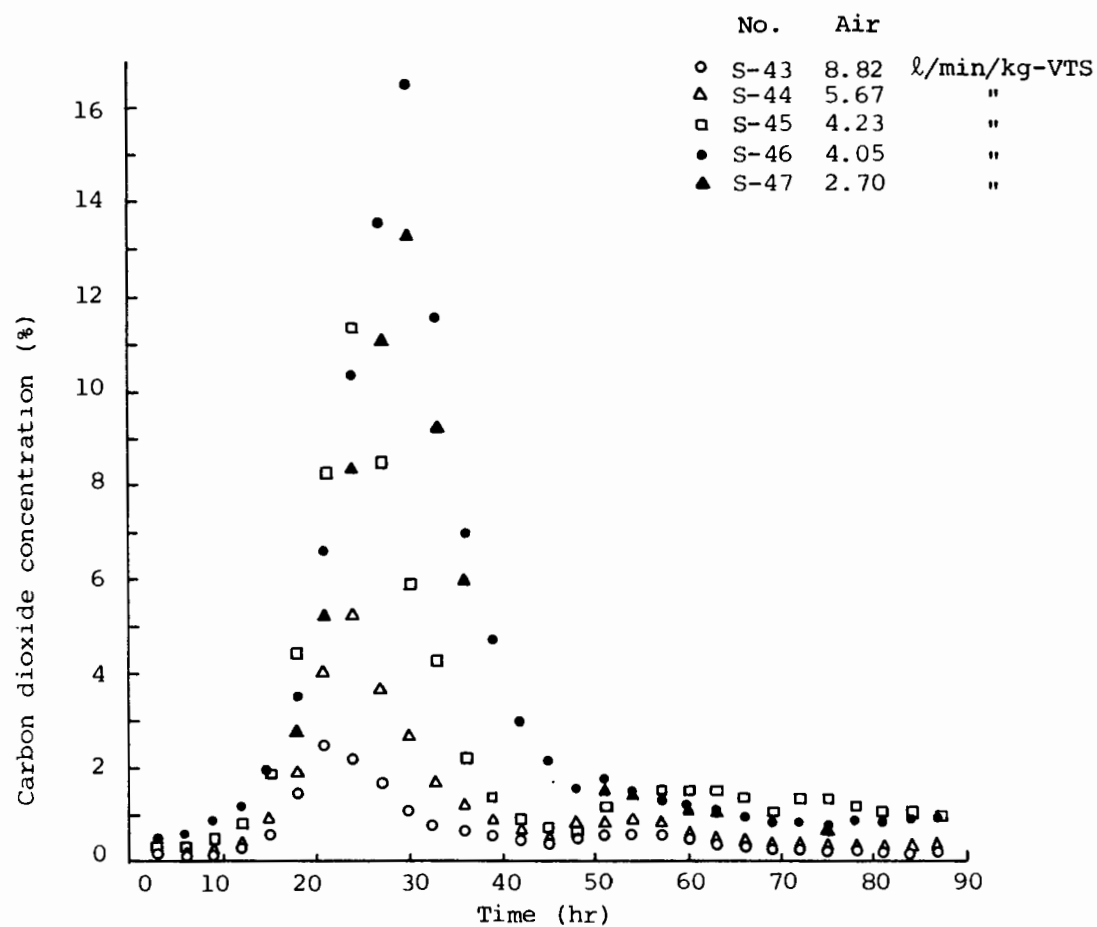


Fig. 2-3 Change of concentrations of carbon dioxide and ammonia gas (1/2)

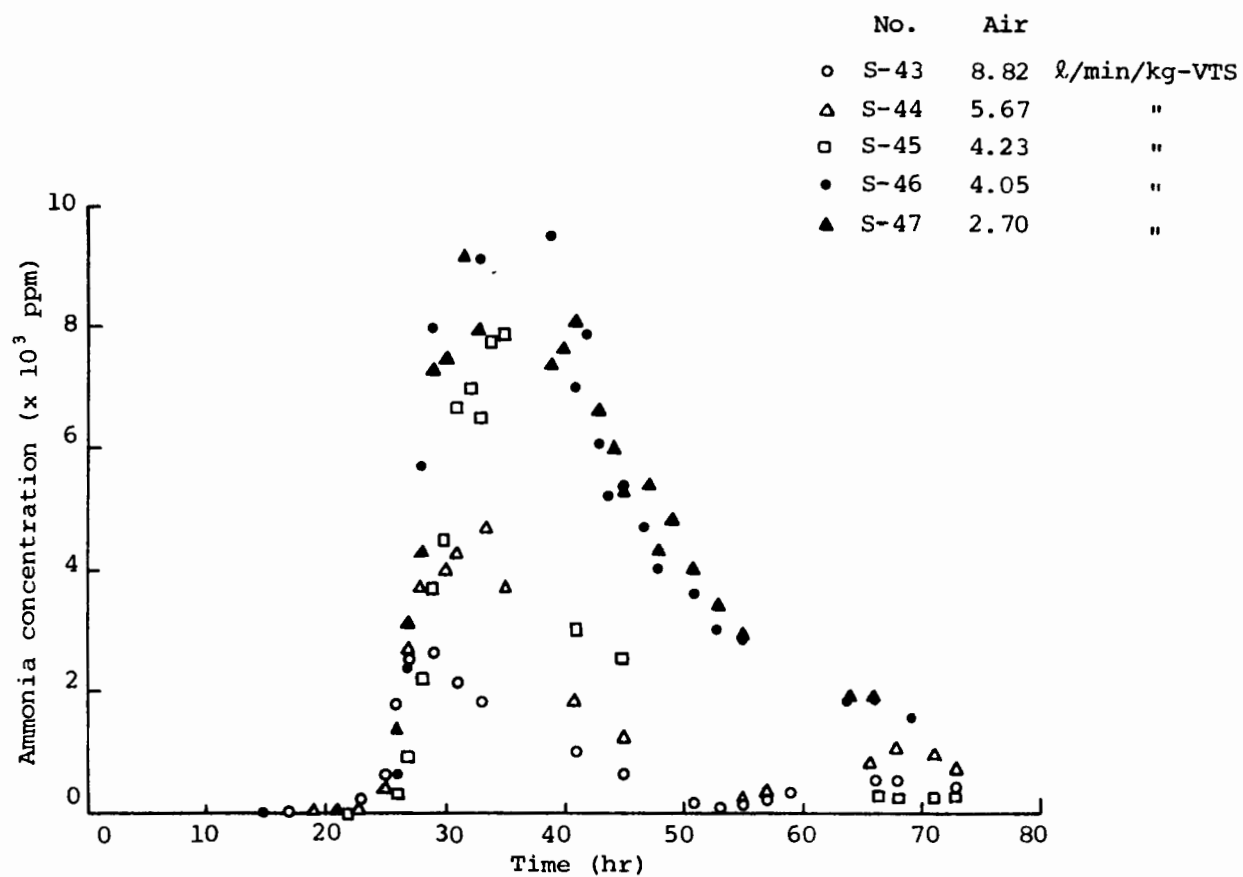


Fig. 2-3 Change of concentrations of carbon dioxide and ammonia gas (2/2)

4.2.3 Discussion of Aeration

The initial aeration rate in the horizontal scoop type composting experiment was set at 8 - 13 l/min/kg-VTS(cake) in order to obtain the optimum temperature-rising rate of 1 - 3°C/hr which was found in the preliminary experiment in the small size fermentor. As a result, aerobic composting proceeded favorably. This experiment was performed at an average daytime temperature of 0 - 5°C in the winter. As for summer season, the microorganism activity is increased due to higher temperature so that higher aeration rate would be required.

Since there is a tendency for aerobic composting to complete rapidly within about 10 days, it is considered that the aeration rate after about 10 days of operation can be reduced depending upon the fermenting temperature.

4.2.4 BOD Removal Rate

The cumulative BOD removal rate with the change of fed mixture is shown in Fig. 2-4. On the 5th day after the start of composting, a high BOD removal rate of 50 - 80% was observed, but after that, almost no change in BOD removal rate was found. On the contrary, the BOD removal rate in composting the chaff-added mixture that had been studied until last year was about 10% on the 5th day after the start of composting, and was remarkably low in comparison with that of non-added compost.

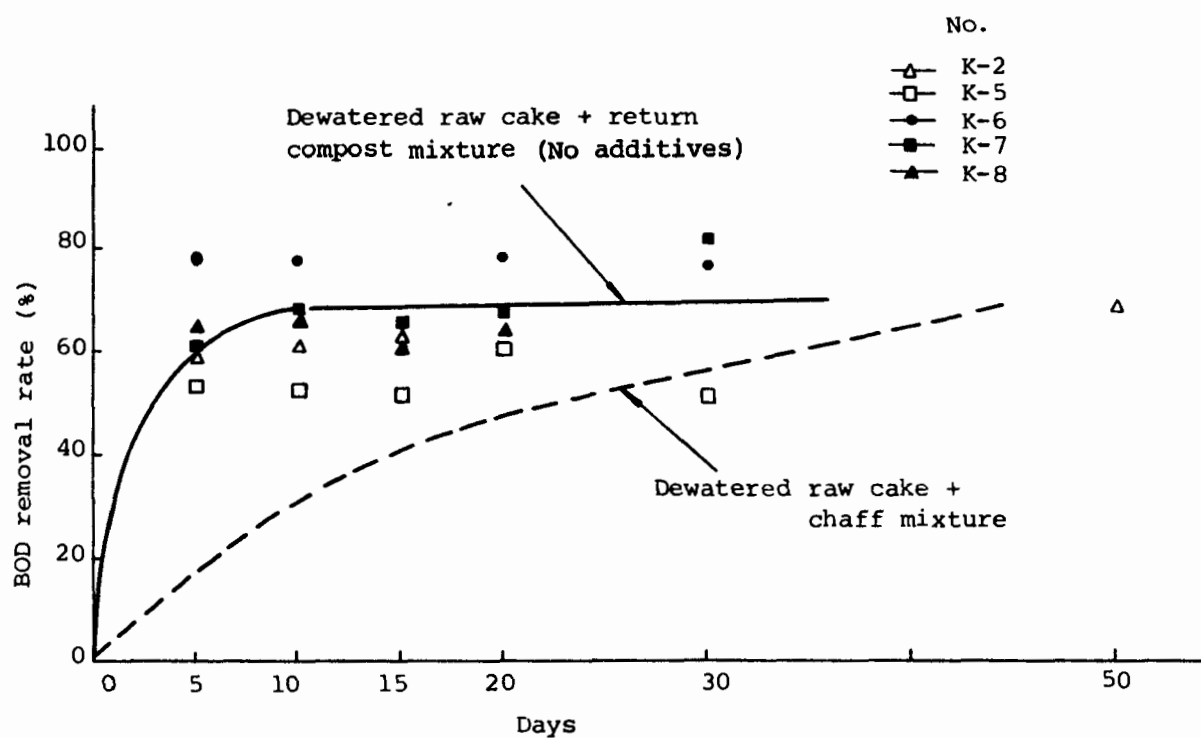


Fig. 2-4 Change of BOD removal rate

4.2.5 Change in Physical and Chemical Properties of Mixture

(a) Change of moisture content

The change of moisture content over time is shown in Fig. 2-5. The initial moisture content (about 50%) of the fed mixture was rapidly lowered during the early fermenting stage, and it became 7 - 15% lower than the initial moisture content 5 days later. Then, it fell to 39 - 43%, and on the 30th day when aerobic composting was completed, it further decreased to 30 - 36%. Therefore, the mixture became easy to handle and could be used as water-content adjusting material for return compost.

(b) Change of pH

The change of pH is shown in Fig. 2-6. The fed dewatered-raw cakes showed a weak acidity of 5.3 - 5.7; return compost, a weak alkalinity of 7.9 - 8.2; and the mixture, a neutrality of 7.2 - 7.8. The pH of the fed mixture, for which fermentation was started around neutrality, indicated the weak alkalinity of 8.0 - 8.5 on the 5th day after the start of fermentation, but after that, its pH was lowered little even when the secondary fermentation proceeded.

(c) Change of BOD

The change of BOD with the elapse of days is shown in Fig. 2-7. The BOD's of the dewatered raw cakes, the return compost and the fed mixture were 310 - 390 mg O₂/g-DS, 23 - 33 mg O₂/g-DS, and 46 - 81 mg O₂/g-DS, respectively. Depending on the properties of dewatered cakes or the mixing ratio of return compost, the BOD value of the fed mixture varied, but it rapidly fell to 20 - 30 mg O₂/g-DS on the 5th day after the start of the primary fermentation, then it had a tendency to attain a constant value. The BOD after the primary fermentation was considerably low and about 1/2 or less of the BOD (45 - 70 mg O₂/g-DS) of the dewatered digested cakes including polymer and chaff-added mixture. This means the easily-biodegradable organic matters were considerably decomposed and the mixture was stabilized.

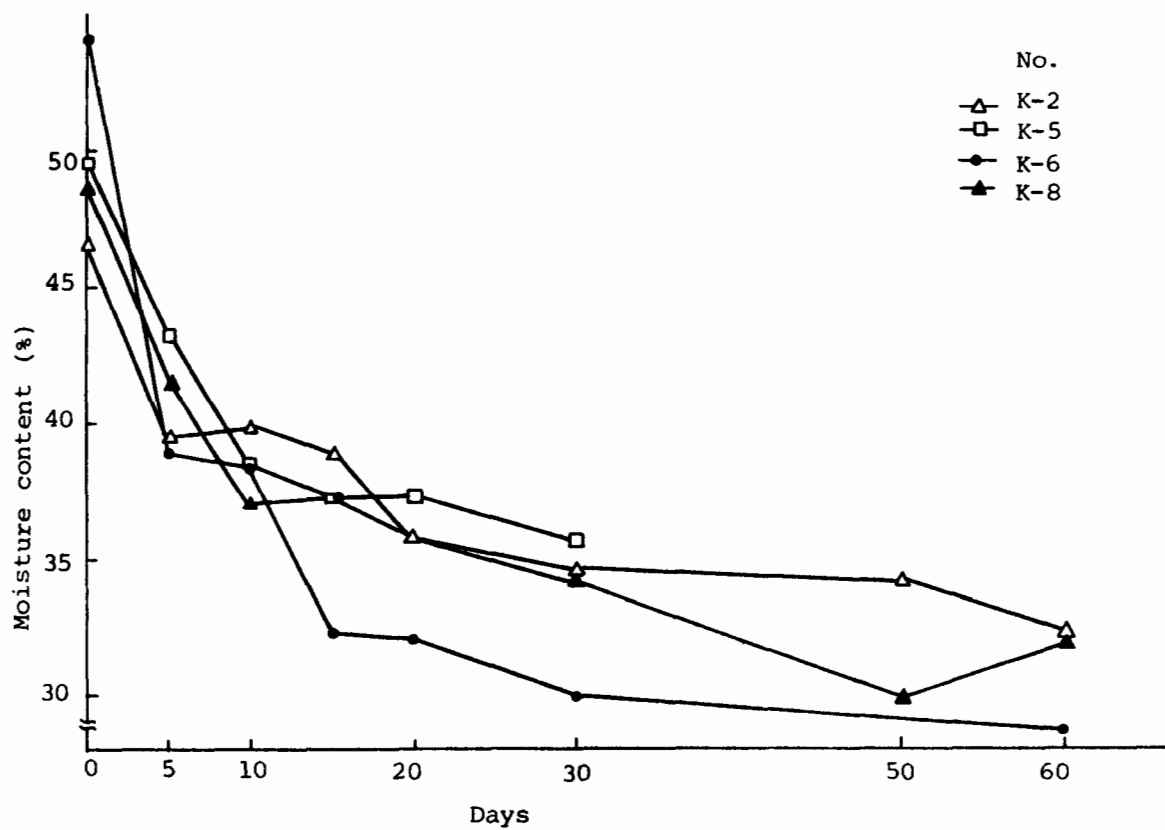


Fig. 2-5 Change of moisture content

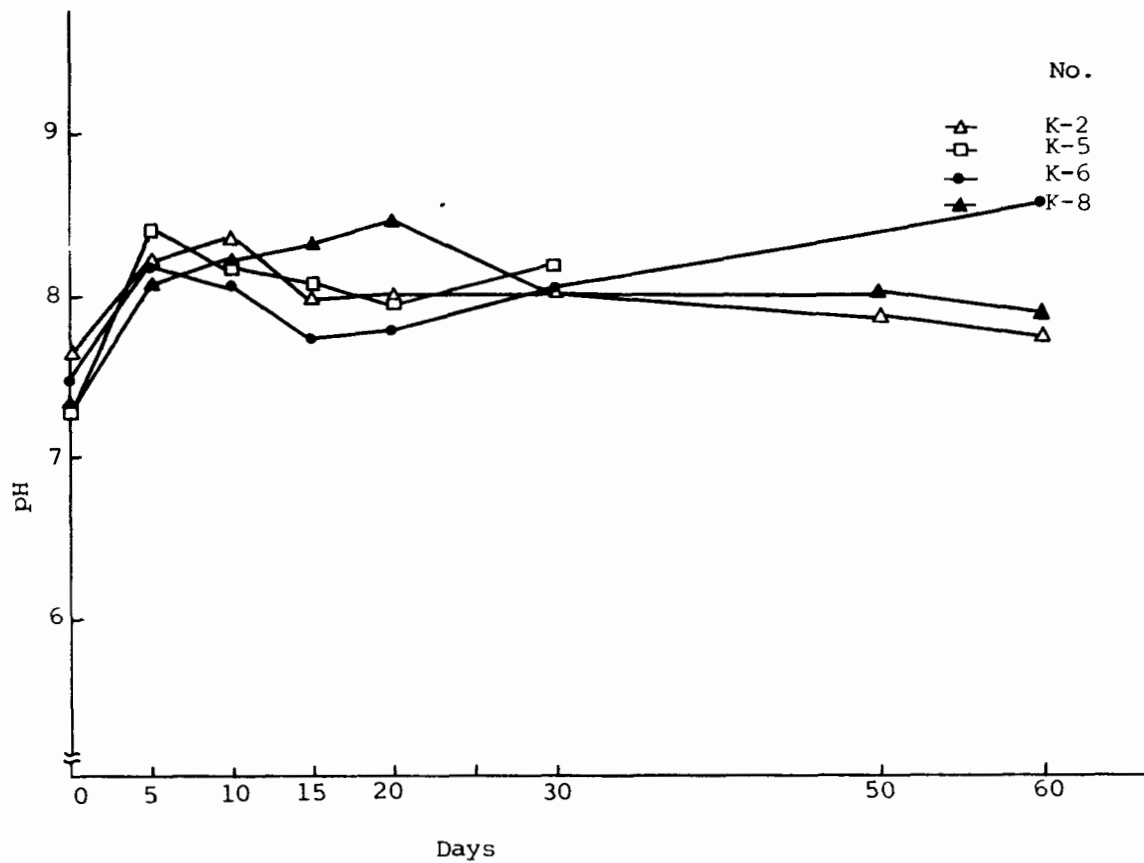


Fig. 2-6 Change of pH

(d) Change of ignition loss

The change of ignition loss with the elapse of days is shown in Fig. 2-8. The ignition loss of the dewatered raw cakes, the return compost and the fed mixture were 83 - 87%, 42 - 44%, and 46 - 51%, respectively. Since they were rapidly decomposed during the 5 days of the primary fermentation period, the ignition loss decreased remarkably by 5 - 7% of the initial value, then decreased gradually. The decomposition rate of organic substances was about 7 - 9 kg/day until the 5th day after the start of composting, and then 1-2 kg/day.

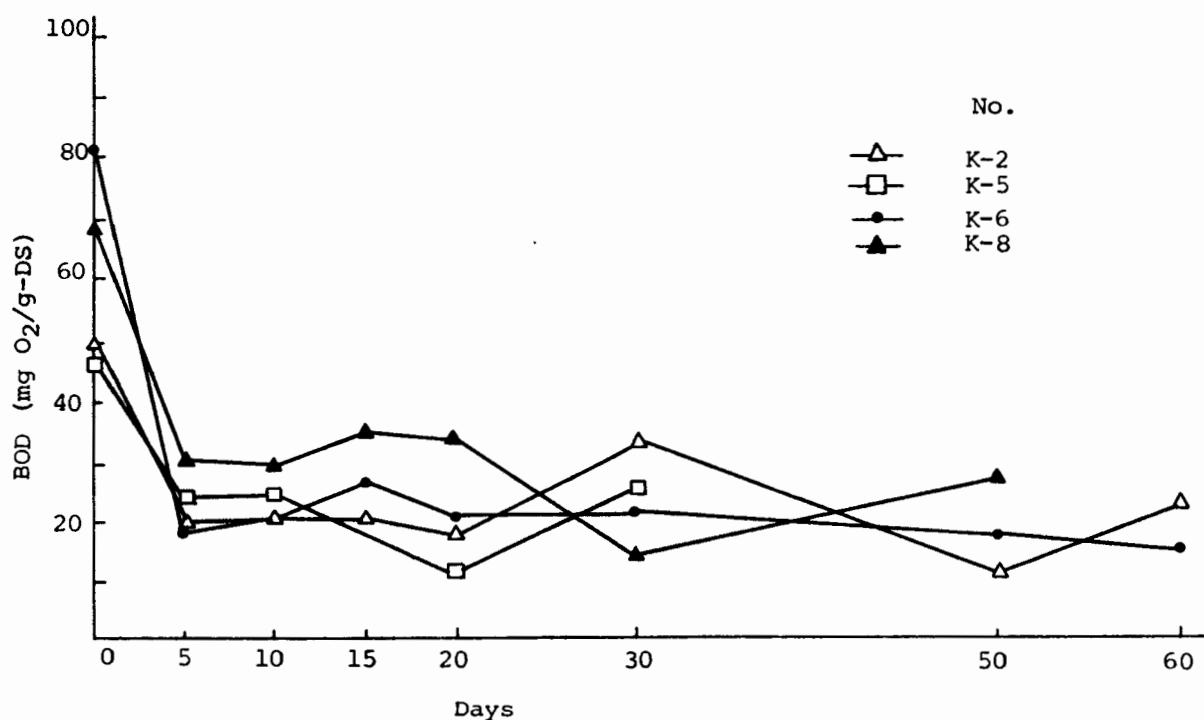


Fig. 2-7 Change of BOD

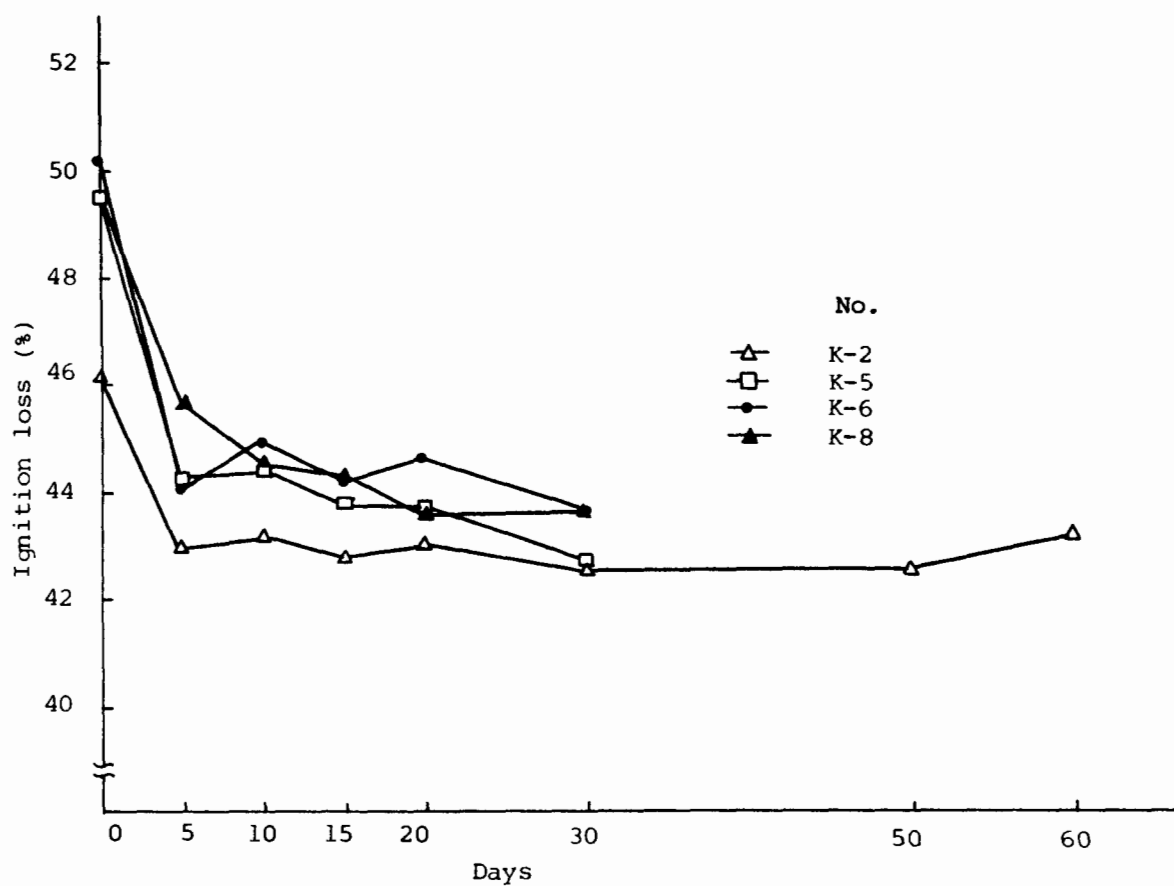


Fig. 2-8 Change of ignition loss

(e) Fertilizer ingredients

The fertilizer ingredients of the composted dewatered raw cakes are shown in Table 2-3. The compost contains 3.4% total nitrogens, 3.7% phosphoric acid and 0.6% potassium, and both total nitrogen and phosphoric acid contents are characteristically about two times those in the chaff compost made of dewatered digested cakes.

Table 2-3 Fertilizer ingredients

Item Experiment No.	Days (day)	Moisture content (%)	T-N (%)	P ₂ O ₅ (%)	K ₂ O (%)	NO ₃ -N (ppm)	CaO (%)	MgO (%)	CEC (me)
K-5	30	34.81	3.37	3.65	0.28	19	0.94	1.72	77.1
K-8	30	33.21	3.47	3.71	0.91	29	0.83	1.08	-
Dewatered digested cake-chaff	60	58.16	2.13	1.34	0.57	411	0.26	0.41	44.4

(Based on the weight of dried
compost except moisture content)

4.2.6 Young Plant Test of Compost

In order to investigate the effect of compost upon the germination and growth of rareripe, small shantung greens, a young plant test was conducted. The four kinds of composts used in this test were: Chaff compost made of polymer-added, dewatered digested cakes; sawdust compost made of polymer-added, dewatered digested cakes; compost made of a mixture of polymer-added dewatered raw cakes and polymer-added dry digested cakes; and compost of no additives.

(a) Method

Test soil was placed in pots and both test and control soils were thoroughly mixed. The water content of the soil was adjusted to make it up to about 70% of the maximum water content. Seeds were sown and their germination and state of growth were investigated.

(b) Materials

1) Analysis of test samples (%)

Test section	Water content	T-N	T-N (Dry based)
Compost No. 13, (Dewatered digested cake + chaff)	48.05	1.04	(1.92)
Compost No. 14, (Dewatered raw cake + dry digested cake)	23.74	2.30	(3.01)
Compost No. 15, (Dewatered digested cake + sawdust)	59.39	0.78	(1.92)
Compost No. 16, (Dewatered raw cake + return compost)	32.07	2.18	(3.21)
Control bark compost	65.15	0.27	(0.77)

2) Test soil and crops

Humus volcanic soil (Sampled at Hamadayama, Suginami-ku, Tokyo)

Table 2-4 Test soil and crops

pH		CEC me	Substitutive base			Absorption coefficient		T.C %	Y ₁	Maximum water content	Volume weight g
H ₂ O	KCl		CaO	MgO	K ₂ O	N	P ₂ O ₅				
5.60	5.05	28.1	12.10	0.82	0.20	859	2,732	8.25	0.4	103.8	400

Volume weight Weight of dried soil per pot (500 ml)

Rareripe, small shantung greens 25 seeds/pot

3) Test section and amount used

Table 2-5 Test section and amount used

Test section	Amount of compost used based on the weight of dried soil(g)			
	(5)	(10)	(20)	(40)
Compost No. 13 (Dewatered digested cake + chaff)	9.6	19.2	38.5	77.0
Compost No. 14 (Dewatered raw cake + dry digested cake)	6.6	13.1	26.2	52.5
Compost No. 15 (Dewatered digested cake + sawdust)	12.3	24.6	49.2	98.5
Compost No. 16 (Dewatered raw cake + return compost)	7.4	14.7	29.4	58.9
Control bark compost	14.3	28.7	57.4	114.8
Non-added section	0	0	0	0

a) As common fertilizers, use ammonium sulfate, calcium superphosphate and potassium chloride containing 75 mg each of N, P_2O_5 and K_2O respectively.

b) In the Non-added Section, use common fertilizers only.

4) Cultivation

Placement of soil	January 27, 1981
Addition of compost & adjustment of water content	January 28, 1981
Sowing	January 28, 1981
Investigation of output	February 27, 1981

(c) Results

The test results are as shown in Table 2-6.

The germination started two days after sowing, and there were no differences in the germination starting day among the test sections.

Table 2-6 Results of young plant test

Test section	Amount fed g	Germinating force %	Germination rate %	Number of seed leaves		Leaf		Fresh leaves	
						L	W		
				2 Feb.	6 Feb.	cm	cm	Weight g	Index No. g
Compost No. 13	5	94	100	20.0	24.5	9.0	2.3	7.9	82.3
	10	96	100	22.5	25.0	10.0	2.6	10.3	107.3
	20	94	100	21.0	25.0	10.1	2.8	9.9	103.1
	40	98	100	20.0	24.0	8.3	2.5	6.5	67.7
Compost No. 14	5	96	100	19.0	25.0	8.7	2.2	6.0	62.5
	10	96	100	20.0	25.0	9.0	2.4	6.9	71.9
	20	94	100	21.0	24.0	8.4	2.4	5.6	58.3
	40	96	100	20.5	24.0	5.7	1.8	3.7	38.5
Compost No. 15	5	98	100	21.0	24.5	10.0	2.3	9.4	97.9
	10	94	100	24.5	25.0	9.8	2.4	9.3	96.9
	20	96	100	25.0	-	7.9	2.3	8.5	88.5
	40	96	100	21.0	24.0	6.0	1.7	5.7	59.4
Compost No. 16	5	96	100	18.0	25.0	9.8	2.4	8.9	92.7
	10	98	100	24.0	24.0	8.5	2.4	8.1	84.4
	20	96	100	23.5	25.0	6.4	2.1	5.2	54.2
	40	96	98	24.0	24.5	5.0	1.3	4.1	42.7
Control bark compost	5	94	100	20.0	25.0	9.0	2.4	10.1	105.2
	10	94	100	23.0	25.0	10.0	2.5	12.0	125.0
	20	96	100	25.0	-	9.9	2.5	11.9	124.0
	40	96	100	23.5	24.5	10.2	2.9	12.6	131.2
Non-added		98	100	23.0	24.5	9.3	2.2	9.6	(100.0)

As for the growth after germination, no differences in growth due to the amount of compost used were found in the control bark section, but in 4 compost sections, compared with the control bark compost section or non-added section, a delay in growth was observed in proportion to increase in the amount of compost. This delay in growth was a little more remarkable in Compost No. 14 & No. 16 sections than in Compost No. 13 & 15 sections. This tendency in growth affected the output.

This is due to the difference in the amount of ingredients contained in the fed compost. Table 2-7 shows the amount of Nitrogen in the fed compost and a delay in growth due to toxic substances was not found in this test.

Table 2-7 Amount of fed nitrogen

Test section	Amount (Dry based) g			
	(5)	(10)	(20)	(40)
Compost No. 13 (Dewatered digested cake + chaff)	100	200	399	799
Compost No. 14 (Dewatered raw cake + dry digested cake)	151	301	603	1,205
Compost No. 15 (Dewatered digested cake + sawdust)	96	192	384	768
Compost No. 16 (Dewatered raw cake + return compost)	160	320	641	1,282
Control bark compost	39	77	155	310

5. CONCLUSIONS

The results obtained in the experiment were as follows:

- (1) Composting of dewatered raw cakes is successfully performed through the return of compost and aeration.
- (2) It was found that the ammonia concentration in off-gas in the primary fermentation process (after about 10 days) was 5,000 - 8,000 ppm maximum at an aeration rate of 5 l/min kg-VTS.
- (3) There was a tendency for VTS and BOD to rapidly decrease and to be stabilized in the primary fermentation process.

In the future, the optimum combination of the moisture content of return compost and that of dewatered cake to be composted will inevitably be found.

UNITED STATES PAPERS

RECENT DEVELOPMENTS IN MECHANICAL COMPOSTING OF MUNICIPAL SLUDGE.....	629
Atal E. Eralp, Municipal Environmental Research Laboratory, ORD, USEPA and Harry A. J. Hoitink, Ohio Agricultural Research and Research Center, Ohio State University, Wooster, Ohio	
THERMAL PROCESSES FOR CONVERSION OF SLUDGE: STATUS, EPA RESEARCH, FUTURE DIRECTIONS.....	661
Joseph B. Farrell, Municipal Environmental Research Laboratory, ORD, USEPA	
TECHNOLOGY ASSESSMENT OF THE VERTICAL WELL CHEMICAL REACTOR.....	681
John M. Smith and Jeremiah J. McCarthy, Municipal Environmental Research Laboratory, ORD, USEPA	
FLOW MANAGEMENT BY POROUS PAVEMENT AND INSITUFORM CONTROL OF FILTRATION.....	697
Carl A. Brunner, John N. English, and Richard Turkeltaub, Municipal Environmental Research Laboratory, ORD, USEPA	
SEQUENCING BATCH REACTORS FOR MUNICIPAL WASTEWATER TREATMENT.....	721
Edwin F. Barth, Municipal Environmental Research Laboratory, ORD, USEPA	
ANAEROBIC TREATMENT OF MUNICIPAL WASTEWATER.....	749
Irwin J. Kugelman, James A. Heidman and Donald Brown, Municipal Environmental Research Laboratory, ORD, USEPA	
NATIONAL SURVEY OF MUNICIPAL WASTEWATERS FOR TOXIC CHEMICALS.....	773
Jesse M. Cohen, Lewis Rossman and Sidney A. Hannah, Municipal Environmental Research Laboratory, ORD, USEPA	
CONTROL OF SPECIFIC ORGANIC AND METAL CONTAMINANTS BY MUNICIPAL WASTEWATER TREATMENT PROCESSES.....	803
Dolloff F. Bishop, Albert C. Petrusek, and Irwin J. Kugelman, Municipal Environmental Research Laboratory, ORD, USEPA	
AN INDUSTRIAL PERSPECTIVE ON JOINT MUNICIPAL-INDUSTRIAL WASTE- WATER MANAGEMENT.....	849
Gerald N. McDermott, Senior Engineer, The Procter & Gamble Company, Cincinnati, Ohio	

(continued)

UNITED STATES PAPERS (continued)

COST-EFFECTIVENESS AND WATER QUALITY JUSTIFICATION FOR ADVANCED WASTEWATER TREATMENT (AWT) FACILITIES.....	865
Robert J. Foxen, Office of Water, USEPA	
EFFECTS OF MULTIPLE DIGESTION ON SLUDGE.....	885
Wilbur N. Torpey, Consultant, New York, New York	
John F. Andrews, University of Houston, Texas	
James V. Basilico, Office of Research and Development, USEPA	
RESEARCH SUPPORTED BY THE NATIONAL SCIENCE FOUNDATION RELATING TO TREATMENT OF WASTEWATER AND MANAGEMENT OF RESIDUAL SLUDGES.....	907
Edward H. Bryan, National Research Foundation, Washington, D.C.	
REMOTE SENSING OF SEPTIC SYSTEM PERFORMANCE USING COLOR INFRARED AERIAL PHOTOGRAPHY.....	919
David W. Hill and Rebecca B. Slack, Region, IV, USEPA and E. Terrance Slonecker, Environmental Photographic Interpretation Center, USEPA, Warrenton, Virginia	
TWO-PHASE ANAEROBIC DIGESTION OF ORGANIC WASTES.....	929
Sambhunath Ghosh, Ph.D., Institute of Gas Technology, Chicago, Illinois	

RECENT DEVELOPMENTS IN MECHANICAL COMPOSTING OF MUNICIPAL SLUDGE

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INTRODUCTION

Sewage sludge composting is an aerobic, microbiological decay process. Under favorable environmental conditions, microorganisms contained in the sludge will begin to break down the large organic molecules also present in sewage sludge. Using the organic matter as a substrate, the microorganisms raise their metabolic rates and hence, the surrounding temperatures. Elevated temperatures in the compost mixture, as well as microbial antagonism are primarily responsible for the elimination of pathogenic organisms. The resulting material, after completion of the process, is an earthy, humus-like material which can be used as a soil conditioner.

Composting can effectively transform municipal organic waste into an agriculturally and horticulturally useful product that is easy to handle and readily accepted by the public. Several composting methods have been extensively researched and tested, and have been found to be reliable for stabilizing and minimizing the pathogen content in sewage sludge and septage. The two most widely used and accepted methods of sewage sludge composting in the United States are the static aerated pile method developed by the U. S. Department of Agriculture at Beltsville, Maryland, and the windrow method.

In recent years several windrow and static pile composting systems have replaced manual labor with automated machines in order to reduce cost and reliability. Several others covered their systems in order to reduce odor and protect their systems against extreme weather conditions.

In some European composting operations, within-vessel systems have been developed and operated for many years. Within-vessel composting is intended to provide a system with operational control that is superior to that available with conventional windrow or static-pile composting techniques. There are additional advantages, in that: the composting sludge is sheltered from

the environment; land requirements are lower than for windrow or static-pile composting; and surrounding areas are protected from odor and other undesirable results of open composting sites.

Often enclosed and mechanized systems are viewed as a black box performing a composting operation in a different and hopefully in an efficient way. This misunderstanding results in skepticism toward the system or in an unreasonably high expectation of the system. In order to evaluate a mechanical system, it is important to realize that regardless of the degree of mechanization the basics of the composting are the same, i.e., the process is aerobic biological degradation.

Development of an effective composting system follows three major steps.

1. Understanding of the major factors affecting the composting and developing control strategies for optimization of the process.
2. Development of the hardware to effectively implement the optimization concepts developed in the first step. This involves the development of configuration of the system and all the equipment needed in the operation.
3. Evaluation of the developed system.

Although more research is needed for complete understanding of the composting process, there is enough information available for private industry to develop and promote mechanical composting systems. About 10 mechanized systems, most of them enclosed, are presently marketed in the United States.

During the last eight years, the Ultimate Disposal Section supported developments in windrow and aerated pile composting systems. Recent projects, however, deal with optimization of composting and evaluation of systems developed by private companies. The material to be presented in this paper is based on the two ongoing research projects cooperatively conducted by the Ultimate Disposal Section of the Wastewater Research Division, United States Environmental Protection Agency, and a) Dr. Melvin F. Finstein, Rutgers, The State University of New Jersey, b) Dr. Harry A. J. Hoitink, Ohio Agricultural Research and Development Center.

In the Rutgers project, the optimization of composting using temperature as a control measure is being investigated. In the Ohio Agricultural Research and Development Center project, the concepts advanced at Rutgers are being evaluated in a highly automated enclosed system at the facilities of Compost Systems Company in South Charleston, Ohio.

BACKGROUND (1,2)

Major factors affecting biological activity and, therefore the composting process, are:

1. Nutrients and their availability
2. Moisture content
3. Oxygen supply and aeration
4. Temperature

Nutrients. Generally municipal sludge includes all the necessary nutrients for the growth of aerobic organisms although not always at the right proportions. An important parameter in terms of nutrients is the Carbon:Nitrogen Ratio (C/N). It is important because it provides a useful indication of the probable rate of organic matter decomposition. Microorganisms use about 30 parts of carbon for each part of nitrogen. Thus, an initial C/N ratio of 20 to 35 would be most favorable for rapid conversion of organic wastes into compost. Sewage sludges usually have C/N ratios of less than 15. Although decomposition will be rapid at this ratio, nitrogen may be lost as ammonia. The addition of woodchips or other organic bulking materials raises the C/N ratio, ensuring the conversion of available nitrogen into organic constituents of the biomass. The subsequent removal of the woodchips for reuse then lowers the C/N ratio, allowing N to mineralize.

Moisture Content. Sewage sludges can be composted aerobically over a wide range of moisture contents, 30% and higher, if aeration is adequate. However, excessively high moisture contents should be avoided in most aerobic composting systems, because water displaces air from the pore spaces and can quickly lead to anaerobic conditions. On the other hand, if the moisture content is too low (less than 40%) stabilization will be slowed because water is essential for microbial growth. The most favorable moisture content for composting sludge (22% solids) with woodchips by the aerated pile method is from 55 to 65% in the sludge-chip mixture.

Oxygen Supply and Aeration. In composting sewage sludge, aeration is essential for the growth of thermophilic microorganisms to ensure rapid decomposition, odor abatement, and stabilization of the residual organic fraction which remains as compost. Aeration also provides for lowering the moisture content of composting materials that may have initially been too high. The forced aeration system used with the Aerated Pile Composting Method provides for internal oxygen levels of from 5 to 15%. Within this range, maximum temperatures are attained to ensure pathogen destruction and rapid stabilization. Proper control of the aeration rate is essential because too high a rate can lead to excessive heat loss, cooling of the pile, and incomplete composting and stabilization.

Temperature. Temperature profoundly affects the growth and activity of microorganisms and, consequently, determines the rate at which organic materials are composted. Most of the microorganisms in sewage sludge are mesophilic; that is, they grow best in the temperature range of 20 to 35 °C. However, as temperatures increase during composting, a specialized group of microorganisms becomes predominant. These are thermophilic aerobic organisms that develop only at higher temperatures and grow fastest at 45 to 65 °C. They generate sufficiently high temperatures to destroy human pathogens.

In this paper the work on the correlations between temperature, aeration and moisture content will be presented. The experiments on the effects of major nutrients, Carbon to Nitrogen ratio and alternative bulking agents are still continuing and will be presented later.

RUTGERS EXPERIMENTS (3,4)

In composting, a basic process control objective is to maximize microbial activity at the expense of the waste being treated. This is equivalent to maximizing metabolic heat output. To approach this objective it is necessary to consider that, in the self-heating ecosystem, temperature is both effect and cause. The temperature is a function of the accumulation of heat generated metabolically, and simultaneously the temperature is a determinant of metabolic activity. The interaction between heat output and temperature is the centerpiece of rational control of the composting process.

Soon after organic material is assembled into a self-insulating mass the temperature starts to increase as metabolic heat accumulates. At first mesophilic growth is stimulated by the higher temperatures but, as inhibitive levels are reached, this leads to a self-limiting condition. Because the elevated temperature now induces thermophilic growth the pattern is repeated in a second, hotter stage. At peak thermophilic temperatures the metabolic activity is relatively slight. In sum, the system is prone to self-limit via the excessive accumulation of heat. It has been reported that the temperature most conducive to organic matter decomposition is greatest at 52 to 60 °C, and that a steep decline starts above this upper boundary.

One of the purposes of the Rutgers study was to devise a practical means of controlling temperature in field-scale composting, and to evaluate such an approach to process control.

Composting materials. Sludge was obtained from the Camden County Municipal Utilities Authority, Jackson Street Sewage Treatment Plant, Camden, New Jersey. Approximately 90% of this material was primary sludge (not subjected to biological treatment). The remaining 10% consisted of "partially digested" sludge derived from a separate area.

Routine practice at the treatment plant is to add 1 kg of chloride based polyelectrolyte polymer conditioning agent per metric ton of dry solids followed by dewatering in a belt filter press. The resultant sludge cake has a moisture content at 103 °C of approximately 75% (wet weight basis of expression) and the oven dry material has a volatile solids content of approximately 75%.

To provide porosity, sludge cake and virgin woodchips (nominally 2.5 x 2.5 x 0.6 cm) were combined (approx. 1:1.8, w/v) and mixed in an industrial pug mill.

Composting pile. A pile consisted of a mixture of sludge and woodchips, with a base and cover of woodchips only. The pile contained approximately 6 metric tons (Figure 1).

Ventilation system. Corrugated plastic hose (10 cm internal diameter) served as ventilation duct work. Blowers capable of delivering up to 15 m³ of air per minute were operated in the force pressure mode.

Temperature control system. A temperature controller with an adjustable temperature set point continuously received and interpreted a signal from a thermistor in the pile (Figure 2,3). When the signal indicated a temperature less than the set point the controller actuated the blower on a periodic schedule preset with a timer. When the signal indicated a temperature greater than the set point the controller directly actuated the blower, which remained in operation continuously until the temperature was lowered to less than the set point. Thus the control system was based on the feedback of temperature information from a selected position in the pile.

Pile temperature. Representative temperatures are shown in Figure 4. In all of the piles the thermistor and a thermocouple were both at position 1. The temperature data from the period of feedback control are summarized in Table 1. During this period the median temperature in pile A exceeded the controller setting for this pile by 3 °C, while the excess values for piles B and C were 7 °C and 2 °C, respectively.

In pile A, the temperature at position 1 during the period at feedback control was essentially that of the assigned set point (45 °C). The only appreciable departure from the set point occurred between hr 92 to 136. During this period the peak temperature was 53 °C, at hr 112. The departure occurred while blower operation was continuous.

Control was generally less precise at the lateral positions (e.g., A 3 and A 12). In this direction the least precise control was at the outermost position. Higher temperatures occurred in the uppermost areas of the pile (e.g., compare the temperatures at positions A 1, A 6 and A 13).

Similarly, in piles B and C during the period of feedback control, there was a close correspondence between the set points (55 °C, and 65 °C respectively) and pile temperatures near the thermistors (positions B 1 and C 1). Lateral to these positions, and in the upward direction, control was less precise. In pile C the ventilation scheduled by timer generally induced early, precipitous, temperature declines.

Performance at selected temperature set points. Three piles, each consisting of 6 metric tons of sludge, were made on 10 May 1979, and the main part of the experiment was terminated on 31 May. The set points assigned to the temperature controllers were: pile A, 45 °C; pile B, 55 °C; pile C, 65 °C. During the experimental period, the ambient temperature varied between 8.9 °C to 30.6 °C and total rainfall was 13 cm in 12 occurrences.

Blower operation. In pile A blower operation first exceeded that scheduled by timer at hr 12, indicating that the controller had responded to a thermistor temperature of > 45 °C (Figure 3). This marks the start of the period of temperature feedback control. Blower operation was nearly continuous from hr 80 to 150. Feedback control terminated at hr 352, and blower operation reverted to that scheduled by timer (7% of the time).

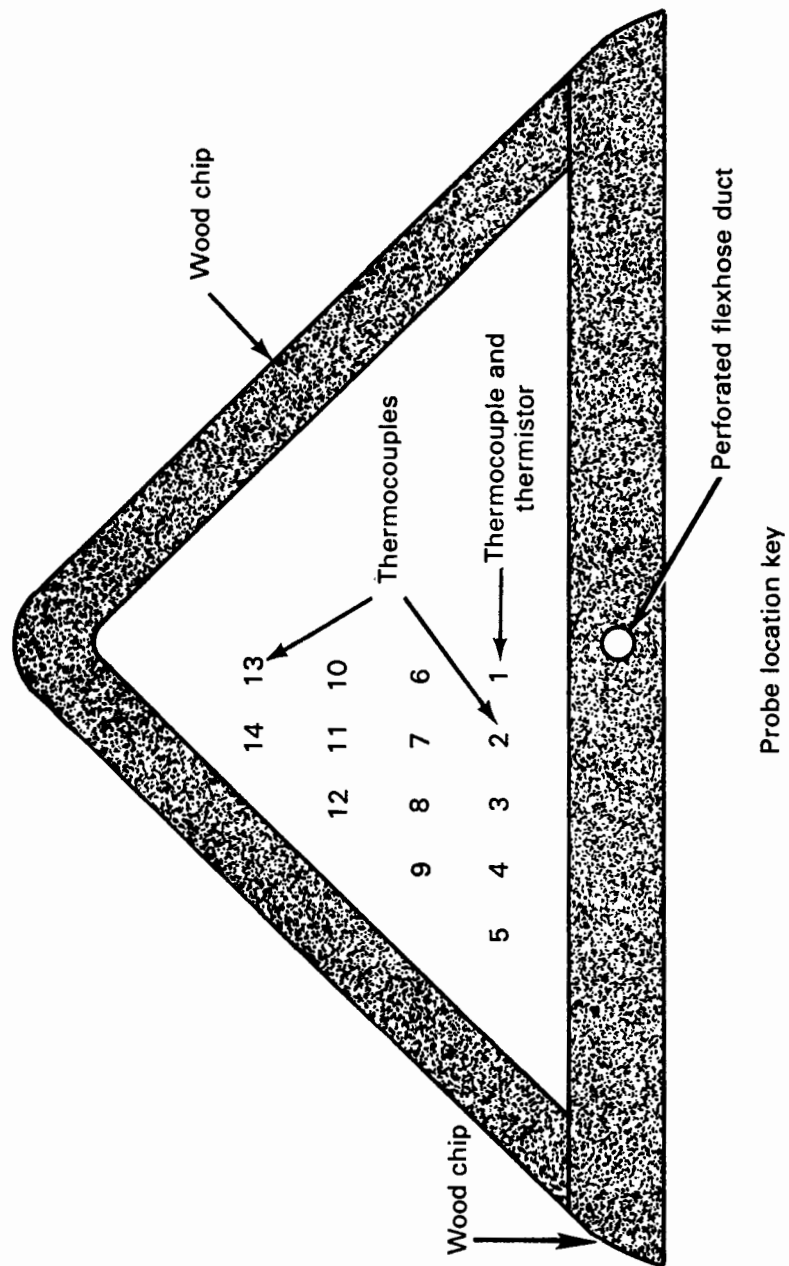


Figure 1. Six-metric-ton composting pile cross section

Table 1. Temperature During the Periods of Blower Demand

<u>Controller set point (°C)</u>	<u>Period of blower demand (hr)</u>	<u>Pile tempera- ture (°C)</u>		<u>Observations ^a</u>	
		<u>Range</u>	<u>Median</u>	<u>No.</u>	<u>% > 60°C</u>
45 (pile <u>A</u>)	12 to 352	25 to 63	48	1020	1.8
55 (pile <u>B</u>)	26 to 214	18 to 78	62	641	58
65 (pile <u>C</u>)	28 to 84	64 to 74	67	182	100

^a 60 °C is considered the threshold to significant self-limitation

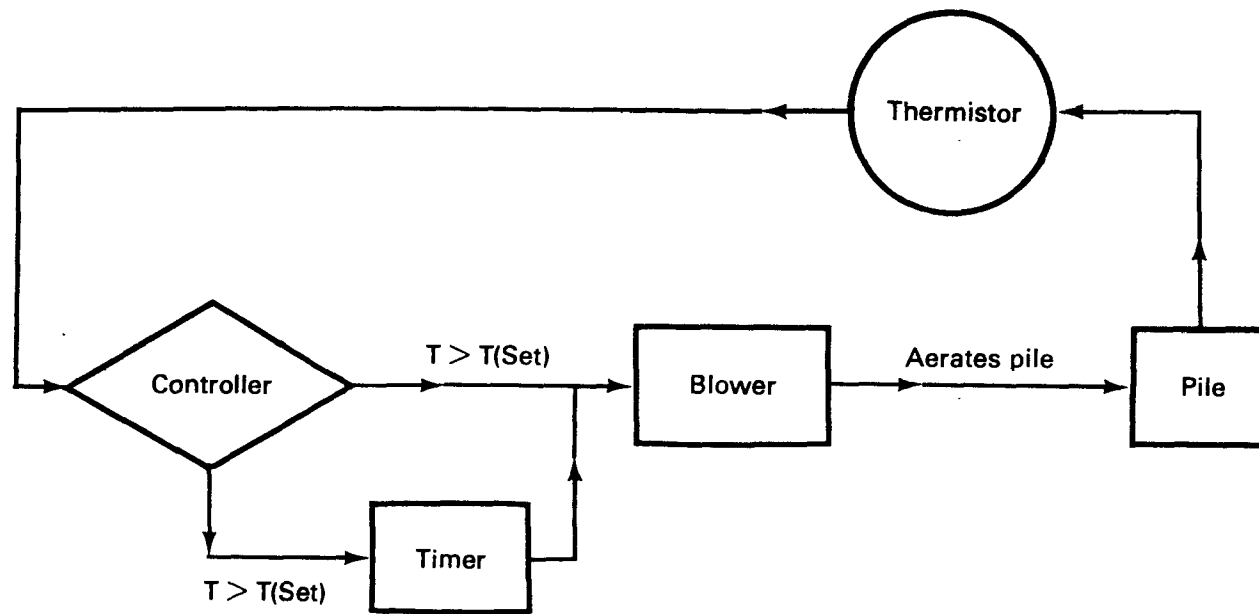


Figure 2. Schematic of the temperature feedback-blower control system

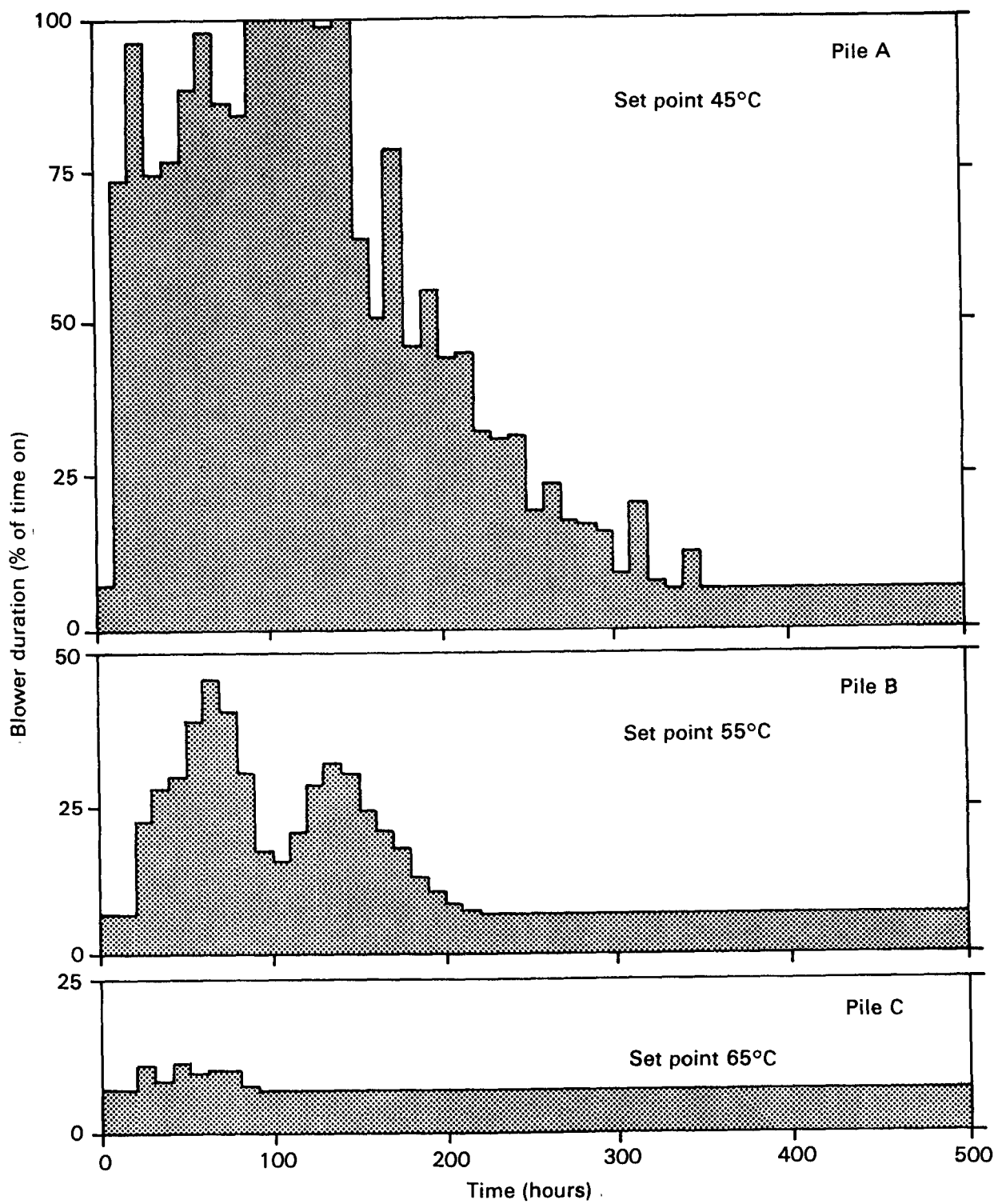


Figure 3. Mean blower operation

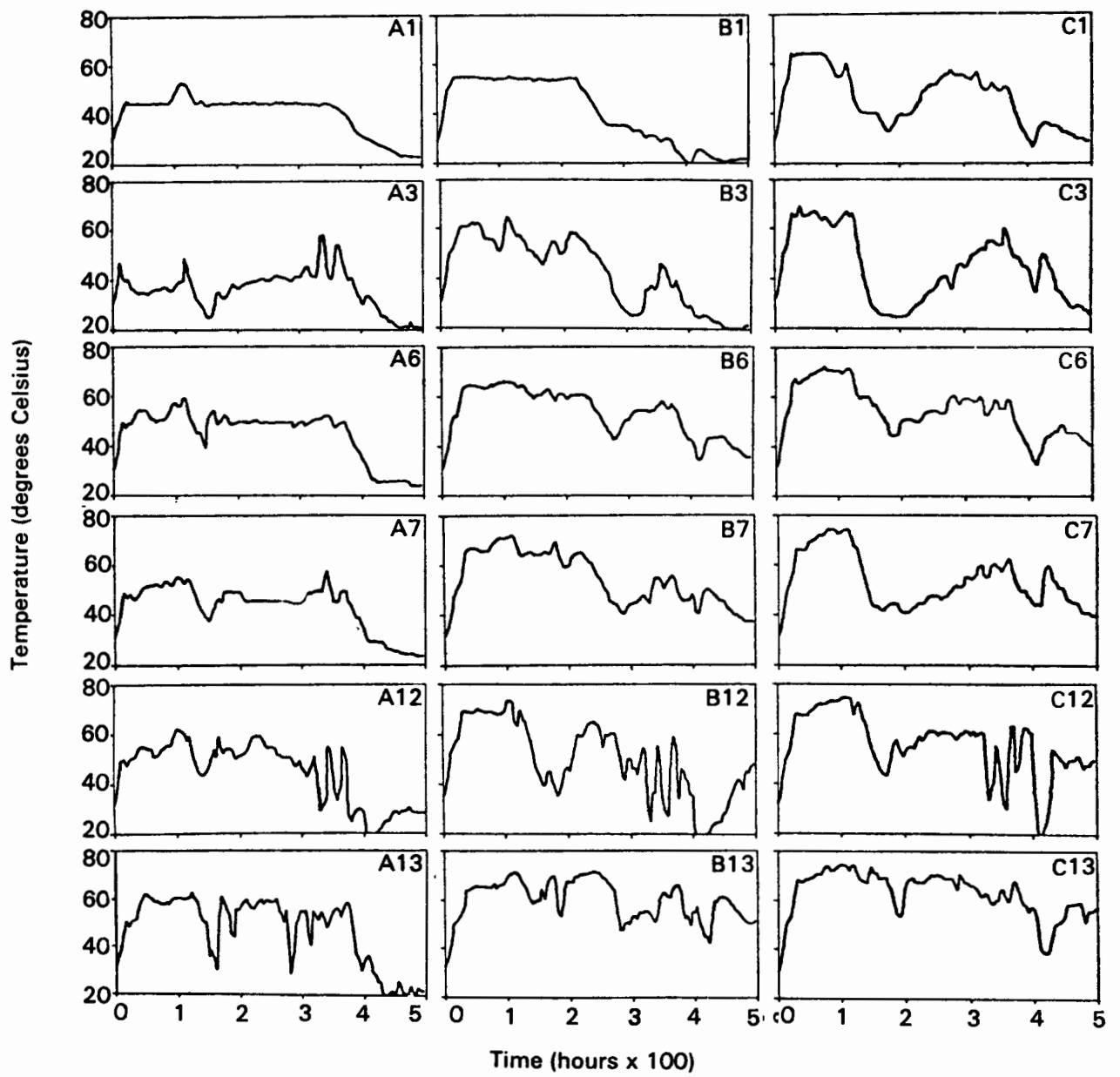


Figure 4. Temperatures at selected points

The decrease in moisture content during composting indicates organic matter decomposition. This is because drying and decomposition are linked via heat output and vaporization. The observation that pile A dried faster than piles B and C therefore represents a field-scale demonstration of the heat output-temperature interaction concept, which requires the most extensive microbial heat output to occur in the coolest pile.

For pile B, the period of feedback control was from hr 26 to 214, during which blower operation peaked at 45% at hr 70. For pile C, the period of feedback control was at hrs 28 to 84, and the peak was 12% at hr 50.

Carbon dioxide. Where the record is complete (piles A and C) the highest CO₂ levels occurred prior to the initiation of temperature feedback control (Figure 5). During the period of feedback control the CO₂ values averaged approximately 2, 3, and 4% for piles A, B, and C, respectively. During the terminal period of timer-scheduled blower operation the CO₂ values were lower.

pH. The starting pH was 6.3 and this increased, at a different rate in each pile, to approximately 8.2 (Figure 6). This was followed by a slight decline. A secondary increase in pile A started after hr 380.

Moisture content. In pile A, the starting moisture content of 76% decreased to 22% in 15 days (Figure 6). In piles B and C, the decrease was to 40% in 18 days.

Discussion

A practical means of controlling temperature was devised in the form of the temperature-feedback control system in conjunction with forced-pressure ventilation.

A definitive comparison of pile A (coolest), pile B (intermediate), and pile C (hottest) based on CO₂ output and heat output is not possible in this open system, as these measurements of decomposition cannot be reliably quantified from the data. However, the results of the odor test indicate that the rate of decomposition was A > B > C. The moisture content data provide a similar indication as the decrease in moisture content during composting indicates organic matter decomposition. This is because drying and decomposition are linked via heat output and vaporization. As indicated previously, the observation that pile A dried faster than piles B and C therefore represents a field-scale demonstration of the heat output-temperature interaction concept, which requires the most extensive microbial heat output to occur in the coolest pile.

OHIO AGRICULTURAL RESEARCH AND DEVELOPMENT CENTER AT SOUTH CHARLESTON, OHIO(5)

One of the main purposes of this research was to demonstrate if the temperature controlled composting developed at Rutgers and explained previously here can be implemented in a mechanized enclosed tank reactor. For this purpose an existing composting facility operated by Paygro, Inc., has been used with the permission of the owners. Although Paygro, Inc., operates

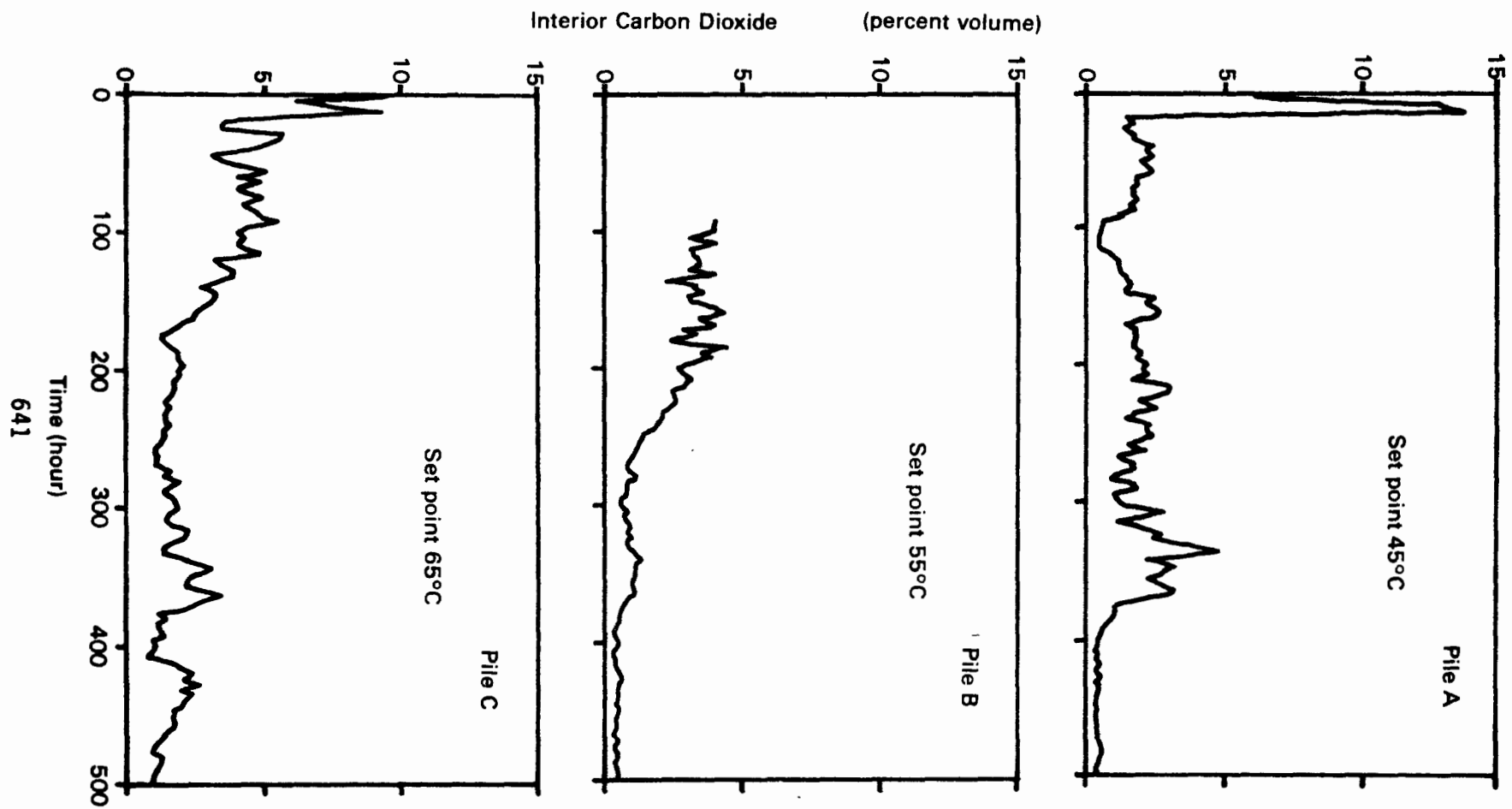


Figure 5. Carbon dioxide content in the three piles

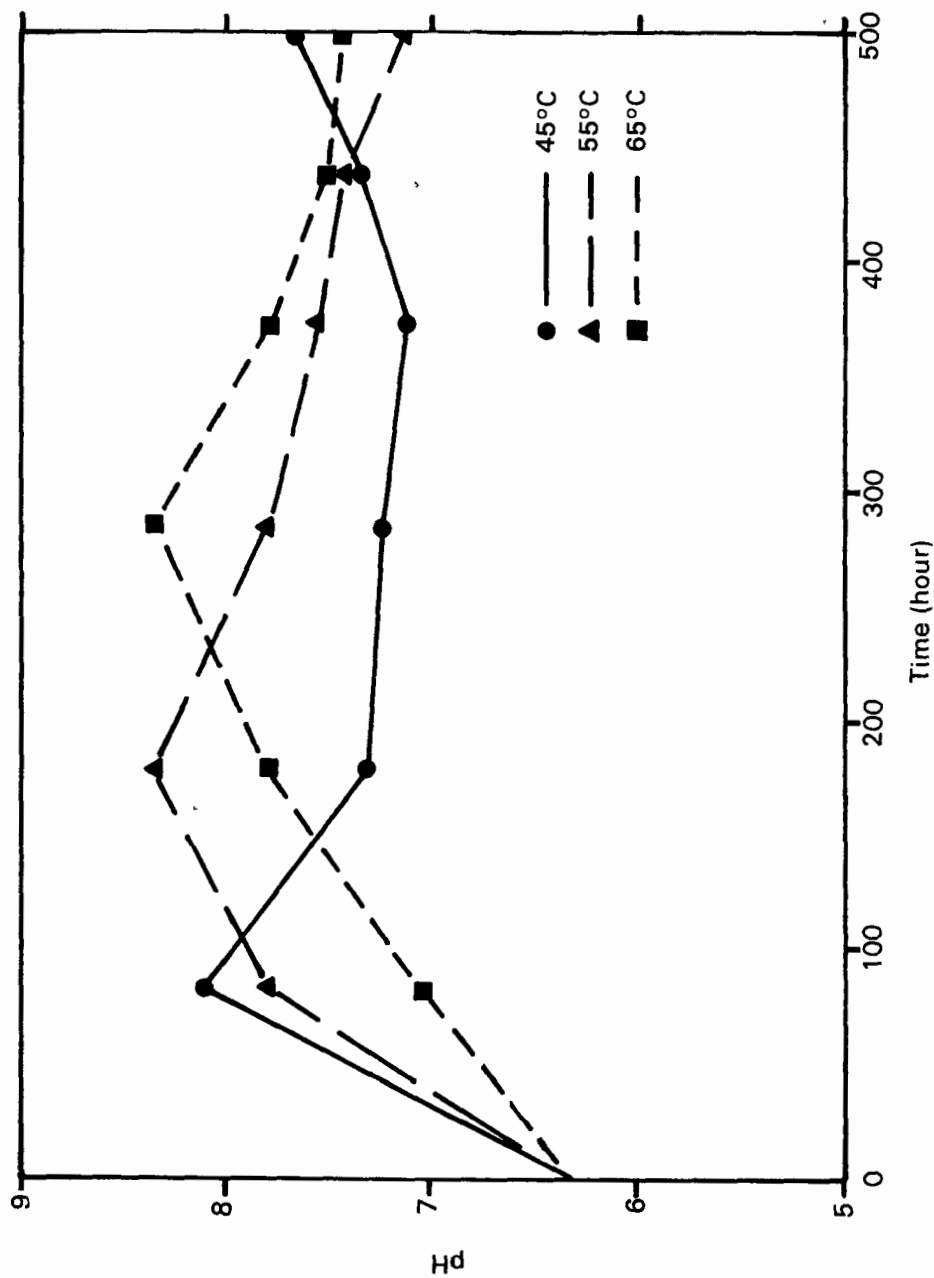


Figure 6A. pH in the three piles

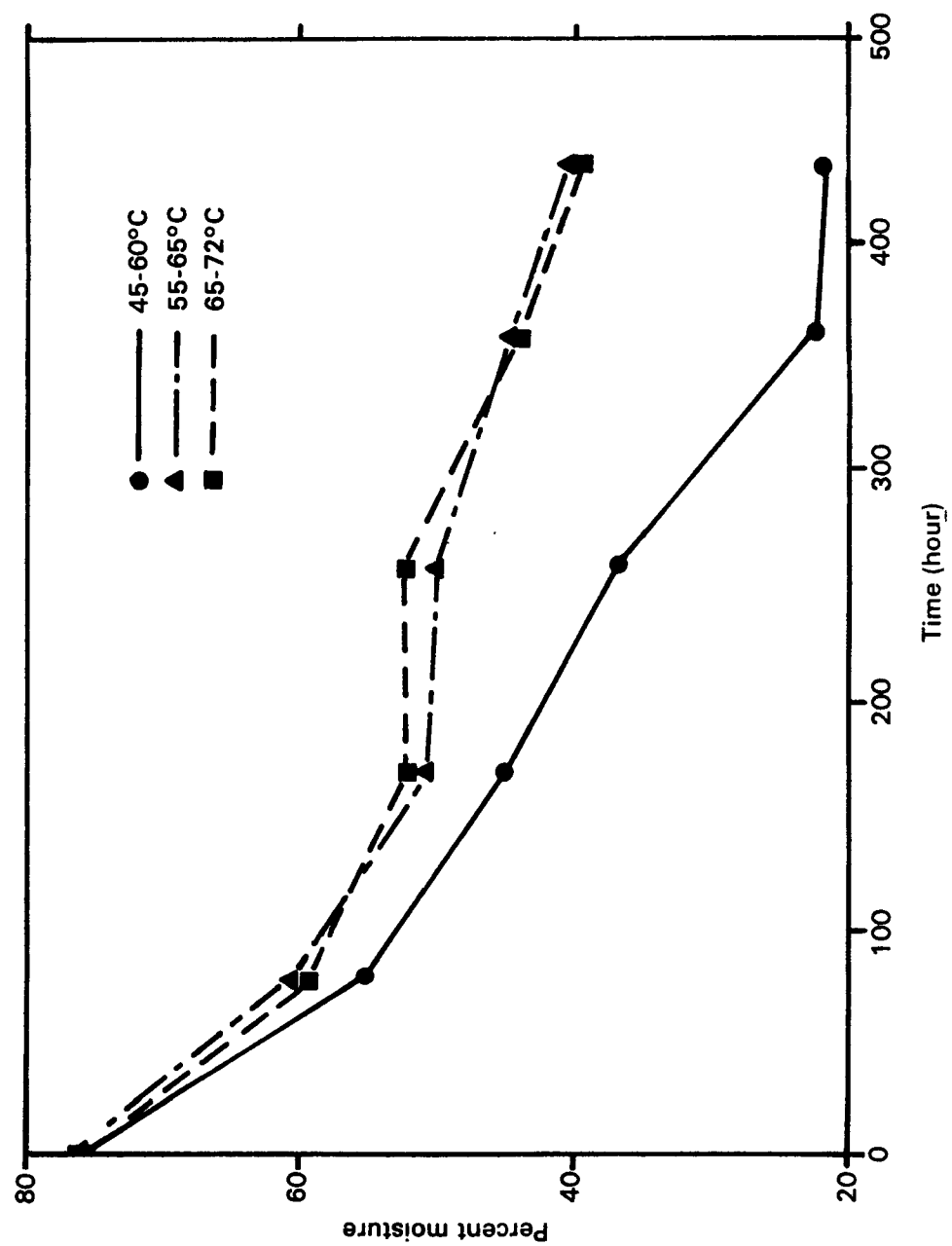


Figure 6B. Moisture content in the three piles

mechanical equipment and provides other needed services, the experiments were designed, operated and evaluated by a research team from Ohio Agricultural Center.

Description of Paygro Bioreactor

The bioreactor was constructed in 1972 and has been in operation since then. Until recently the system has been used mainly for composting feed lot manure and bark. The composted materials have been bagged and sold to the horticultural market. During the last year portions of the facilities have been used for research with municipal sludge. A diagrammatic layout of the facilities is shown in Figure 7.

The composting operation for the manure consists of feeding a mixture of wet manure and sawdust, used as bedding in the cattle pens, to the metering hopper via front end loader.

The metering system consists of a flat bottom hopper and incline drag conveyor, all hydraulic driven with variable speed controls. An operator controls the feed rate from the metering system to the center conveyor system.

A center conveyor, tripper car and indexing conveyor transports the material to the reactors.

The reactors are two parallel 122 m long structures, each 6 m wide by 3 m deep. The reactor walls support the tripper car, indexing conveyor and the digging machine.

Air is forced through the reactors by a series of fans located along the reactor wall. One 7-1/2 HP fan provides 1.4 to 2.4 m³/s of air for a reactor contents of approximately 226 m³. The fans are designed to blow air up through or pull it down into the reactor contents.

Air is equally distributed through a perforated plate and a gravel support bed located in the bottom of the reactor structure. The material is aerated for 14 to 21 days. While in the reactor the mix is removed and remixed 1 to 2 times before leaving the reactor for storage and bagging. There is no requirement for recovery of any bulking agent for reuse. The material enters the reactor at 40 to 50% dry solids and leaves at 60 to 70% dry solids.

The material is removed by the use of the digging machine and trailing conveyor (extractoveyor). This equipment travels the entire length of the reactors on rails and is hydraulically operated. The extractoveyor discharges the material to the center conveyor. The center conveyor either relocates the material by the use of the tripper car and indexing conveyor or discharges the finished compost material to storage.

Experiments

Raw municipal sludge was received from the Columbus Southerly Plant on May 26 and 27, 1981. It was mixed with recycled compost and hammer-milled

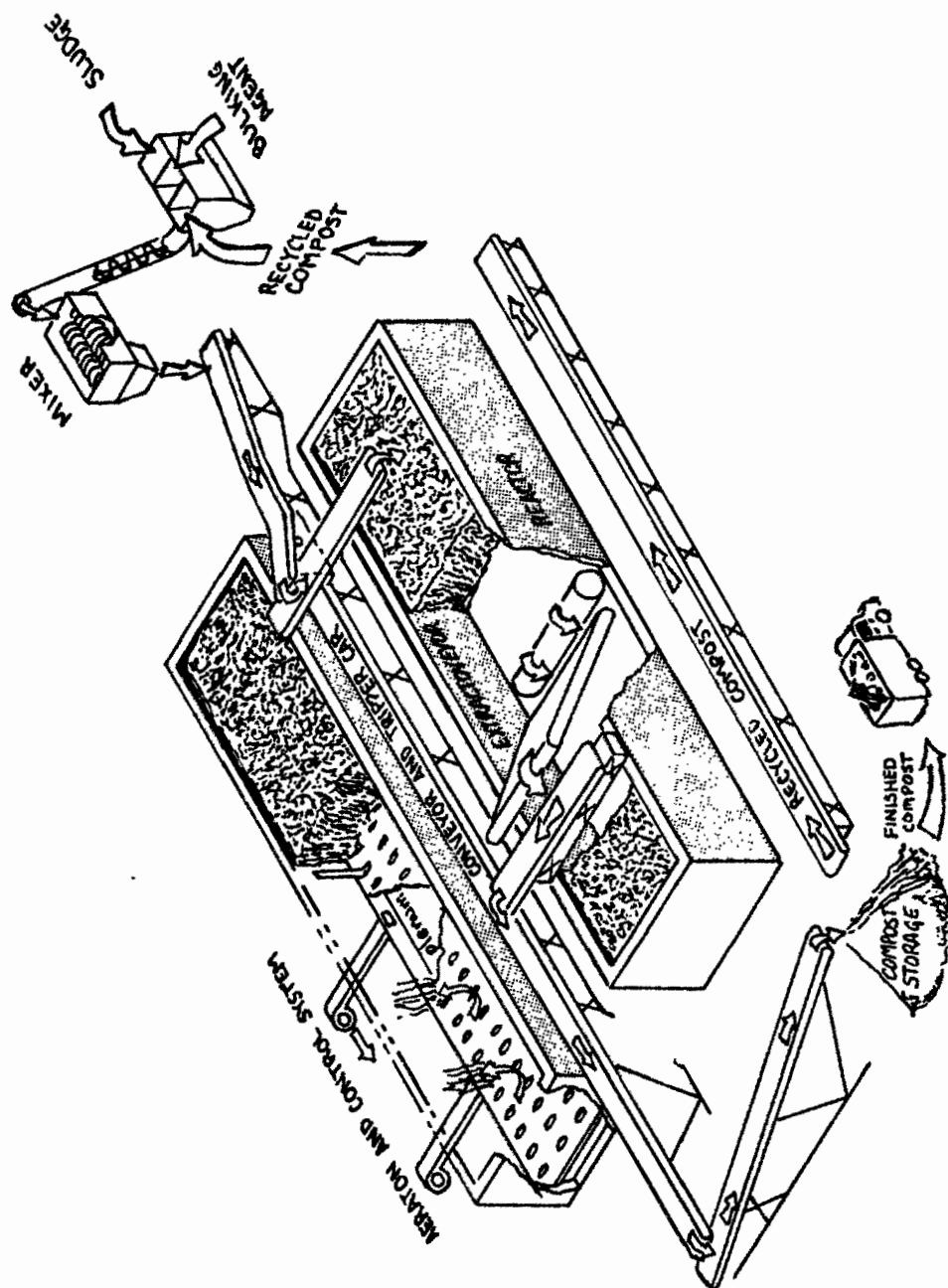


Figure 7. Layout of Paygro Composting Facilities

bark (approximately 80% red oak) at volume ratios of 1:2:8:1:6 (sludge: recycled compost:bark). The mixture (Table 2) was loaded into the reactor on May 28, 1981. The mean percent dry solids of the reactor feed was 40.8 and the mean percent volatile solids was 72.9.

The reactor feed was placed in two adjacent 12 m sections (A and B) of the reactor, which were separated by a plastic wall. Each section was aerated with a separate fan to be operated under different aeration schemes. However, an air leak was discovered between the sections on the first day. Aeration for each section therefore could not be controlled separately. Fans were controlled by 30 min timers and were on 60% of the time (18/30 min) throughout the trial.

Automatically collected CO₂ data are not yet available for this trial. However, the percent CO₂ in exhaust air reached a high of 14% on the first day and remained below 7% throughout the rest of the trial. The sum of O₂ and CO₂ was constant throughout the trial.

Free airspace (FAS) values for the reactor feed, the reactor product and the compost after 10 weeks of curing were 49, 52, and 45%, respectively.

Temperature was recorded at the fans and 40 locations in each of the Sections A and B (Figure 8).

The compost was turned twice. It was removed on June 1 and re-entered the reactor on June 2. The second turn occurred on June 10 and 11. It was finally removed from the reactor on June 18. The total composting period in the reactor therefore was 326 hrs (18 days). The actual time elapsed from the start to the final removal was 23 days.

Differences in the volatile solids and dry solids levels between Sections A and B were not significant (Table 3). Drying occurred at significant levels (5%) throughout the 18 day period.

The mass balance is presented in Table 4 (pooled data from Sections A and B). The percent loss in volume of the total mass in 18 days was 22.4%. The percent loss in wet weight was 40.8. The percent loss in dry weight was 15.1 and the percent loss in volatile solids (of original) was 21.1.

The distribution of the moisture (percent dry solids) with depth in the reactor at the time of the second turn (day 11) and at removal (day 18) is presented in Tables 5 and 6. The surface of the compost did not dry throughout the trial. Considerable drying occurred at the 15 cm depth (less condensation than at the surface). There was little difference (5% level) in drying at deeper levels in the reactor. No significant wall effect was detected, except for near the end of the trial, when samples from within 15 cm from the wall of the reactor were significantly wetter than all others.

Significant differences (5%) in levels of volatile solids destruction were not observed among the various depths in the reactor at either the 11 or the 18 day period (Tables 7 and 8). Wall effects were either not detectable or did not show a consistent pattern.

Table 2. Description of Components and Reactor Feed Mixture

Component	Mixing ratio ^a (volumes)	Bulk density kg/m ³	Mean % solids	Mean % volatile solids ^b
Raw sludge ^c	1	1063	14.5	72.7
Recycled compost	2.8	605	58.0	65.3
Bark	1.6	414	57.2	84.5
Reactor feed ^d	-	679	40.8	72.9

^a Determined by compacting (0.7 kg/cm², 10 lbs/sq in) samples into a 28.3 liter (1 cu ft) square box.

^b Mean of 10 or more samples of approximately 100 gm wet weight each.

^c Raw municipal sludge (98.4 M tons net weight) was obtained (5/26/81) from the Southerly Treatment Plant, Columbus, Ohio.

^d Total volume = 439 m³.

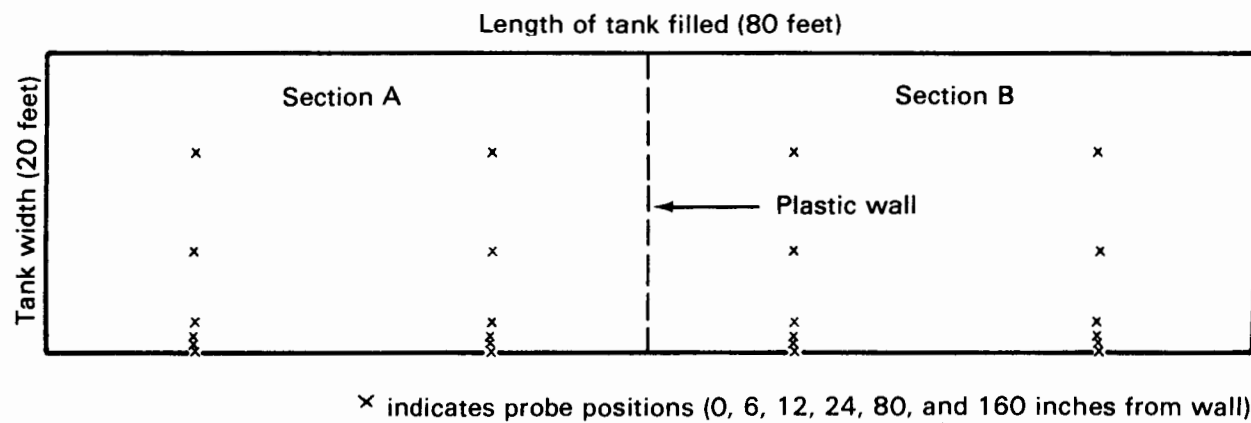


Figure 8. View of tank showing temperature probe positions

Table 3. Changes in Percentages Solids and Volatile Solids During Composting
in Two Sections (A and B) in the Reactor

	Section sampled	Number of samples	Percent solids			Percent volatile solids		
			Mean	95% Conf.	Int.	Mean	95% Conf.	Int.
Reactor feed	A B	18	40.8	±	1.3	72.9	±	1.6 ^a
First turn (day 4)	A B	10	43.3	±	0.4	73.3	±	1.3
Second turn (day 11)	A	10	52.0	±	0.4	69.9	±	1.1
	B	10						
Reactor product (day 18)	A	10	58.4	±	1.4	64.2	±	0.7
	B	10						

^a Means followed by a common line are not significantly different according to Duncan's new multiple range test (P = 0.05) (For description of Duncan's test, see reference 6).

Table 4. Materials Balance for Composting Trial 3

	Volume ^a (m ³)	Bulk ^b density (kg/m ³)	Wet ^c weight (M tons)	% Dry ^d solids	% Volatile ^d solids	Weight dry ^e solids (M tons)	Weight ^e water (M tons)	pH
Reactor feed	439.4	678.5	298.2	40.8	72.9	121.7	176.5	5.5
Reactor product	341.0	517.7	176.5	58.5	64.2	103.3	73.2	6.5
Loss during composting ^f	98.4		121.7			18.4	103.3	
% loss	22.4		40.8			15.1	58.5	

^a Measurements were made in the reactor (error < 4%).

^b See Table 1 for explanation.

^c Calculated from bulk density and volume.

^d Means of a minimum of 10 samples of approximately 100 g wet weight each.

^e Calculated from wet weight and percent solids or volatile solids.

^f After 18 days in the reactor.

Table 5. Mean Percent Solids Distribution in th Reactor
at Time of Second Turn (Day 11)

Depth (inches from surface)	Distance from wall (inches from east wall)						Means for depth ^a
	6	24	80	107	133	160	
2	38.0 ^b	39.3	36.9	40.5	40.4	37.6	38.8
6	50.9	53.1	47.4	47.0	46.4	45.3	48.3
31	52.4	52.9	50.6	51.7	50.0	49.6	51.2
52	51.9	50.0	51.1	51.7	52.2	51.4	51.4
73	49.9	49.7	52.1	54.6	52.3	51.8	51.6
94	50.3	54.0	53.0	52.2	52.0	55.0	52.7
Means for distance from wall ^c	48.8	49.8	48.5	49.6	48.9	48.5	

^a Means (of 24 samples each) joined by a common line are not significantly different according to Duncan's new multiple range test $LSD_{.05} = 1.5$.

^b Means of four values.

^c No significant difference among means ($P = 0.05$).

1 inch = 2.54 cm

Table 6. Mean Percent Solids Distribution in the Reactor
at Time of Removal (Day 18)

Depth (inches from surface)	Distance from wall (inches from east wall)						Means for depth ^a
	6	24	80	107	133	160	
2	41.2 ^b	40.8	41.9	41.3	41.0	40.6	41.1
6	59.6	54.3	50.6	50.0	54.9	54.9	54.0
31	60.5	58.1	56.7	56.0	56.6	57.5	57.6
52	62.7	60.2	57.6	56.3	57.6	55.9	58.4
73	58.6	60.1	60.1	57.7	56.7	58.3	58.6
94	57.2	55.1	60.0	62.6	61.2	60.3	59.4
Mean for distance from wall ^c	56.6	54.8	54.6	54.0	54.7	54.6	

^a Means (of 24 samples each) joined by a common line are not significantly different according to Duncan's new multiple range test $LSD_{.05} = 1.6$.

^b Mean of four values.

^c No significant difference among means ($P = 0.05$).

1 inch = 2.54 cm

Table 7. Mean Percent Volatile Solids Distribution in the Reactor
at Time of Second Turn (Day 11)

Depth (inches from surface)	Distance from wall (inches from east wall)						Means for depth ^a
	6	24	80	107	133	160	
2	69.7	70.1	69.9	69.8	69.8	67.9	69.2
6	69.6	69.5	67.6	68.7	70.1	70.6	69.3
31	72.9	71.5	67.1	71.9	67.7	71.6	70.5
52	67.9	72.1	70.6	69.8	70.2	72.3	70.5
73	70.4	70.1	70.5	67.6	69.9	72.8	70.2
94	75.1	69.6	68.3	67.4	69.6	68.7	69.8
Means for distance from wall ^c	70.9	70.6	68.5	69.2	69.6	70.7	

^a Means (of 24 samples each) joined by a common line are not significantly different according to Duncan's new multiple range test ($P = 0.05$).

^b Mean of four values.

^c No significant difference among means ($P = 0.05$).

1 inch = 2.54 cm

Table 8. Mean Percent Volatile Solids Distribution in the Reactor at Time of Compost Removal (Day 18)

Depth (inches from surface)	Distance from wall (inches from east wall)						Means for depth ^a
	6	24	80	107	133	160	
2	62.9 ^b	66.9	65.0	68.3	66.2	66.0	65.9
6	64.6	65.2	63.6	63.4	63.7	62.4	64.4
31	64.1	61.8	62.6	64.0	61.8	64.3	64.4
52	65.3	65.3	65.3	65.2	63.8	63.7	63.9
73	63.7	65.0	65.2	63.1	63.7	62.5	63.9
94	64.6	63.1	66.8	63.7	63.1	64.9	63.1
Means for distance from wall ^c	63.8	64.5	64.7	64.6	63.7	64.0	

^a Means joined by a common line are not significantly different according to Duncan's new multiple range test $LSD_{.05} = 1.4$.

^b Mean of four values.

^c No significant difference among means ($P = 0.05$).

1 inch = 2.54 cm

Table 9. Mean Temperatures of Compost in Various Thermocouple Positions for Selected Time Periods

Total hours elapsed	Depth (inches)	Distance from wall (inches) ^a				
		Wall	6	12	24	80
88	6	- ^b	-	-	-	-
	31	-	-	-	-	73.1
	52	31.4	33.5	41.7	44.0	71.4
	94	20.3	22.2	24.6	35.2	59.3
First Turn						
184	6	34.3	47.8	49.9	51.1	58.9
	31	-	-	-	-	59.0
	52	32.9	36.0	34.7	49.4	54.5
	94	22.4	18.4	24.6	30.9	34.9
Second Turn						
160	6	45.7	53.0	53.6	54.6	59.2
	31	-	-	-	-	59.5
	52	39.4	41.5	41.8	40.3	56.5
	94	25.3	26.4	24.0	29.3	37.0
Removal						
432 (total)	6	39.1	49.7	51.2	52.4	59.0
	31	-	-	-	-	62.5
	52	34.4	36.9	38.3	45.7	59.0
	94	22.7	21.5	25.7	31.5	41.3

^a Means of readings taken every 4 hr in four locations for the wall, 6", 12" and 24" positions. Means of eight locations for the 80" position.

^b Missing data.

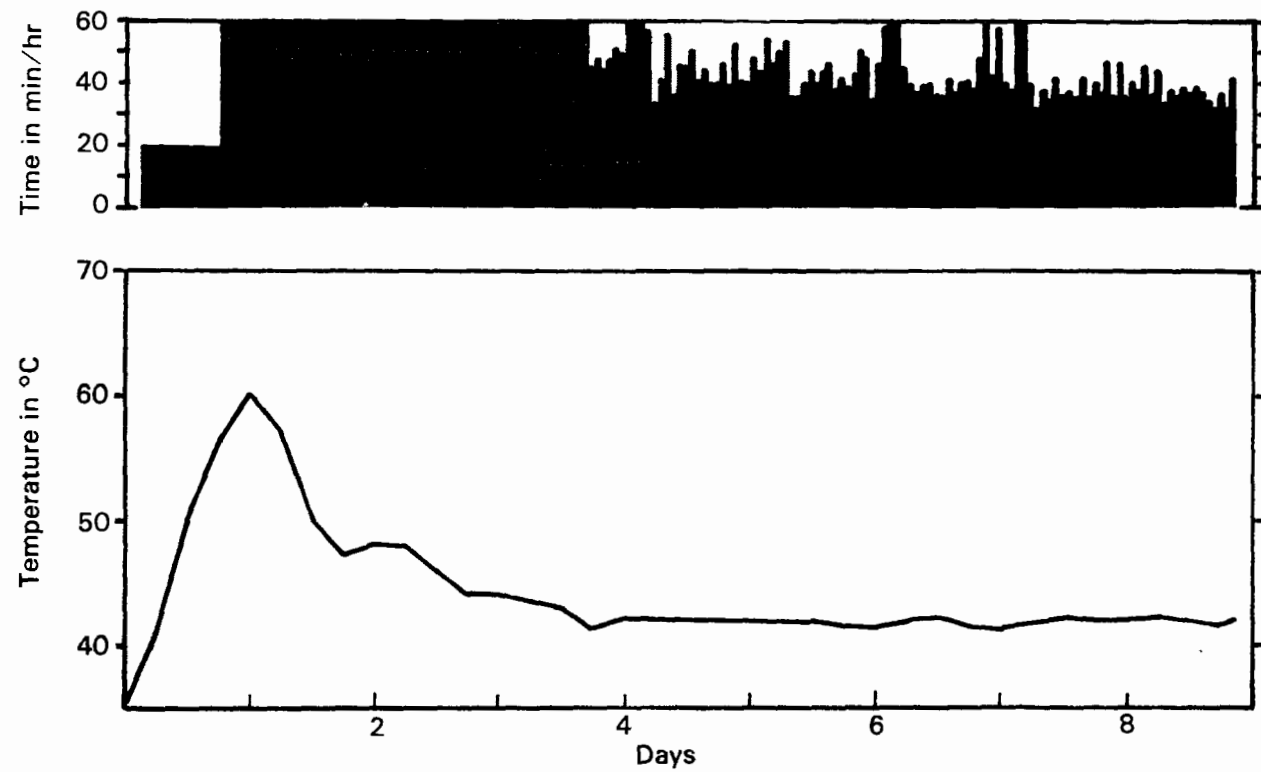


Figure 9. Mean temperatures and fan operation time for the pile with set temperature of 65 °C

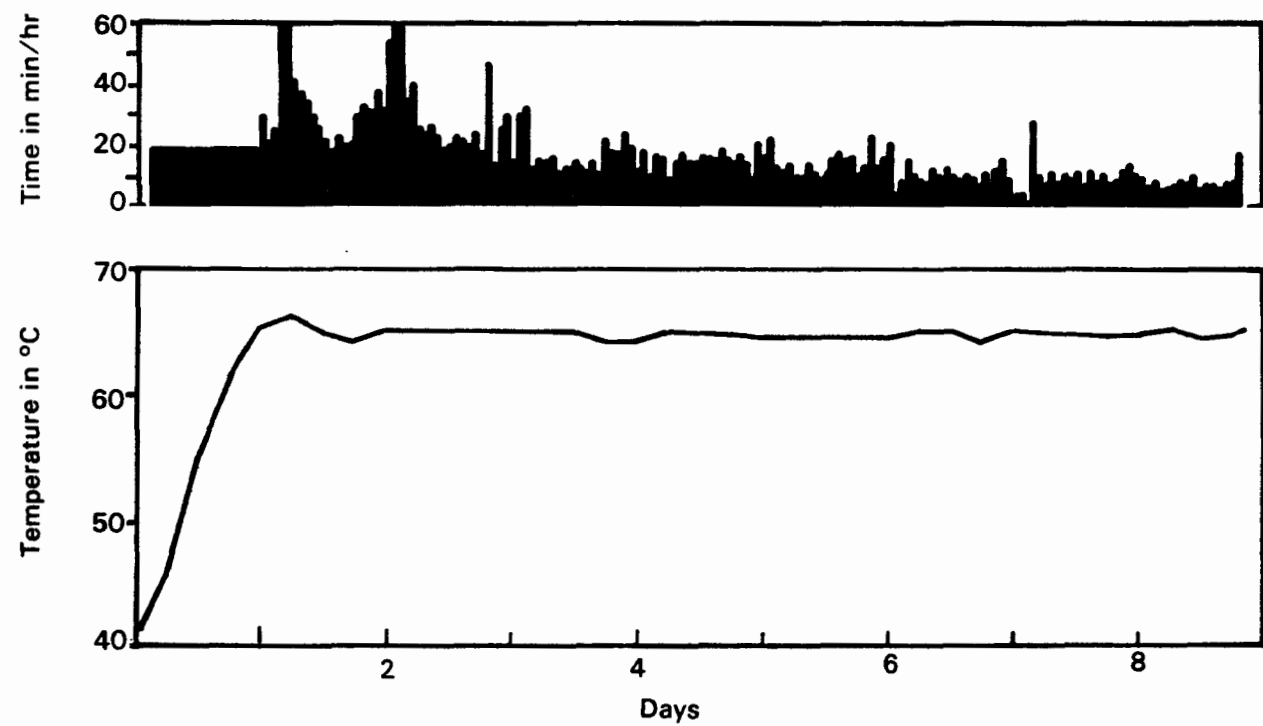


Figure 10. Mean temperatures and fan operation time for the pile with set temperature of 45 °C

Mean temperature readings for the various thermocouple positions in the reactor (combined means of Sections A and B) are shown in Table 9. The temperature of the compost in the bottom foot of the reactor was near that of the air temperature at the fan. At a depth of 127 cm the temperature was similar to that of the rest of the compost at higher locations. Wall effects were obvious up to 30 cm into the reactor.

A comparison of the Paygro bioreactor performance published accounts of the aerated static pile shows the following:

Method	Composting days	% loss in wet weight
Paygro	18	40.8
Beltsville	42	31
Washington	42	29

A comparison of the data presented here with data from Rutgers experiments shows that drying was similar to that of Rutgers results obtained at 55 and 65 °C in 18 days, but less than that obtained at 45 °C.

Test of Automated Equipment

In a second set of experiments, two 12 m reactor sections were separately aerated. The feedback temperatures were set at 45 and 65 °C. The main purpose of the experiments was to evaluate the performance of reactors at two different temperatures and to test the operation of feedback mechanism and the computerized data collecting system. The experiments have just been completed in September 1981 and most of the data analyses have not been completed. However, the mean temperatures and frequency of air blowing operation over a period of 9 days (Figures 9 and 10) indicate that, the control mechanism operates successfully, expected temperatures are maintained, and blower capacities are adequate.

CONCLUSIONS

The research at Rutgers and Ohio Agricultural Research Center has not yet been completed. However, from the results obtained thus far and presented here, the following conclusions can be reached:

1. Two independent sets of experiments confirmed that temperature controlled aeration in the forced pressure mode can maintain the temperature of a composting pile at a desired level.

2. By maintaining the temperature at 45 to 65 °C range, weight losses obtained were higher than the weight losses obtained in uncontrolled composting operations.

3. In an enclosed mechanized system, the temperature was maintained more uniformly throughout the experimental period and throughout the composting pile cross section.

4. In an enclosed mechanized system, temperature controlled forced aeration system appears to provide larger weight losses and drying than the open systems.

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**THERMAL PROCESSES FOR CONVERSION OF SLUDGE:
STATUS, EPA RESEARCH, FUTURE DIRECTIONS**

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INTRODUCTION

"To live is to change, To be perfect is to have changed often" (John Cardinal Newman). Cardinal Newman's sage observation referred to growth of the human spirit, but his words are equally appropriate to technology. What appears appropriate in one decade may in the next decade be thoroughly outmoded by advances in technology, by new knowledge, or by a change in economic conditions or incentives.

Thermal processing of sludge, which has meant incineration of sludge in years past, has been adversely affected by new knowledge and concern about air pollution, and by the increase in cost of fuel relative to other costs. Interest in it has declined and other methods, particularly land application and landfill have been advocated. These alternatives, however, have not proved to be panaceas. Other concerns, among them protection of the land from contamination, have arisen, that have made these latter methods seem inappropriate for certain contaminated sludges. It is evident that despite some problems, there are many situations where properly conducted thermal methods are the best solutions to the sludge disposal problem.

The U.S. Environmental Protection Agency (EPA) has continued a modest thermal conversion program over the years, aimed at reduction of pollution from conventional incineration and development of new approaches to thermal conversion. The following presentation reviews the status of thermal conversion processes, and outlines current EPA research and development efforts and directions.

In order to conduct a discussion of thermal conversion, it is desirable to classify the subject matter in an orderly fashion. The usual approach, organization by the processing procedure (e.g., incineration, starved-air combustion, pyrolysis, etc.) is unwieldy, so a different approach is used herein. The subject matter is organized by the nature of the desired product of the thermal conversion process. Most processes have two or more streams leaving the process, so a choice of the desired stream, usually not a difficult one, must be made. The scheme proposed is the following:

Nature of Desired Product

- (a) Thoroughly combusted gases
- (b) Combustible gases
- (c) Combustible liquids
- (d) Decontaminated water
- (e) Solid products

The classification of processes under these major headings is shown in Table 1. There are many more potential processing routes to the desired product than are indicated. Only those which have been seriously proposed are presented in the table. The discussion that follows expands on each topic outlined in Table 1.

Table 1. Classification of Thermal Conversion Processes by the Desired Product

I Thoroughly Oxidized Gases

- (a) Incineration of sludge
- (b) Addition of low-grade fuel to sludge incinerator
- (c) Addition of sludges to other combustion processes
(solid waste incinerator, cement manufacture, power plants)
- (d) Symbiotic processing

II Combustible Gases

- (a) Partial combustion of sludge
- (b) Partial combustion of sludge with another waste or low-grade fuel
- (c) Symbiotic processing

III Liquefied Products

- (a) Pyrolysis of sludge
- (b) Pyrolysis of sludge with gaseous reactants (H_2 , CO)

IV Decontaminated Water

- (a) Wet oxidation in a deep well

V Solid Products

- (a) Sludge drying by Carver-Greenfield process
- (b) Add sludge to mix and make bricks
- (c) Coincineration of sludge and solid waste ash followed by fusion, to make aggregate

OXIDIZED GASES

The conversion of sludge solids to oxidized gases and an inert residue is merely the time-honored process of incineration. A variety of options are available and are discussed below.

Incineration of sludge

Estimates have been made that 25 percent of the sludge in the United States is incinerated. Generally, undigested sludge is dewatered to about 20-25 percent solids and is incinerated in multiple hearth furnaces. Usually, addition of high grade supplemental fuel, such as natural gas or No. 2 fuel

oil is required to reach combustion temperatures; if afterburning is required, more supplemental fuel is needed. Fuel requirements are substantial, particularly if the sludge cake solids are near or below 20 percent; excessive fuel requirements have caused many communities to shut down their incinerators and use alternative sludge disposal methods, such as lime treatment followed by landfilling. Besides the disadvantages of the high cost of fuel, incineration has a bad public reception because of the conviction of the public that incineration, particularly the stack discharges, pollute the environment.

EPA research on conventional incineration has shown that incineration stack discharges create some pollution but that particulate discharges are small and the established standards (0.65 g/kg dry feed) are easily met. Certain metals, particularly lead and cadmium, are poorly retained; as much as 10 percent escape collection, whereas bulk particulate losses (volatiles-free basis) are about 0.2 percent. Cadmium and lead escape because: (1) the process of combustion converts much of them into a form, gaseous or particulate, in which they are swept out of the furnace, and (2) in the cooled gases that approach the wet scrubbers, they are present as exceptionally fine particles. Both of these effects contribute to large percentage losses.

Despite the unfavorable results with lead and cadmium, environmental hazard is slight, primarily because quantities of these substances are low in sludge. Nevertheless, to overcome public uneasiness and to permit incineration to be used when cadmium and lead concentrations are high, a demonstration is appropriate in which particle collectors that are efficient in the fine particle range are used, and good removal of lead and cadmium is demonstrated. Additionally, basic studies are needed to investigate whether temperature level, oxygen concentration, and equipment design can be manipulated to lessen the initial transport of cadmium and lead into the gas phase that leaves the furnace.

Addition of low-grade fuel to sludge incinerator

The need to add supplemental fuel to a sludge incinerator is a serious economic obstacle. One alternative is to dewater sludge with more costly dewatering devices, such as diaphragm filter presses, which produce a much drier cake. Another alternative is to add a cheap low-grade fuel, such as refuse-derived fuel (RDF), woodchips, or even coal to the incinerator to supply the necessary heat.

Experiments in which coal or woodchips were added to a multiple hearth sludge incinerator were conducted at Minneapolis, St. Paul (1). The investigators found that fuels savings were considerable. When ground bituminous coal (100% through 8 mesh) was added to the sludge cake, it could be used to replace from 50 to 80 percent of the natural gas normally required. The coal could be used to replace nearly all of the natural gas, but this increased the possibility of thermal excursions with subsequent damage to the incinerator from overheating. The authors found that use of coal unduly complicated the plant required for sludge combustion, since equipment had to be explosion-proof, storage was needed, and operating labor requirements increased. Their experiments with woodchips were successful, and handling problems were fewer than with coal. Use of anthracite coal was unsuccessful; much of it passed

through the furnace unburned (volatile solids in ash increased from 0-4% to 20-30%).

The use of RDF in the form of fluff was demonstrated at Contra Costa (2) in the incineration mode and in the starved-air combustion mode (see a following section). Best throughput rates were achieved when the fluff was inserted on the third hearth and sludge cake introduced on the top hearth. Emissions were lower when sludge and RDF fluff were premixed and introduced on the first hearth but throughput rates were lower.

A facility has been designed and installed at Duluth, Minnesota, in which sludge is to be burned in fluidized bed incinerators, with RDF, probably as fluff, to be added as supplemental fuel. This startup has been delayed by several years, but startup is now imminent.

The technology in which a low-grade supplemental burnable material, such as coal, RDF, or woodchips, is introduced into a sludge incinerator appears sufficiently well established to permit design of an operating facility with an excellent likelihood of success. Despite the adequate state of knowledge, there are very few planned or operating installations (other than Duluth - see above). Either potential users are not aware of the state-of-the-art or have made the decision that other approaches are more promising.

Addition of sludge to other combustion processes

Sludge cake could be destroyed by addition to any combustion process, provided it does not degrade the discharged gases, the product, or the process equipment. Power plants using coal or oil could also burn sludge; however, there has been no interest from this quarter even though there is no "product" that might be degraded. Concern about pollution in the discharged gases and about possible harm to equipment may be the reason for lack of interest. The fact that power plants are utilities which can "pass through" fuel costs to customers may be a factor. Cement manufacturers have shown interest in destroying organic pollutants but have shown no interest in sludge, possibly because they fear contamination of the product by the sludge ash.

The only combustion process that has been applied in a significant way to sludge combustion is solid waste incineration (3). The disadvantage of high cost of fuel is eliminated but other disadvantages are introduced. Sludge and solid waste must be mixed properly. A recent concern has been air pollution, in particular, the fear that dioxin is formed in solid waste incineration. In at least one case (Hempstead, N.Y.), startup of an incinerator is being held up because of concerns about formation of this chemical (4).

Another reason for slow activity in this potentially attractive route for sludge disposal has been the method of financing sludge disposal facilities. A solid waste-sludge incinerator is federally supported by EPA's Construction Grants Program only to 75 percent of the cost of a facility for destroying sludge alone. Because a facility for destroying a municipality's solid waste and sludge will be much more costly than a facility to destroy sludge alone, municipalities are inclined to search for a sludge only disposal method rather than provide capital funds from their own resources for the extra increment of

cost needed for the codisposal facility.

Symbiotic processing

Symbiosis is defined as the living together of two dissimilar organisms in close association. Symbiotic processes can be defined as two separate but intimately related processes, related for example by use of the product of one (or both) in the other process. The coincineration facility at Ansonia, Connecticut, is an excellent example of a symbiotic process. The plant in Krefeld, Germany, uses similar technology. Here, flue gases from a solid waste incinerator are used to dry sludge, after which the dried sludge is disposed of by combustion in the incinerator.

The tying together of two processes in a symbiotic relationship generally involves little development work because the processes are not substantially changed. However, process rates and material flows have to be coordinated for a successful symbiosis. Other than the Ansonia plant and a plant at Harrisburg, Pennsylvania (steam generated by solid waste burning dries the sludge), there are no known plants in the United States of this type. However, opportunities appear to abound, and more applications may be expected in the future.

COMBUSTIBLE GASES

Sludge can be converted to combustible gases in a process that involves sludge either alone or supplemented by other combustibles such as solid waste.

Process for sludge alone

Sludge can be converted to combustible gases by pyrolysis, where heat is transmitted indirectly to the sludge, by contact with reactive hot gases (e.g., steam reforming), or by partial combustion. Partial combustion, also called starved-air combustion, has been the route that appears the most practical and has received the most attention.

In the United States, an incinerator, designed for either incineration or starved-air mode, has been built at Arlington, Virginia, using sludge cake produced by a filter press as feed. This unit is not yet in use. It is planned to use starved-air combustion both at the City and County of Los Angeles plants on completely dried sludge. EPA plans to evaluate performance when these facilities come on stream.

Processes which include non-conventional feed supplements

Because the sludge and its decomposition products are not thoroughly combusted, heat release in the primary reactor per unit mass of feed is less for starved-air combustion than for incineration. Consequently, there is a need either to lessen the heat requirement in the primary reactor by using a very dry sludge cake, or alternatively to increase the heat release by adding a dry product that adds to the heat of combustion. One process (2) uses RDF as a supplemental fuel added to a conventional multiple hearth furnace. This process has been proposed for Memphis, Tennessee, although a final decision has not yet been made. Rochester, New York (5) appears to be going ahead with

its plans to utilize this process, using RDF as supplemental fuel. No additional development work on this processing approach appears to be needed at this time. EPA will attempt to evaluate process performance and report on results when the Rochester plant goes on stream.

Three processes that produce combustible gases by adding another fuel to sludge are outlined in summary form in Table 2. Two processes, the Purox process and the Sanoplex have many similarities. In the Purox process, compressed slugs of pre-ground solid waste (with added sludge cake) are charged to the vertical shaft kiln. The mass is partially combusted using oxygen. The resulting fuel gas is cleaned and used in the vicinity for power generation and/or steam production. Experiments with sludge have proven successful, but no installations have been made. Evidently, economic considerations are not sufficiently attractive.

The Simplex process (6) uses premanufactured fuel briquettes charged to a vertical shaft kiln, and carries out partial combustion using oxygen. The briquettes are a mixture of a caking coal and RDF. A fuel gas very similar to the Purox gas is thus produced. The process appears very similar to the Purox process. However, its inventors claim that gas production rates per unit reactor volume are much greater because the briquettes allow much greater gas velocities in the fuel bed. In the Sanoplex version of the Simplex process, the briquettes include sludge. EPA-sponsored research has shown that acceptable briquettes can be made in the following formulations:

<u>Ratio of Coal/Waste</u>	<u>1:1</u>		<u>2:1</u>	
Sludge percent solids	20	40	20	40
Ratio of RDF to DSS	10:1	5:1	7:1	3:1
Percent DSS in briquettes	3.5	6.3	3.4	8.8

DSS = dry sludge solids

RDF = refuse derived fuel

Note in the above table that acceptable briquettes can be made for RDF to DSS ratios ranging from 10:1 to 3:1. For typical communities in the United States, the potential RDF to DSS ratio is about 10:1. Consequently, there is ample RDF to dispose of all a community's sludge.

Before further development work on this process is undertaken, economic assessments of the process are needed along with expressions of interest from potential users. The process requires the interest and support of power producers, groups concerned with solid waste disposal, and groups concerned with sludge disposal. Unless all three groups actively support the process and see economic justification for it, successful completion of the development and ultimate use are unlikely.

The Vigil-UCD sludge gasifier resembles gas producers used before 1945 in many parts of the world to convert low grade fuels into burnable gases. Many of these units were portable and provided fuel for internal combustion engines that powered trucks and buses. This technology disappeared from the scene when cheap natural gas and gasoline became available. The Vigil-UCD process

Table 2. Processes That Produce Combustible Gases By Partial Combustion of Sludge Mixed with a Combustible Additive

Name	Description	Products
Purox	Slugs of ground solid waste and sludge cake are charged to a vertical shaft kiln, and partially combusted with oxygen	350 BTU/ft ³ * clean fuel gas and a fused residue
Sanoplex (a version of Simplex Process)	Briquettes of a caking coal, RDF and sludge cake are charged to a vertical shaft kiln, and partially combusted with oxygen	350 BTU/ft ³ * clean fuel gas and a fused residue
Vigil-UCD Sludge Gasification System	Briquettes of source-separated paper and sludge cake are charged to a down-draft gasifier, and partially combusted with air	140 BTU/ft ³ * fuel gas and a char residue *BTU/ft ³ x 0.0372 = MJ/m ³

(Vigil is the inventor, rights tentatively assigned to University of California, Davis) combines source-separated paper and sludge cake into pellets and gasifies them in a conventional downdraft gasifier. The gas can be used locally to produce steam or can supply a gas turbine or a gas engine for electric power generation. Electric power generation is attractive in some states (e.g., California) where laws require utilities to purchase power generated by such processes.

Pellets containing up to 15 percent dry weight of sludge were tested in a pilot-scale gasifier (6a). A schematic drawing of the unit is shown in Figure 1. Such gasifiers are much less expensive than incinerators because construction is cheaper (firebrick is not needed) and capacity per unit volume is higher. Gas with a low heating value (dry basis, 0 °C, 1 atm) of 5.2 MJ/m³ (140 BTU/ft³) was produced in the experimental runs. An economic comparison was made among the four alternative sludge disposal routes shown in Figure 2 (6b). For communities below 10,000 persons, land application and landfill showed marked advantages. As community size increased, the relative economics of the thermal processes improved. For a population of 50,000 (see Figure 3), co-gasification was more economical than the land based options unless the land application sites were nearby, and much more economical than incineration.

A small-scale demonstration of the Vigil-UCD gasification system is planned in FY 1982. A run of long duration is planned in which the gasifier system will be used to drive a gas engine. Performance, maintenance, and overall economic data will be obtained.

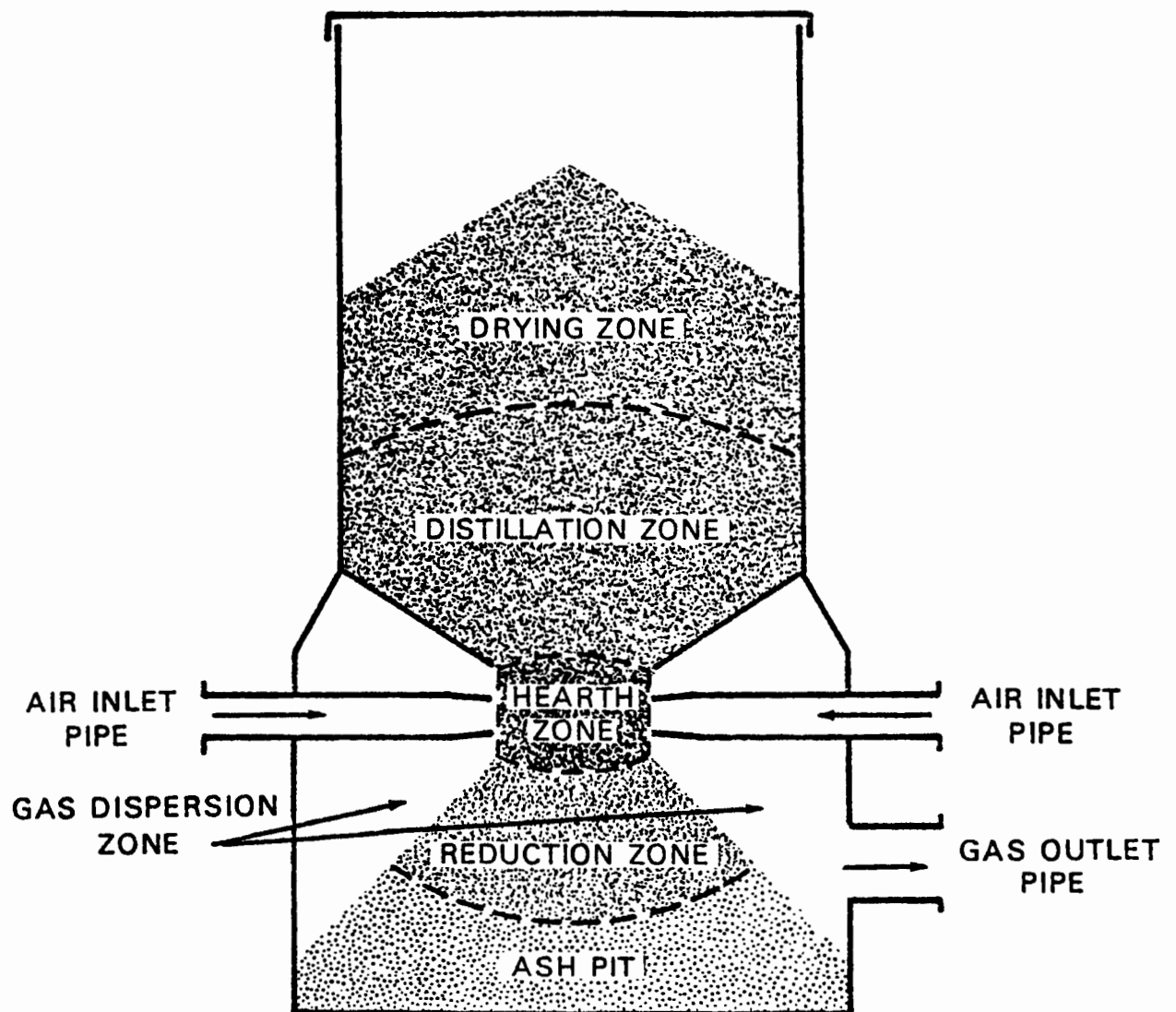


FIGURE 1. SCHEMATIC OF A DOWNDRAFT GASIFIER.

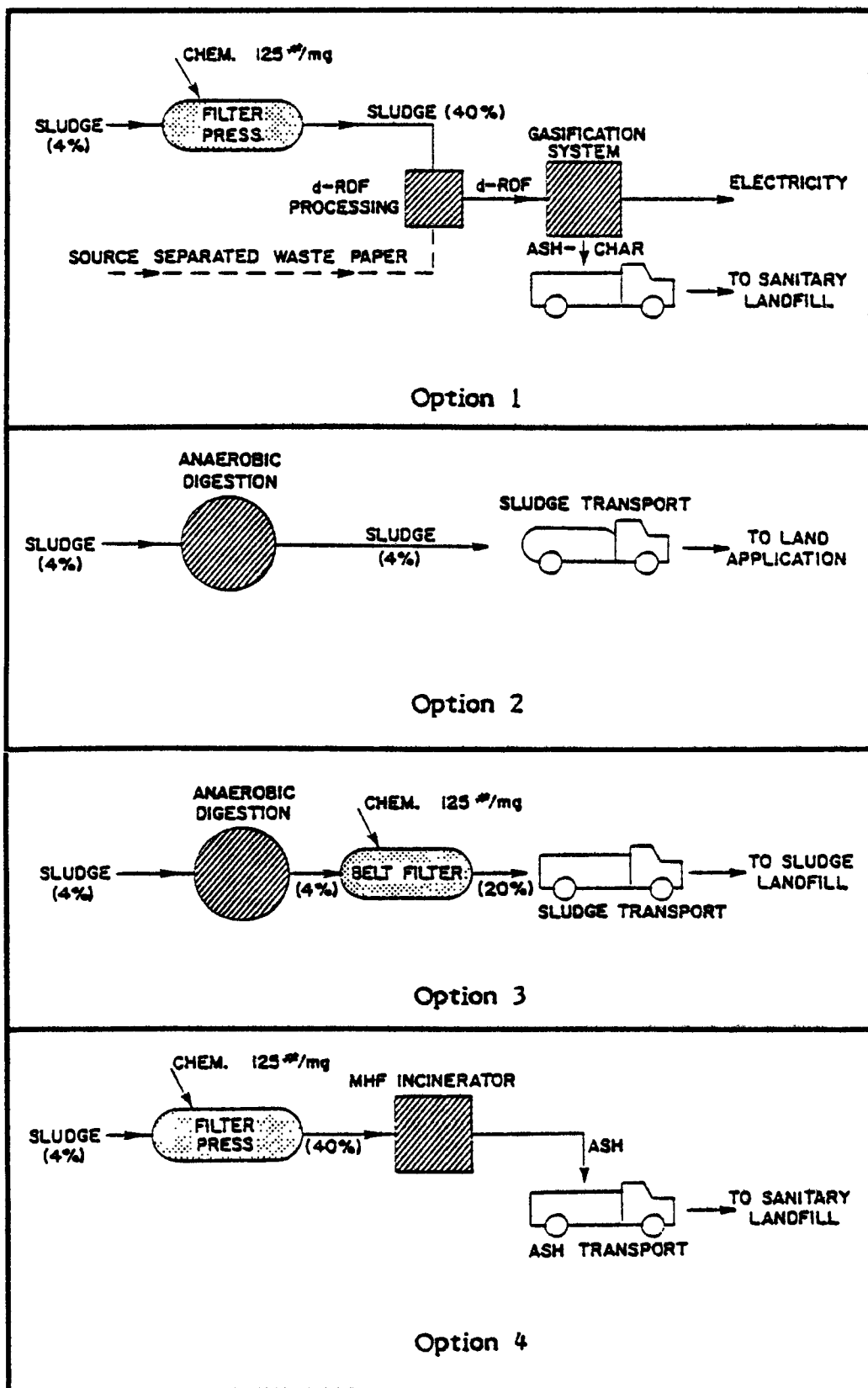


FIGURE 2. SLUDGE PROCESSING AND DISPOSAL OPTIONS.

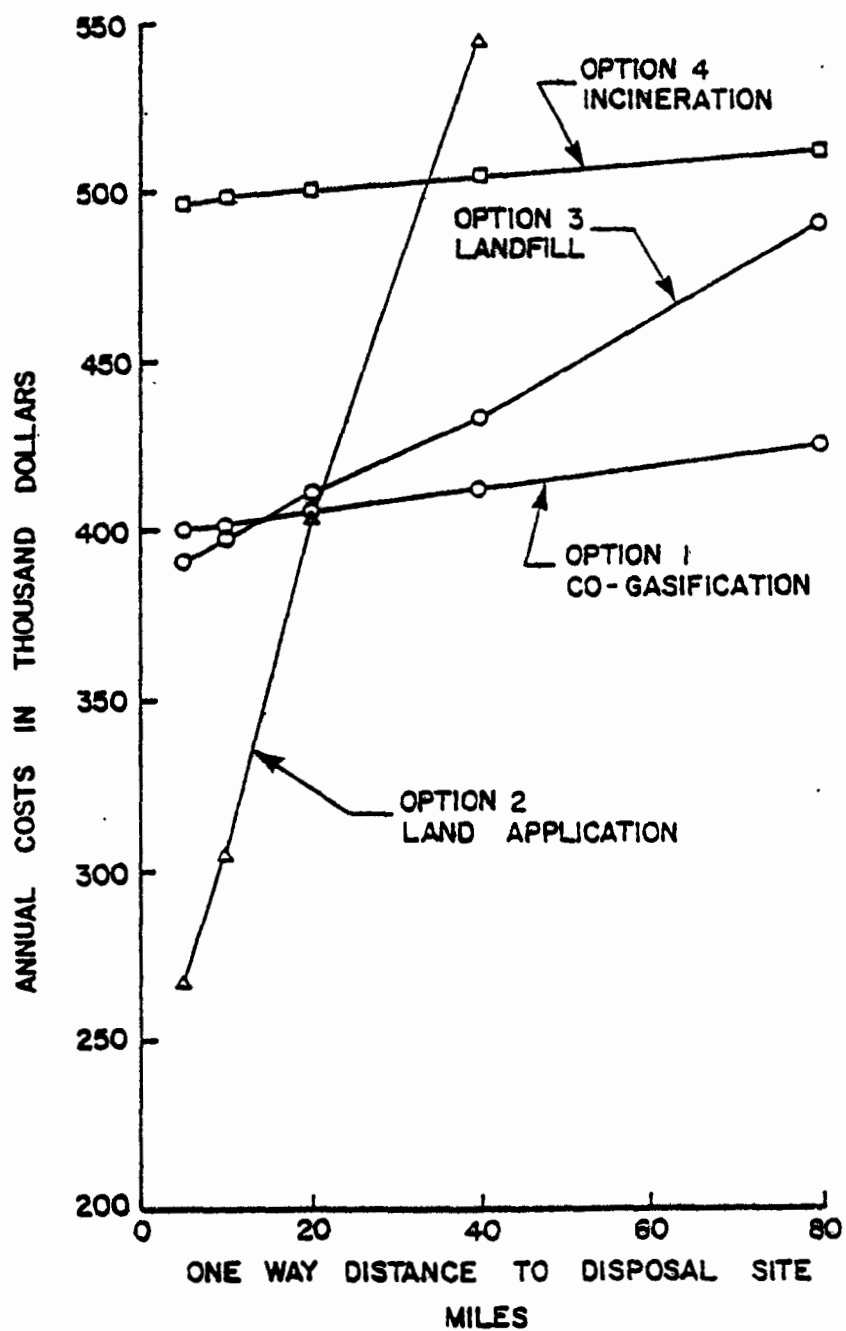


FIGURE 3. ANNUAL COSTS OF PROCESSING AND DISPOSAL OF SEWAGE SLUDGE BY VARIOUS METHODS FOR A COMMUNITY OF 50,000 PERSONS.

Symbiotic processing

Symbiotic processing (see earlier discussion) is possible for processes that produce combustible gases. Staff of Wegman Engineers (7) are reportedly working with wastewater authorities of Central Contra Costa, California, on a starved-air combustion process in which sewage sludge cake is processed at about 20 percent solids under starved air conditions in a multiple hearth furnace. The deficiency in energy needed for drying and decomposition of the sludge is provided in part by combustible gas produced in an adjacent modular solid waste incinerator, also operated under starved-air conditions. Using this combination of processes, it is not necessary to set up a refuse separation facility in order to produce RDF. This concept is still in the planning stage. Our laboratory will be following progress with great interest.

LIQUEFIED PRODUCTS

When sludge or sludge-solids waste mixtures are subjected to high temperatures and high pressure, sludge is converted to gaseous, liquid, and solid products. By appropriate selection of catalysts, reactants such as CO and H₂, temperature, and pressure, product mix can be varied to emphasize liquid or gaseous products. Liquid product processes, discussed below, are favored by high pressures.

Work on such processes is at an early stage and no processes are near to commercialization. EPA has invested a small amount of funds into two processes, one that produces a heavy oil with a large asphalt-like fraction and another that produces a fraction in the motor fuel boiling range.

Pyrolysis at high pressure

A high pressure pyrolysis process for sludge liquefaction has been developed by Battelle Northwest Laboratory (BNW). Experiments were carried out on a batch basis in a stirred autoclave held at reaction temperature for one hour. During this period, gaseous products more volatile than water evolved and were condensed outside of the autoclave, along with some water vapor. After the reaction period, the residue was separated by centrifugation into an aqueous supernatant and a char cake. The char cake was extracted with solvent to produce a heavy oil and a char or ash. The heavy oil was vacuum distilled to remove more volatile materials. This step greatly increased the viscosity and asphalt-like character of the heavy oil.

Reaction conditions and yields are shown in Table 3. Results indicate an oil and asphalt yield of as much as 37 weight percent (ash-free basis). Standardized tests for asphalt showed that the product had good potential for use as asphalt.

All three product fractions have potential for fuel use. Empirical chemical formulas and heating values are given below:

Table 3. Reaction Conditions and Yields in Sludge Liquefaction Experiments¹

Run	Sludge ² Charged(kg)	Reaction Temp (°C)	Pressure ³ (atm)	Reaction Adjunct (5% by wt)	Steam Volatile Oil (g)	Synthetic Asphalt (g)	Total Yield
HS-3	2.61	320	112	Na ₂ CO ₃	330	268	23
HS-4	2.52	345	153	Na ₂ CO ₃	480	440	37
HS-5	2.41	295	79	Na ₂ CO ₃	0	250	10
HS-6	3.99	320	112	Na ₂ CO ₃	450	330	20
HS-7	2.01	320	112	Na ₂ CO ₃	0	310	15
HS-8	2.28	320	112	Na ₂ CO ₃	250	50	13
HS-9	2.49	320	112	CaO	450	430	35
HS-10	1.33	320	112	Na ₂ CO ₃	250	164	31

1. Autoclave was 1 hour at reaction temperature

2. Dry ash-free basis

3. Estimated as saturated steam pressure at reaction temperature

4. Weight of light oil and synthetic asphalt as percent of dry, ash-free sludge

<u>Product</u>	<u>Empirical Formula</u>	<u>Heating Value</u>	
		<u>MJ/kg</u>	<u>BTU/lb</u>
Volatile oil	$C_{11}H_{1.9}N_{0.3}O_{0.9}S_{0.04}$	39.2	16,900
Asphalt fraction	$C_{11}H_{1.6}N_{0.5}O_{0.9}S_{0.04}$	34.8	15,000
Char (includes ash)	--	12.5	5,400

If the asphalt fraction can be used as a fuel extender, which appears likely, economics could be improved because the process could be simplified (for example, by eliminating the vacuum distillation step). Preliminary economic analysis for a plant to produce a light oil and asphalt shows a payout of the initial capital investment in 10 years. If both the light oil and the asphalt fraction were used as a fuel oil, the payout time is estimated to be half as long (5 years).

The work carried out so far indicates feasibility of the process but economics are uncertain. More batch experiments followed by operation of a continuous pilot plant will be needed to firm up data on yields, process conditions, and product qualities. The fuel production option looks more attractive than the asphalt option. Decisions on further work efforts are awaiting recommendations of a critical review of thermal conversion approaches but this approach seems more promising than some others.

Pyrolysis at high pressure with gaseous reactants

Worcester Polytechnic Institute (WPI) has proposed a high pressure liquefaction process which uses thermal conditions similar to the BNW process. However, in the the WPI process, a solid catalyst is used in the presence of hydrogen. In preliminary experiments, the sludge was suspended in a liquid carrier (a paraffin oil). All of the organic substances were converted to liquid and gaseous products. The need to supply hydrogen increases the cost of the process substantially. WPI has proposed to supply the hydrogen by steam reforming part of the sludge to form hydrogen.

Considerable experience has been obtained on performance of the WPI process using solid waste as feed, but information on sludge is very limited. A short experimental program will be carried out using sludge as the feed. A decision on whether to go further than this with the process awaits the aforementioned review of thermal conversion processes.

DECONTAMINATED AQUEOUS STREAM

A sludge stream can be treated directly by strong oxidizing chemicals to decontaminate it and oxidize its organic content. By elevating the temperature, oxygen from the air can also be used for this purpose. This latter process is called "wet oxidation" and has been available for many years. The Zimpro "high oxidation" process is commercially available, and its effects are well documented in the literature. Only rarely does this method appear to be cost effective. Studies for Boston and New York showed that other thermal destruction methods are less costly. Important disadvantages are: (a) the

high pressures needed (1500 psi), (b) need for expensive metals or alloys to limit corrosion, and (c) high requirement for electrical power.

A modification of this process, the Wet-Ox process uses a different reactor design but does not differ in any substantial way from the Zimpro High Oxidation process. The claim is made that more efficient contact in the stirred reactor allows use of lower temperatures and pressures to achieve the same result as the Zimpro high oxidation process. Recent work with the Wet-Ox process for destroying industrial waste has been published (8).

A novel approach to wet oxidation has been proposed by the Vertical Tube Reactor Corporation. In their process, a deep vertical concentric double pipe assembly (ca. 6000 ft deep) is used to produce the high pressures needed to maintain water in the liquid state at the thermal conditions for wet oxidation. No high pressure equipment is needed at the surface. Heat exchange between incoming and exiting liquid is efficiently accomplished by the pipe within a pipe assembly, without the need for a cumbersome above ground double pipe heat exchanger. Air under moderate pressure is pumped into the sludge as it is pumped downward. A third concentric pipe near the bottom of the pipe assembly provides a chamber for addition of heat for startup and removal of heat when the oxidation commences and excess heat is generated.

The process should show much lower capital investment and much lower operating cost than an above-ground wet oxidation plant. More usable surplus thermal energy should be generated. The principal drawback is the inaccessibility of the equipment. If anything goes wrong, it is a major task to fix it.

Another presentation at this symposium will explain in more detail the proposed VTR installation at Longmont, Colorado. Consequently, it will not be discussed further here. The EPA is planning to assist in the installation of the Longmont facility and is supporting the costs of an operation and evaluation period. This process holds great promise for many areas of the country, particularly those where geology is favorable for well drilling.

SOLID PRODUCTS

Drying

The simplest thermal process for producing a solid product from sludge is thermal drying. The dried product is accepted as a fertilizer, and, in the users' mind, no longer has a "fecal connection." If metals and organic contaminants are low, sludge makes an acceptable fertilizer. The main drawback is the cost to evaporate the water from the sludge cake. There has been a great revival of interest in removal of water by the Carver-Greenfield process in which sludge is dewatered as it is carried by an oil through a multiple-effect evaporator. This process was evaluated by the City of Los Angeles as part of the LA/OMA (Los Angeles, Orange County Metropolitan Area) sludge disposal research program for possible use in one of their processing plants (9).

Operation demonstrated good thermal efficiency, low loss of carrier oil, and few operating problems. As a result of this work, both the City of Los Angeles and the Los Angeles County Sanitation Districts plan to use this process in their processing scheme (9). Both authorities plan to dewater the sludge to dryness by this method, to be followed by starved-air combustion of the dried sludge in multiple hearth furnaces.

The Carver-Greenfield process needs no further development. Evaluation of results when the Los Angeles plants come on stream is appropriate.

Manufacture of bricks

The author has heard over the course of several years suggestions that sludge be used in processes that require water, for example, casting concrete foundations. The user would receive a credit close to the dewatering cost of sludge which would reduce cost of manufacture. The important concern is that the sludge must not degrade the quality of the product. Most of these suggestions have not become a reality because of concerns about degradation of product quality. Recently, the National Science Foundation has funded an investigation by Dr. James Allamen, University of Maryland, to determine the effect of sludge addition on quality of brick. Dr. Allamen's investigation so far indicates a reduction in strength and a reduction in density as sludge content increases. The investigation is continuing and will perhaps disclose conditions where sludge addition will produce a competitive product.

Manufacture of aggregate

For several years, the Franklin Institute Research Laboratory (FIRL) in Philadelphia has been developing a method called the Ecorock process for producing an aggregate suitable for an asphalt hot mix used for road construction. They have thermally processed the residue from solid waste incineration into an aggregate and submitted it to the U.S. Department of Transportation for tests, which included testing in actual use on highways. The material was rated very highly in all tests. The EPA has supported experiments to determine whether sludge cake could be included in the aggregate.

The basic Ecorock process is shown schematically in Figure 4. Sludge cake is introduced into the dryer along with coarsely ground incinerator residue. Drying and burnout occur in two separate rotary kilns. Temperature reaches 1700 °F in the burnout kiln. Further elevation in temperature to 2200 °F occurs in the fusion furnace. The resultant fused product is cooled and crushed to the desired size.

Bench scale experiments were conducted to determine the effect of sludge cake on the properties of the Ecorock. The most difficult step in this research was obtaining a representative sample of incinerator residue. Results of the tests are shown in Table 4. From the table, it appears that ratios of sludge to residue up to 0.4:1 produce a suitable aggregate.

A large pilot facility capable of processing, on a dry weight basis, 28 tons per day of solid waste residue (82% solids) and 20 tons of sludge cake (20% solids) has been built at Philadelphia's Northeast Sewage Treatment Plant.

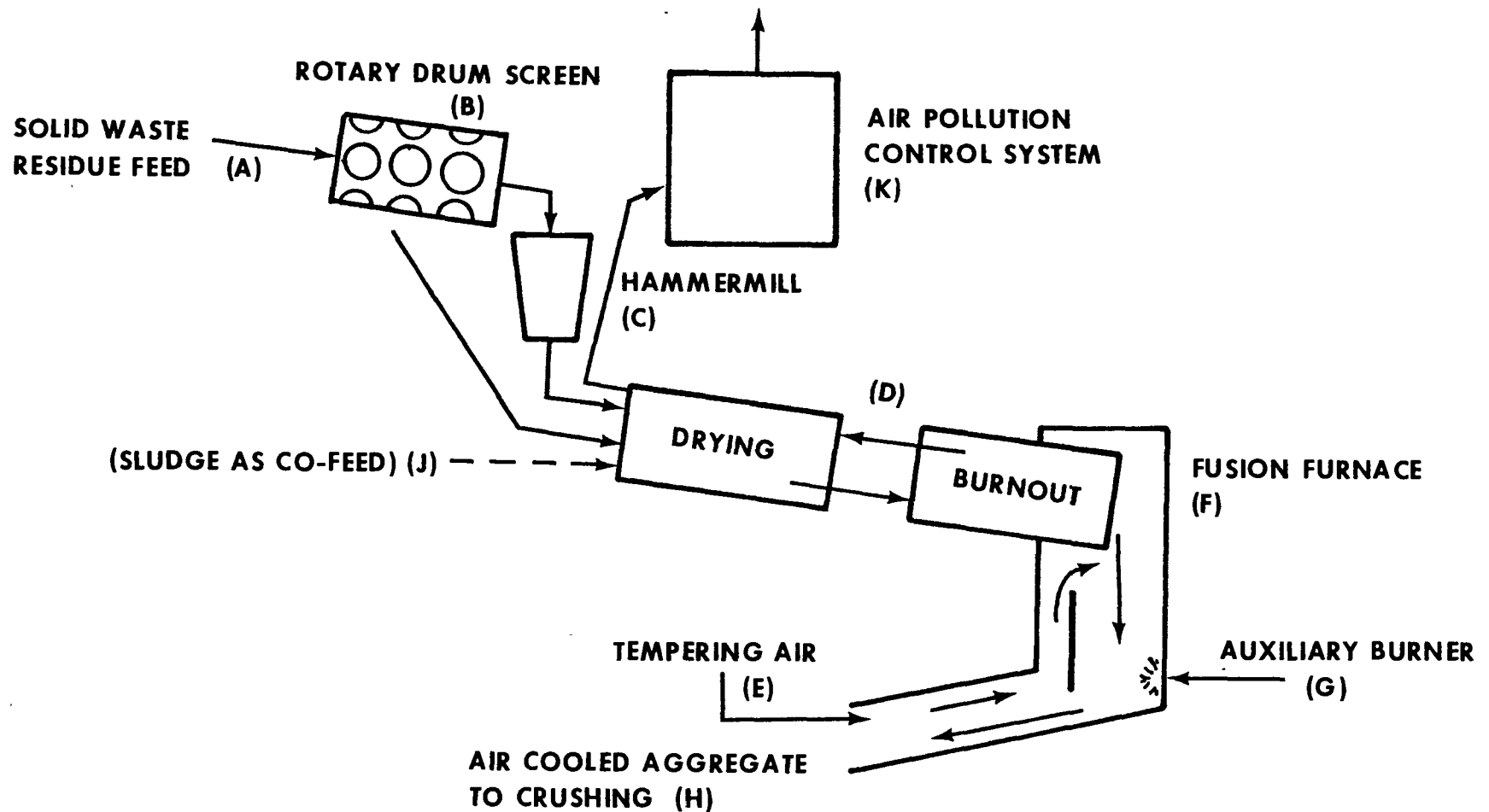


FIGURE 4. RESIDUE FUSION PROCESS INCORPORATING SEWAGE SLUDGE DISPOSAL (MOD II).

Table 4. Summary of Mixing, Handling, Drying, Burning and Fusion Experiments on Wet Sludge/Incinerator Residue Mixtures

Sludge/Incinerator Residue*

<u>Wet Ratio</u>	<u>Dry Ratio</u>	<u>Observations</u>
Pure sludge		Pasty solid dried into a hard cake. Burned with considerable odor and smoldering from lack of air. Aggregate formed at 2,000 °F was light weight and of very low strength.
10/1	2/1	Wet mixture resembles and handles like pure sludge. Ecorock may be acceptable as a light weight low strength aggregate.
5/1	1/1	The presence of incinerator residue is barely apparent in the wet stages. Smoldering occurred when burnout was occurring. Dusty appearance during final burning stages. Fused aggregate appears hard.
4/1	0.8/1	Approximately the same as above.
3/1	0.6/1	Wet mix tumbles and flows because of the presence of incinerator residue. No smoldering during burnout. High concentration of residue readily fuses with sludge ash during fusion. Aggregate looks very good; hard, tough, and uniform.
2/1	0.4/1	Mixing, flow, drying, burnout and fusion proceeded as well as with pure incinerator residue.
1/1	0.2/1	Same as above.

*Sludge at 20% solids

Plans are to start operation in fiscal year 1982. EPA will join with the City in evaluating the process.

A LOOK INTO THE FUTURE

In the next few years, the pressure to modify and improve incineration practice, particularly with respect to particulate removal and fuel economy, will continue. It is expected that many installations will modify their existing facilities to the starved-air combustion mode using extra dry sludge cake or supplemental fuel addition.

Coincineration will make slow progress, primarily because solid waste incineration is making slow progress. We need showplaces that demonstrate that incineration of solid waste is not a messy, polluting operation.

Conversion of sludge to liquid fuel might be used at the largest sewage treatment plants. Information on the effect of scale of operation on economics is needed. More developmental work is needed before this kind of information can be developed. Consequently, it is not possible to predict the future of such processes yet. Because developmental work is expensive, and the "universe" of plants that might use the process is small, efforts will be made to limit development work to a single outstanding candidate process.

It is not expected that processes like the Ecorock process or deep well oxidation can be used everywhere, but they appear to have a place. Within a year or two, firm information should be available on the potential of these processes.

One thing can be stated with assurance. Thermal conversion will remain an important disposal option for sludge, but existing procedures will see radical change.

ACKNOWLEDGMENTS

The assistance of Dr. Atal Eralp and Mr. Howard Wall in assembling material for this presentation is gratefully acknowledged. The work reported is primarily the work of EPA grantees and contractors. Their contributions are also sincerely appreciated.

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TECHNOLOGY ASSESSMENT OF THE VERTICAL WELL CHEMICAL REACTOR

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TECHNOLOGY ASSESSMENT OF THE VERTICAL WELL CHEMICAL REACTOR

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ABSTRACT

The vertical well chemical reactor (VWCR) is designed to oxidize high strength organic wastes using wet combustion principles. Its vertical concentric configuration uses little space compared to above ground wet oxidation vessels and promotes efficient heat exchange. Natural pressurization from the weight of the liquid above results in safer, more economical operation. Full size reactors are expected to extend to 6000 foot depths underground and operate at maximum temperatures and pressures exceeding 650°F and 1500 psi respectively.

COD reductions of waste sludges have approached 50 percent at pilot scale. Eighty percent reductions are expected for full scale where deeper wells will be drilled and higher temperature and pressures attained. The fate of metals in the VWCR including distribution within the particulate and soluble wastewater fractions for various oxidation conditions is not well understood and more work needs to be done in this area.

Construction is to begin late 1981 on a full scale sludge demonstration VWCR plant at Longmont, Colorado. Major operational items to be addressed (based on pilot plant experience) include pit corrosion, scale formation, and leaking joints.

The VWCR is a potentially desirable treatment process which can stabilize organic wastes, significantly reduce sludge volume, oxidize toxic materials, kill pathogenic organisms, and permit energy recovery from high strength wastes.

INTRODUCTION

The vertical well chemical reactor (VWCR) is designed to oxidize high strength organic wastes using wet combustion principles. If sufficient air (oxygen), temperature and pressure are present, organic substances can be oxidized in a liquid state. The oxidation reaction proceeds exothermically and if organic content of the waste is high enough, combustion may be thermally self-sufficient and even produce energy as heat.

Figure 1 is a process flow diagram showing a vertical section of the VWCR. Two concentric tubes constructed from 300 series stainless steel serve as the reactor vessel. They are surrounded by a liquid filled heat exchange jacket which can independently add or remove heat to maintain the required temperature.

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Air is also injected at several downcomer locations along the waste fluid path. It assists fluid flow and provides oxygen for oxidation. Insulation to minimize heat losses within the reactor and to the surrounding earth completes the basic VWCR design.

Waste is introduced into the downcomer tube at the earth's surface. As it flows downward, effective wet oxidation begins at 350°F and continues as temperature and pressure increase to their maximum. Upflowing oxidized waste is gradually cooled as it transfers heat to the downflowing fresh waste. Any excess heat which may result from the exothermic oxidation reactions is removed by the heat exchange jacket.

The VWCR is designed with no moving parts below ground level and needs no high pressure vessels. Its components below the surface are the stainless steel downcomer and upcomer concentric reactor tubes, air lines, heat exchange jacket, and associated temperature thermistors and pressure measuring devices. The reactor tubes are subject to the potential scaling and corrosion characteristic in any high temperature and pressure liquid environment. VWCR operation efficiency, in addition to design and waste composition considerations, is dependent on the reliability and accuracy of downhole measuring equipment and on the heat exchange efficiency between the fresh and oxidized waste.

DEVELOPMENT STATUS

Laboratory Bench Scale Research

Initial laboratory scale testing began in 1973 by the Vertical Tube Reactor Corporation, Englewood, Colorado. A 2.7 liter stainless steel laboratory batch reactor employing similar temperature conditions upon which a full scale VWCR is based was used to oxidize organic materials over a wide range of COD concentrations. Under EPA/MERL sponsored research since 1979, primary and secondary sludges from several municipal wastewater treatment plants have been oxidized in the laboratory reactor.

Batch reactor tests using municipal wastewater have been run on wastewater from Montrose, Colorado. Montrose wastewater contains a high portion of industrial (candy) wastes which range from 200-2500 mg/l COD in strength.

For municipal organic wastes without a significant refractory component, it has been found that the extent of COD reduction is primarily a function of reactor operating conditions and less a factor of specific waste make-up. Figure 2 summarizes general COD removal experience using the laboratory reactor while oxidizing both high COD wastewater and sludge. General trends are summarized in Table 1. These trends generally agree with pilot and full-scale studies reported in the 1960's by investigators using conventional above-ground pressure vessel equipment for wet oxidation (1).

Pilot Scale Research

A VWCR pilot plant has operated intermittently since 1977 at Lowry Bombing Range, near Denver, Colorado. It consists of a stainless steel 1-3/4 inch downcomer and 2 inch diameter upcomer within a 2-1/2 inch reactor casing. The reactor casing, air line, heat exchanger lines and insulation are all suspended in a 5-inch, 1500 foot standard American Petroleum Institute well casing.

Most pilot scale effort has concentrated on solving structural, mechanical and other operational problems. Pit corrosion and scale formation, heat exchange line plugging, reactor-heat exchanger interface design, and leaking joints were the major engineering problems encountered.

Table 2 presents a summary of municipal sludge COD reduction data from the pilot plant tests and compares them to batch reactor results under similar operating conditions. The close pilot and batch treatability results experienced under similar operating conditions has supported the use of a laboratory batch reactor to model pilot plant treatability expectations. Limited solids, metal and off-gas data have also been taken during pilot studies. More measurements need to be made on these parameters before conclusions can be made.

Full Scale Research

EPA's Office of Research and Development recently received a preapplication for federal assistance from Longmont, Colorado to build and evaluate a VWCR to treat its sludge. The Longmont wastewater treatment plant is a secondary biological process treatment plant with flow equalization and anaerobic sludge digestion.

The proposed VWCR for Longmont has an 8 inch nominal diameter reactor casing to be suspended 6000 feet. It will operate at temperatures of 500 to 650°F, influent diluted sludge strengths of 5,000 to 10,000 mg/l COD and reaction times of 25 to 100 minutes. Oxidation efficiency is expected to be 75 percent. Figure 3 shows the proposed Longmont process schematic.

The overall objective of the Longmont program is to demonstrate technical and economic feasibility of a full scale VWCR for oxidation of municipal sludge and selected industrial wastes. The program involves three basic phases over at least two years. Phase I includes design, construction and initial operation of the VWCR at Longmont. Phase II is designed to provide long term reliability and operating information about processing municipal sludge. It will also evaluate the impact of sludge treatment with the VWCR on the other Longmont wastewater treatment plant operations. Phase III will investigate treatment and disposal of complex industrial wastes. In addition, an energy recovery system will be designed to convert excess combustion heat into usable energy for treatment facility use. During all phases, system components will be monitored for energy efficiency, operation and maintenance characteristics, and material durability.

TECHNOLOGY EVALUATION

Theory

The VWCR process utilizes the fact that any burnable substance can be oxidized in the presence of liquid water and sufficiently high temperatures and pressure. Oxygen for the VTR oxidation process is supplied by air.

Wet oxidation is the result of at least three chemical reactions: (1) heterogeneous (two-phase) oxidation; (2) hydrolysis; and (3) liquid phase oxidation. Initially, destruction of the solid organic waste is predominantly the result of heterogeneous (two-phase) oxidation by direct contact between adsorbed oxygen gas and organic solids. At elevated temperatures, the solids are quickly reduced to simpler organic colloids by hydrolysis. Hydrolysis will split organic polymers from the colloidal or soluble organic matter but will not destroy them.

Ultimate destruction of the organic matter is by liquid phase oxidation, i.e., wet oxidation following individual collisions and interactions between absorbed oxygen and organic compounds in solution.

Wet oxidation reactions are exothermic and release energy as heat. The quantity of heat released depends on waste make-up. If the heating value (proportional to organic strength) of the waste is high enough, the temperature required for oxidation can be sustained by oxidation itself.

The liquid in the reactor cannot be allowed to vaporize and change into the gaseous phase. The maximum allowable temperature of the waste is just below the boiling point of the liquid. This temperature depends on the pressure of the liquid's saturated vapor. Pressure down hole in the VWCR is in turn a function of weight or head of liquid above it. Thus, the maximum allowable temperature in the VWCR varies with depth and roughly follows the saturated vapor pressure - temperature curve relationship for water.

In summary, four important parameters control performance of the wet oxidation process: feed solids concentration, pressure, temperature and air supply. The COD test is normally used as a measure of process efficiency. Average wet oxidation efficiency is 70-90% COD reduction. Some organic matter in the form of low molecular weight compounds such as organic acids, aldehydes and acetates will be observed in the effluent. Final oxidation products are highly dependent on the degree of oxidation and composition of the waste.

Process Capabilities and Limitations

Because it is not necessary to supply energy for the latent heat of vaporization, wet air oxidation is particularly applicable to oxidize organic sludges which can supply some or all of the heat required to maintain combustion temperatures but cannot be separated readily from water. The concentric tube configuration of the VWCR is naturally conservative and is conducive to good heat transfer because of tube proximity. Furthermore, once the surrounding earth reaches equilibrium, it may act as a heat envelope and buffer external temperature variations.

Like many high temperature - high pressure processes, corrosion and scale formation in the reactor and heat exchange surfaces are inherent problems with the process. Measures such as repassivation of the stainless steel surfaces with an acid can be taken to minimize corrosion and deposition. The extent of scaling and deposition and the degree to which it can be controlled is yet to be characterized for the VWCR.

Compactness of the VWCR configuration makes down hole maintenance and replacement difficult. Delays in pilot operation to date have come from difficulties in replacing components which, for various reasons, had to be changed or redesigned. Experience gained during pilot scale testing will be applied to full scale design at Longmont, hopefully to minimize engineering problems.

Oxidation using the VWCR may provide a good approach for treatment of toxic and hazardous wastes, landfill leachates in particular. In independent but related laboratory tests, wet oxidation using a moderate (527°F) and high (608°F) temperature has been demonstrated to be an excellent method for destroying and detoxifying certain organic compounds including phenols, acrolein, dinitrotoluene, and diphenylhydrazine (2). Other laboratory tests have indicated that virtually all organic substances can be completely solubilized and broken down ("reformed")

to low to medium molecular weight compounds in supercritical water (above 705°C and 220 atm)(3). Subsequent rapid and complete oxidation of the reformed products is expected but just beginning to be tested under these supercritical conditions (4). The fluid which exists under supercritical conditions is neither a liquid or a gas and has a density of only 0.2-0.5 g/m³. Thus, much of the natural hydrostatic head of the VWCR would be lost when operating in a supercritical regime and the reactor would have to be pressurized.

Design Considerations

Mathematical models simulating thermodynamic conditions within the reactor have been developed to be used with wastewater treatability results to design a VWCR:

Hydrodynamic Analysis: Flow velocities, temperatures, and pressures at various down hole locations are estimated from given waste treatability data, VWCR physical dimensions, heat exchange properties, and desired reaction temperatures. VWCR size and depth options are considered from this model.

Heat Flow Analysis: Heat loss from the VWCR to the surrounding earth is estimated as a function of time and reactor geometry and insulation. Heat exchanger design criteria is established from this model.

Heat Exchanger Analysis: Heat loss from heat exchanger lines to the earth is estimated as a function of reactor geometry, operation, and insulation. Net energy surplus from (or required energy input to) the heat exchanger fluid serving the VWCR is estimated from this model.

Verification of model predictions should come from demonstration plant operation at Longmont.

When considering feasibility of a VWCR, a subsurface geological investigation is necessary both to determine expected well drilling costs and to estimate the surrounding earth's thermal and structural properties. A geophysical survey of available sources of data can provide much information.

Precautions must be taken to minimize effects of corrosion and deposition. A weekly nitric acid passivation operation has been recommended. The effect of down hole temperature and pressure as well as the influence of acid addition on wastewater treatment efficiency is not known at this time. The most desirable procedure for acid addition over long term VWCR operation will be developed during demonstration plant operation.

Energy Considerations

In the absence of other data, a value of 6,000 BTU/lb COD oxidized can be used to estimate the heat value of a waste. Energy considerations are best illustrated by example:

Consider a 10 mgd plant producing 10 tons dry sludge daily.

Assuming wet sludge consists of 5 percent solids the:

$$10 \times 2000 \times 1/.05 = 4 \times 10^5 \text{ lbs wet sludge produced daily}$$

Assuming further that 20% of its heat is lost to the surrounding earth then:
 $6000 \times (1.0 - 0.2) = 4800 \text{ BTU/lb COD effective heat value}$

For an influent/effluent temperature differential of 5°F , and a specific heat of $1 \text{ BTU/lb mass } ^{\circ}\text{F}$, the heat loss $Q = MC\Delta t$ is:
 $4 \times 10^5 \times 1 \times 5 = 20 \times 10^5 \text{ BTU/day}$

The COD required to be oxidized to make up this heat loss assuming 75% of the COD is oxidized in the VWCR is:
 $4 \times 10^5 / (4800 \times .75) = 556 \text{ lb COD oxidized/day}$

Thus, for steady state conditions where heat losses just equal heat production through the VWCR, COD of the sludge must be at least:
 $556 / 4 \times 10^5 = 1390 \text{ ppm or mg/l for thermal self sufficiency}$

This example well illustrates how several variables can affect calculations estimating the COD concentration required for thermal self-sufficiency of the waste. Note that the sludge solids concentration, specific heat of sludge, heat loss, percent sludge oxidation and influent/effluent temperature differential were assumed. If, for example, the temperature differential had been 2°F , the mg/l COD required for thermal self-sufficiency would have dropped to 556 mg/l COD for the waste. It is also important to keep in mind that the example assumes sufficient conditions already exist in the reactor for oxidation to proceed. That is, initial heat to bring the reactor up to the required oxidation temperature has been supplied, there is sufficient air for oxidation, and there is sufficient pressure in the reaction zone for the liquid reaction to proceed at the required temperatures.

Operation and Maintenance Requirements

Operation and maintenance requirements for full size VWCR treatment have been estimated based on pilot plant experience. Major labor expenditures are for pump and valve maintenance, coarse screen cleaning, and sludge disposal.

Structures and reactor casings, as well as air and heat exchange lines, are estimated to have a 40 year life. Mechanical/electrical equipment is estimated to last 15 years. The stainless steel upcomer and downcomer are conservatively estimated to have a 10 year life. Their life span will ultimately depend on the extent and controllability of corrosion and deposition experienced. Nitric acid qualities and acid wash operation have yet to be determined. This is one of the most important areas to be investigated at the Longmont VWCR demonstration plant.

Unlike most existing wet oxidation processes which operate at low to intermediate wet oxidation levels, VWCR operation is expected to provide a high degree of wet air oxidation. It is expected that liquor strength will not be as great as that from heat treatment processes which do not stabilize the waste to such an extent. Similarly, odor treatment is not expected to be a significant design factor for such high degrees of sludge oxidation.

ASSESSMENT OF NATIONAL IMPACT

The vertical well chemical reactor employs chemical oxidation to oxidize organic materials in water solution or suspension. In general, the desirability

of using a VWCR is influenced by plant size, site-specific geological conditions, wastewater characteristics and sidestream treatment.

It is estimated that VWCR can be a competitive process for wastewater treatment plants of size 20 mgd and larger. Well drilling costs will normally constitute the single largest capital outlay item. Thermal self-sufficiency depends largely on the actual temperature differential between influent and effluent. The VWCR will most likely be applied to oxidation of municipal sludges. The high degree of oxidation expected from the VWCR is expected to stabilize liquor and odor sidestreams to a great extent. Refractory components in liquor sidestreams need to be measured.

In summary, the VWCR is a technology for treating organic wastes which can significantly reduce sludge volume, toxic materials, or pathogenic organisms, or provide the potential for energy recovery from high strength wastes.

CONCLUSIONS AND RECOMMENDATIONS

1. Configuration of the VWCR has both advantages and disadvantages:

a. The VWCR uses little space compared to above ground wet oxidation configurations;

b. The concentric tube configuration promotes efficient heat exchange between influent and effluent streams;

c. The vertical tube configuration allows natural pressurization of the waste from weight of the liquid above it. The below ground natural pressurization is safer, simpler and cheaper than above ground mechanical pressurization; however, it is less flexible. Pressure at any point down-hole in the reactor is relatively constant, and therefore, the maximum allowable temperature (not exceeding the waste boiling temperature) is fixed for any depth, approximately following the saturated vaporization curve for water;

d. Reactor tube size limits the amount of air which can be added to support oxidation. Standard operating procedure involves sludge dilution to meet maximum air and temperature limitations; and

e. VWCR configuration and compactness make down-hole accessibility difficult. Mechanical reliability and maintenance of the VWCR system are important considerations.

2. Appropriate bench or pilot scale treatability tests using the waste to be oxidized are very important. The degree and rate of waste wet oxidation is significantly influenced by temperature and pressure. Temperature and pressure requirements affect VWCR dimensions and ultimately costs.

3. COD reduction experienced using a batch laboratory reactor has been close to that obtained at pilot plant scale under similar operation conditions. This supports the use of a bench scale reactor to model pilot COD reduction rates. Experience with the fate of metals or toxics is less definite and more study needs to be undertaken in this area.

4. The VWCR is especially applicable to wastes having a high organic content so that a thermally self-sustaining reaction can be maintained. The minimum organic concentration for autothermal conditions will depend largely on the actual temperature differential between influent and effluent wastes.

5. When considering the feasibility of a VWCR, a subsurface geological investigation is necessary to identify aquifers, estimate well drilling costs, and determine the earth's thermal properties.

6. Sludge stabilization trends using a laboratory reactor designed to simulate VWCR oxidation conditions generally agree with historical above ground pilot and full scale wet oxidation observations which indicate that as pressure and temperature increase:

- a. The rate and extent of COD reduction increases;
- b. The particulate waste fraction approaches an inert, readily settleable ash; and
- c. The soluble waste fraction becomes more biodegradable.

7. The VWCR is not yet fully developed in that all process variables normally expected in full scale application have yet to be characterized.

- a. The efficacy of the acid wash system to control reactor sealing and corrosion has yet to be demonstrated;
- b. Verification of heat transfer and heat flow models which influence VWCR design and predict energy surplus or deficits is not complete; and
- c. Long term operation at thermally self-sufficient conditions has not been carried out. Such operation will help define realistic operational variables and maintenance requirements.

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Table 1. Waste Stabilization Trends Using Laboratory Reactor Data

- The extent of COD reduction significantly increases with temperature up to at least 600 °F.
 - Increasing batch reaction time from 1/2 to one hour effects about 10 percent greater COD removal at 400 °F and approaches no difference at 650 °F.
 - The particulate (ash) COD decreases to almost zero about 600 °F confirming expectations of an inert ash.
 - The portion of effluent BOD remaining (as a fraction of total COD) increases with reaction temperature indicating the refractory reactor effluent is mostly biodegradable for the wastes tested.
-

Table 2
A Comparison of
VWCR Pilot Plant and Laboratory Batch Reactor
COD Reduction Data

<u>Test Dates</u>	<u>Flow Rate Gal/Min</u>	<u>Reaction Time Min*</u>	<u>Bottom Reactor Temp °F</u>	<u>Influent COD MG/L</u>	<u>Effluent COD MG/L</u>	<u>Pilot Plant COD Reduction %</u>	<u>Laboratory Reactor COD Reduction, %</u>
July 24-25,80	4.0	30	440	350	248	29	28
Sept 11-22	4.1	25	400	100	73	27	18
Nov 6-21	4.2	25	420	740	548	26	24
Dec 7-16	4.5	20	440	600	468	22	22
Dec 20-23	4.5	20	420	880	695	21	22
Mar 19-23,81	4.5	28	500	1,340	784	45	51
Mar 20	4.5	28	500	1,110	657	41	--
Mar 23	4.5	28	510	1,063	567	47	51

*Estimated time that sludge was above 350°F while flowing through pilot plant.

Table 3. Preliminary Life Cycle Cost Estimate for a VWCR
System Containing an Eight Inch Diameter Reactor*

Project Capital Costs (and expected service life)	\$ 600,000
Well drilling and casing (40 yr)	
Vertical well chemical reactor	
- reactor heat exchange lines, air lines and casing (40 yr)	\$ 300,000
- reactor upcomer and downcomer (10 yr)	\$ 300,000
Mechanical/electrical equipment (15 yr)	\$ 300,000
VWCR building (40 yr)	\$ 40,000
Existing WWTP modifications to accommodate VWCR	\$ 60,000
Construction Cost	<u>\$1,600,000</u>
20% drilling contingencies	\$ 320,000
20% non-construction costs (engineering, supervision, etc)	<u>\$ 320,000</u>
Total Capital Cost (PW)	<u>\$2,240,000</u>
Replacement Costs	
Reactor upcomer and downcomer ($\$300,000 \times .50245$) =	\$ 151,000
Mechanical/electrical equipment ($\$300,000 \times .35615$) =	<u>\$ 107,000</u>
Total Replacement Costs (PW)	<u>\$ 258,000</u>
Salvage Value	
Well drilling and casing 20/40 ($600,000 \times .25245$) =	\$ 76,000
VWCR heat exchange lines, air lines and casing 20/40 ($300,000 \times .25245$) =	\$ 38,000
Mechanical/electrical equipment 10/15 ($300,000 \times .25245$) =	\$ 50,000
VWCR building 20/40 ($40,000 \times .25245$) =	<u>\$ 5,000</u>
Salvage credit (PW)	<u>(\$ 169,000)</u>
Operation and maintenance costs	
$\$154,250 \times 10.49186$	<u>\$1,618,000</u>
Total Present Worth	\$3,947,000
Equivalent Annual Costs ($\$3,952,000/10.49186$)	\$ 376,000/ yr.

*See Figure 3 for definition of system components.

The VWCR system is sized to treat sludge generated by a 7.5 mgd WWTP.

Discount rate = 7-1/8%; 20 year life cycle period. Potential energy recovery credits are not included in O&M.

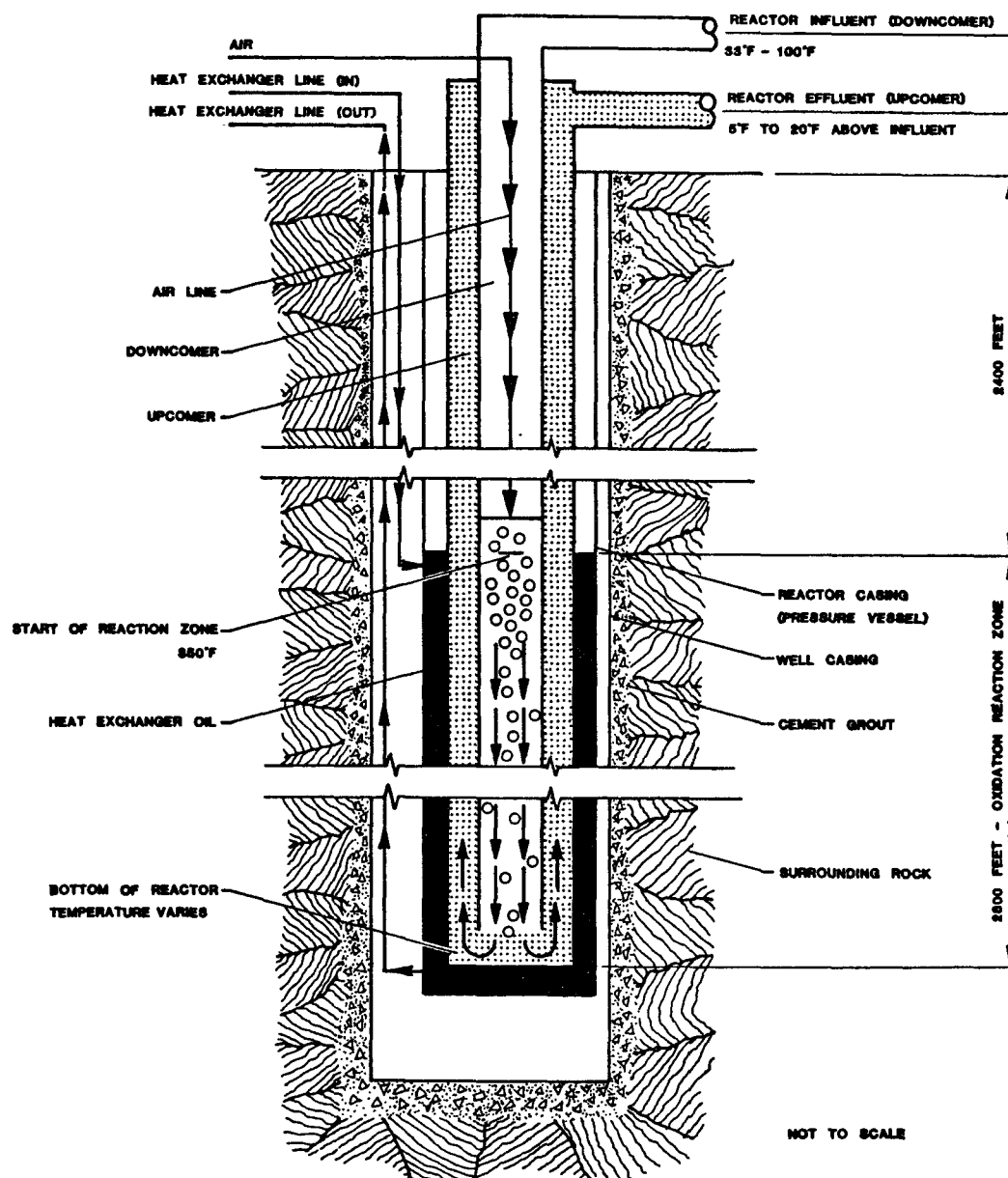


Figure 1. Typical vertical well chemical reactor profile.

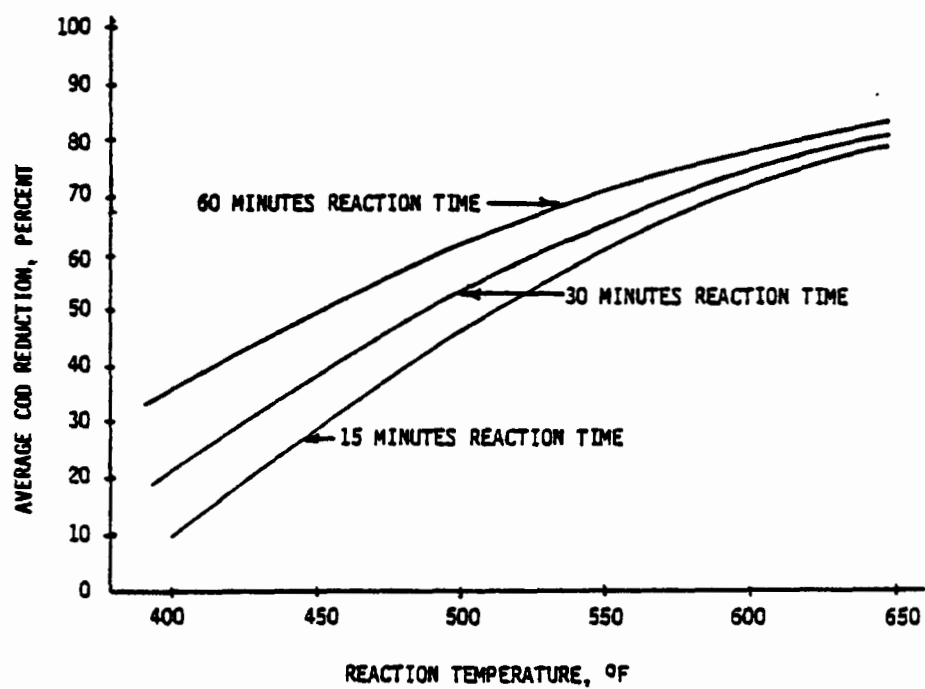


Figure 2. Typical COD reduction versus reaction temperature and time using laboratory reactor data.

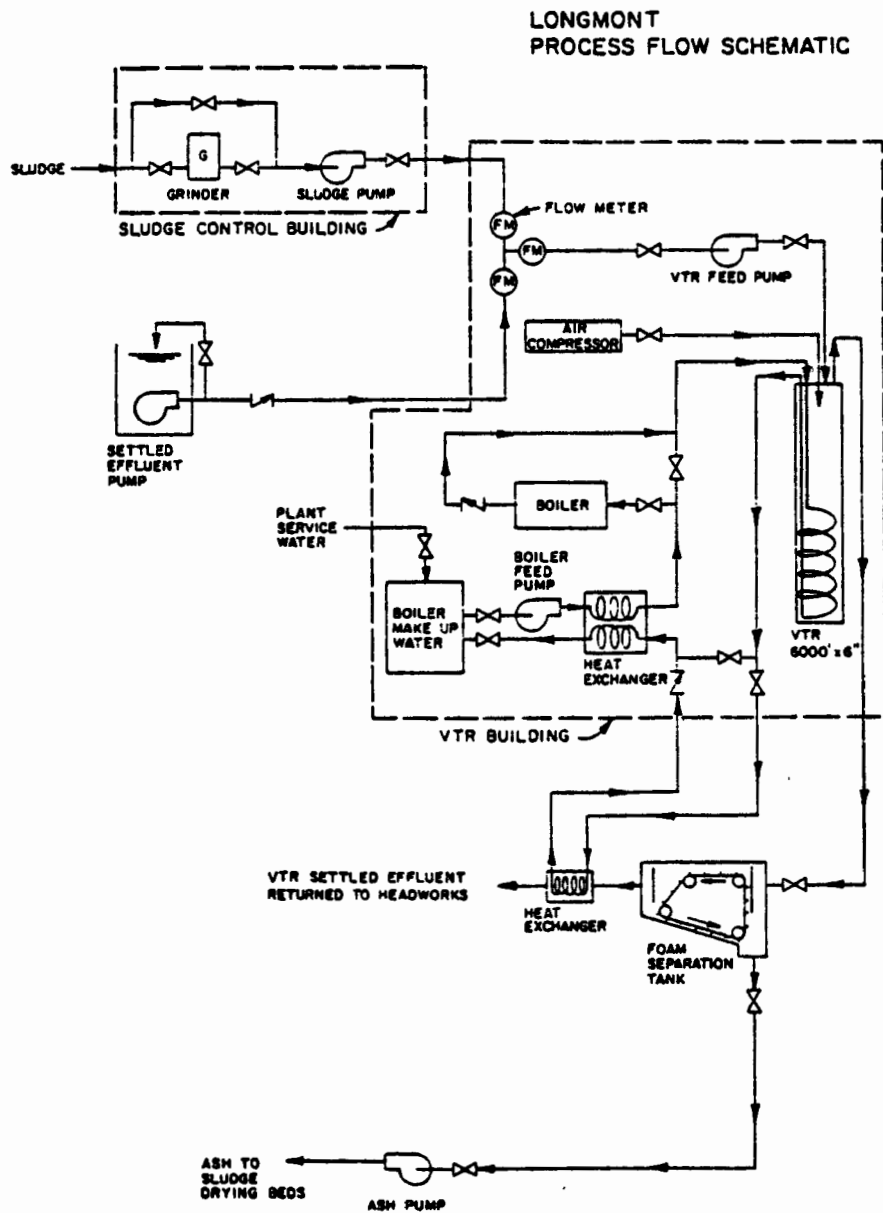


Figure 3. Proposed Longmont VWCR process flow schematic.

FLOW MANAGEMENT BY POROUS PAVEMENT AND INSITUFORM CONTROL OF INFILTRATION

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Cincinnati, Ohio

ABSTRACT

The runoff from large paved urban areas and the infiltration of storm-water into sanitary sewers both can constitute large volumes of polluted waste. These wastewaters either cause environmental problems directly from discharge to receiving waters or indirectly by overloading sewage treatment plants and interfering with proper operation. Porous pavement provides a possible solution for reducing or totally eliminating the runoff from parking lots and low traffic volume streets. Insituform is a novel method for rehabilitating leaky sewers that has the potential for a much higher degree of infiltration reduction than has been possible using more conventional methods.

STORMWATER MANAGEMENT BY POROUS PAVEMENT

Urban Stormwater Problem

Urbanization produces significant changes in the hydrologic characteristics of a watershed that can result in two major problems, flooding from increased runoff and water quality degradation. Consequently, various communities have enacted legislation which allows no increase in runoff rates, or water quality degradation beyond pre-developed conditions.

Stormwater management has generally consisted of collecting all the runoff in a conveyance system of sewers and channels which are tributary to a nearby receiving water. High peak flows result, however, which can create severe flooding problems downstream. In addition, impervious areas generate urban pollution that is difficult to control because they do not have capacity to assimilate pollutants. The major method to control runoff excess is the use of detention and retention ponds. In areas where a number of contiguous impervious surfaces exist or are planned, the use of porous pavement to restore the infiltration and storage capacity of an urban watershed becomes a viable flow and pollutant control alternative.

Porous Pavement as a Control Technique

Porous pavements provide storage that can be used to reduce runoff to preurbanization levels. They capture the initial runoff or "first flush" volume, which most studies indicate to be the most degraded in terms of

pollutant concentrations, and most of the subsequent flows which are likely to be less polluted.

Porous pavements may be used in areas that are already urbanized, such as downtown areas of most cities, as well as in existing suburban shopping centers where the storm sewer network was installed prior to excessive impervious cover development. Under these conditions, the storm sewers may become overloaded and the disposal of excess runoff becomes a problem that porous pavements could solve. This benefit is enhanced in areas with combined sewerage because the probability of the sewer overloading and the resultant discharge of raw sewage into the receiving waters is reduced.

In areas being developed, porous pavement has the potential for reducing both storm sewer requirements and the volume of runoff.

If the stormwater requires treatment, it may be stored in porous pavement systems isolated from the natural ground by an impermeable membrane until the treatment plant capacity becomes available. Thus, treatment plant capacity does not need to be expanded. Also, detention of highly polluted initial runoff by the porous pavement, and dilution by less polluted subsequent runoff might result in acceptable pollutant concentrations throughout the storm duration.

Natural vegetation and drainage patterns can be retained by the use of porous pavements. The clearing of trees from large areas for parking lots is unnecessary, and their aesthetic benefits need not be lost. Other potential benefits include construction cost reductions due to elimination of curbs and gutters, groundwater recharge, traffic safety resulting from skid resistance and improved visibility on wet pavements.

Porous Pavement Development

The earliest applications of porous pavements were for nonstorage purposes and consisted of a thin layer of open-graded asphaltic mix placed on top of a regular pavement to provide improved drainage and reduce the possibility of skidding or hydroplaning. Highways and airport runways utilizing porous asphaltic pavements with no storage capacity have been constructed in California, Kentucky, New Mexico and several other states. The use of porous pavements as stormwater management systems was initiated by the Franklin Institute in Philadelphia, Pennsylvania in 1971 under the sponsorship of the Environmental Protection Agency (EPA). The intent of this study was to combine the porous asphalt mix with a porous base and porous subbase in order to collect and store the runoff rather than to remove it to a stormwater collection system.

Existing porous pavements may be of two types: open-graded asphalt concrete overlying a crushed stone base course, such as is shown in Figure 1, and lattice work concrete blocks shown in Figure 2. Porous asphalt pavements can be underlain by a gravel base course with whatever storage capacity is desired. The whole system can be isolated from the natural ground by an impermeable membrane such as a polyethylene liner, in which case some type

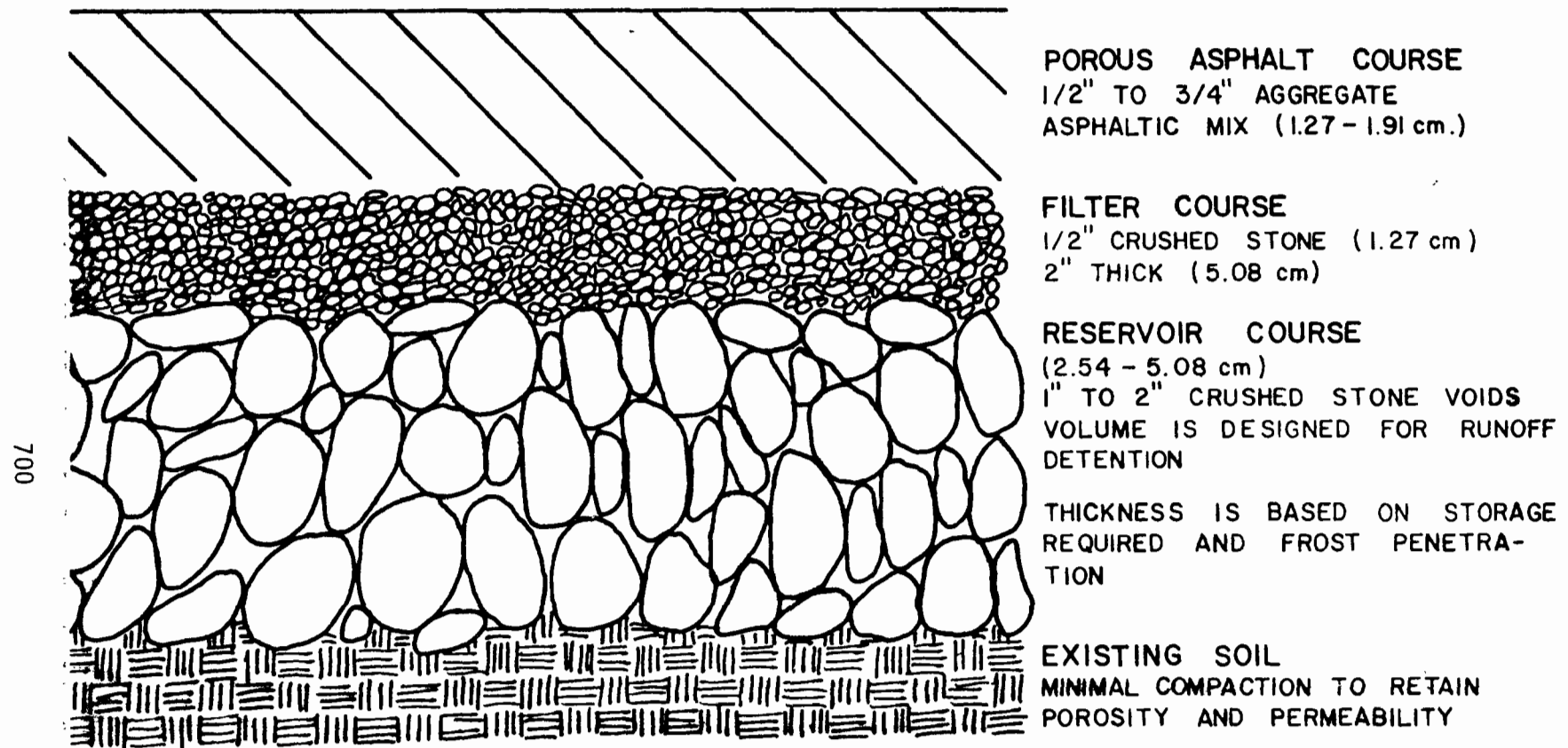
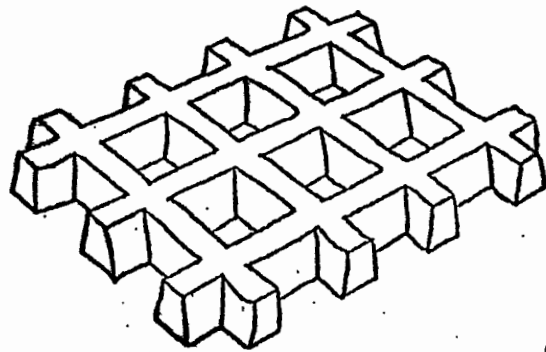
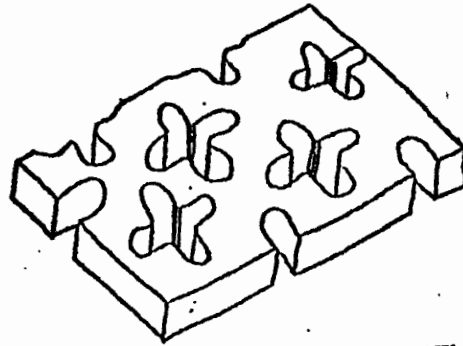


Figure 1. Porous Asphalt Paving Typical Section

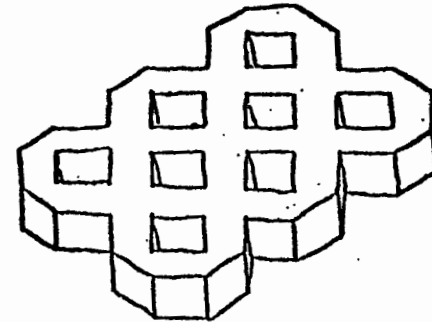
"LATTICE" TYPE PAVERS



"GRASSTONE"
BOIARDI PRODS.

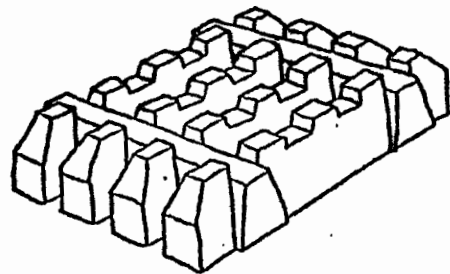


"GRASSCRETE" (POURED IN PLACE)
BY BOMANITE CORP.

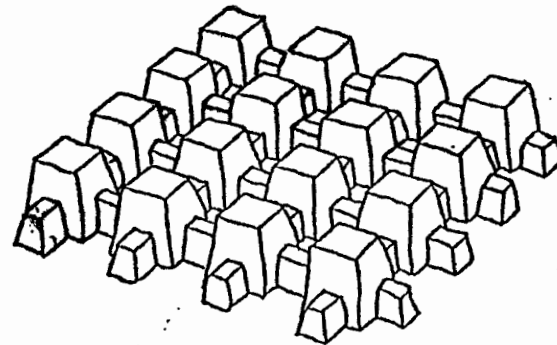


"TURFBLOCK"
PAVER SYSTEMS, INC.
WAUSAU TILE

"CASTELLATED" TYPE PAVERS



"MONOSLAB"
GRASS PAVERS, LTD.



"CHECKER BLOCK"
HASTINGS CO.

FIGURE 2. Concrete Porous Pavements

of an artificial drain would be needed. The result in these situations is primarily attenuation of flow with very little flow reduction. In the more common situation the porous pavement system would be allowed to drain to the natural soil at all points of contact. The latter arrangement does not preclude the use of artificial drains, which still might be needed in the case of highly impervious natural soil.

Lattice work concrete blocks will not be discussed at length here. With grass planted in the interstices they can provide an aesthetic and practical solution to water control problems. These also are underlain with a porous base and can be underdrained. Whether the grass will survive is a function of the intensity of use of the area. Residential parking areas for apartment complexes are normally vacant during the daytime, and, generally, driving speeds are minimal. Under these conditions, the grass suffers no detrimental effects. In heavier use areas the surface in the interstices can be kept lower than the concrete surface so that the grass is kept trimmed but not worn out by traffic passing over it. The lowered level can create a hazard to pedestrians by catching the heels of shoes.

State of the Art of Porous Asphalt Pavements

Numerous porous asphalt pavements have been installed in Pennsylvania, Delaware, Texas and in several other states. However, for only a few of the sites are any data available on design, hydrology, construction methods, operation and maintenance of the site, and problems encountered in construction and maintenance. The only scientifically instrumented asphaltic porous pavement area was part of an earlier EPA project at Woodlands, Texas (1). Tables 1 and 2 includes general information on sites which have some technical data available. The pavements at most of these sites appear to have performed adequately, but continuous monitoring to obtain and document definitive information on their long term porosity and structural integrity has not been carried out.

An EPA project (2) designed to accumulate what data are available on the design, construction, and operation of existing asphaltic porous pavement sites concluded that:

- The design of porous pavements has to be undertaken with extreme care, particularly in areas where the natural soils do not have sufficient permeability to naturally drain the stored runoff within a reasonable time.
- The use of porous pavements is presently limited to parking lots only. Ideally, these parking lots should be located on soils which have very low runoff potential or which have a high percolation rate.
- Infiltration and velocity reductions in porous pavements will result in some suspended particulate removal and some chemical pollutant reduction. Preliminary data indicate that nutrients and concentrations of some heavy metals can also be reduced. In any case, the removal of water from the surface runoff regime prevents

Table 1. Existing Porous Pavement Areas (2)

Number	Location
1A	South College Avenue Parking Lot. Newark, Delaware
1B	Orchard Road Parking Lot. Newark, Delaware
2	Marine Sciences Center. Lewes, Delaware
3	Woodlands, Texas
4A	Bryn Mawr Hospital. Bryn Mawr, Pennsylvania
4B	Bryn Mawr Hospital, Lot No. 2. Bryn Mawr, Pennsylvania
5*	Havertown Hospital. Havertown, Pennsylvania
6*	Newton, Savings Association Parking Lot, Washington Crossing, Pennsylvania
7	Travelodge. Tampa, Florida
8*	Salisbury State College. Salisbury, Maryland
9	Powell Ford Park. New Castle County, Delaware
10*	Coney Island Housing Project. North of Nathans, New York
11*	Korman Interplex. Philadelphia, Pennsylvania
12	Bell Telephone Company, West Goshen Township, Chester County, Pennsylvania
13	Bell Telephone Company. Newtown, Pennsylvania
14	Hollywood Hospital. Perth, Australia
15	Hamersley Headquarters Telecom. Perth, Australia
16*	Zurich Hilton. Zurich, Switzerland

*Identified existing sites for which data were not available

Table 2. Technical Data for Existing Porous Pavement Areas

Porous Pavement Site	Installation Date	Design Storm (in.)	Area (acres)	Slope %	Soil	Topography		Available Outflow	Type of Use
						Permeability ft./Day	Side Flow		
1A	1973	6.0	0.64	2.0	----	0.50-1.00	from North	Southwest Corner	Light Parking
1B	1974	---	1.37	---	Brown Clay, Slightly sandy, Silt	576.0	----	Northwest and Southwest corners with french drains	Light Parking
2	1974	---	5.00	---	Clay lenses, 3 ft. in diameter	-----	----	-----	Parking and service Roads
3	1975	---	0.50	---	Poor, replaced w/ 3 ft. of sand	-----	----	Nearby Creek	Light Parking
4A	1975	5.0	0.75	1.4	-----	1.00	none	3-4 inch drains at subbase for each section	Parking 100 cars
4B	1977	---	0.22	3.5	-----	-----	none	-----	Parking 33 cars
7	1973	---	---	---	-----	-----	----	-----	Parking Lot
9	1974	4.4	0.93	1.8	Clay, silt, gravel, silty sands & Silty Clay	0.050 0.120 0.003	none	Catch basin & 1-12" RCCP	Parking 109 + cars
12	1977	---	2.76	2.1	Glencig Channery Silt loam	-----	runoff from building	Retainage trench along N.W. boundary	125 employee cars, 100 company vehicles
13	1976	7.2	4.00	---	Readington Silt loam	-----	Roof drains onto pavement	Outlet stream to Newton Creek	Parking and delivery
14	1978	---	---	---	-----	-----	-----	-----	-----
15	----	---	2.29	---	Limestone sand	-----	-----	-----	Parking Lot

Table 2. (Continued)

Porous Pavement Site	Asphalt Mix Thickness inches	Base Thickness inches	Drains Installed	Remarks/Comments
1A	2.5	12	30 ft. trench, 15 ft. X 3 ft. L-shaped 3/4 inch stone	Asphalt was laid on hot day, trouble with rolling, trouble with trucks disturbing subbase, had to regrade.
1B	2.5	12	45 ft. trench, 15 ft. X 3 ft. L-shaped 3/4 inch stone	Northwest Corner
2	2.5	6	30 ft. trench, 15 ft. X 3 ft. 3/4 inch stone	Southwest Corner
3	2.5	12	5-4" drains to nearby creek	-----
4A	2.5	10-14	-----	Pavement Failed
4B	2.5	12	-----	6 1/2" layers of 2" stone 5' back from end of lot
7	---	---	-----	Breakdown of asphalt by large trucks parking, water ponding
9	2.5	12	40 ft. trench 6.5 ft. X 3 ft. in end of lot	In place for 5 years, excellent condition water runs off end of lot
12	2.5	16	-----	Mud from heavy equipment plugged porous pavement lot, fork lifts gouge pavement
13	4.0	8-20	-----	-----
14	1.2 2.0	4-6	-----	Limestone base is less expensive than crushed rock
15	1.2 2.0	4-6	-----	-----

Notes: 1 inch = 2.54 cm
1 foot = 30.48 cm
1 acre = 0.405 hectare

the introduction of pollutants into the receiving water that could create problems in downstream areas.

- If it is known, or the possibility exists, that water infiltrating in the ground could reach a water supply aquifer, adequate precautions should be taken to determine that the surface runoff is not still contaminated when it reaches the aquifer. If there is a possibility of adversely affecting the aquifer, the porous pavement areas should be designed to be sealed off from the aquifer recharge zone.
- The aggregate in porous pavements should contain a minimum of two percent passing the Number 200 sieve to provide stabilization of the coarse fraction. Consequently, the following size gradation is recommended for the open graded asphalt mix.

<u>Sieve Size</u>	<u>Percent Passing Through</u>
1/2 in.	100
3/8 in.	90-100
# 4	35-50
# 8	15-32
# 16	2-15
# 200	0-3

- The total asphalt cement content for the mix is suggested to be between 5.5 and 6.0 percent. However, the actual percentage must be determined in the field, particularly if the characteristics of the aggregate are not known from previous experience. Also, dry aggregate should be used to avoid vapor release after the aggregate is coated. Insulated covers must be used on all loads during hauling to prevent the asphalt from crusting on the surface of the load. Also, medium to light weight vibratory rollers are somewhat better for compaction of the open graded asphalt mix.
- A two-inch (5.08 cm) course of 0.5 in. (1.27 cm) gravel was found to be desirable to stabilize the top of the gravel reservoir underlying the open-graded asphalt mix. This is referred to as the filter course in Figure 1. The gravel reservoir, composed of one to two in. (2.54 to 5.08 cm) crushed stone, is designed to control the total volume of runoff computed for the area based on a preselected design storm and a hydrologic analysis of the area. Both the filter and reservoir gravels should be pre-washed to remove excessive fines which may tend to affect the permeability of the underlying soil. Because the length and width of the base reservoir are generally limited by the dimensions of the parking lot, the only variable dimension is the depth. If sufficient depth cannot be obtained due to physical limits, additional relief drainage structures such as french drains and pipe drains may be installed. If additional structures are required, the cost of these structures can be quite expensive.

Consequently, conventional drainage schemes could become cost-competitive and viable alternatives. If the subbase does not drain at a sufficient rate, relief drainage structures can be incorporated, or additional excavation, or replacement with material having more desirable drainage characteristics can be contemplated.

- The total thickness of the base reservoir should be the largest depth requirement for the bearing strength of the wet subbase, the hydrologic storage requirements, or the frost depth for the site.
- The initial costs of porous asphalt can be as high as 35 to 50 percent above the cost of conventional paving. However, the major reason for this difference is the required use of new technology involved in porous pavement production, primarily in gradation requirements and the narrow limit on asphalt cement content in the hot mix. If curbs, gutters, or storm sewers are not required, the total cost of the parking lot can be comparable to or cheaper than a conventional parking lot, especially if the aggregate source for the asphalt mix as well as for the base reservoir are easily available. It is anticipated that with the construction of additional porous pavement areas, technology transfer should be facilitated. Construction crews will become more familiar with the process, and contractors will be able to bid lower on porous pavement jobs.
- Existing porous parking lots were constructed without curbs and gutters around the surfaced area. Porous pavements operate efficiently and there is less chance of debris accumulation on the parking lot if curbs are eliminated.
- For most efficient operation of porous pavements it is desirable that the subbase not be compacted or be only minimally compacted. This will retain the original permeability of the soil which can be substantially reduced after compaction.

A recently completed EPA study of two asphaltic porous pavement parking lot sites in Rochester, New York (3) was designed to demonstrate the structural integrity and permeability of the pavements under severe environmental and heavy truck traffic conditions. Results of the study indicated:

- Peak runoff rates were reduced by as much as 83%.
- The pavement, which was subjected to 100 freeze/thaw cycles, showed no observable structural degradation. In addition, the water drained through the pavement without problems during the winter. Through observations and flow monitoring, the structural integrity of the porous pavement installed, where heavy load vehicles were parked, was not impaired.
- Clogging did result from runoff carrying a heavy sediment load. Clogging during the test study was relieved through cleaning.

- The cost of constructing a porous pavement parking lot utilizing an impermeable membrane and underdrains, \$18/yd² (\$22/m²), is slightly higher than that of a conventionally paved lot with storm-water inlets and subsurface piping, \$16/yd² (\$19/m²). If subsurface soil conditions are adequate to allow passage of the rainfall that infiltrates through the porous pavement, and if there is no ground-water pollution problem, then the impermeable membrane and underdrains are not needed and costs for both types of pavements would be the same.

Current Demonstration of Porous Pavements at Austin, Texas

Although Table 1 indicates a number of porous pavement parking lots have been constructed, the design and construction of porous pavements is in an early stage of development and only a few engineers and contractors have any experience with this type of pavement. This situation is likely to continue until sufficient demonstration pavement areas are installed and long term data is obtained on: (1) runoff quality and quantity changes; (2) the effects of continuous saturation of the subgrade; (3) maintenance and potential for plugging; and (4) the economics of using these pavements under existing regulations. An EPA project is currently underway with the City of Austin, Texas to obtain some of this information on the following seven types of parking lot surfaces:

- Porous asphalt
- Gravel
- Lattice concrete
- Conventional concrete
- Grass (natural condition for control)
- Conventional asphalt
- Conventional asphalt with trench storage

Characteristics to be investigated include: (1) construction feasibility and design life; (2) runoff control; and (3) water quality control. The resulting information will be used to develop design criteria for porous pavement construction. Emphasis in this project is on the asphaltic type of pavement.

Each pavement is sloped to a collection barrel that is instrumented with flow devices and recorders to measure both surface runoff and underflow. The porous asphalt and gravel lots were constructed over an impervious limestone area and all underflow moves horizontally and does not enter the groundwater. The lattice concrete and grass lots allow potential percolation through the soil and into the groundwater. However, the soil in the area is clayey and slow to drain. Test wells are included at these sites to obtain samples of the water for quality analyses and rain gauges are located near each study area to accurately record rainfall intensity and duration.

With regard to the porous asphalt there was a lack of experience by the City of Austin paving department and the asphalt suppliers, in paving

with, and producing the required open-graded mix. This lot is located on the grounds of one of the City's maintenance facilities and serves as parking space for the employees. The dimensions are 80 by 200 ft (24 by 61 m). Figure 1 shows the type of pavement construction. The thickness and particle size of each illustrated course are given in Table 3. AC-20 asphalt was used in the porous pavement design and the characteristics of the mix are shown in Table 4. The depth was designed to store the 25 yr storm which is 4.2 in/24 hr (10.7 cm/24 hr).

Table 3. Thickness and Particle Size of Pavement Courses at Austin, Texas

Course	Thickness (in.)	Particle Size
Porous Asphalt	1 3/4	See Table 4
Filter Course	2	3/4 in. Stone
Reservoir	8 to 42	1-2 in. Stone

To convert in. to cm multiply by 2.54

Table. 4 Characteristics of Asphalt Porous Pavement Mix

Sieve Size	Percent Passing
1/2 in.	100
1/2 - 3/8 in.	7.6
3/8 in. - #4	59.3
#4 - #8	21.1
#8 - #16	3.4
#16 - #200	8.2
#200	0.4
Percent AC-20 Asphalt	5.5
Specific Gravity	2.221
Stability	645 psi (4440 kN/m ²)

The mix was transported in covered trucks in order to maintain the temperature near 300°F (149°C) and was placed on the rock base using standard paving equipment. Since the degree of compaction is critical in obtaining a structurally sound, but highly permeable pavement, much discussion took place at the site between the pavement designer and the foreman of the paving construction crew. Pneumatic and two different size flat rollers, 2

and 8 ton (1820 and 7280 kg), were available for use. It was concluded after several trial and error compaction techniques that 2 passes with the 2 ton flat roller and 3 with the pneumatic roller would produce an optimum compromise of strength and permeability. Since there exists a trade-off between structural integrity and inherent permeability as a result of the degree of compaction employed, it was decided to vary the compaction in several sections of the parking lot and evaluate future performance of the porous pavement in order to correlate the results with construction techniques.

The permeability characteristics of the porous asphalt surface were determined after 18 months of vehical use by in situ tests using an infiltrometer designed and fabricated specifically for this study. The infiltrometer consists of a 6-inch (15.2-cm) inside diameter clear PVC pipe 18 inches (45.7 cm) high with a stopper mechanism which allows a timed release of a known volume of water through a 3.5 inch (8.9 cm) diameter hole at the base of the infiltrometer. The permeabilities of the porous asphalt surface ranged from 152 in/hr (386 cm/hr) to 5290 in/hr (13,437 cm/hr) with an average rate of 1766 in/hr (4486 cm/hr). This rate compares to an average rate of 70,000 in/hr (177,800 cm/hr) for the 2-inch (5 cm) reservoir base course. Notable exceptions to this average rate occurred on the west side of the lot, throughout the driveway path, and in the northeast corner. Lower than average infiltration rates on the west side of the lot can be attributed to rolling the asphalt at a temperature higher than 180 F (82.2 C). The driveway path has more traffic than other parts of the lot and may be compacted more than normal. The northeast corner of the lot was noted to have soil accumulating in the pores of the porous asphalt surface, possibly transported there by tires of vehicles.

Table 5 compares the Austin permeability results with others reported in the literature. There is a wide variation in the results which is probably related to use of different types of permeability testing techniques rather than to the use of different compaction methods. The Austin results were measured with a very low hydraulic head most closely simulating actual conditions. In all cases very high degrees of porosity were initially obtained with asphaltic porous pavement.

Table 5. Asphalt Porous Pavement Permeability Test Results

Project Location	Porosity (in./hr)	
	Average	Range
Austin, Texas	1766	152 - 5290
Woodlands, Texas ⁽¹⁾	2000	-
Rochester, New York ⁽³⁾	-	170 - 1980

To convert in./hr to cm/hr multiply by 2.54

The data from the Austin, Texas project combined with what information can be obtained from asphaltic porous pavements constructed in the early 1970's will be used to develop a design methodology for porous paving systems which can be used by planners, engineers, and building plan reviewers.

Conclusions

Porous pavements of various types constitute a methodology for significantly reducing runoff from parking lots and lightly traveled roads or for accomplishing flow storage and attenuation. Costs can be competitive with conventional pavement if curbs, gutters and storm sewers can be eliminated. Installation of collection drain piping within the base material to move the underflow laterally to an appropriate discharge point is necessary in locations underlain by impervious soils. Information is available to construct these pavements and more is presently being obtained, but it has not yet been satisfactorily organized and has not reached a large segment of pavement designers and constructors. More data on groundwater pollution potential would be useful. The most significant lack of information concerns pavement life and long term maintenance. Although some installations have been in existence for about a decade, their recent porosity has not been measured and the cost of maintenance has not been adequately documented.

INSITUFORM CONTROL OF INFILTRATION

Introduction

The infiltration of groundwater into sanitary sewers through leaky joints, and cracks and breaks in sewer pipes can add significantly to the amount of flow to sewage treatment plants. This is especially true during wet weather when the groundwater level is elevated. Also, during wet weather there can be inflow of water from roof leaders and other types of drains. The result of the combination of infiltration and inflow (I/I) is the overloading of treatment plants with a reduction in operating effectiveness.

Since the passage of the 1972 Clean Water Act Amendments, there has been a determined effort in the United States to reduce the effects of I/I at sewage treatment plants. There has been greatly increased activity in the use of sewer rehabilitation techniques. A survey conducted by the EPA in 1980 (4) indicated, however, that these techniques were not nearly as effective in reducing I/I as estimated by engineers before the rehabilitation was carried out.

There are a number of possible reasons that have been proposed for the ineffectiveness of sewer rehabilitation methods. One significant reason appears to be the lack of completeness in sealing all potential leaks. The sealing of only those leaks which appear to be the largest in volume apparently results in an increase in water level outside the sewer line and the initiation of new leaks at unsealed locations that formerly did not leak. A rehabilitation technique of reasonable cost that could seal all unwanted openings in the pipe wall should decrease measurably the amount of infiltration. A method conceived in England over a decade ago, but only recently introduced to the United States, has a good potential for completely sealing all leaks. It involves the complete lining of the inside of the pipe with a thin plastic layer and has the commercial name "Insituform". The plastic layer has sufficient strength to cover openings into a sewer such as service laterals and to prevent their further discharge. The method has, therefore, the capability of eliminating inflow from illegal connections. The method also produces little surface disturbance since installation can be made in most cases through existing manholes.

To assess the effectiveness of Insituform, two full-scale evaluations were undertaken, one at the Village of Northbrook, Illinois under a cooperative agreement between the Village and EPA. The information presented here is largely from that project (5). The other evaluation is being conducted in

Hagerstown, Maryland. No information is yet available from the Hagerstown project.

Description of the Lining Procedure

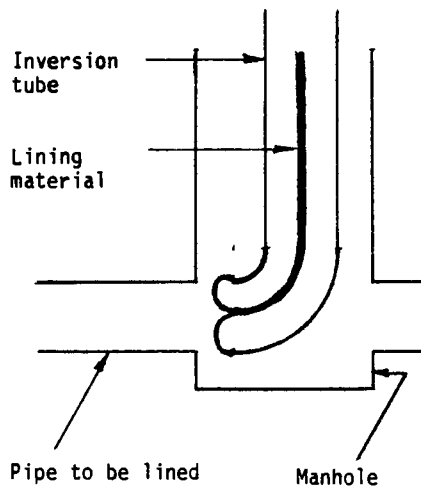
The concept of the Insituform process is to form a long tube or bag of partially polymerized thermosetting resin impregnated on a felt backing and closed at one end, and to invert this tube into the pipe to be lined, by filling with cold water. The felt tube has a thin film of polyurethane on what is initially the outside. Upon inversion of the tube into the pipe, the polyurethane film forms the pipe surface which is very smooth. This film also provides a leak proof barrier to the water being used for inversion. After inversion, hot water is pumped into the tube to cure the liner by completing polymerization. Figure 3 shows the important steps in installation of the liner. After installation the end of the tube is cut off with a power saw and the wall of the manhole near the pipe is finished with a sand-resin mixture. Reconnection of service laterals is carried out with a rotary cutter. For pipes too small to enter, a television camera-cutter combination has been developed.

Because this lining procedure is very new and improvements continue to be made, it is difficult to place precise limitations on the range of pipe diameters for which Insituform is practical. About 36 in. (91 cm) diameter is presently the largest size that can be lined with the water inversion method. Variations have been utilized for larger pipes in Europe including air inversion. Manhole covers in the United States can accommodate equipment for lining pipes up to about 24 in. (61 cm). Larger pipes would require more complicated preparation and, therefore, higher cost. The present minimum size is six in. (15 cm). Smaller sizes present problems during inversion and do not accommodate the available TV-cutter equipment.

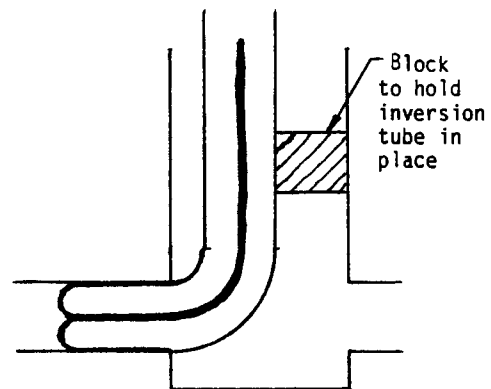
Specialized equipment and specific procedures have been developed for installation that allow for rapid completion of the lining of a pipe section. The felt bags or tubes can be prefabricated away from the site to fit the pipes involved. Impregnation can be conducted on-site or off with shipping in a refrigerated truck. Impregnation is not a lengthy procedure if done at the site. The water pressure for inversion is provided very simply by maintaining a sufficiently high column of water in an inversion tube. The flattened liner is led into the top of the inversion tube and the open end of the liner is firmly attached to a shoe at the tube base which has an appropriately sized round opening. The pressure from the water in the inversion tube inverts the liner similarly to the blowing up of a balloon. A rope is attached to the closed end of the liner that is used to control the rate of inversion. Too rapid an inversion can result in momentary head loss and poor contact of the liner with the pipe. A hose is also attached to the end of the liner to be used after inversion is complete to supply hot water at the far end of the liner segment for curing. A boiler and pump are used to produce and circulate the hot water.

At Northbrook, two consecutive 12 in. (31 cm) diameter vitrified clay pipe segments were lined. Segment lengths were 150 ft (46 m) and 432 ft

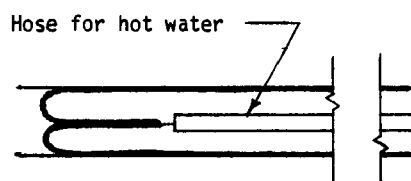
1. Lining material is threaded into inversion tube and attached at bottom



2. Lining material being inverted after inversion tube is placed against pipe opening



3. Lining material nearly inverted



4. Completed lining ready for cutting of end of tube

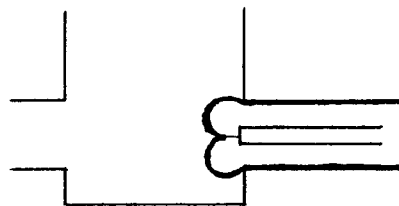


Figure 3. Steps in Lining with Insituform

(132 m). There were manholes at the entrance to the first segment, between the two segments and at the outlet of the second segment. There were two six in. (15 cm) service laterals entering the first pipe segment. The sewer was a sanitary sewer installed in 1962 and had many offsets, and radial and longitudinal cracks. Some sections were deteriorated to the point of not being circular.

Before lining, the sewer segments were cleaned well since any remaining debris would not be forced out during inversion, but would increase roughness of the lining, reduce flow capacity and reduce strength. Very sharp protrusions could break the liner and interfere with curing.

The felt tube was built up from two, 3 mm layers of densely needled polyester fiber. Thickness can be increased by adding 3 mm layers. Impregnation was conducted at the site by filling the liners with the appropriate amounts of a catalyzed thermosetting isophthalic-acid-based resin and passing the liner through a system of conveyers and rollers to thoroughly wet all of the felt. Inversion was carried out using a 6 m static head. The two pipe segments could have been lined with one length of liner and one inversion, but two were used on successive days to allow observation by a larger number of interested observers.

The desired curing temperature for the chosen resin was approximately 180°F (82°C). The curing cycles for the two segments are shown in Table 6. Considerable time was required to heat the water to curing temperature. Cooling is purposely slowed to an hour or more to prevent rapid contraction that might cause separation of the liner from the pipe. Small thermocouples placed between the inside of the original pipe wall and the outside of the liner at both ends of each length being lined were used to determine whether curing temperature was being reached through the total thickness of the liner. The conditions shown in Table 6 would be typical of most installations.

Results at Northbrook

The success of the lining process was tested by observing the infiltration into the upstream pipe segment just before and just after lining and by observing exfiltration from the downstream segment just before and just after lining. The infiltration test was carried out by plugging the upstream manhole and measuring flow over a weir at the downstream end of the pipe segment. Groundwater levels were approximately 35 cm above the crown at the outlet. The average value of infiltration before lining was 19,500 gpd (74 m³/day) and after lining was about 100 gpd (0.4 m³/day). The small amount of inflow after lining is believed to have resulted from leakage at the upstream manhole since television (TV) inspection of the pipe showed no breaks in the lining. The exfiltration test was carried out by plugging the line just above the manhole at the upstream end of the pipe and the line just below the downstream manhole. The pipe segment was then flooded to 3 ft (91 cm) above the crown at the upper manhole. The rate of fall of the water was measured. Before lining, the rate of exfiltration was 3,800 gpd (14 m³/day) and after lining 270 gpd (1.0 m³/day).

The small amount of exfiltration after lining is believed to have occurred in the manholes.

An indirect measure of the effectiveness of lining was an obvious increase in the leakage into the adjacent manholes through cracks in the manhole walls. This observation confirms the experience with sewer grouting methods where, after grouting, leakage appears at points that did not formerly leak. The cause is ascribed to higher groundwater level resulting from the elimination of the most significant leaks. Where the condition of manholes is questionable, their rehabilitation should also be considered.

Table. 6 Resin Curing Cycles

	Temperature Range (°F), (°C)	Curing Time (hr)
Upper pipe segment	55-160, 13-71	0.75
	160-185, 71-85	0.5
	185, 85	2
	185-100, 85-38	<u>1</u>
		4.25 Total
Lower pipe segment	55-160, 13-71	1.25
	160-185, 71-85	0.75
	185, 85	2
	185-100, 85-38	<u>1.5</u>
		5.5 Total

One of the advantages claimed for the insitu lining of sewers is the increased smoothness that, in relation to carrying capacity, compensates for, or more than compensates for, the slight decrease in pipe diameter. At Northbrook the Manning coefficient was measured after lining. In the upstream segment the value was 0.0078; in the downstream segment the value was 0.0085. These values are comparable with those for other plastic pipe such as PVC. Accurate values for the Manning coefficient were not obtainable before lining so a comparison of maximum water carrying capacities before and after lining was not possible at Northbrook.

A number of physical properties of samples of the lining material used at Northbrook were measured. These are shown compared to PVC in Table 7.

Table 7. Properties of Insituform Compared to PVC

Property	Insituform	PVC
Tensile Strength (psi)*	5,420	7,200
Modulus of Elasticity (psi)	475,000	400,000
Flexural Strength (psi)	9,320	11,000
Compressive Strength (psi)	15,500	9,000
Coefficient of Thermal Expansion (cm/cm °C)	5.96×10^{-5}	5.2×10^{-5}

* To convert psi to kN/m² multiply by 6.895

It can be concluded from this comparison that the two materials are very similar in their physical characteristics.

Insituform Costs

Approximate costs for long total length installations of Insituform are shown in Table 8. These figures include the fabrication and installation of the liner and cutting of openings to service laterals. Additions for pipe cleaning and inspection and for bypass pumping of sewage must be made as shown at the bottom of Table 8. In addition, there are other relatively small costs for setting up of equipment (mobilization) and for traffic control.

Table 8. Insituform Costs

Sewer Diameter (in.)*	Liner Thickness (mm)	Cost Per Lineal Foot (\$)	Cost Per Meter (\$)
6	3	33	108
	6	39	128
12	6	47	154
	12	55	180
18	6	57	187
	12	66	217
24	9	74	243
	15	86	282

Note: Add \$1,900 per line for bypass pumping. Add \$1.90 for 6 in. to 15 in. pipe and \$2.50 for 15 in. to 30 in. pipe per foot for preliminary cleaning and inspection.

* To convert in. to cm multiply by 2.54

A realistic experienced cost cannot be obtained directly from the installation at Northbrook because of the demonstrative nature of the project which limited the total length to be lined and increased installation time and cost for sewage bypass. An estimate can be made for Northbrook, however, based on Table 8 information. For the two pipe segments lined and with \$1,500 estimated for mobilization and traffic control, the cost would be approximately \$55/ft (\$166/m).

In making this estimate it is assumed that other sewers in the area would be lined at the same time to take advantage of the large-order prices shown in Table 8. An estimate was also made at Northbrook for the cost of complete replacement of the pipes as an alternate to lining. This cost was \$79/ft (\$260/m). At Northbrook, therefore, the cost of insitu lining was about 70% of the cost of replacement. It is important to remember, however, that sewer replacement usually causes severe disturbance to or prohibits traffic entirely on the streets involved, and often causes significant financial loss to businesses in the area or inconvenience to residents. Because insitu lining requires little or no surface disruption, the inconvenience is minor and of only a day or two duration.

The Washington Suburban Sanitary Commission (WSSC) became interested in Insituform and has made cost estimates for both Insituform lining and sewer replacement for a number of sewers being considered for rehabilitation in Maryland (6). These estimates are shown in Table 9. At these locations the ratio of the cost of Insituform to the cost of replacement ranged from 36% to 76%. For the sum of the costs for the sewer pipe segments, the ratio was 44%, significantly lower than at Northbrook. This probably reflects the greater congestion of some of the Maryland sites, requiring higher cost for sewer replacement. The costs indicate, as in Northbrook, that insitu lining has potential for reducing significantly the cost for rehabilitation of badly deteriorated sewers.

Table 9. Comparative Costs of Insituform and Sewer Replacement at WSSC

Pipe Diameter In.)*	Length (ft)**	Insituform Cost	Sewer Replacement Cost
6	556	\$ 27,495.00	\$ 56,000.00
8	10,304	482,269.26	1,028,000.00
10	10,445	492,999.34	1,365,000.00
12	4,015	225,312.35	422,000.00
15	1,667	103,173.40	184,000.00
18	782	68,240.80	90,000.00
24	3,272	336,847.60	800,000.00
TOTAL	31,041	\$1,736,337.75	\$3,945,000.00

* To convert in. to cm multiply by 2.54

** To convert ft to m multiply by 0.305

At Northbrook the condition of the pipes was too poor to consider specific point repair and grouting. The same was true for the pipe segments considered by the WSSC and shown in Table 9. Had grouting been practical, it would have been a cheaper alternative. Grouting is not as effective as Insituform and in a number of documented cases was essentially ineffective (4). The cost effectiveness of grouting in terms of actual reduction in I/I can, therefore, be very low.

Conclusions

Insitu lining of badly deteriorated sewer pipes provides a new rehabilitation alternative that reduces infiltration essentially to zero. The cost is substantially less than for sewer replacement and the surface disturbance is greatly reduced. The cost associated with surface disturbance can be very significant, especially in the centers of cities and other densely populated areas. The procedure is more expensive than grouting and therefore, would probably not be the method of choice where deterioration was minor and where grouting would be judged adequate. Care must be taken, however, in estimating the effectiveness of grouting. A survey of past grouting results indicates that the effectiveness, and resulting cost effectiveness, is likely to be greatly overestimated.

Because insitu lining is less than ten years old and experience in the United States is of very short duration, there are no dependable data on lining life or long term maintenance requirements. Continuing observation of Insituform installations is necessary to determine conclusively the technical and economic feasibility of this sewer rehabilitation method.

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SEQUENCING BATCH REACTORS FOR MUNICIPAL WASTEWATER TREATMENT

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INTRODUCTION

Antiquated techniques and modern hardware technology can be combined to provide an alternative process for municipal wastewater treatment.

At the present time, when continuous flow activated sludge is considered the paradigm of secondary treatment, it may seem contemptible to suggest a batch process may have superior attributes.

Historical technology is replete with examples of batch treatment of municipal wastewater. Sidwick and Murray (1) have outlined the evolution of batch processes into continuous flow processes in England. Early, in land treatment practices, it was recognized that intermittent irrigation of wastewater was necessary for reaeration of the soil to occur. In the year 1897, ways to conserve land were investigated. Vessels were constructed which contained various fine media (3 mm) to try to duplicate soil surfaces. The first of these were operated as simple fill and draw in accordance with intermittent land application experience. Later, a cyclic process was instituted which consisted of 2-hour fill, 1-hour stand full and 5-hour drain. The cycle was repeated twice daily. Seventy-eight percent removal of organic matter was reported. The need to supply oxygen in the proper amount for active biological oxidation was the key finding of this early work. From this point on, two divergent developments led to continuous flow processes. One utilized larger media with greater void space so that wastewater splashing from surface to surface entrained oxygen from the surrounding air. This, of course, is the now familiar biological trickling filter. The other retained the concept of a flooded vessel containing media, but instituted continuous flow with suitable valving and supplied air by an external blower. This technology was the progenitor of the Hays Process (2) and Imhoff's contact aerators (3). Conceptually, Imhoff discussed a variation of contact aerators which described rotating biological contactors before their actual invention.

The precursor to the various, now familiar, continuous flow activated sludge processes was actually a fill and draw system operated as a batch process. Ardern and Lockett (4) in 1914 were among the first to show the benefit of retaining substrate adapted organisms for efficient treatment. Working with 2.3 liter flasks containing Manchester, England raw wastewater, they showed that the batch aeration period to achieve nitrification could be reduced from 5 weeks to 9 hours if the sludge that accumulated from each

batch were retained in the flask after decanting the nitrified liquid. They coined the term "activated sludge" to describe the resultant biological mass.

In later studies, these same investigators (5) showed good nitrification in 4 hours and suggested a full-scale operational cycle as shown in Table 1.

Alvord (6) described some of the automatic dosing siphons and mechanical devices used to switch flows in early batch systems. Figure 1 is a view of the 10 bay, 900 m³/d (0.25 mgd) intermittent batch sand filter constructed on the shore of Lake Michigan in 1901 for the Village of Lake Forest, Illinois.

Table 1. Batch Treatment of Manchester Wastewater (modified from Ref. 5)

	Analytical Results, mg/l		Operational Cycle, hours
	Raw	Effluent	
kMnO ₄ (Oxygen Absorption)	124	18	Filling, 1 Aeration, 4
NH ₄ -N	37	18	Settlement, 2 Discharge, 1
Organic-N	12	1.9	-- 3 cycles per day
NO ₂ -N	--	1.4	-- 20 percent activated
NO ₃ -N	--	14	Sludge by volume

The building on the hillside housed the control mechanism for intermittent gravity application. Figure 2 is an inside view of the building showing the flow switching mechanism on the floor above the dosing chamber (7). When influent flow increased the liquid level in the dosing chamber, it caused the iron ball, resting on a plate attached to a float mounted pole, to elevate to the top level of one of the 10 inclined planes. The ball then rolled down the plane and activated a counter weighted valve operator leading to one of 10 discharge pipes to the intermittent sand filter. The ball rested on the valve operator until the discharge lowered the level of the dosing chamber, causing the next float position to lower to accept the ball, and thereby release the valve operator. There was no discharge until the influent flow again increased the liquid level in the dosing chamber and caused the ball to rise to the next incline. The only design constraint was to insure that the dosing chamber discharge rate exceeded the influent rate. There was no need for external energy or operator, and the continuously circulating ball unerringly dosed each bay in sequence.



Figure 1. Village of Lake Forest's Intermittent
Sand Filters, Year 1901

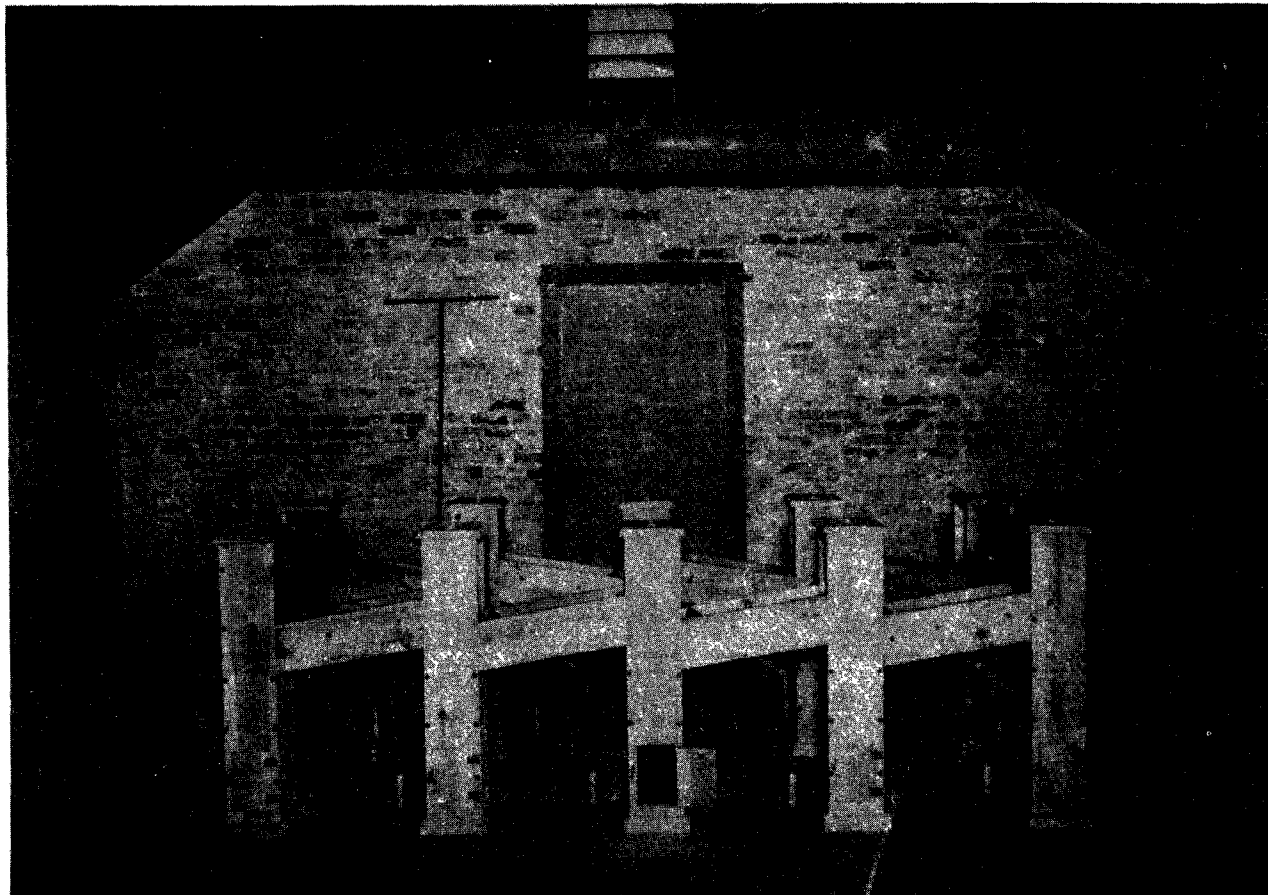


Figure 2. Control Device for Intermittent
Sand Filters, Year 1901

Fill and draw or batch systems were never applied to any great extent for municipal treatment. By 1920, when larger facilities were being constructed, batch systems were no longer considered viable. Batch systems, at that time period, would have required a high degree of manual operator attention, and reliable process valving, timing and switching technology was not available to counter that deficiency.

Lack of implementation of batch treatment is not due to early operational or process failures, in reality batch treatment for mainstream application is a long forgotten successful technology. Developments of new hardware devices, since the year 1900, such as motorized valves, pneumatically actuated valves, electronic and mechanical timers, solenoids, level sensors, and flow meters, coupled with an omnipotent microprocessor or process controller can revitalize this technology.

PRESENT DAY BATCH PROCESSES

Many present day industrial processes are batch processes, such as dairy, steel, pharmaceutical and antibiotic, fine chemical, cosmetic, and paint.

In some instances, state regulatory agencies require toxic wastes, such as cyanide, be detoxified by batch operation to insure the effluent can be monitored before discharge.

Most modern municipal treatment facilities utilize batch treatment in one guise or another, such as anaerobic digestion, aerobic digestion or intermittent discharge lagoons. Tchobanoglous (8) calls attention to the fact that the classical biological oxygen demand test (BOD) is a batch process.

Renewed interest in main stream batch treatment resulted from a mentorial article by Pasveer (9). He reported on the conversion of a continuously operated oxidation ditch, treating the waste from Sancta Maria Hospital, Noordwijkerhout, Holland, into a discontinuous discharge process with intermittent aeration. The operational change resulted in the control of a filamentous activated sludge, improved clarification and yielded 90 percent denitrification.

Goronszy (10) has applied this concept of single tank treatment systems in Australia. Extended aeration processes are operated with continuous inflow but discontinuous discharge. The liquid level in the tank varies and various type decant devices are used. Aeration is provided in a cyclic manner to encourage nitrification and denitrification.

These two approaches provide a high degree of treatment due to the fact that they are both lightly loaded systems. Since the volume of the tank is large in comparison to the influent flow, the possibility of short circuiting influent to effluent is minor.

Irvine, the doyen of current investigations of batch process, has suggested a uniform terminology and united the batch concept with modern control strategies (11).

BATCH TREATMENT CONCEPTS

Batch Compared With Continuous Flow

Most simply stated, the conceptual difference between batch and continuous flow is that continuous flow processes have spacially related unit operations, whereas unit operations are timed sequentially in batch processes.

The generic batch process can have several modifications, such as single reactor, multiple reactors in series or parallel, and sequencing reactors, all with variable liquid levels. The most simple of these would be a variable volume single reactor as depicted in Figure 3, which would be suitable for a rural or small industrial situation where no flow occurs for part of the day. The figure shows the different time dependent modes that would occur in a complete cycle of a 1-tank reactor. The modes are labeled according to Irvine's (11) recommendation. The percent of maximum volume and cycle time are only illustrative. For any batch system, Fill and Draw must occur in each cycle, but React, Settle and Idle could be eliminated depending on the objective of the treatment. Alternatively, other modes could be inserted by varying time and operational controls within a cycle. Systems of this nature have a great deal of flexibility. Table 2 compares batch and continuous flow processes. In several instances drastic differences can be noted and their impact on design, operation and effluent quality are apparent.

Since the discharge is periodic, the possibility exists, within constraints, to hold effluent until some predetermined specific residual is obtained.

The cyclic organic and hydraulic loading coupled with inherent equalization allow control to be exerted over substrate tension in the reactor by suitably matching oxygen supply.

No recycle pumping is required since the mixed liquor is always in the reactor, thereby saving on energy. All the active microbial mass is available and not shunted through an inactive period in a clarifier.

Liquid solids separation occurs under near ideal quiescent conditions. During the settling period there is no hydraulic motion due to inflow, outflow or recycle, as in continuous flow systems. The effluent exit piping must be sized larger than influent piping because flow accumulated over a long time span is discharged in a much shorter time span.

Sequencing Batch Reactors

In situations where the influent flow is continuous, two or more tanks operated in sequence would be needed to treat the flow. A two-tank sequencing batch reactor (SBR) schematic is shown in Figure 4. As one tank progresses from the FILL to the REACT mode, flow is switched to the second tank,

SINGLE TANK SBR

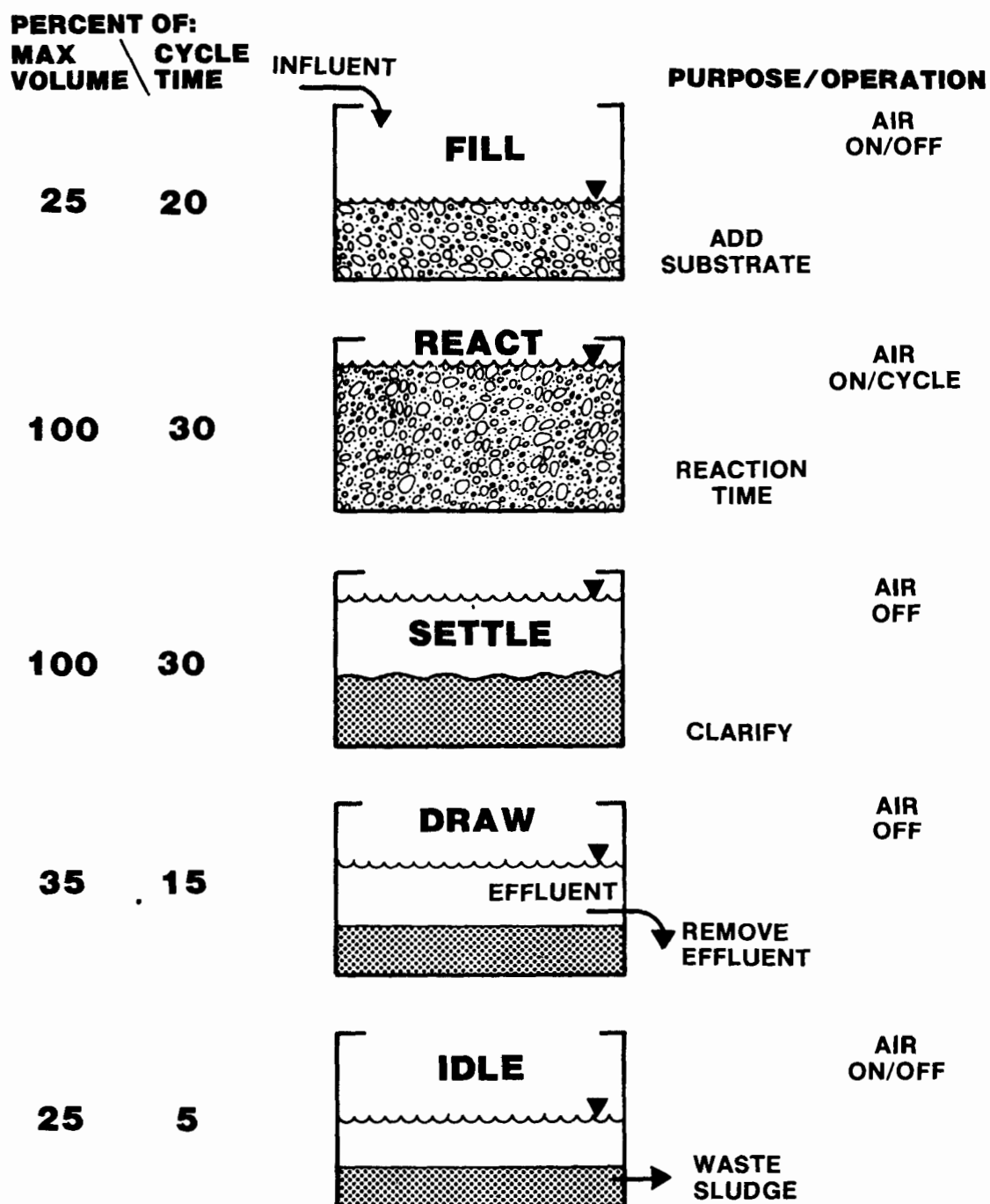


Figure 3. Single Tank Batch Reactor

PLAN VIEW OF SBR

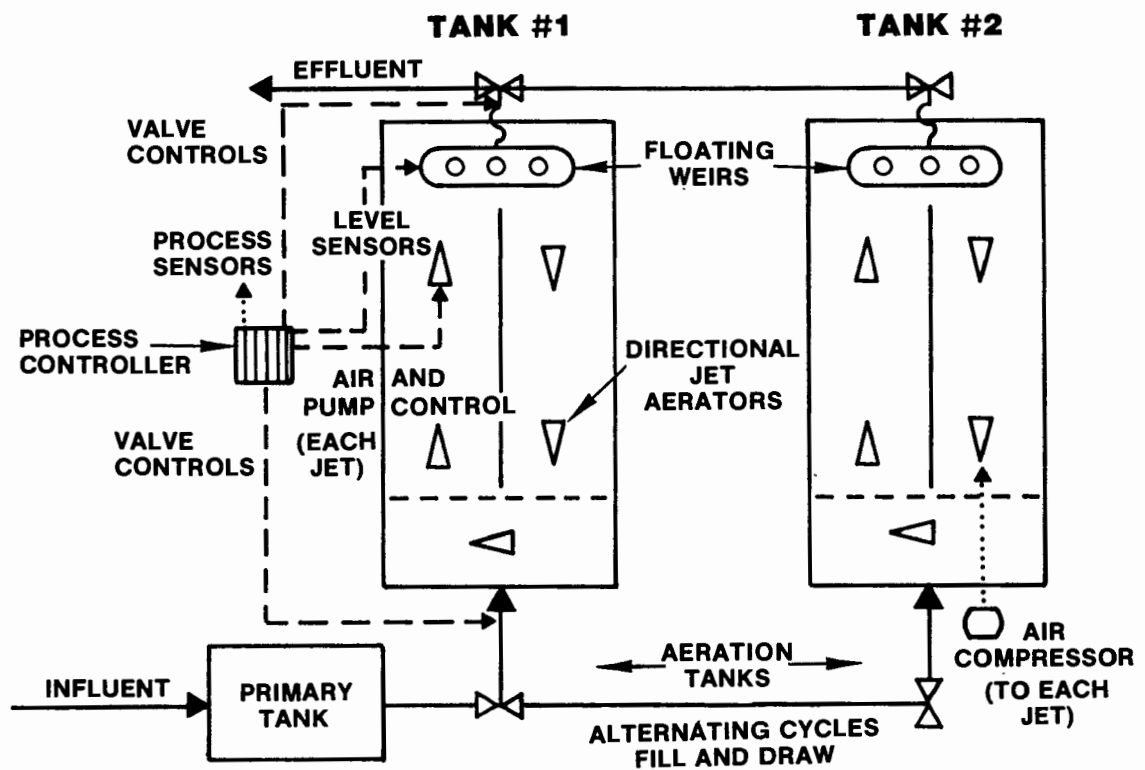


Figure 4. Sequencing Batch Reactors

Table 2. Comparison of Batch and Continuous Processes

Parameter	Batch	Continuous
Concept	Time Sequence	Spacial Sequence
Inflow	Periodic	Continuous
Discharge	Periodic	Continuous
Organic Load	Cyclic	Even (by convention)
Hydraulic Load	Cyclic	Even (by convention)
Aeration	Intermittent	Continuous
Mixed Liquor	Always in Reactor, no Recycle	Recycles through Reactor and Clarifier
Clarification	Quiescent Hydraulics	Hydraulic Motion
Flow Pattern	Perfect Plug	Complete Mix or Approaching Plug
Equalization	Inherent	None
Flexibility	Considerable	Limited
Hydraulic Sizing	Variable	Uniform

which starts a new cycle in the FILL mode. When capital costs and operational controls are considered, Irvine and Richter (12) indicate a three-tank system is near optimum for flows above 3,785 m³d (1 mgd).

Control of Substrate Tension

SBR's are dynamic processes and a great deal of selective control can be implemented. Figure 5 shows the time profile of soluble organic carbon, in an idealized reactor, with three different schemes for aeration during the FILL mode. A great deal of control over the substrate tension that the activated sludge is exposed to is possible. Ketchum (13) has discussed this attribute in relation to bulking sludge. High substrate tension during part of a cycle places filamentous organisms at a competitive disadvantage compared with floc forming organisms. Apparently, filamentous organisms cannot store substrate during anoxic periods but floc forming organisms can; therefore, at commencement of aeration, these latter organisms have a growth advantage. Goronszy (14) has shown a direct correlation between anoxic and aerobic periods with sludge volume index (SVI) for domestic wastewater. Increase in aeration time produced higher SVI's.

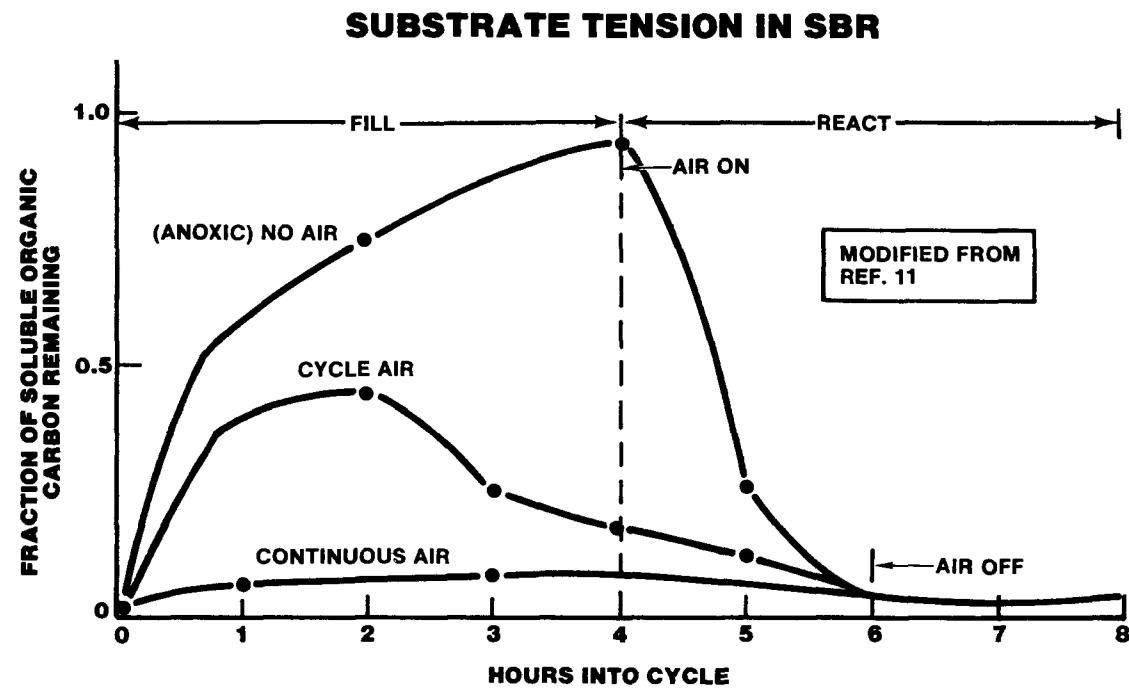


Figure 5. Time Profile of Soluble Organic Carbon

Figure 5 also illustrates other facets of SBR technology. With continuous air supply during both FILL and REACT, the soluble substrate is always at a low concentration, very similar to a complete mixed activated sludge process. With no air supply during FILL and air during REACT, the process conditions resemble plug flow. Air supplied in a cyclic fashion is intermediate between these two process regimes. If the influent waste contained appreciable nitrates, the anoxic FILL could be used to force denitrification and consequently reduce aeration requirements. If a waste required some degree of anaerobic conversion before aerobic biodegradation could occur, provision could be made for this by selection of the most suitable aeration scheme during FILL.

Mixed Liquor and Oxygenation Requirements

The concentration of activated sludge mixed liquor also changes in a dynamic fashion as shown in Figure 6. Since the mixed liquor never leaves the reactor, its concentration varies with the mode of the cycle. During FILL the initial concentration becomes diluted by the incoming waste volume, a slight increase due to synthesis would be noted during REACT, pronounced thickening occurs during SETTLE with slight densification during IDLE. Sludge wasting could occur during any mode but IDLE would usually be the most appropriate. Other modes and aeration schemes could change the generalized pattern shown in Figure 6. For instance, prolonged aeration during FILL, REACT or IDLE could show an influence from endogenous respiration (15).

Aeration requirements also change with time in an SBR reactor. Figure 7 shows oxygen uptake rates in the reactor liquid and specific oxygen uptake rates of the activated sludge during FILL and REACT modes with municipal strength wastewater. Initially from a concentration standpoint, there is a high oxygen demand due to the high mixed liquor solids content, as noted in Figure 6. Reduction of this concentration demand is a function of dilution by incoming flow, substrate concentration and the oxidative activity of the activated sludge. A stable demand is reached near the end of the REACT mode. The specific oxygen uptake varies from an endogenous level at the end of REACT to a moderate metabolic rate near the end of FILL. Other aeration schemes or mode sequences would alter the shape of these curves.

Flexibility of Modes Within a Cycle

The flexibility of the SBR concept can be noted by considering alternative modes within a cycle for the purpose of denitrification, as shown in Figure 8. A jet pump, which can serve as an aerator or mixer, or both, is installed in the reactor. The figure shows how the different modes would be controlled to achieve the sequence of events necessary to accomplish carbonaceous removal, nitrification, denitrification and liquid-solids separation. Refinements within a cycle can be considered, such as addition of wastewater during ANOXIC to increase denitrification rate, or a short aeration period after ANOXIC to provide dissolved oxygen and remove nitrogen bubbles adhering to the activated sludge before SETTLE.

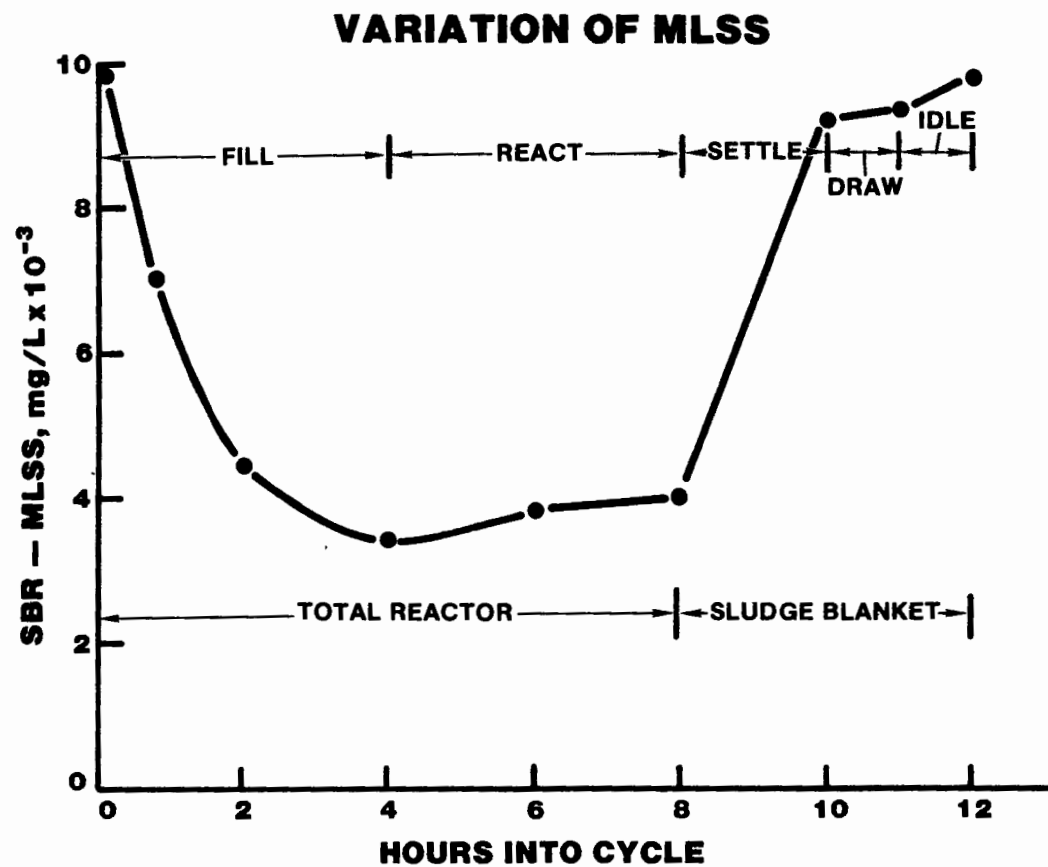


Figure 6. Time Profile of Mixed Liquor Suspended Solids

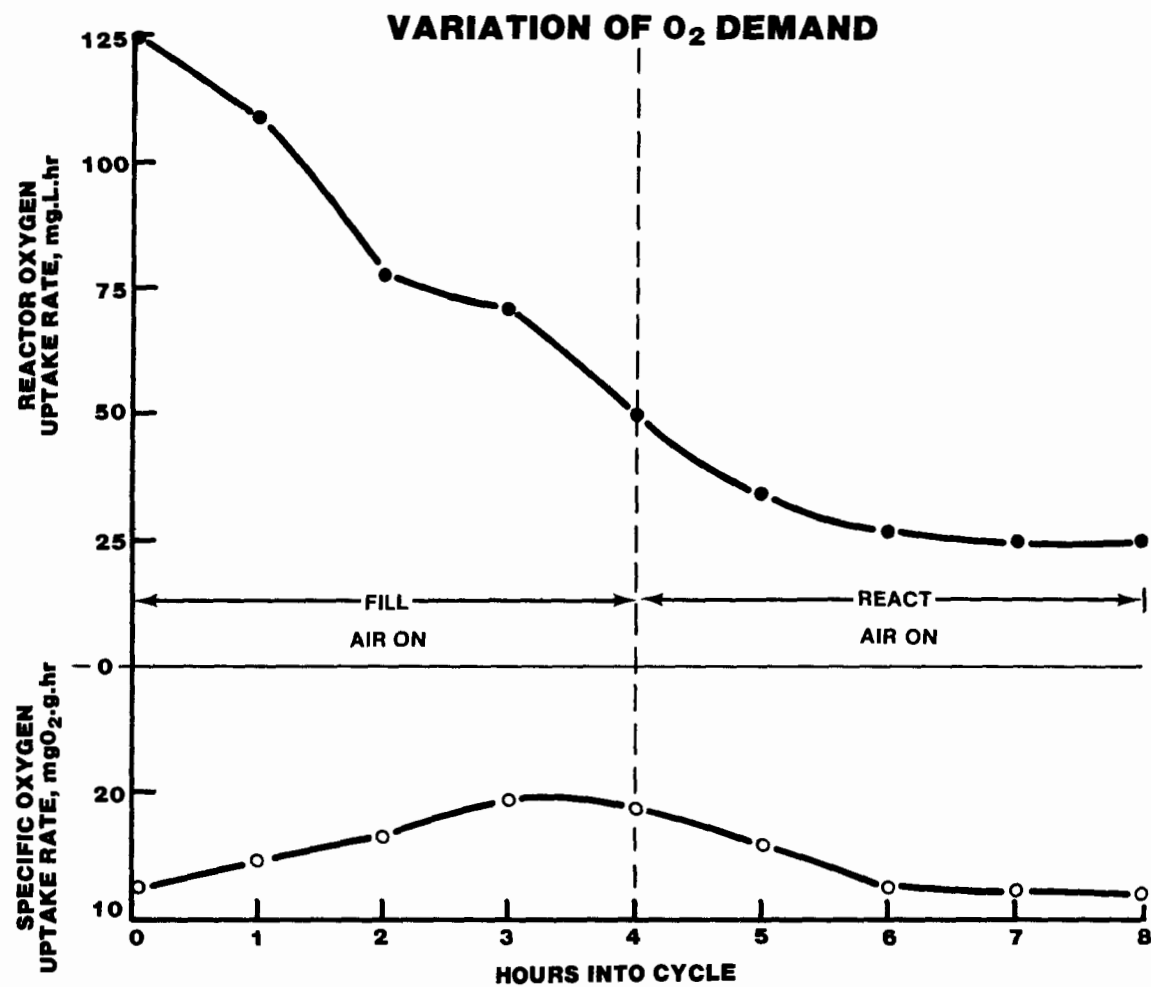


Figure 7. Oxygen Uptake Rates in Batch Reactor

NITRIFICATION/DENITRIFICATION IN SBR

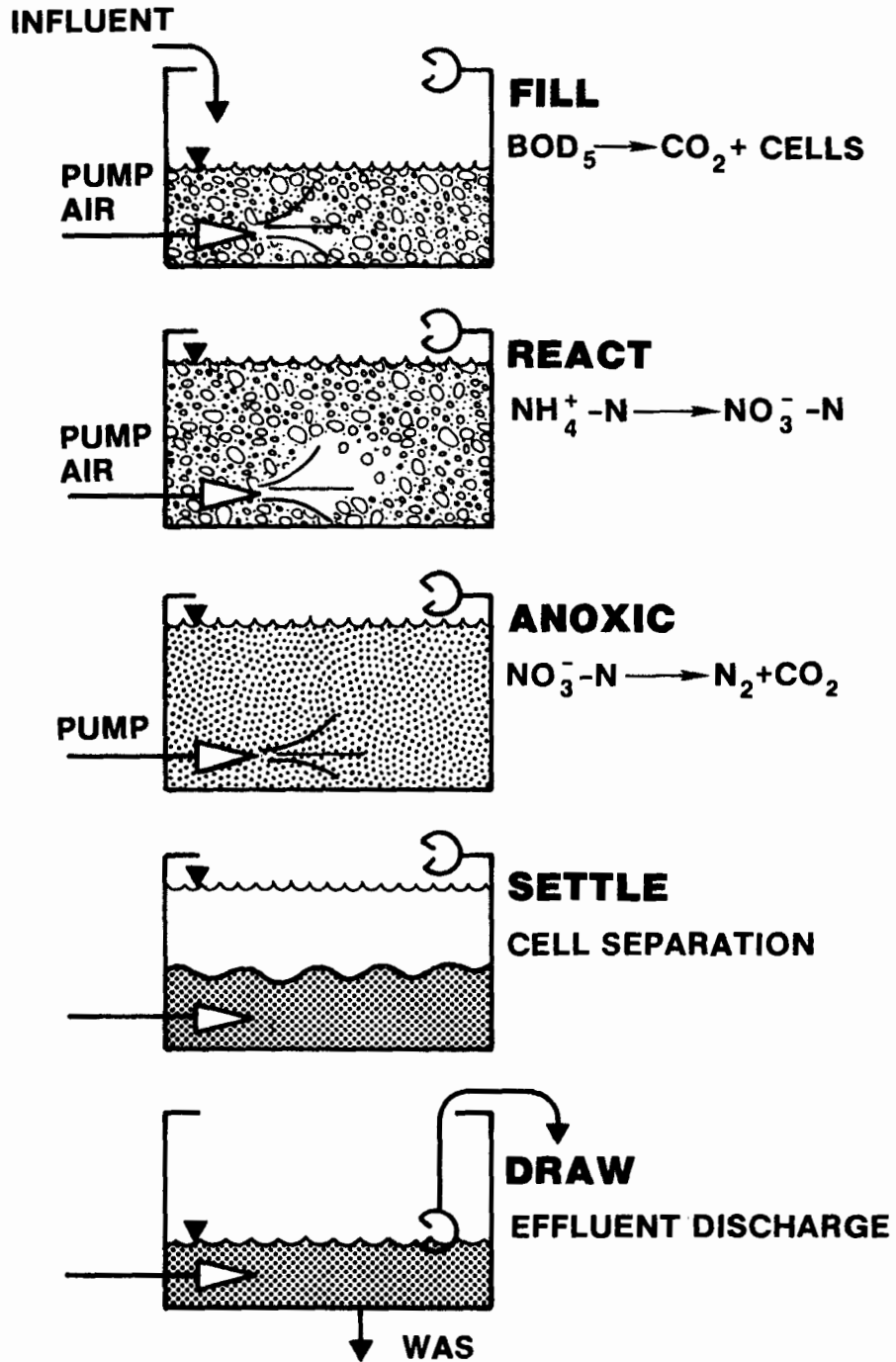


Figure 8. Denitrification Cycle for SBR

Sizing SBR Processes

Ketchum, et al. (16), Irvine and Richter (12), and Irvine, et al. (17), provide design equations for sizing sequencing batch systems. These calculations show that sequencing batch systems can be designed with considerably less total volumetric requirements than continuous flow processes due to inherent equalization, plug flow hydraulics and elimination of external clarifiers. Designs can be based on constant influent flow, constant substrate concentration or variable flow, variable concentration, with these parameters either in or out of phase.

OPERATION AND CONTROL OF SEQUENCING BATCH REACTORS

Sequence Control

A three-tank SBR system could be operated under the regime given in Table 3. In a 24-hour period, Tank #1 would complete two full cycles, with Tanks #2 and #3 each completing one full cycle. Effluent would be discharged from one or the other tanks each four hours, and at any one instant, one tank would be in the FILL mode.

Table 3. Sequence of Events in a Three Tank System

Progressive Hours	Tank Number		
	1	2	3
1			Settle
2	{	{	{
3			
4			
5			
6	{	{	{
7			
8			
9			
10	{	{	{
11			
12			
13			
14	{	{	{
15			
16			
17			
18	{	{	{
19			
20			
21			
22	{	{	{
23			
24			

Control of the mode sequencing could be managed by several routes, depending on site conditions or effluent requirements. Time clock-solenoids, set by empirical observations, would be the most simple way to control in a situation where flow is relatively constant. Varying flow situations could be managed by a flow totalizer on the influent or liquid level switches in the tanks to initiate valving changes. Large variations in substrate could be handled with in-tank, non-specific probes, such as dissolved oxygen, or more specific, off-line open-loop control sensors, such as soluble organic carbon.

Aeration Management

Any of the above can be mated together for specific application. Added to this rather complex control strategy is the need to switch aeration from tank to tank or within a cycle. These management functions can be very conveniently programed into microprocessor or process controller units. In any programed operation a SCRAM function is at the top of program hierarchy to oversee total plant control and warn operations staff of out-of-limits events (18).

Since SBR processes characteristically operate with varying liquid levels, aeration efficiency can be reduced due to poor oxygen transfer at reduced depths. As the discussion of Figure 5 indicated, aeration can be started at the beginning of FILL, at some point during FILL, or at the beginning of REACT. The biological reaction rate can be controlled by the oxygen supply rate, but in final analysis, enough oxygen must be transferred to satisfy the influent mass load of oxygen demand. Ketchum (19) has discussed a step supply approach using two aeration systems of different capacity. While a single aeration system of constant capacity would be the most simple to control, the stepped approach would be most economical with respect to capital and operating costs. In a step air supply design, the larger unit would be utilized only to meet the large oxygen demand occurring near the middle of FILL, which is a resultant of the high sludge concentration and increasing organic content (Figures 5 and 7). The lesser capacity aeration unit would commence at the beginning of REACT.

Liquid-Solids Separation

Australian (10) experience has shown that sludge bulking is not as critical to performance of batch systems as it is to continuous flow systems. The increased surface area and quiescent conditions, compared with a continuous flow clarifier, allows sludge of significantly poorer settleability to be accommodated. Mixed liquor concentration is much easier to control in batch systems because during SETTLE, there is less solids flux and the reactor sludge concentration during FILL or REACT does not depend on a recycle flow.

Irvine (20) has commented on how this feature of batch process could improve municipal compliance and provide appropriate technology for developing nations.

Variable water levels during the various modes of a cycle in SBR's require special attention to decant devices which are activated during DRAW. These can range in form from a submerged outlet pipe with automated valves, weir troughs connected to flexible couplings, floating weirs, movable baffles, tilting weirs or floating submersible pumps. The two common features of all these approaches are provision for scum control and insuring effluent take-off is uniformly distributed across the tank. This latter provision is necessary since the DRAW mode is the peak hydraulic flow within the cycle and short circuiting could cause uncontrolled suspended solids loss.

APPLICATION OF BATCH TREATMENT TECHNOLOGY

Commercial Wastewater

Witherow, et al. (21) has described the conversion of an anaerobic lagoon, of earthen construction, into an extended aeration SBR for treatment of meat packing wastewater. Hydraulic flow was about $75 \text{ m}^3/\text{d}$ (20,000 gpd) and occurred in a single 8 hour period during the day, for 6 days each week. Conversion was accomplished by installing a floating aerator and submerged automated outlet valve, both controlled by sequence timers. Sludge wasting was done manually.

The cycle employed was:

Aerated FILL and REACT	18 hours
SETTLE	2 hours
DRAW	4 hours

The process conditions were:

F/M	0.06 kg BOD ₅ /kg MLVSS/d
Detention time	10 days
SRT	64 days
MLSS	3,350 mg/l
SVI	217 ml/g

The efficiency was evaluated for one year. The yearly average values for influent and effluent are given in Table 4. The average data indicates efficient pollutant removal with the exception of rather high total suspended solids (TSS) in the effluent. These average data are not indicative of actual performance since the TSS values were near 200 mg/l during the two month startup period, but declined to about 25 mg/l afterward.

The long hydraulic detention time and high sludge age, coupled with periodic addition of high strength organic material yielded good nitrification and subsequent denitrification.

Table 4. Batch Treatment of Meat Packing Wastewater

Item	Influent, mg/l	Effluent, mg/l	Removal, Percent
BOD ₅	714	17	98
COD	1,630	121	93
TSS	535	65	88
NH ₄ -N	13	2	95
NO ₂ +NO ₃ -N	0.4	3	-
TKN	79	8	90
TP	11	3	72

Small Community Wastewater

Goronszy (22) has reported on sequentially aerated, discontinuous discharge single tank processes that have minimal operator attention. Nitrification and denitrification are encouraged in these applications to protect water quality and for utilization of the hydroxyl ion, produced by the denitrification reaction, to buffer low alkalinity wastewaters.

These processes are sequenced by a variable time selector process controller. Typically, to achieve nitrification and denitrification, a facility for 4,000 inhabitants would have the following cycle:

Aerated nitrification period	4.5 hours
Anoxic denitrification period (mixed)	3.0 hours
Quiescent settling period	2.5 hours
Decant period	0.5 hours

Process conditions are:

F/M	0.05 kg BOD ₅ /kg MLSS/d
Detention time	1.7 days
MLSS	5,000 mg/l
SVI	140 ml/g

The effluent quality from a facility of this type is shown in Figure 9.

Estimates of the relative cost of the various components of a sequentially aerated continuous flow system are estimated by Goronszy (21) to be as follows:

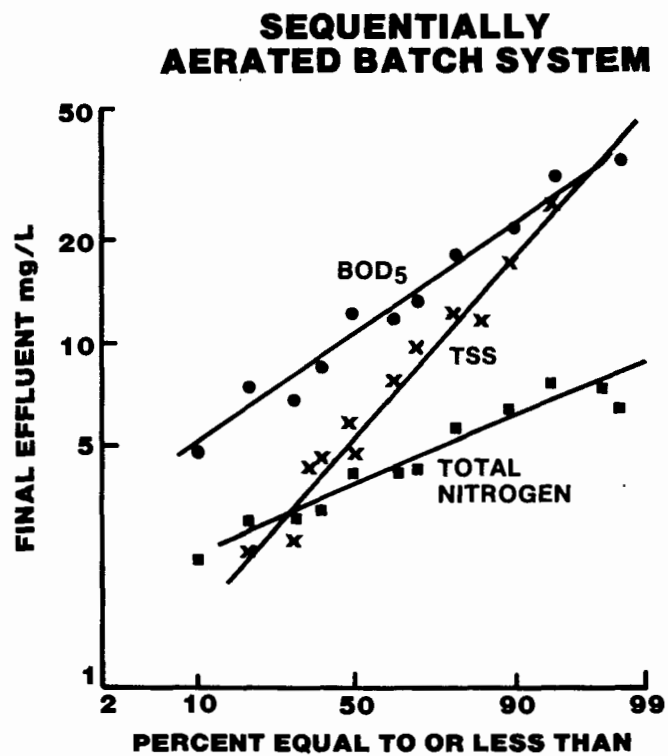


Figure 9. Effluent Quality of Sequentially Aerated, Discontinuous Discharge Batch Reactor

<u>Component</u>	<u>Percent of Total</u>
Aeration tank	53
Aeration and mixing system	32
Decant and control system	15

Rural SBR Systems

Individual onsite SBR's with effluent dispersion systems have been constructed in the Washington, D.C. area since 1975 (23). An ordinary concrete septic tank, or other suitable tank, connected to a dwelling by gravity piping, is fitted with air diffusers. A timer controlled panel board sequences a pump, blower, level control and alarm sensor. The usual cycle is 20 hours of aeration and mixing, as daily household activities fill the tank. At approximately 2:00 a.m. (0200 hr), the blower stops and the contents settle for 3 hours. The pump then directs the supernatant to the disposal system, such as a mound, trench or evapotranspiration bed. Since effluent is only pumped once per day, the effluent disposal system has a wet and drain cycle which allows aerobic conditions to prevail.

This type batch process has also been applied to institutional and commercial establishments. Several proprietary firms, such as Flygt Corp. (Norwalk, Conn.) and Environment/One Corp. (Schenectady, N.Y.), have marketed small batch systems. A typical unit, marketed by Eastern Environmental Controls, Inc. (Chestertown, Md.) is shown in Figure 10.

Municipal Application of SBR's

There is a reluctance to installation of batch processes at municipal sites due to the lack of contemporary experience for review by consultants and state regulatory offices. As noted in the Introduction, this approach to wastewater treatment can be considered archaic in view of the 22,000 continuous flow municipal facilities now installed in the United States.

Recognition of the virtues of SBR's and the emergence of reliable process control hardware lead the USEPA to fund a development project, conducted by The University of Notre Dame, to re-evaluate municipal batch treatment. The project involved the conversion of the existing 1,500 m³/d (0.4 mgd) Culver, Indiana, continuous flow activated sludge facility into a two-tank SBR. The new flow scheme is the same as shown in Figure 4.

Primary effluent is directed to each tank on an alternate basis as dictated by liquid level sensors in each tank. Pneumatic compression pinch valves control tank switching. Aeration and mixing are provided by directional jet aerators. Effluent is decanted during DRAW by submersible pumps attached to flotation devices which are swivel mounted to allow changes in elevation of the pump suction. All these functions are programmed into a process controller which operates and monitors all aspects of treatment.

The controller programs for both Tank-1 and Tank-2 operate independently in order to have greater mode flexibility within a cycle. However, a Main

Key:

- | | |
|-----------------|------------------|
| 1 Control panel | 7 Influent line |
| 2 Air intake | 8 Alarm sensor |
| 3 Air filter | 9 Pump shut-off |
| 4 Blower | 10 Pump |
| 5 Effluent line | 11 Air-diffusers |
| 6 Junction box | |

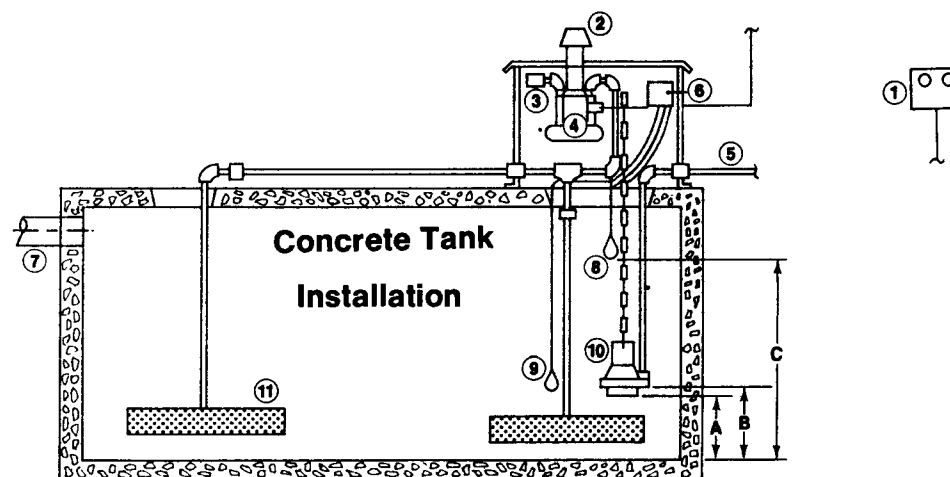


Figure 10. Rural Aerobic Batch Unit

program monitors the Tank-1 and Tank-2 programs, on a time-sharing basis, to insure correct sequencing. A SCRAM program overrides all three programs to warn of unprogramed events or emergency valving changes.

The initial evaluation at the Culver facility used an anoxic period for part of the FILL mode and this was programed into the controller as FILL-1 and FILL-2. During FILL-1 only the mixing action of the jet pumps was activated. During FILL-2 the jet pumps mixed and aerated simultaneously. A typical cycle for the two tanks was as follows:

	Hours in Mode					
	FILL-1	FILL-2	REACT	SETTLE	DRAW	IDLE
Tank-1	1.1	1.8	0.8	0.8	0.7	0.8
Tank-2	1.2	1.9	0.6	0.8	0.6	0.9

The difference in times between the modes of the two tanks is due to diurnal flow variation, since the system is controlled by liquid level sensors. At the design flow of 1,500 m³/d (0.4 mgd) each cycle is completed in 6 hours and both tanks complete 4 cycles each day.

The process conditions at Culver are:

F/M	0.2 kg BOD ₅ /kg MLSS/d
SRT	20 days
MLSS	2,200 mg/l (70 percent volatile)
SVI	110 ml/g

One year of data and operational experience has been documented for BOD₅ and SS control. Table 5 provides monthly average wastewater and SBR effluent constituents for the month of March 1981. The City of Culver has an effluent phosphorus limitation of 1 mg/l. To achieve removal of phosphorus, ferric chloride is dosed directly into the primary effluent channel leading to Tank-1 and Tank-2.

Very high efficiency for BOD₅, SS and TP is achieved at Culver with the SBR process. Nitrate nitrogen of about 2 to 5 mg/l is usually present in the Culver raw wastewater due to run-off from fertilized farm land infiltrating sections of the collection system. Table 5 shows that this influent nitrate is denitrified during the anoxic FILL-1 mode of the SBR.

Process conditions to achieve nitrification and denitrification have recently been implemented. The anoxic period for FILL-1 has been reduced to provide a longer period of dissolved oxygen supply for nitrification. The efficiency of the SBR process during the month of August 1981 is given in Table 6.

Nitrification and denitrification occur almost simultaneously, as evidenced by the fact that neither ammonium or nitrate nitrogen reach high concentrations in the reactors. Apparently as ammonium nitrogen is trans-

formed to nitrate, during the low load period at the start of FILL, there is subsequent denitrification due to the high mixed liquor concentration and differential D.O. content of the reactor. When mixed liquor is pumped through the aerated jet nozzle a high D.O., suitable for nitrification is present; then as the mixed liquor circulates as a bulk liquid localized low D.O. conditions can occur.

Once nitrification is established in the Culver SBR ammonium nitrogen concentration in the reactor can never reach influent levels due to the dilution of influent by the residual reactor contents left after IDLE, and the biological transformation to nitrate. Accordingly, the nitrate concentration cannot reach high levels, and even a low kinetic rate for denitrification would yield fairly efficient nitrogen removal.

During the remainder of the demonstration a more detailed evaluation of nitrogen removal capabilities of the SBR will be carried out.

Table 5. Constituents of Culver, Indiana Process Streams

Location	Milligrams per Liter				
	BOD ₅	SS	TP	NH ₄ -N	Oxidized Nitrogen
Raw Wastewater	173	136	6.3	20	2.8
Primary Effluent	132	81	5.2	19	2.6
SBR Tank-1	8	7	0.4	19	0.4
SBR Tank-2	9	9	0.4	18	0.4
Overall Removal, Percent	95	94	94	5	86

Table 6. Efficiency of Culver SBR for Nitrification and Denitrification

Location	Milligrams per Liter				
	BOD ₅	SS	TP	NH ₄ -N	Oxidized Nitrogen
Raw Wastewater	118	133	5.8	16.5	1.7
Primary Effluent	92	64	4.8	14.0	1.7
SBR Tank-1	6	3	0.9	0.3	1.7
SBR Tank-2	6	3	1.0	0.4	1.2
Overall Removal, Percent	95	98	84	98	-

Several new municipal facilities, at about the 3,785 m³/d scale, are being constructed under the Innovative/Alternative section of the USEPA Construction Grants Program.

FUTURE DEVELOPMENT OF SBR TECHNOLOGY

1. All prior SBR technology has been with systems that are basically low-rate, long SRT operations. An effort should be to evaluate this technology under high hydraulic and organic loadings to obtain a wider band of design data for BOD₅ and SS removal. This is necessary to fully evaluate the position of SBR technology in the municipal area.
2. Theoretically, any internally staged process for biological phosphorus removal, such as Air Product's Anaerobic/Oxic or the Bardenpho process of Envirotech, Inc. could be designed as an SBR. Bench-scale studies have shown feasibility of this approach (24). Design would be much simpler since only overall tankage capacity needs to be known. Modification of cycle times could provide most efficient treatment as operation was optimized.
3. Treatment plant internal recycle streams are relatively small in volume but high in strength. Treatment of these streams in an off-line SBR process could upgrade the effluent quality of many facilities. This same situation relates to leachates from landfill operations which produce a low volume, high strength discharge. Since the character of leachate changes with age of the landfill modification of the modes in a cycle could account for changing strength or quality.
4. Use of SBR's to serve as an equalization process would utilize a cycle that had only FILL and DRAW modes. An SBR could serve to equalize both flow and organic loading prior to a continuous flow process. Combined sewer overflow is an obvious candidate for application.
5. SBR's can also be envisioned as tertiary processes. Bench-scale work indicates that lower residuals of phosphorus can be achieved with the SBR approach compared with continuous flow systems (25).

Due to more efficient contact and high utilization of precipitant, less chemical was required and consequently less sludge was produced.

The possibility of monitoring the effluent for a specific component's residual before discharge is an attractive feature of this approach.

6. Presently SBR technology utilizes electro-pneumatic-mechanical valving for flow control or flow switching. With microprocessor control, these functions may be amenable to fluidic devices that contain no moving components.

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ANAEROBIC TREATMENT OF MUNICIPAL WASTEWATER

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ABSTRACT

Application of anaerobic biological processes to the treatment of dilute wastes at ambient temperature appears economically feasible through the application of the fluidized bed reactor concept. When applied to municipal wastewater, cost savings of approximately twenty percent appear possible because of the significant reduction in secondary sludge levels and lower power requirements. Preliminary data from a small pilot plant is presented here. Excellent suspended solids removal has been achieved but organic removal has not met secondary standards.

INTRODUCTION

It has been realized for many years that anaerobic biological treatment systems have significant advantages over aerobic systems. These include the low yield of biological sludge per unit of organics stabilized, the production of methane (an energy rich end product), and lower operational power requirements (no oxygen supply is required). Until recently these have been manifest as economic advantages only in situations where the waste contained high concentration of organic material. The primary reason has been the exclusive use of the complete mix-suspended growth - no recycle reactor concept. In order to achieve high efficiency with such reactors, large reactor volumes are required, as well as operation at elevated temperatures. The large reactor volume insures a high biological solids retention time (SRT) when no recycle is used, since a long hydraulic detention time is also provided.

Recently, film flow reactors have been successfully applied to anaerobic treatment of wastes of moderate strength at room temperature (1, 2). In such systems, the waste is passed through a reactor packed with a stationary inert medium. The microorganisms attach to the packing and are retained in the unit thus providing for very long SRT values. These units, however, do not appear suitable for dilute wastewater at ambient temperature, i.e., domestic sewage.

Various studies with municipal sewage have yielded effluents which do not meet secondary effluent standards at detention times below 12 to 24 hours (3, 4, 5, 6, 7, 8). In these reactors, solids removal appears to be the major mechanism operative when treating municipal wastewater.

The problem with the fixed film reactor type referred to above may be that the spaces in the packing have to be so large to prevent plugging that good contact between the microorganisms and the soluble organics in the bulk liquid cannot be obtained in a short detention time. A reactor type which obviates this problem is the fluidized bed or expanded bed.* In this system (Figure 1) flow is upward through a packing of small granular material such as: filter sand, granular activated carbon, diatomaceous earth, etc., at a rate sufficient to insure fluidization. Generally, effluent recycle is used to achieve a high enough total flow rate to maintain fluidization under all conditions. In this system, as biomass and solids accumulate in the bed, the degree of expansion automatically changes preventing plugging. However, effective mass transfer from the bulk solution to the microbial surfaces is maintained because the distance between the surfaces of adjacent media particles is quite small, and the medium is not stationary but swirls and migrates through the liquid.

The potential effectiveness of the fluidized bed system for treatment of dilute wastes versus the stationary packed bed reactor has been demonstrated in both aerobic and anoxic systems (9). Successful BOD removal and denitrification have been achieved in fluidized bed systems at detention times below 30 minutes. Recent data to be reviewed below have given promise that anaerobic treatment of municipal wastewater in fluidized or expanded bed reactors can produce an effluent which meets secondary standards at detention times competitive with activated sludge systems.

PRELIMINARY STUDIES ON ANAEROBIC FLUIDIZED BED TREATMENT OF MUNICIPAL WASTEWATER

Jewell (10, 11) reported on an upflow expanded bed reactor (Jewell's terminology) with the support media consisting of a mixture of PVC particles and ion exchange resin with diameters less than 1 mm. This laboratory study utilized a 1-liter reactor with 5.1 cm (2 in) I.D. After 50 days startup operation, which included seeding with anaerobic sludge, experiments with primary effluent as feed were conducted for a period of 200 days. The primary effluent was a weak domestic waste with an average effluent COD of 186 mg/l. Primary effluent was blended with recycle, with the recycle pumping rate maintained constant at about 100 ml/min. Except for some shock loading studies, the temperature was maintained at 20°C. Effluent quality was monitored by unfiltered COD and SS measurements.

During the 200-day study, the hydraulic retention time (HRT) was varied from 24 hours down to a low of 0.08 hours. For the first 95 days the HRT was 4 hours or greater, and after approximately a 20-day period of

*In this discussion, the terms fluidized and expanded beds are used interchangeably, even though there is a technical difference in these terms.

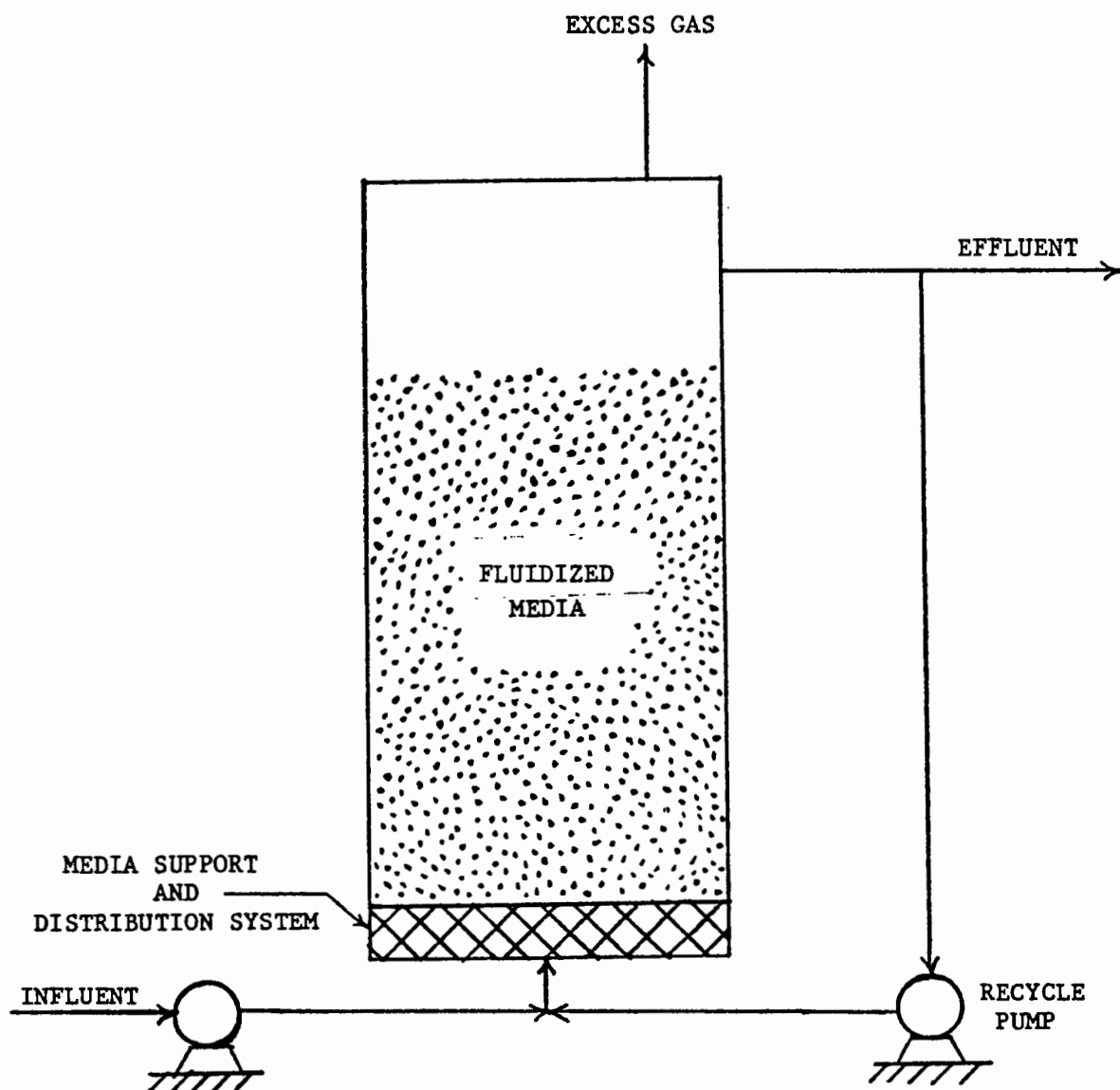


Figure 1. Schematic Diagram of a Fluidized Bed System.

operation at HRTs of 2 + 0.5 hours, the HRT was again returned to 8 hours for several weeks. During the last ten days of the study the HRT was varied from 0.25 down to 0.08 hours. The data indicated that the anaerobic expanded bed system could treat primary effluent and consistently produce a secondary effluent of excellent quality (COD of ~ 30 mg/l and SS of ~ 4 mg/l) when operating at an 8-hour HRT at 20°C. Good effluent quality was also obtained during operation at a 4-hour HRT. The data suggest that long term operation at HRTs of 1-2 hours may also be possible.

The effective biomass concentration in the reactor was reported to vary from 20 to 30 kg VSS/cu m. The cell yield was estimated at about 0.15 g VSS/g COD destroyed. If a reactor removed 150 mg/l of COD, had a 4 hour HRT, effluent VSS of 6 mg/l and a net yield of 0.15, it would take 252 days to accumulate a biomass concentration of 25 kg VSS/cu m. Alternatively, if a reactor were operating at equilibrium with a 4-hour HRT, a reactor VSS of 25 kg/cu m, effluent VSS of 6 mg/l, and no deliberate sludge wasting, the SRT would be 694 days. These calculations are intended to illustrate the long SRTs which are characteristic of the system investigated by Jewell. Although the system produced acceptable effluent quality during the brief periods of operation at 1-2 hour HRTs, it is not known what would happen over a long time period if operation were continued under these conditions. In view of the long SRT's associated with equilibrium operation under a given set of conditions, the successful operation for a few days at the high loadings does not ensure that the same effluent quality would be achieved at the new equilibrium conditions which ultimately develop.

Switzenbaum and Jewell (12) also evaluated the expanded bed concept in a laboratory study with a feed of glucose and nutrient salts. This small scale study used 5.1 cm (2 in) I.D. columns with a fluidization media of aluminum oxide particles that were approximately 0.5 mm in size. The bed was expanded from an initial volume of 400 ml to an operating volume of 500 ml. Three reactors were operated at 10, 20 and 30°C, respectively with steady state feed concentrations ranging from 200 to 600 mg/l of COD. Solids concentrations in the reactor were reported between 15,000 to 38,000 mg/l TVS. At feed concentrations of 200 and 400 mg/l, the COD removals resulting from a combination of cell synthesis and CH₄ production were as shown in Table 1.

These data show that fluidized bed systems are operable over the range of wastewater temperatures which are encountered throughout most of the United States. On the average, about 80 percent of COD removal resulted from CH₄ formation. Whether results from municipal wastewater treatment will be comparable to those obtained from glucose at the lower temperatures has not yet been ascertained.

DESIGN, ENERGY and ECONOMIC CONSIDERATIONS

Process design of an anaerobic fluidized bed system involves selection of: detention time, size and density of medium, recycle flow, expansion range, overall reactor height, degree of staging. All of these parameters are interrelated and their selection will be influenced by capital, energy, operation and maintenance costs. At present, only a limited amount of

Table 1. Soluble COD Removals Reported by Switzenbaum and Jewell (12).

HRT hours	Soluble COD Removal %					
	Feed ^{10°C} , mg/l		Feed ^{20°C} , mg/l		Feed ^{30°C} , mg/l	
	200	400	200	400	200	400
6	73	83	83	88	79	82
4	70	81	74	86	72	83
2	55	65	72	81	66	77
1	50	54	57	67	61	70

theoretical and practical information which can aid in making these selections is available. In this section a brief review of the information is given.

Fluidization

Extensive studies have been conducted on fluidization and expansion of beds of granular material. A review of most of the data has been presented by Cleasby & Brunann (13). Figure 2 illustrates the effect of particle size and specific gravity on the upflow rate required for fluidization by water at 2 different temperatures. This figure is based on the equations presented in the above review. Figure 2 is only applicable to clean particle systems. In actual practice the growth of a bacterial film on the inert particle will change its hydrodynamic characteristics. Figure 2 can be used to make a preliminary judgment on the specific gravity and particle size to be used for a given range of hydraulic retention times, bed depths and recycle ratios.

Effect of Bacterial Film

Changes in particle characteristics resulting from bacterial growth will impact fluidization characteristics of the bed. The degree of impact can be estimated by calculating changes in bed characteristics which would result from a uniform coating of bacterial growth of different thicknesses developing around spherical support media. The results of such calculations for one set of assumed parameters is shown in Table 2. A bacterial specific gravity of 1.50 (dry weight basis) with a film concentration of 0.15 gm/cu cm represents a bacterial film with an apparent specific gravity of 1.05 ($0.15 + (1. - 0.15/1.5)$). One of the most interesting observations in the study by Switzenbaum and Jewell (12) was the extremely thin bacterial film thicknesses encountered. Film thickness was estimated by viewing the particles under a light microscope with a calibrated ocular. The thicknesses ranged from a minimum of .007 to .014 mm. It was also reported that the unattached entrapped biomass comprised between 4 to 6 percent of the total biomass present. Thus a film thickness of .015 mm was used in the calculations on Table 2. The changes resulting from 0.015 mm assumed bacteria thickness in Table 2 indicate that the thin dense films reported by Switzenbaum and Jewell (12) should have very little impact on the fluidization characteristics of the bed as a whole. For example, the 15 micron film modeled in Table 2 would decrease the fluidization velocity of a 0.5 mm particle by only 0.52 m/hr (1.7 ft/hr) i.e., from 10.03 m/hr (32.9 ft/hr) to 9.51 m/hr (31.2 ft/hr).

However, for anoxic denitrification systems Jeris (14) reported that 0.65 mm activated carbon particles reached sizes of 3 to 4 mm as a result of accumulations of biomass. The influence that various film thicknesses would have on the support particles and the resulting changes in the bed characteristics in the absence of some positive mechanism to limit the particle size can be discerned from Figure 3. This Figure is also based on the same model for spherical particles that was used in Table 2, although some of the parameter estimates are different in this example. These results

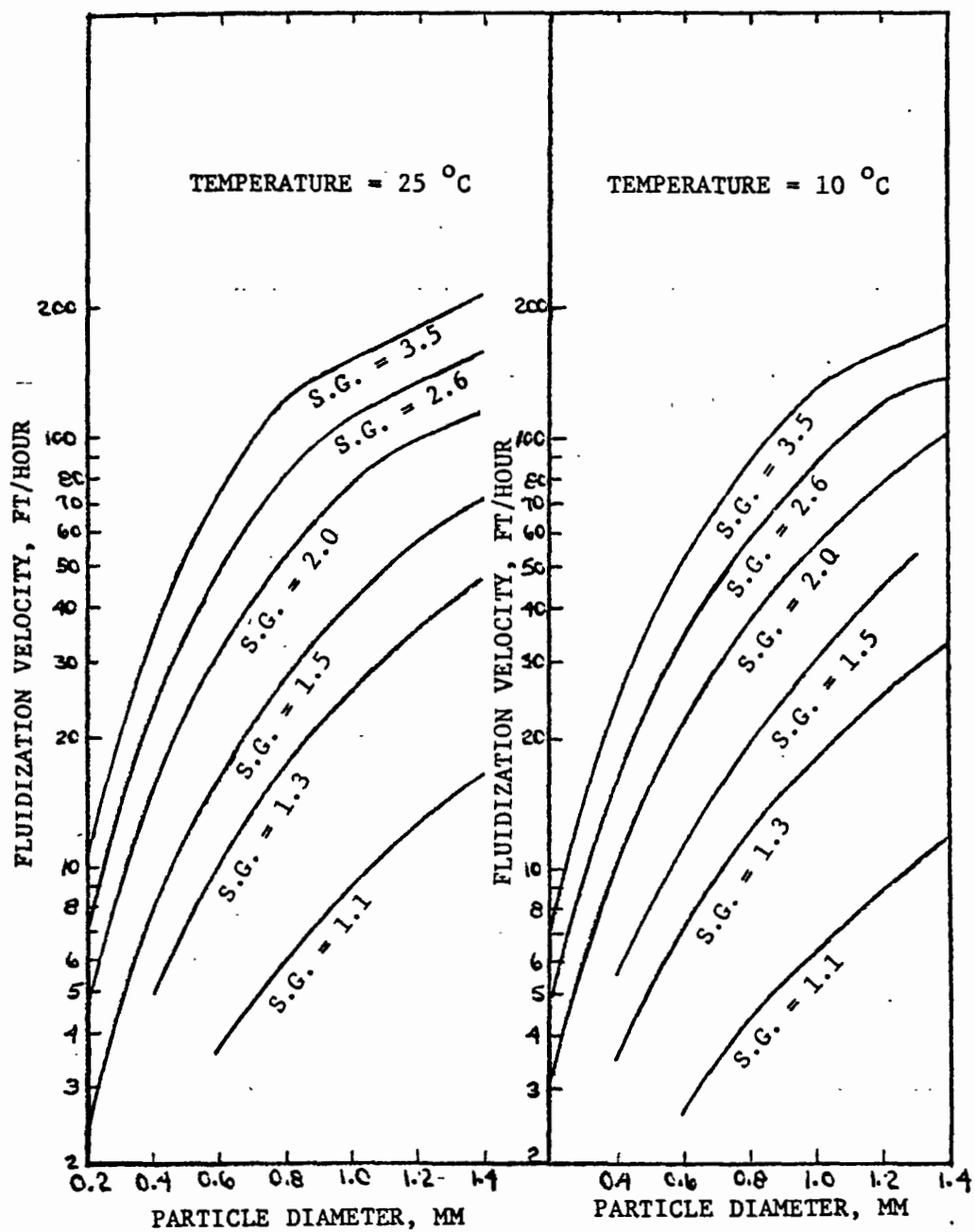


Figure 2. Fluidization Velocity vs. Particle Diameter at Different Specific Gravities.

Table 2. Fluidized Characteristics for Spherical Particles

Temperature = 20.0 Degrees C

Media Specific Gravity.....	2.650
Bacteria Specific Gravity.....	1.500
Bacteria Film Thickness, mm.....	0.015
Bacteria Film Concentration, gm/cu cm.....	0.150
Initial Porosity of Unfluidized Bed.....	0.400
Final Porosity of Unfluidized Bed.....	0.400

INITIAL VALUES:

Particle Diameter, mm	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
Particle Volume, cu mm	0.0042	0.0141	0.0335	0.0654	0.1131	0.1796	0.2681	0.3817	0.5236
Particle Mass, mg	0.0111	0.0374	0.0888	0.1734	0.2997	0.4759	0.7104	1.011	1.388
Particle Number Per Liter	1.43×10^8	4.24×10^7	1.79×10^7	9.17×10^6	5.31×10^6	3.34×10^6	2.24×10^6	1.57×10^6	1.15×10^6
Fluidization Velocity, gpm/sq ft	0.773	1.617	2.729	4.097	5.709	7.557	9.637	11.94	14.46
Fluidization Velocity, ft/hr	6.20	12.97	21.89	32.86	45.79	60.62	77.30	95.78	116.0
Reynolds Number	0.1049	0.329	0.741	1.389	2.323	3.588	5.229	7.289	9.811
Fluidization Head Loss, ft/ft	0.990	0.990	0.990	0.990	0.990	0.990	0.990	0.990	0.990

FINAL VALUES:

Particle Diameter, mm	0.230	0.33	0.43	0.53	0.63	0.73	0.83	0.93	1.03
Particle Volume, cu mm	0.00637	0.0188	0.0416	0.0779	0.1309	.2037	0.2994	0.4212	0.5721
Particle Specific Gravity	2.102	2.252	2.338	2.393	2.432	2.461	2.483	2.500	2.514
Particle Mass, mg	0.0134	0.0424	0.0973	0.1866	0.3184	0.5012	0.7433	1.053	1.438
Unexpanded Particle Number Per Liter	9.42×10^7	3.19×10^7	1.44×10^7	7.70×10^6	4.58×10^6	2.95×10^6	2.00×10^6	1.42×10^6	1.05×10^6
Bacteria in Unexpanded Bed, mg/l	3.08×10^4	2.24×10^4	1.76×10^4	1.44×10^4	1.23×10^4	1.06×10^4	9.41×10^3	8.43×10^3	7.64×10^3
Ratio of Unexpanded Bed Volumes	1.521	1.331	1.242	1.191	1.158	1.134	1.117	1.103	1.093
Fluidization Velocity, gpm/sq ft	0.6821	1.484	2.556	3.886	5.461	7.275	9.32	11.59	14.08
Fluidization Velocity, ft/hr	5.472	11.90	20.51	31.17	43.81	58.35	74.75	92.96	112.9
Reynolds Number	0.1064	0.3321	0.7456	1.397	2.334	3.602	5.246	7.31	9.836
Head Loss Per ft of Media									
Initially Present, ft	1.006	0.999	0.997	0.9957	0.9947	0.994	0.993	0.9931	0.9928
Bacteria Per Liter of Media									
Initially Present, kg	0.0469	0.0297	0.218	0.0172	0.0141	0.0121	0.0105	0.0093	0.0083

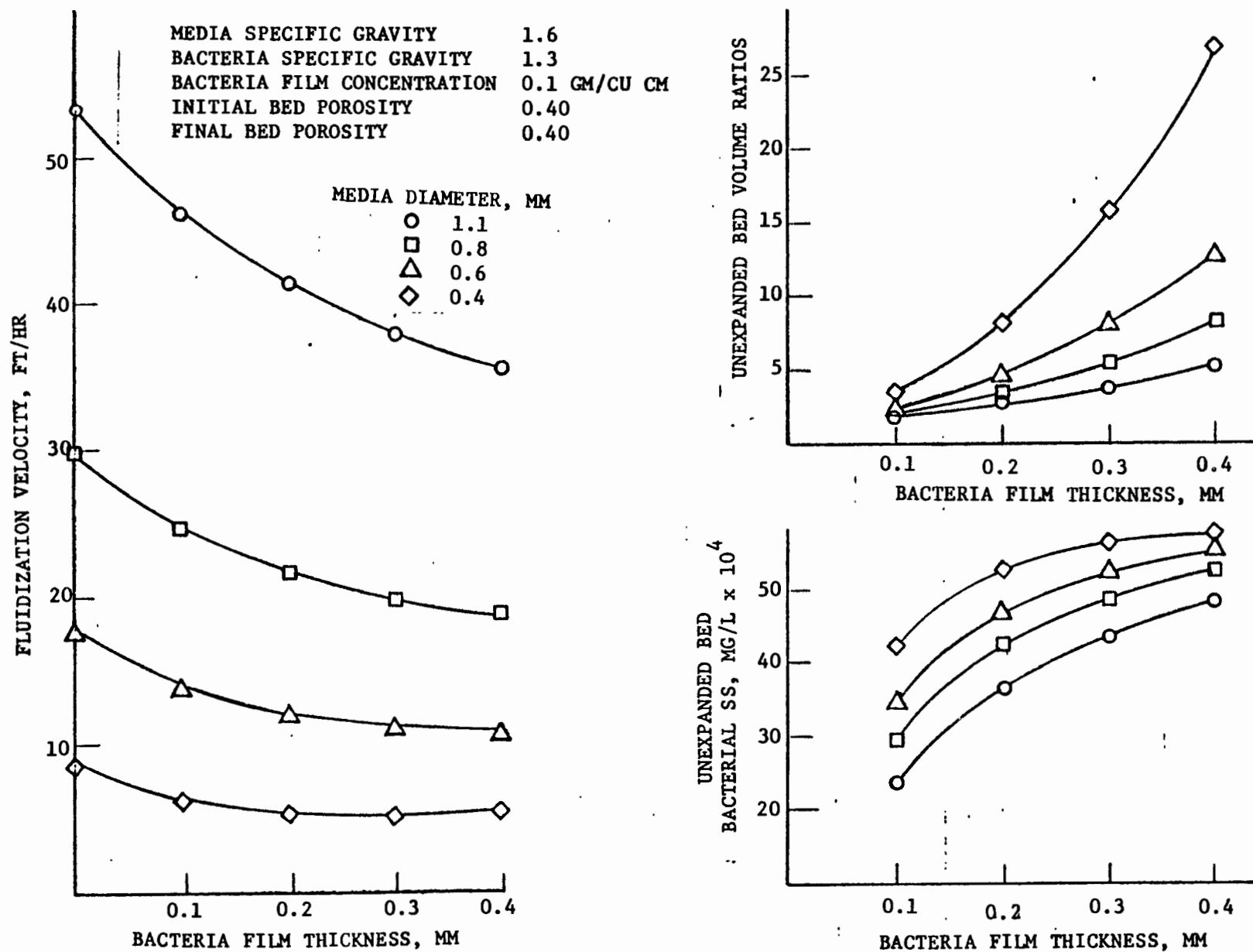


Figure 3. Influence of Bacteria Film Thickness on Fluidization Properties and Bed Characteristics.

illustrate that in designing fluid bed systems it is important to know the nature and thickness of the bacterial growth to be expected. This will influence the optimal media size and density, the amount of bed expansion observed, the need to control media-bacteria particle size, and the importance of diffusional considerations within the films in controlling the biofilm kinetics (15).

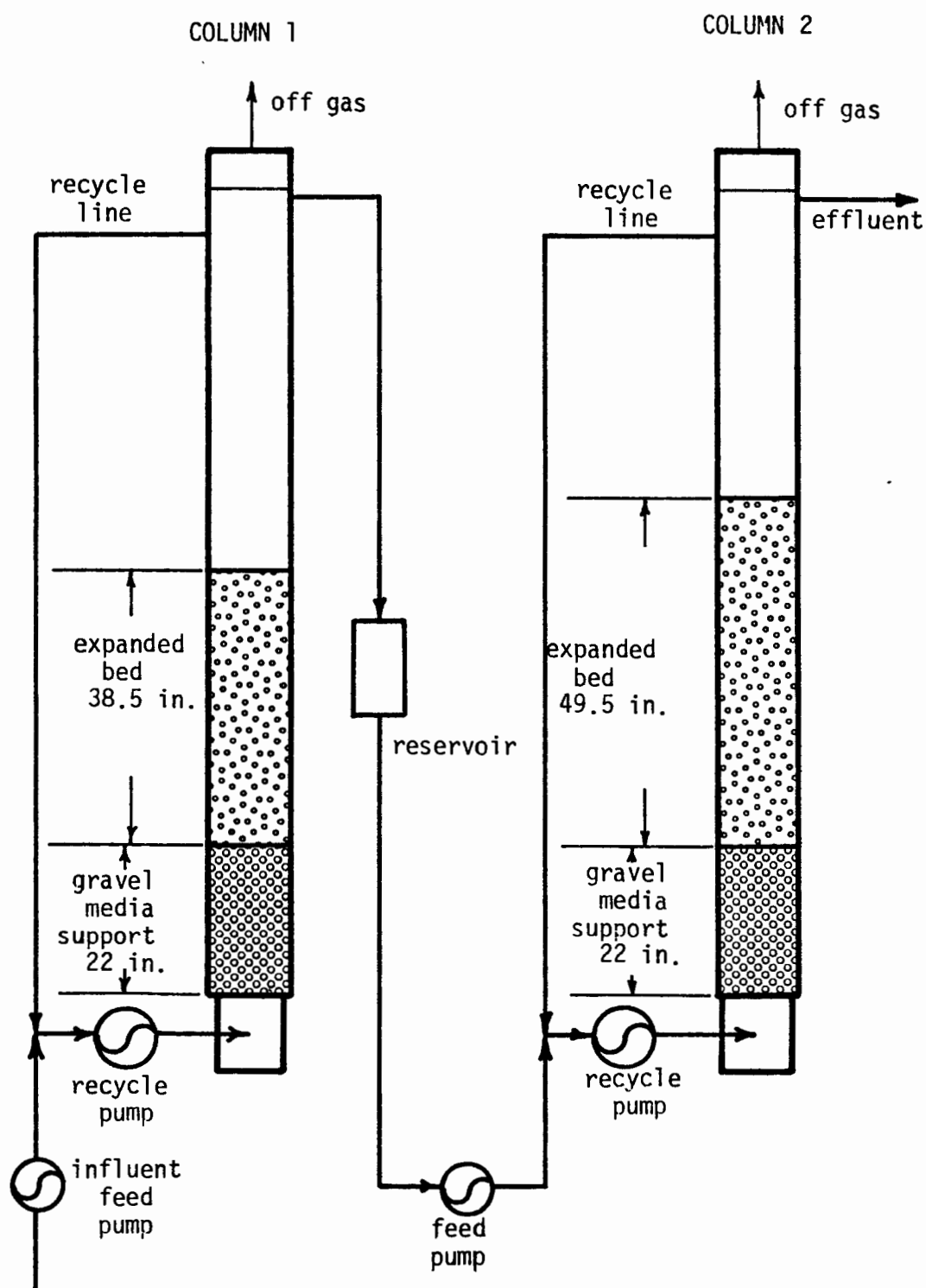
Recycle Effect on Pumping Energy

As an operational expedient, the systems studied by Jewell (10,11) and Switzenbaum and Jewell (12) used a very high recycle rate to maintain bed expansion. The recycle rate in Jewell's system was maintained at 100 ml/min which corresponds to an upward velocity of 70.4 m/day (1730 gpd/sq ft). For operation at a 4-hour HRT the recycle: influent pumping ratio was 24:1. Switzenbaum and Jewell used even higher recycle flows (211 m/day or 5200 gpd/sq. ft) for studies with the aluminum oxide media. Although the fluidized bed system will save energy compared to activated sludge because no oxygen is added to the reactor, it is clear that the energy savings could be negated through excessive pumping requirements.

The energy required to fluidize a bed of granular material can be estimated from the head loss required to fluidize the bed, which is equal to the bouyant weight of the bed particles. As an example, consider a reactor containing 3.05 m (10 ft) of silica sand of specific gravity 2.65 with a porosity of 0.40. The head loss through the bed is 3.02 m (9.9 ft). If the design called for 2-hour HRT and no recycle pumping was contemplated, the particle sizes would have to be exceedingly small (≈ 0.2 mm) to insure fluidization; as shown by the curves in Figure 4. Assuming a wire to water pumping efficiency of 65%, the energy expended to overcome the headloss through the bed (excluding the losses in the distribution system) would be

$$\frac{(3.02 \text{ m}) (9.806 \text{ newton/kg}) (1000 \text{ kg/cu m})}{(3600 \text{ sec/hr}) (1000 \text{ watt/kw}) (0.65_{\text{eff}})} = 0.0126 \text{ kwh/cu m (47.8 kwh/MG)}$$

If the proposed design called for using sand particles of approximately 1 mm size, the minimum fluidization velocity would increase to 30.5 m/hr (100 ft/hr) (Figure 4) and providing a 2-hour HRT in the 3.05 m (10 ft) bed would require that the recycle: influent pumping ratio rise to greater than 19:1 to achieve more than minimum bed expansion. In this case, the pumping requirements at an overall efficiency of 65 percent would rise to 0.252 kwh/cu m (955 kwh/MG) of wastewater treated, excluding the additional losses in the distributor system. The distributor losses will vary with the type of distribution system and flow rates chosen and will probably add an additional 0.3 to 1/2 m (1 to 4 ft) of head loss to the system. The first situation above, i.e. no recycle requires low pumping energy but could lead to instability because, with small particles, small changes in flow yield large changes in degree of expansion. There is little chance of bed instability with high recycle but pumping costs may be excessive.



NOTE: Both columns are 6 in. diameter by 120 in. tall cast acrylic cylinders. 1 in. = 2.54 cm.

Figure 4. Experimental Fluidized Bed Pilot Plant.

It can be seen that the energy requirements for fluidized beds will be determined by the HRT required, the size and specific gravity of the media selected, and the extent to which the bacterial film characteristics alter the particles behavior. For the thin films observed by Switzenbaum and Jewell, silica sand particles of around 0.3 to 0.4 mm size should produce acceptable fluidization characteristics and bacterial concentrations (Table 2), and result in a head loss of 8.5 to 13.7 m (28 to 45 ft) for a 2-hour HRT. Pumping 3785 cu m/day (1 mgd) with a head loss of 15.2 m (50 ft) requires 242 kwh/day at an overall efficiency of 65 percent, so the head loss for such a system would be reasonable.

In actual practice it is not clear which flow control strategy would be optimal for plant operation. One approach is to incorporate a flow equalization basin ahead of the reactor to insure that it receives a relatively constant hydraulic loading and a more uniform organic loading. This approach will minimize the amount of recycle pumping required. Alternatively, it may be more desirable to pass the incoming flow directly through the system and vary the recycle ratio as required for adequate bed expansion. If the ratio of maximum flow to minimum flow and maximum organic concentration to minimum concentration are synchronously varied by 3:1, the organic mass loading would vary by 9:1 during the day. The optimal combination of flow equalization, reactor size and recycle rate can only be calculated when considered in conjunction with the expansion characteristics of the media selected and the biological kinetic response of the attached growth (Percent Removal of BOD vs HRT).

Sludge Production from Anaerobic Reactors

Biological sludge production in anaerobic systems is substantially less than in aerobic systems. McCarty⁽¹⁶⁾ lists the following growth constants and endogenous respiration rates:

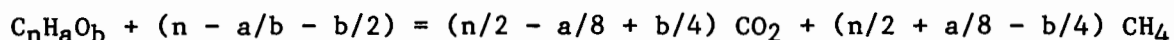
Waste	Growth Constant g/g	Endogenous Respiration Rate (day) ⁻¹
Fatty Acid	0.054	0.038
Carbohydrate	0.240	0.033
Protein	0.076	0.014

The combination of a low cell yield coupled with extremely long SRT's in the reactor will lead to a very low net sludge production. This net solids production may be low enough to obviate the need for final clarifiers and still meet secondary effluent standards. The data presented by Jewell⁽¹⁰⁾⁽¹¹⁾⁽¹²⁾ suggest that the net sludge production is low enough so that the excess solids can be discharged in the effluent. If gas bubble formation and subsequent attachment to the particles tends to float media from the reactor, some stripping and final settling may be required. The settling velocity of a 0.4 mm particle of 1.1 specific gravity is approximately 21.3 m/hr (70 ft/hr) so settling these particles can be accomplished in clarifiers/settling tanks with high overflow rates. Larger or more dense particles will, of course, settle faster.

Methane Production and Recovery

In contrast to an anaerobic sludge digester where the high sludge feed concentrations make the amount of CH_4 existing in solution negligible in comparison to the amount which is recovered in the overlying gas phase, the amount of CH_4 which leaves the reactor in a dissolved phase from an anaerobic fluidized bed reactor can represent a substantial part of the CH_4 formed.

Methane production from the anaerobic decomposition of any organic compound can be accurately predicted by a number of techniques. Symons developed the following equation:



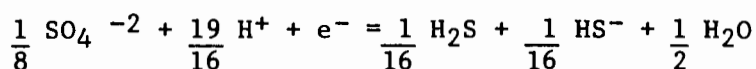
Equal proportions of methane and CO_2 result from the decomposition of carbohydrates and also from acetic acid. Proteins, fats and long chain acids will yield gas compositions higher in CH_4 than CO_2 . Typical municipal wastewaters have total organic carbon concentrations (TOCs) in the primary effluent of 80 to 180 mg/l. If 85 percent of this TOC were converted to CO_2 and CH_4 in an anaerobic system in the ratio of 40:60, the carbon in the methane produced would range from 41 to 92 mg C/l. A comparison of these values with the methane solubility data in Table 3 shows that in all cases the quantity of methane produced which exists as dissolved methane gas must be considered in any design situation where recovery of the methane from the gaseous space overlying the reactor will be practiced. These data show that the amount of CH_4 which remains dissolved in the liquid phase can be a significant fraction of the total CH_4 production. Of course, the partial pressure of the methane in the gaseous phase will affect the equilibrium solubility concentration. Whether the dissolved CH_4 concentration will tend toward the equilibrium concentration dictated by the overlying partial pressure, remain near the saturation concentrations shown in Table 3, or be somewhat supersaturated will be influenced by the reactor design, the hydraulic residence time, and the degree of gas transfer across the gas-liquid interface. In contrast to an anaerobic sludge digester with an overlying atmosphere of 25 to 35 percent CO_2 , the CO_2 overlying an anaerobic fluidized bed reactor will be much less. Assuming influent TOC of 80 to 180 mg/l, the CO_2 production would be 27 to 62 mg/l as C. When these values are compared to the solubility limits in Table 3 it is clear that the equilibrium partial pressure of CO_2 will be quite small. The actual values will depend upon wastewater pH and mass transfer across the gas-liquid interface, but should be less than 10 percent of the off-gas volume. Also the N_2 concentration in the overlying gas volume could be 5 to 15 percent of the total gas volume because of evolution of the nitrogen gas initially dissolved in the wastewater.

Another consideration in estimating methane production is the sulfate concentration of the wastewater. The sulfate concentration in natural waters varies from 5 mg/l to 250 mg/l⁽¹⁸⁾. In anaerobic systems the sulfate can serve as a terminal electron acceptor in biologically mediated reactions. This can be represented by the following half reaction⁽¹⁹⁾:

TABLE 3. SOLUBILITY OF CARBON DIOXIDE AND METHANE GASES

Temperature °C	Solubility*, mg/l		Solubility as C, mg/l	
	CH ₄	CO ₂	CH ₄ -C	CO ₂ -C
10	29.6	2318	22.2	632
15	26.0	1970	19.5	537
20	23.2	1688	17.4	460
25	20.9	1449	15.7	395
30	19.0	1257	14.3	343

*When the pressure of the gas plus that of the water vapor is 760 mm Hg



According to Bryant⁽²⁰⁾, methanogenesis in natural ecosystems does not occur when sulfate is present. Conversion of acetate to CO₂ with sulfate reduction to sulfide is thermodynamically more favorable than acetate conversion to CO₂ and CH₄. With wastewaters containing influent COD's of 200 to 250 mg/l and SO₄ concentrations of 200 mg/l (133 mg/l as O₂), the majority of the organic material could be oxidized through sulfate reduction with a corresponding decrease in methane formation. Hydrogen sulfide gas is extremely soluble in water (3850 mg/l at 20°C), whereas most heavy metals form insoluble sulfides. The partitioning of the H₂S gas between the liquid and overlying gas phase will depend on the distribution of sulfur species and the degree to which the equilibrium conditions predicted by Henry's law are approached.

ECONOMIC PERSPECTIVE

Application of anaerobic fluidized bed treatment of wastewater envisions replacement of the activated sludge system with an anaerobic reactor. This can potentially result in cost savings in the following areas: a) a reduction in energy demand as air or oxygen need not be supplied, b) a reduction in capital cost for the secondary system because it may be possible to use smaller reactors, smaller final clarifiers; and no air supply system will be needed, c) all of the organics in the wastewater can be converted to methane which can be used on site as a fuel or sold to a utility company, and d) a reduction of capital and operating costs for sludge handling. The first three of these are somewhat speculative and may not yield significant economic advantages. The energy reduction resulting from the curtailment of aeration will range from 0.132 to .264 kwh/cu m (500 to 1000 kwh/MG) depending on the SRT of the activated sludge system. However, as illustrated above, perhaps 1/4 to 1/2 of this power will be required to pump the recycle. Although the data presented by Jewell and co-workers^(10, 11, 12) indicates successful operation at HRT lower than conventional activated sludge this must be demonstrated in a real world situation. Also to be demonstrated is the possibility that smaller final clarifiers can be used. Thus the only sure saving is that extensive air supply equipment will not be required. Even this saving must be modified because some degree of post-aeration will be needed with an anaerobic system to strip sulfides and methane and raise the effluent dissolved oxygen. In a conventional treatment plant with anaerobic digestion for stabilization of both primary and secondary sludge 60 to 70 percent of the incoming degradable organic carbon can be converted to methane. With an anaerobic reactor substituted for the aerobic secondary treatment process, over 90 percent conversion to methane should occur. However, much of the extra methane will be dissolved on the liquid and must be stripped under controlled conditions to be recovered. In addition, as indicated previously, some of the organics will be stabilized through sulfate reduction which will reduce the methane yield.

The major economic effect of adaption of anaerobic treatment will be the reduction in sludge disposal costs. Table 4 presents data on sludge

TABLE 4. SLUDGE QUANTITIES AND VOLUMES REQUIRING PROCESSING PER MILLION GALLONS TREATED IN A TYPICAL ACTIVATED SLUDGE PLANT WITH PRIMARY CLARIFICATION

	Case No. 1	Case No. 2
Influent BOD ₅ and Suspended Solids, mg/l of each	200	250
Primary Sludge, lb	1001	1251
Secondary Sludge, lb	636	837
Unthickened Primary Sludge Volume at 4% Solids, gal	3000	3750
Thickened Primary Sludge Volume at 9% Solids, gal	1334	1667
Unthickened Secondary Sludge Volume at 1% Solids, gal	7626	10036
Thickened Secondary Sludge Volume at 3% Solids, gal	2542	3345
Thickened Combined Sludge Volume at 5.5%, gal	3569	4552
Primary Sludge Volatile Solids, lb	651	813
Activated Sludge Volatile Solids, lb	477	628

Design Assumptions:

Primary Clarifier Solids Removal 60%
 Primary Clarifier Sludge 65% Volatile Solids
 Primary Clarifier BOD Removal 35%
 Cell Yield 0.75 lb. VSS/lb BOD₅ Removed
 Cell Decay 0.07 days⁻¹
 Soluble Effluent BOD₅ 3 mg/l
 SRT 5 days
 Effluent Suspended Solids 15 mg/l
 Effluent Solids are 75% Volatile

quantities produced at a conventional treatment plant. The preliminary data produced by Jewell et.al.(10, 11, 12) indicates that an anaerobic secondary treatment unit could meet secondary effluent standards without any sludge wasting except the 10-15 mg/l biomass in the effluent. It can thus be seen that the sludge quantity in mass and volume is considerably less from a conventional primary plant followed by an anaerobic secondary than from a conventional primary plus aerobic secondary plant. Using the EXEC/OP cost program, (21, 22) the effect of this reduction in sludge quantity was calculated for 3,780m³/day (1MGD) and 37,800m³/day (10MGD) treatment plants. For the smaller plant using gravity thickening, lime stabilizations, vacuum filtration and hauling of filter cake costs are reduced by 33 percent for sludge handling and disposal and by 15 percent for overall plant capital, operation and maintenance. For the larger plant using gravity thickening, anaerobic digestion, elutriation, vacuum filtration and cake hauling, costs for sludge handling and disposal are reduced by 46 percent and overall plant capital and operation and maintenance by 18 percent. Thus, use of anaerobic secondary treatment can leave a significant effect on the treatment cost for municipal sewage treatment.

SUMMARY

The results reported by Jewell (10, 11) and Switzenbaum and Jewell (12) have demonstrated that better than secondary effluent quality can be obtained from a laboratory anaerobic expanded bed reactor treating primary effluent at 20°C. The process was also shown to provide good COD removal with a glucose feed when the temperature was 10°C and the HRT was 4 hours or greater. Since wastewater temperatures in much of the United States fall to 8 to 12°C during wintertime operation, the response at lower temperatures is quite important. Previous studies by O'Rourke (23) with homogenized primary sludge established that methane fermentation was drastically reduced at 15°C and that efficient digestion could not be accomplished even at a 60-day retention time. The lipid fraction of the waste was not utilized. However, there was a measurable reduction in the total COD due to the methane fermentation of formic and acetic acids resulting from cellulose and protein degradation. Whether or not anaerobic treatment of municipal wastewaters at low temperature is economically attractive has yet to be demonstrated.

Because of the limited data available, the long time required for such systems to come to equilibrium, and the scale of the studies reported, there are a number of questions related to anaerobic fluidized bed technology which remain to be answered before the design approach can be optimized. These include: reaction kinetics as a function of temperature; reactor response under dynamic loading; optimal reactor depth, media density and size; need for equalization basins and an overall flow control strategy, effect of degree of bed expansion; net solids production; solids levels attainable in the reactor; biological film properties; effect of biological growth on media expansion characteristics; solids control strategies in the reactor, if any; need for final clarifiers; influence of wastewater sulfate concentration on the desirability and performance of the process; long term process stability and

reliability at pilot scale; need for post treatment to remove sulfides, residual solids and raise dissolved oxygen; and effect of the transient presence of toxic materials.

EXPERIMENTAL EVALUATION OF ANAEROBIC FLUIDIZED BED TREATMENT OF MUNICIPAL WASTEWATER

In order to gather information on some of the uncertainties listed in the previous section a small pilot unit was set up at the Cincinnati Test & Evaluation Facility. The prime purpose was to determine effluent quality as a function of hydraulic detention time and wastewater temperature with a feed of primary effluent. Based on hydrodynamic analysis of fluidization in previous sections, the medium chosen was -40 to +50 mesh silica and with the system flow rate set to produce approximately 10 percent bed expansion. Use of this medium and flow rate requires pumping power which is within the range of economic acceptability. In addition, the bed height required is not excessive for the detention times anticipated. It was decided to start at a low expansion rate to allow for maximum change in expansion due to biomass coating of the media.

A schematic diagram of the final version of the test apparatus is given in Figure 4. It consists of two 15.2 cm (6 inch) diameter columns in series each originally packed with 1.53 m (5 feet) of sand. Supporting the sand is graded gravel used to provide flow distribution. The sand depth given in Figure 4 reflects losses during system renovation discussed below. Two independent positive displacement pumps are used for each column: one for the wastewater, the other for the recycle. Note that the recycle take off point is below the effluent discharge point in each column. Traps are provided to protect the pumps against clogging with any media which escapes the column. The effluent line of each column is trapped to prevent escape of gas. Gas measurement with a wet test meter is provided. The details of this design are the result of several months of trial and error during system start up. Originally only one column was used and when a second was added, the same recycle pump was used for both. This was abandoned when operational problems developed. The location of and size of the traps were changed to avoid problems which developed during sampling.

The system was started by seeding with digester supernatant and primary effluent with the system set on 100 percent recycle. Periodically a mixture of sodium acetate, acetic acid and methanol was added on a batch basis. Periodic analyses were used to determine when the system had to be refed. After several months of operation, during which time the second column was added, a flow through system was started with primary effluent as the feed. The hydraulic detention time was gradually reduced from 5 days to 1.5 days over a 2 month period at which point long term operation was to be initiated. At that time a crack developed in one of the columns. The system was shut down, and the media was removed and stored in a barrel while repairs were made. When the system was restarted, methanol was fed to evaluate the effect of the shutdown and media removal. Rapid removal of the methanol accompanied by gas evaluation occurred, indicating that several days exposure to the air had little effect on the methane bacteria in the biofilm.

Again, just when long term operation was to be started, an operational problem developed. The recycle pump packing sprang a leak, the columns drained, and the pump ran dry. After several days down time, the pump was repaired and start-up took place. It was difficult to fluidize the media at first and in the process much biomass appeared to be flushed out of the system. Despite this, once underway system operation seemed satisfactory.

For the next four months the system was operated under steady conditions with the HRT in the range of 1 to 1.2 days. Table 5 gives data on operating conditions during this period. Tables 6 and 7 provide data on treatment performance.

It can be seen that suspended solids removal was good with the effluent usually meeting the EPA standard of 30 mg/l. However, organics removal was not as good as required for secondary effluent. Organics removal was not affected by temperature changes as the column temperature was 20°C in January and 22°C in April. The consistency in temperature was due to the high rate of recycle (100 to 1). Volatile acids were low generally in the range of 50 to 100 mg/l, with lower concentrations in the effluent from the first column. There was no evidence of gas evolution from either columns because either the quantity of methane produced was less than its solubility, or sulfate reduction accounted for most of the COD decrease.

In May 1981 the detention time was reduced at 2 week intervals first to 21 hours, then to 17 hours, and finally to 12 hours. The system was maintained at this detention time through June 1981. The performance of the system remained the same as at the 1 day detention time: good suspended solids removal, modest organics removal, no evidence of gas production and lower volatile acids in the first column effluent than the second. The latter indicates the possibility that breakdown of the complex organics in the sewage is the rate limiting step in the system rather than the methane fermentation step. Further evidence of this is that each time the detention time was reduced in May, the volatile acid level rose for only one day and then was reduced to below 100 mg/l. As indicated in a previous section, at low temperatures anaerobic breakdown of lipid material is severely retarded. This may account for the high level of organics in the system effluent.

SUMMARY OF EXPERIMENTAL EVALUATION

To date anaerobic expanded bed treatment of primary effluent at detention times down to 12 hours has not been able to produce a treatment level which would qualify for approval as secondary effluent. Suspended solids levels in the effluent have been consistently below 30 mg/l but effluent COD has been about 25 to 50 mg/l higher than aerobic secondary effluent at the same plant site. The rate limiting step may be volatile acid production from complex organics rather than methane production. Methane production rate has not been high enough to exceed the methane solubility in water. Long-term stable mechanical and hydraulic pilot plant operation at conditions envisioned for field application has taken place. Biomass accumulation has not affected fluidization and expansion characteristics of the bed of granular media.

TABLE 5. OPERATING PARAMETERS
ANAEROBIC TREATMENT SYSTEM
JANUARY - APRIL 1981

Parameter	January	February	March	April
HRT (days)	1.2	1.0	1.0	1.2
Flow (L/d)	48	39	39	35
Flow (gpd)	12	10	10	9
Recycle Flow (L/d)	4800	4200	5800	4800
Recycle Flow (gpd)	1300	1100	1500	1300
Bed Expansion (%)	12	6	11	6
Upflow Rate (L/m ² -S)	3.6	3.0	4.4	3.6
Upflow Rate (gpm/ft ²)	5.3	4.4	6.5	5.3

TABLE 6. EFFLUENT QUALITY
ANAEROBIC TREATMENT SYSTEM
JANUARY - APRIL 1981

Parameter	January	February	March	April
pH	8.0	8.1	8.2	8.2
Alkalinity (mg/l as CaCO ₃)	600	530	630	520
TOC (mg/l)	127	-	78	69
COD (mg/l)	185	104	193	129
TSS (mg/l)	37	42	23	30

TABLE 7. REMOVAL EFFICIENCY (% REMOVAL)
ANAEROBIC TREATMENT SYSTEM
JANUARY - APRIL 1981

Parameter	January	February	March	April
TOC	39	-	44	27
COD	58	71	52	45
TSS	80	84	88	75

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NATIONAL SURVEY OF MUNICIPAL WASTEWATERS FOR TOXIC CHEMICALS

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INTRODUCTION

The modern industrial society manufactures, uses, and discharges to the environment tens of thousands of chemical compounds that amount to billions of pounds annually. It was long known that some of these chemicals were being discharged to wastewater for treatment at publicly owned treatment works (POTW). Further, there was a strong presumption that some of these chemicals were incompletely removed and were being discharged with the effluents.

Until a few years ago the identification and quantification of the trace concentrations of organic chemicals in wastewater was beyond the capability of the analytical techniques of the time. As more sophisticated analytical instrumentation and techniques became available, they were quickly applied to wastewater analysis. In the early and mid 1970's wastewaters were analyzed for selected classes of compounds. For example, in 1971, Reichert, et al.,(1) analyzed for polynuclear aromatic hydrocarbons; Schmidt, et al.,(2) in 1971 and Lawrence and Tosine (3) in 1976 analyzed for PCB's. Not until the late seventies were analytical techniques sufficiently advanced to undertake to analyze wastewaters for more diverse classes of compounds. Then Chian and DeWalle (4) and Glaze, et al., (5) identified organic compounds which encompassed a wide range of classes of compounds.

A court imposed Consent Decree issued in 1976 (6) galvanized the attention of U.S. Environmental Protection Agency on a selected list of 129 compounds (now reduced to 127) including some 114 organic and 13 inorganic compounds. In the field of wastewater treatment, implementation of the Decree presented two immediate problems. First there was the need to develop the analytical methods capable of analyzing raw wastewaters and sludges for the diverse classes of chemical compounds represented in the priority pollutant list; and secondly, there was the imperative need to establish a base of information on the numbers and amounts of compounds received at the POTW, the degree of removal accomplished by various treatment processes and the compounds and concentrations accumulated in the sludges.

To meet the above objectives, an EPA funded study was undertaken in June 1978 by the University of Washington and Georgia Institute of Technology and had two principal tasks. The first objective, which was crucial

to completion of the second, was to develop and modify existing analytical methods to enable the analysis of raw wastewaters and sludge. The second objective was to obtain information on priority pollutants in wastewaters in many plants nationally distributed, to provide a first-step overview of the status of priority pollutants in POTWs. This paper summarizes the results of the priority pollutant survey of 25 cities located throughout the United States. Results of the study are described in a report submitted to EPA by F. P. DeWalle, E. S. K. Chian, et al., entitled "Presence of Priority Pollutants in Sewage and Their Removal in Sewage Treatment Plants." (7)

Shortly after the initiation of the present study, a more extensive and comprehensive survey was undertaken by the USEPA Effluent Guidelines Division in Washington, D. C. The survey sampled 40 cities with each plant sampled for five consecutive days. Results of a portion of this study are described in two Interim Reports (8,9).

ANALYTICAL METHODS DEVELOPMENT

State-of-the-art methodology has been developed to measure priority pollutants in wastewater and sludge at concentrations previously unattainable with standard methodology. Practical detection limits derived from minimum measured concentrations of priority pollutant organics were on the order of one ug/l except for pesticides where detection limits were near 10 ng/l.

Low molecular weight volatile compounds were measured using a modified Bellar/Lichtenberg procedure as shown in Fig. 1. In this method, the aqueous sample is spiked with one or more recovery/quantitation standards, diluted if necessary with purgeable organic-free water, and purged with a stream of organic-free helium. The gas stream is then passed through an inert adsorbent trap at room temperature where the entrained organics are retained. Upon completion of the purging, the trap is heated and back-flushed with organic-free helium to desorb the trapped organics. This sample stream is passed directly into a capillary GC column where it is cryotrapped at liquid nitrogen temperatures. When all of the sample has been desorbed and cryotrapped, the coolant is turned off and the cryotrap warmed with a flush of hot air. The GC or GC/MS analysis then ensues.

Recovery of selected volatile priority pollutants spiked into raw sewage and into sludge from several different POTWs are shown in Table 1 and Table 2, respectively. Considerable variation in recoveries is evident for individual compounds in different wastewaters and sludges as well as for different compounds in the same wastewater or sludge. These differences are attributed largely to matrix effects. The recovery/quantitation standards consisting of deuterated volatile compounds were added to each survey sample before analysis. The standards were used to compute expected retention times to identify the priority pollutants and to correct for the variable recovery of similar priority pollutants. It was estimated that a single analyte quantitated against two internal standards would ordinarily vary by 5-10%.

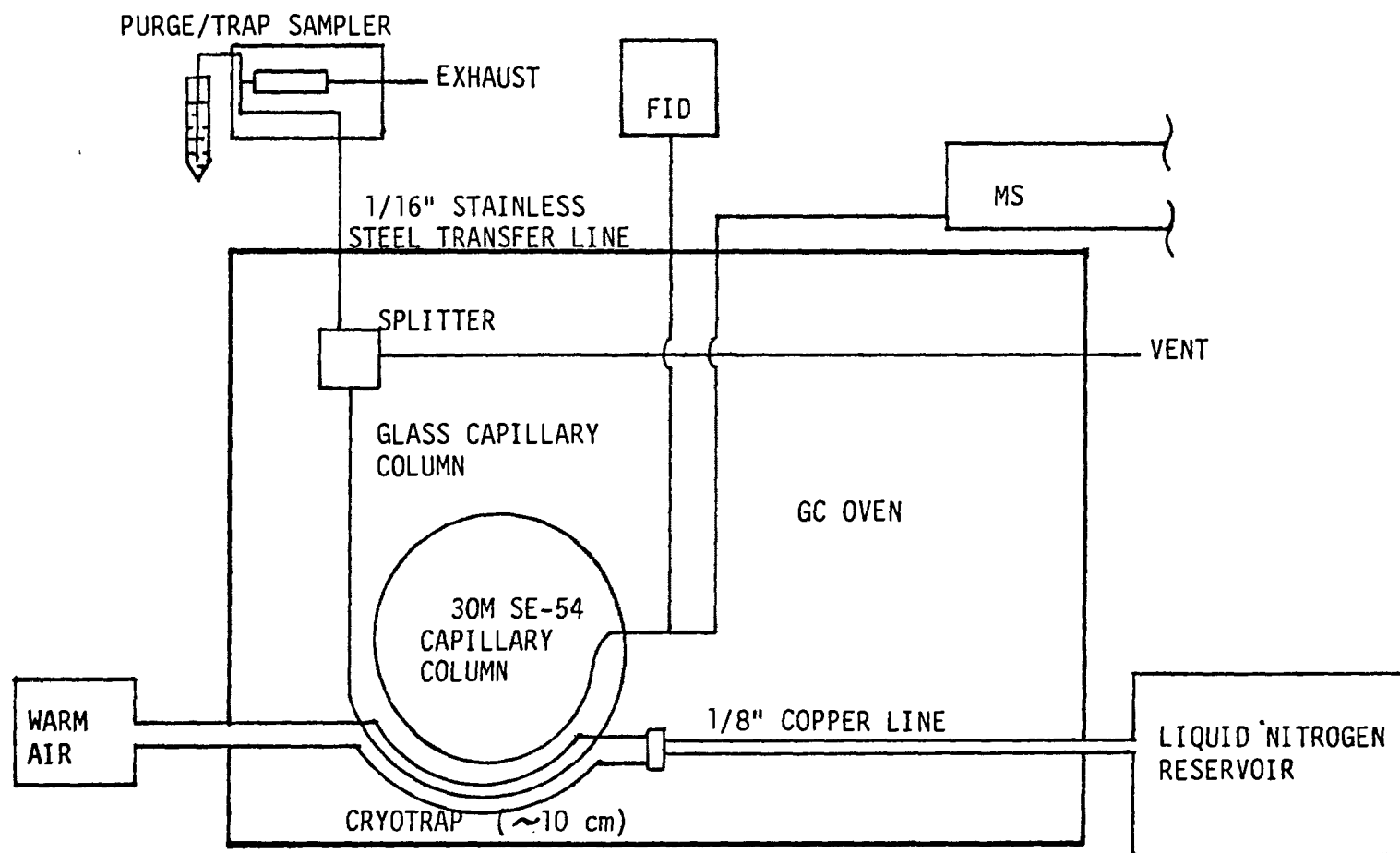


FIGURE 1. SCHEMATIC OF VOLATILE ORGANICS INSTRUMENTATION USING CRYOGENIC TRAPPING AND CAPILLARY GC/MS SEPARATION

TABLE 1. RECOVERY OF SELECTED VOLATILE PRIORITY POLLUTANTS FROM RAW SEWAGE
FROM FIVE POTW

Pollutant Spiked at 20 ug/l	% Recovery for Listed POTW					Mean Recovery \bar{x}	Standard Deviation σ
	Roch- ester	Akron	Green- ville	Chatta- nooga	Seattle		
Ethene, 1,1-Dichloro	70	139.5	102	124	94	105.9	26.9
Methane, Dichloro-	223	243.5	59.5	133	ND	164.8	85.0
Methane, Trichloro-	106.5	117.5	72.5	192.5	43	106.4	56.4
Ethane, 1,2-Dichloro-	109.0	104	103.5	109	82	101.5	11.2
Benzene	89.5	109	97	81.5	90.5	93.5	10.3
Ethene, Tetrachloro-	67.5	67	90.5	240.5	110.5	115.2	72.3

ND = Not Detected

TABLE 2. RECOVERY OF SELECTED VOLATILE PRIORITY POLLUTANTS FROM SLUDGE
FROM THREE POTW

Pollutant Spiked at 20 ug/l	<u>% Recovery for Listed POTW</u>			Mean Recovery \bar{x}	Standard Deviation σ
	Akron	Seattle	Tacoma		
Ethene, 1, 1-Dichloro-	73.5	155	49	92.5	55.4
Methane, Dichloro	80.5	123	188	130.5	54
Ethane, 1,2-Dichloro	89	142	130.5	120.5	27.9
Benzene	127	121.5	115	121	6.0
Ethene, Tetrachloro	80.5	66	75.5	74	7.5
Ethane, 1,1,2,2-Tetrachloro	53.5	213	119.5	128.5	80

Extractable priority pollutant organics were separated from aqueous samples in a stirred liquid-liquid continuous extractor developed at the University of Washington. The resulting extracts were further processed by gel permeation chromatography, florisil chromatography, and cesium silicate coupled chromatography to remove interferences and to fractionate the sample to facilitate analysis for individual compounds. Fig. 2 shows a simplified schematic for separation of extractable organics. Various solvent exchange and concentration steps have been omitted for clarity of presentation. Derivatization with diazomethane was used to improve the chromatographic behavior of the acidic compounds.

Analysis consisted of screening of each fraction by capillary GC-FID, subsequent recombination of the neutral components, capillary GC/MS analysis of the neutral and derivatized acid fractions, and GC/EC analysis of the neutral fraction to pick up low levels of PCB's and pesticides. The GC/MS data were analyzed by an automated data searching routine capable of applying both spectrum matching and GC retention time criteria for qualitative identification, and capable of either single-ion or multiple-ion quantitation.

As with the volatile analyses, the extractables require extensive use of blank and recovery samples to produce reliable quantitative data. Results of recovery tests for selected extractable compounds spiked into wastewaters and sludges are shown in Tables 3 and 4, respectively. In general, recoveries of extractables were lower than those of volatiles, particularly recoveries from sludges. Recoveries were also quite variable between different wastewaters for individual priority pollutants and for different priority pollutants in the same wastewater sample. Appropriate standards were spiked into every sample to allow individual corrections for the variable recoveries.

SELECTION OF CITIES

With some 20,600 municipal treatment systems in the United States, it was important that the small number of cities that would be sampled would constitute a reasonably representative sampling of the country. The number of compounds and concentrations in municipal wastewater was expected to be influenced by the amount and type of industrial waste discharged into the collection system. An important criterion for the selection of the cities was, therefore, based on an analysis of the industrial discharges to POTWs, differentiated by flow and type of industry.

Industry in the United States discharged a total of 14,144 billion gallons of water per year ($53 \times 10^{15} \text{ m}^3$) (U.S. Census 1972) of which 56 percent or 7987 billion gallons/year ($30 \times 10^{15} \text{ m}^3$) was untreated. The major portion was discharged to surface waters of various types but 7 percent or 990 billion gallons/year ($3.7 \times 10^{15} \text{ m}^3$) was discharged to a public sewer. Not only was flow considered but also the type of industry since the degree of contamination of the discharged water varies greatly with category of industry. Pre-treatment before discharge by the industry was also taken into account.

FIGURE 2
EXTRACTION AND CLEANUP SCHEMATIC FOR PRIORITY POLLUTANTS

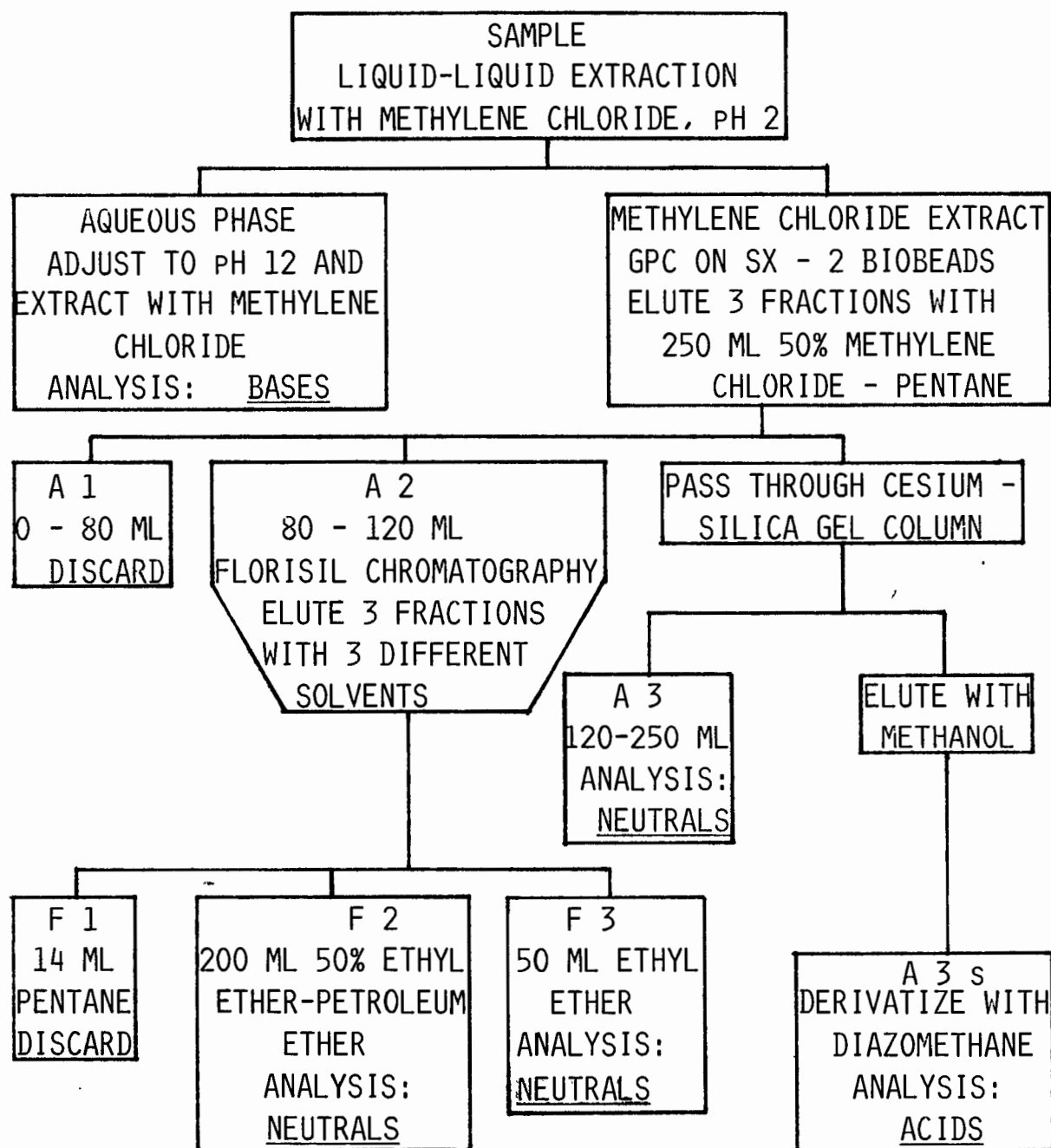


Table 3. Recovery of Selected Extractable Neutral Priority Pollutants
From Spiked Raw Sewage of Four Plants

Pollutant Spiked at 10 or 20 ug/l	% Recovery for Listed POTW					Mean Recovery \bar{x}	Standard Deviation σ
	Winston Salem	Seattle	Chatta- nooga (1st Sampling)	Chatta- nooga (2nd Sampling)	Peoria		
Nitrobenzene	65.0	75.5	73.5	29.5	59.5	60.6	18.5
Dimethylphthalate	54.5	59.0	80.5	16.5	61.0	54.3	23.4
2,6-Dinitrotoluene	68.0	69.5	42.0	30.0	67.5	55.4	18.2
Fluoranthene	28.0	11.5	29.0	21.5	36.0	25.2	9.2
Chrysene	19.0	14.0	54.5	30.0	64.5	36.4	22.2
Hexachlorobenzene	67.5	17.0	72.0	1.5	116.5	54.9	46.2

Table 4. Recovery of Selected Extractable Neutral Priority Pollutants
From Spiked Sludge of Four Plants

Pollutant Spiked at 50 or 100 ug/l	% Recovery for Listed POTW				Mean Recovery \bar{x}	Standard Deviation σ
	Ft. Myers	Greenville	Peoria	Winston Salem		
Di-N-Butylphthalate	53.1	21.5	14.8	16.8	26.6	17.9
Fluoranthene	53.7	25.8	44.0	13.1	34.2	18.2
Naphthalene	66.8	72.6	58.7	23.7	55.5	21.9
Hexachlorobutadiene	55.5	13.8	75.1	18.5	40.8	29.6
Dimethylphthalate	68.7	49.1	84.0	26.1	57.0	25.1
2,4-Dinitrotoluene	58.1	37.0	NF	20.3	28.9	24.7

NF = Not Found

A further criterion was based on the size of the plants and geographical location. Plant sizes ranged from 3.5 mgd (13,200 m³/day) to 309 mgd (1,170,000 m³/day) while the percentage of industrial flow varied from 0 percent to 60 percent. A frequency distribution plot of plant size and percentage industrial flow, Figure 3, compares the 25-plant study with an EPA survey made in 1978 which inventoried 586 plants discharging ≥ 5 mgd (18,925 m³/day or more). The 25-city survey compares quite well with the much larger EPA survey especially when allowance is made for the additional criteria used in making the selection.

The POTW selection also sought to obtain cities with varying death rates on the presumption that the high incidence of cancer and lung diseases would be expected to have a high number and concentration of priority pollutants in their wastewater. And finally, POTW selection considered the type of treatment process. While the majority of the plants used activated sludge, three employed trickling filters and four used aerated lagoons or algal ponds. One of the POTWs practiced land spreading. Twelve of the 25 plants had combined sewers.

SAMPLING PROCEDURE

In the plant survey single 24-hr composite samples were taken from the influent, primary effluent and final effluent. In those plants that were using chlorination an additional sample of the final discharge was taken after chlorination. Grab samples were taken of the sludges of various types such as digested, primary, combined, etc. Nine of the cities were revisited for a second sampling, thus the survey consisted of 34 samplings of 25 cities. Some 30,000 chemical parameters were determined in the collected samples during the survey.

Since almost half of the priority pollutants are volatile to varying degrees - the more volatile compounds having half-lives in open vessels of only 20-30 minutes - a sampler had to be devised which would circumvent losses of compounds by volatilization during the 24-hr compositing time. Based in part on a concept developed in our laboratory, (10) a special sampler was constructed and used in the survey. Sample aliquots were taken proportional to flow on a time delay basis related to the hydraulic detention in the various plant units. The sampler is described more fully by Tigwell, et al., (11).

DISCUSSION OF SURVEY RESULTS

Frequency of Occurrence and Removals in POTWs

Frequency histograms for the number of priority organic compounds found in individual wastewater and sludge samples from the different POTWs are shown in Fig. 4. Over a third of the raw wastewater samples (37%) contained between 30 and 40 organics at detectable concentrations while 83 percent of the samples contained between 20 and 50 organics. The maximum number found in any one sample was 57. The frequency distribution for the raw wastewaters is relatively symmetrical.

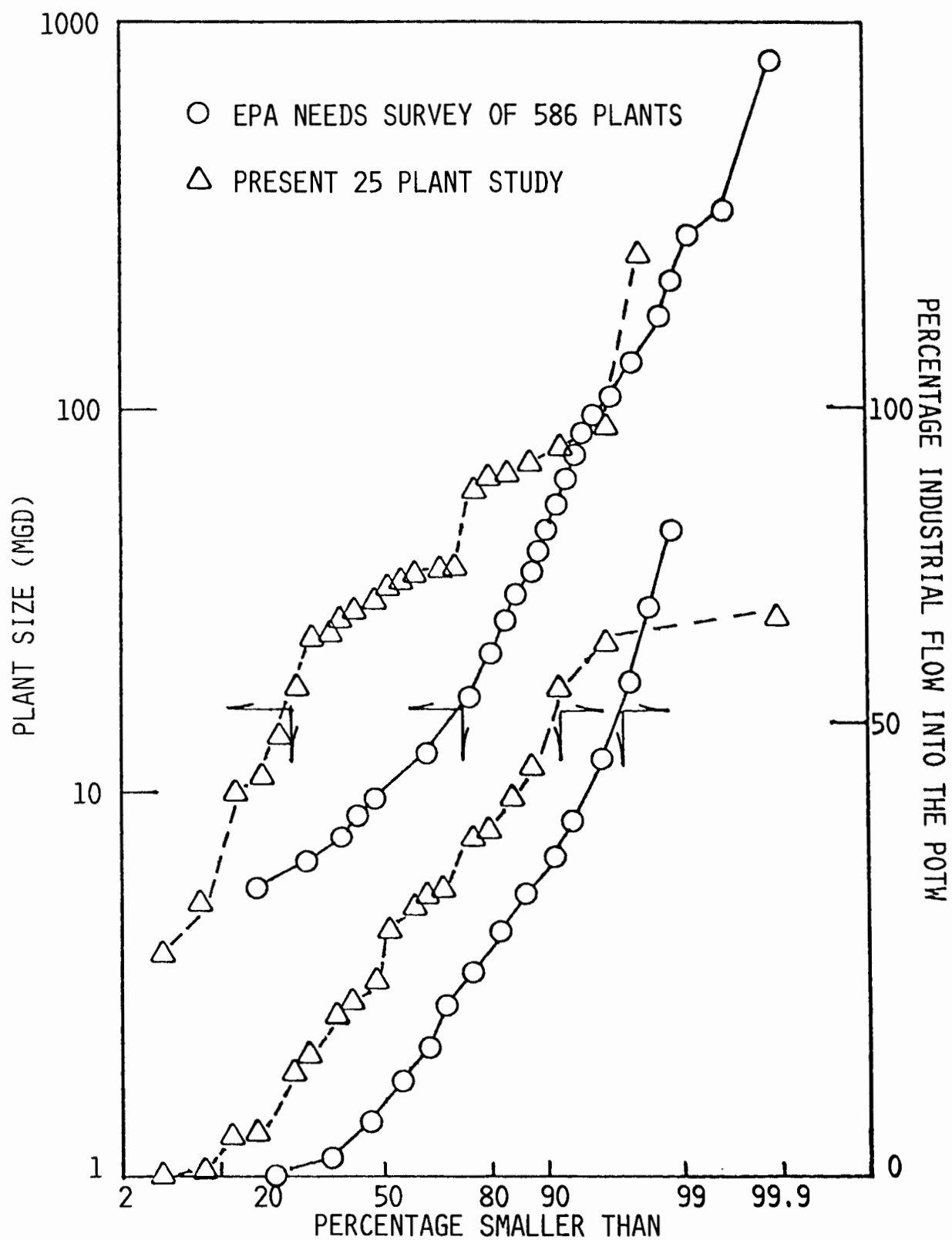


FIGURE 3. FREQUENCY DISTRIBUTION PLOT OF SIZE AND PERCENTAGE INDUSTRIAL INFLOW INTO POTWs

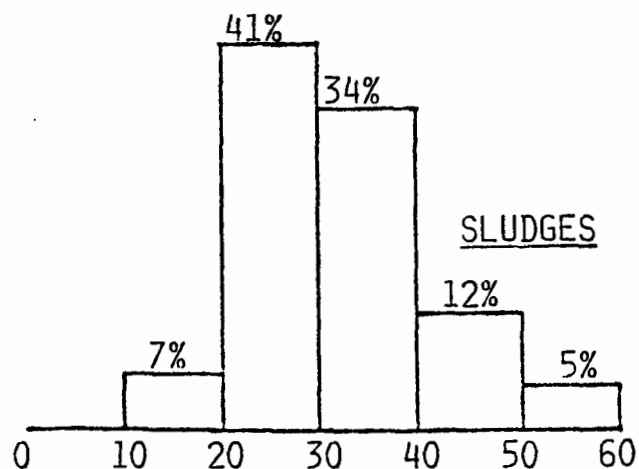
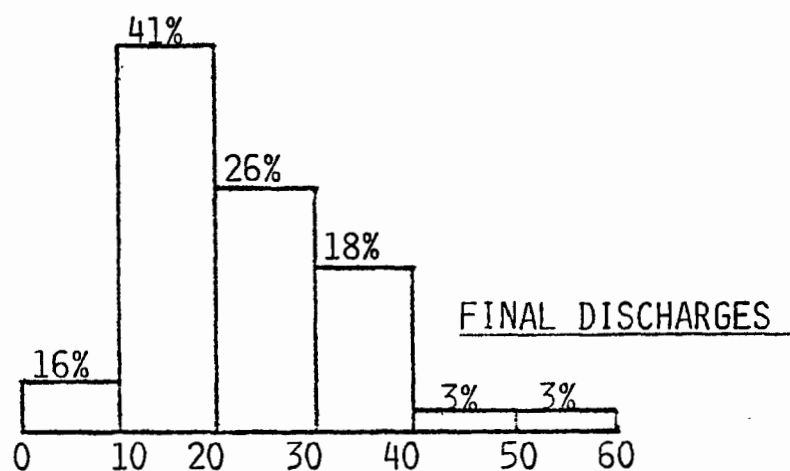
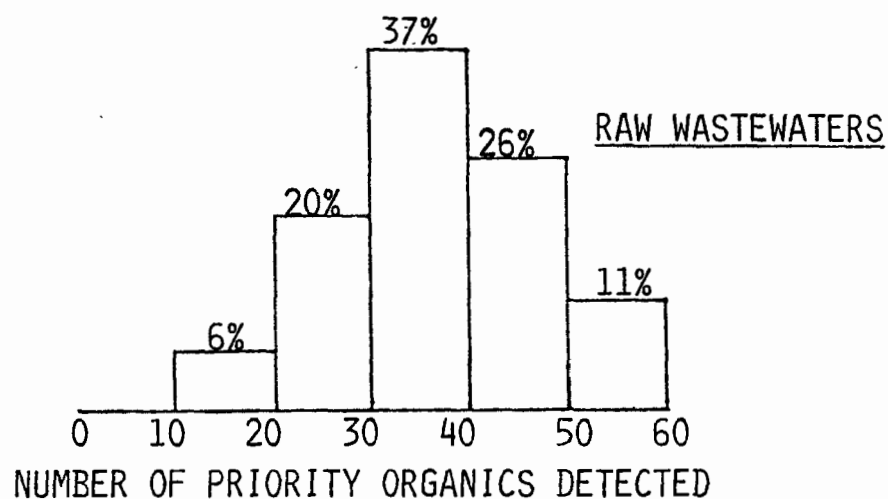


FIGURE 4. DISTRIBUTION OF NUMBER OF PRIORITY ORGANICS DETECTED IN INDIVIDUAL WASTEWATER AND SLUDGE SAMPLES

The final discharge samples are skewed to the lower end in number of organics detected. Forty-one percent of the samples had between 10 and 20 priority organics. Only 6 percent had more than 40 organics present. Three quarters of the sludge samples (75%) had between 20 and 40 organics detected. Again the distribution for the sludge samples is skewed toward the low end.

Figure 5 compares the number of priority organics detected in each raw wastewater sample with the total concentration of all priority organics in the sample. Each point represents one 24-hour composite sample at a POTW. The total concentration of priority organics in a sample was calculated by summing the concentrations of the individual priority organics measured in the sample. The majority of samples from the raw wastewaters from the different cities contained less than 1000 ug/l (1 mg/l) of priority organics. Also as previously shown in Figure 4, the number of compounds detected tends to cluster between 20 and 50. For those samples containing more than 1000 ug/l of priority organics, the data indicate that about the same number of organics are present at higher concentrations rather than a larger number of organics at normal concentration levels. It is obvious from the figure that there is no defined relationship between number and total concentration of priority organics over all the POTW's.

Since many priority organics in municipal wastewaters are believed to be from industrial sources, the number of priority organics in raw wastewaters was plotted against percent industrial flow as shown in Figure 6. The solid line is the least squares fit for the points enclosed by the two dashed lines. There appears to be a generally increasing trend in number of compounds found with increasing industrial flow. The coefficient of determination of this fit is, however, only 0.2.

No correlation was observed between total concentration of priority organics and percent industrial flow. This is most probably due to the varying nature of the industries and different levels of pretreatment provided in the cities surveyed.

The preceding discussion has addressed the numbers and total concentrations of organics in individual wastewater samples. The occurrence of individual priority organics in the collective data from all POTWs will now be reviewed. Figure 7 shows the variation in number of individual organics detected at a specific frequency above concentration levels of 0, 1, 10, and 50 ug/l. Occurrence in raw wastewaters is in the upper graph; occurrence in final discharges is at the bottom. An example of how to interpret the information is shown for the final discharges. As indicated by the upper circle, there were 45 individual compounds detected at least 20 percent of the time. Note that this is not the same as saying that 20 percent of the samples had 45 compounds present since the 45 in question could be occurring in different samples. Seventeen organics were detected at least 20 percent of the time at levels higher than 1 ug/l as shown by the lower circle. A total of 79 different compounds were detected in one or more final discharge samples. Thirteen were found at least once in concentrations greater than 50 ug/l. Only 12 compounds were ever detected in over 50 percent of the samples and of these only 7 were at levels greater than 1 ug/l.

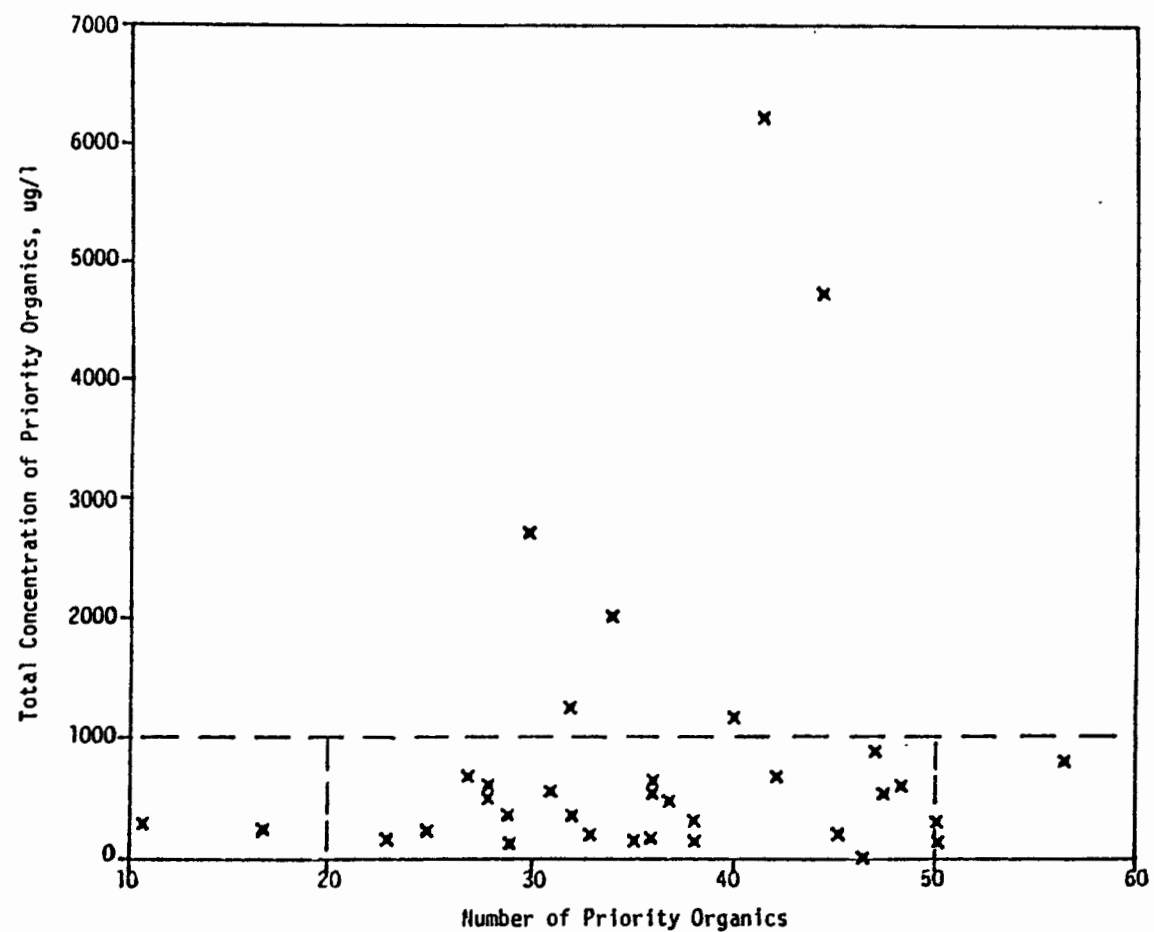


Figure 5. Total Concentration of Priority Organics
v. Number of Compounds in Raw Wastewater Samples

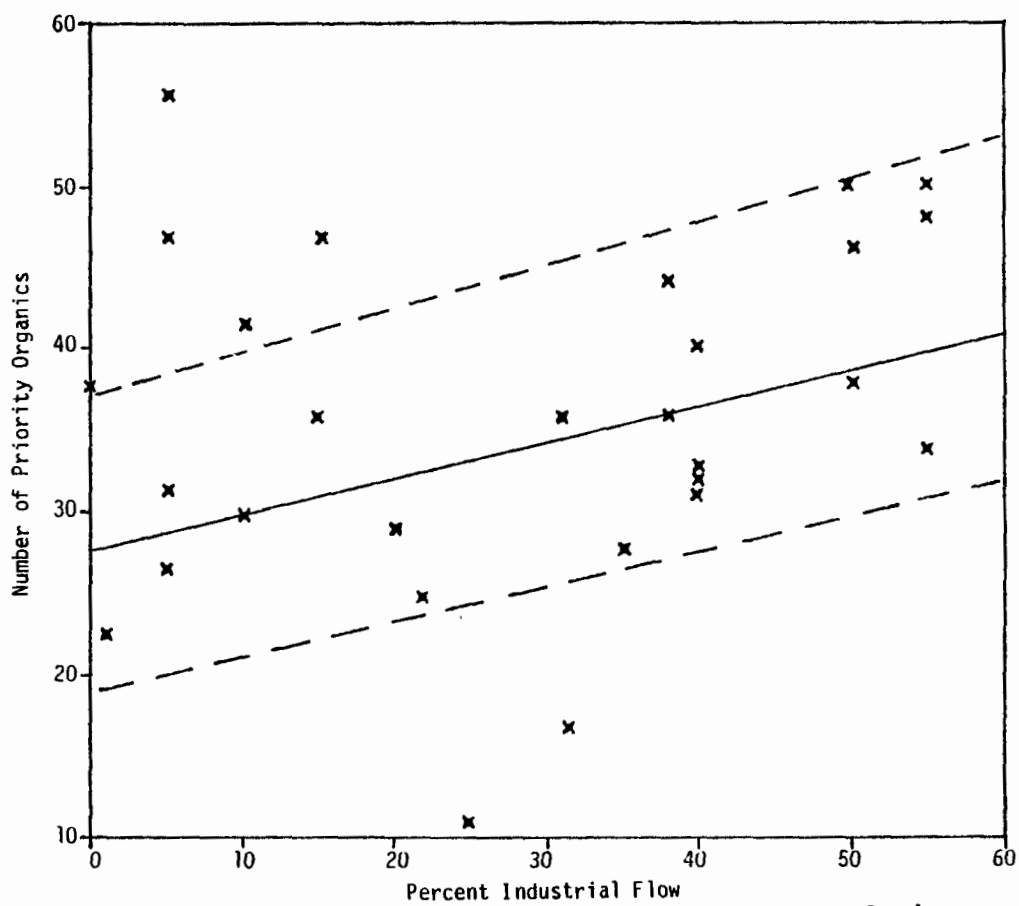


Figure 6. Number of Priority Organics Detected in Raw Wastewater Samples
v. Percent Industrial Flow

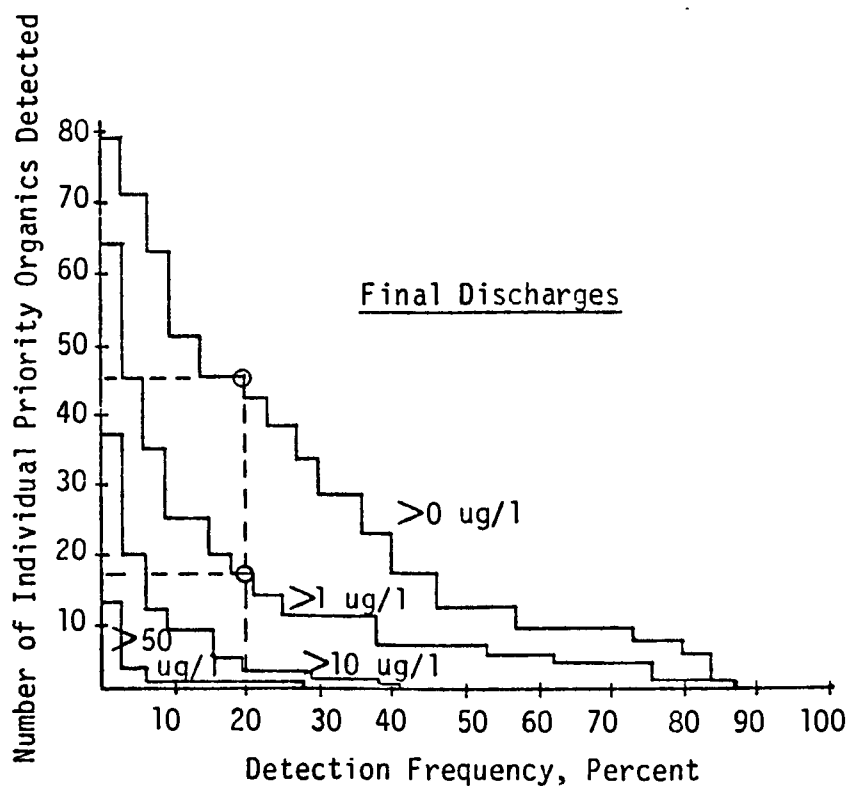
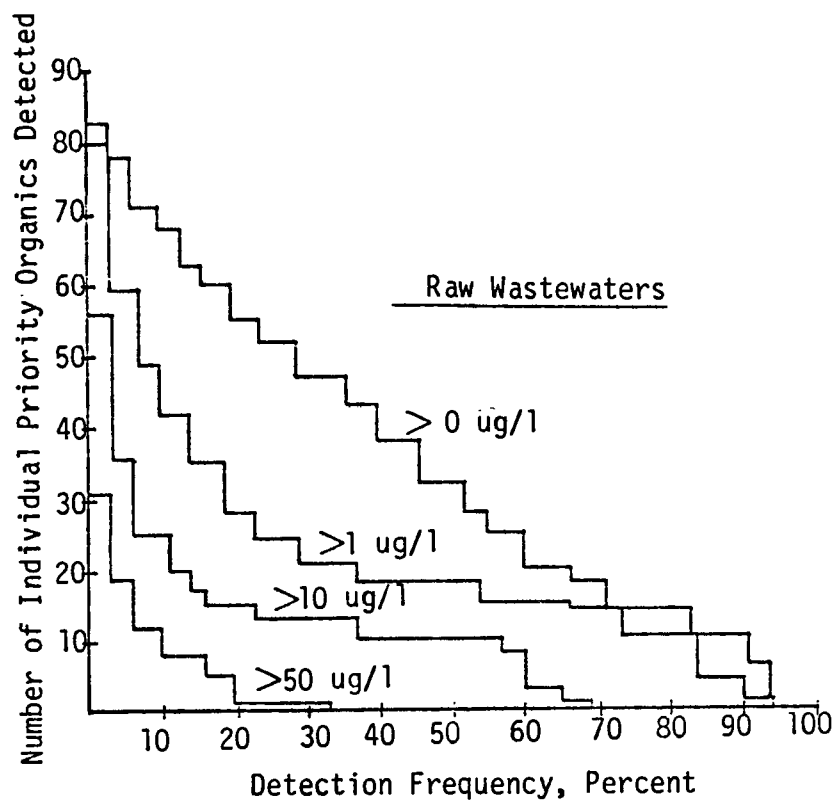


Figure 7. Detection Frequencies of Individual Priority Organics in Wastewaters

A relatively even spread of detection frequencies was observed for organics in raw wastewaters. One compound was detected as much as 95 percent of the time, 32 were detected at least half the time and 83 were detected at least once. Yet only 18 were detected 50 percent of the time at levels greater than 1 ug/l. A total of 31 compounds were found at least once in concentrations greater than 50 ug/l but none were found at this level more than 35 percent of the time.

The detection frequencies for priority organics in sludges, shown in Figure 8, follow those found for raw wastewaters. The sludge samples represent 9 raw primary sludges, 8 raw waste activated sludges, 5 raw combined sludges, 18 digested sludges, and 2 heat-treated sludges. Eighty-two compounds were detected at least once, 26 were detected at least half of the time and one was detected in 89 percent of the samples. Thirty-three compounds were found at least once in concentrations greater than 50 mg/kg but only one compound was found in 25 percent of the samples at that concentration.

Table 5 summarizes the most frequently found compounds at levels of 1, 10, and 50 ug/l in raw wastewaters and final effluents, and at levels of 1, 10, and 50 mg/kg for sludges. These include all of the compounds found above 1 ug/l (or 1 mg/kg) at least 80 percent of the time in raw wastewaters and at least 50% of the time in final discharges and sludges. The total number of priority organics meeting these selection criteria is only 15. Five are chlorinated methane or ethane derivatives while four are phthalates.

The ranges of total concentrations of nine classes of priority organics in individual raw wastewaters and final discharges are shown in Figure 9. The upper bar for each class, labeled R, is for raw wastewater; the lower bar, labeled F, is final discharge. Identified on each bar are the minimum reported discrete value, the 25th percentile, the median concentration, the 75th percentile, and the maximum reported value for the selected class and type of sample. Because of the wide range of values, a logarithmic concentration scale was used. Total concentrations generally ranged over 3 to 4 orders of magnitude with interquartile ranges (25% to 75%) ranging between 1 and 2 orders of magnitude. The positions of the medians indicate that the concentration distributions are skewed to the left with most of the values at the lower end of the concentration range. Final discharge ranges were always lower than raw wastewater ranges, however, no conclusions regarding percent removals should be drawn from Figure 9 since the data do not segregate influent-effluent pairs from a single plant.

Figure 10 was prepared from single plant influent and effluent data pairs to show the interquartile ranges of percent removals for the same 9 classes of priority organics. The left end of each bar represents the 25th percentile while the right end is the 75th percentile of the data set. High variability was observed for the methanes, ethanes, and pesticides. The wide range of removals for the former two classes can be explained by the production of chlorinated methanes and ethanes in final discharges during disinfection with chlorine. In fact, negative removals are sometimes observed where final discharge concentrations exceed those in raw wastewater.

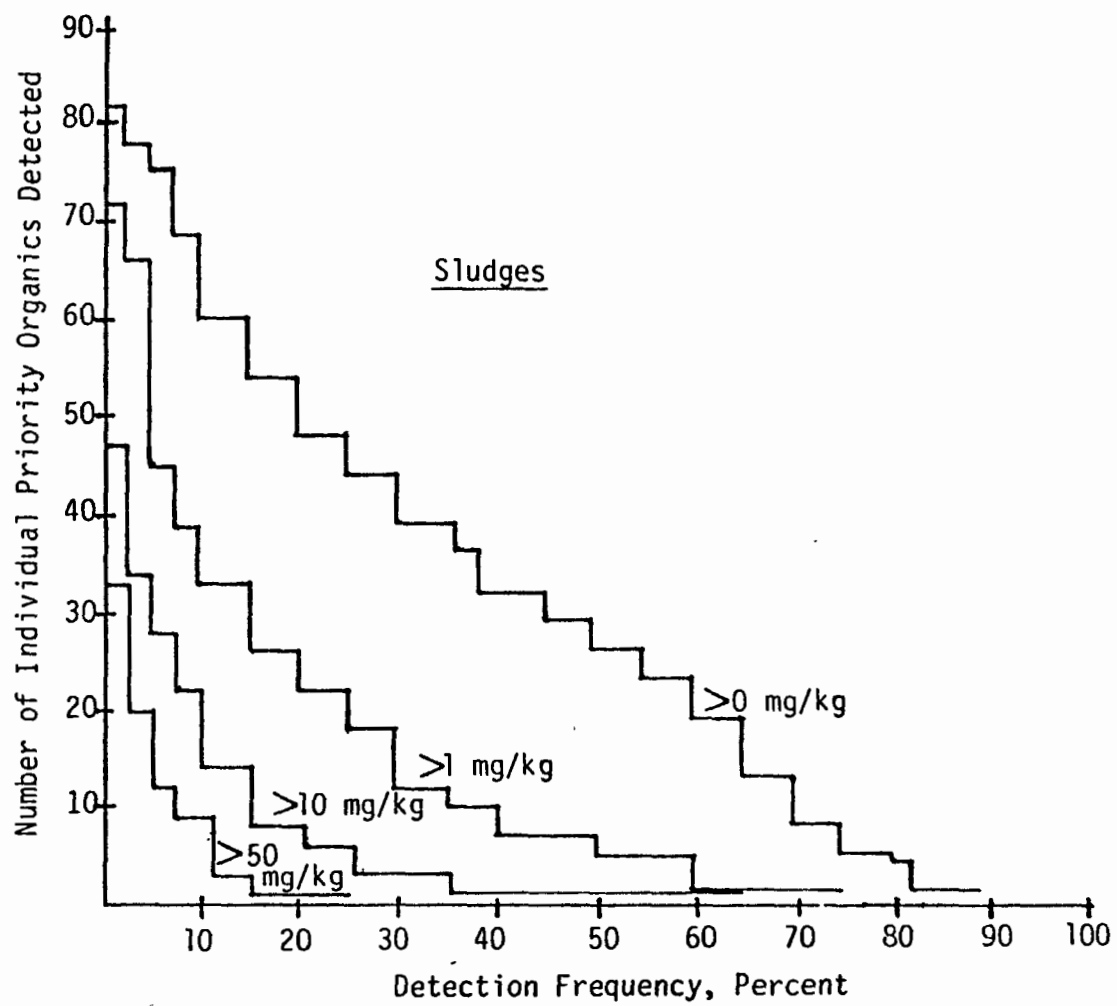


Figure 8. Detection Frequencies of Individual Priority Organics in Sludges

TABLE 5. MOST FREQUENTLY DETECTED COMPOUNDS IN RAW WASTEWATER, FINAL DISCHARGE AND SLUDGE SAMPLES

COMPOUND	PERCENT OCCURRENCE AT INDICATED CONCENTRATION								
	RAW WASTEWATER			FINAL DISCHARGE			SLUDGE (A)		
	≥ 1 ug/L	≥ 10 ug/L	≥ 50 ug/L	≥ 1 ug/L	≥ 10 ug/L	≥ 50 ug/L	≥ 1 mg/kg	≥ 10 mg/kg	≥ 50 mg/kg
METHANE, DICHLORO-	83	57	33	79	41	28	41	12	3
METHANE, TRICHLORO-	74	26	3	55	6	0	3	0	0
ETHANE, 1,1,-TRICHLORO-	90	61	16	53	22	6	5	3	0
ETHENE, TRICHLORO-	90	61	19	47	16	3	26	9	3
ETHENE, TETRACHLORO-	94	65	26	77	19	3	27	8	3
BENZENE, 1,4-DICHLORO-	83	37	6	38	6	0	36	18	5
ETHYLBENZENE	87	23	13	13	3	0	33	3	0
TOLUENE	84	61	23	45	17	3	59	35	11
PHENOL	86	66	31	38	6	3	63	25	13
NAPHTHALENE	86	37	11	24	3	0	65	33	15
PHENANTHRENE	57	9	0	15	3	0	60	20	8
PHTHALATE, DIETHYL	91	57	17	62	29	0	43	23	13
PHTHALATE, DI-N-BUTYL	89	60	9	85	18	3	63	25	13
PHTHALATE,BIS(2-ETHYLHEXYL)	86	69	20	76	38	6	75	63	25
PHTHALATE,BUTYLBENZYL	77	60	11	32	15	0	50	35	18
ALL OTHERS	< 80	< 60	< 20	< 50	< 20	< 10	< 50	< 25	< 15

(A) EXPRESSED ON DRY SOLIDS BASIS.

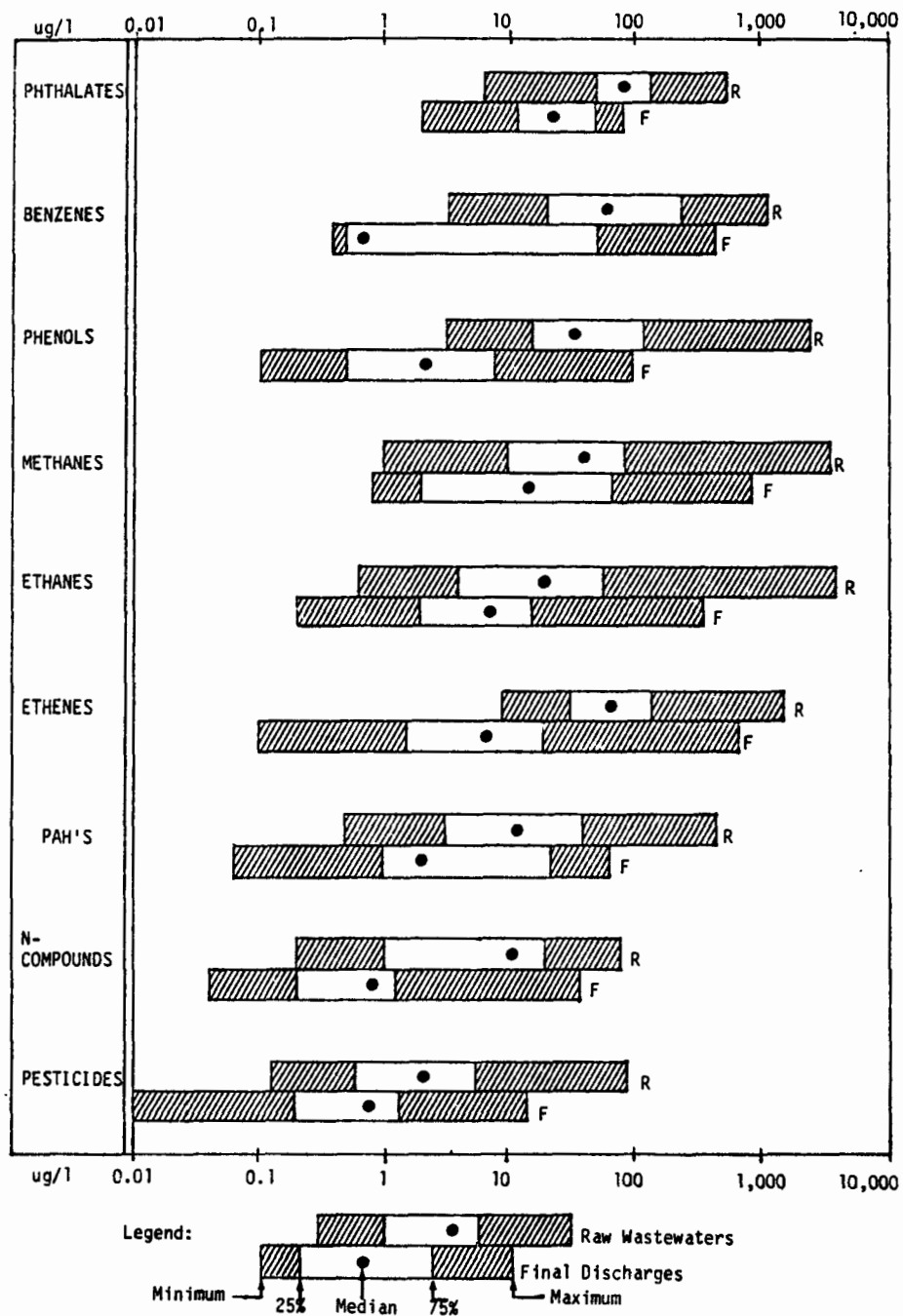


Figure 9. Ranges of Total Concentrations of Nine Classes of Organics in Raw Wastewaters and Final Discharges

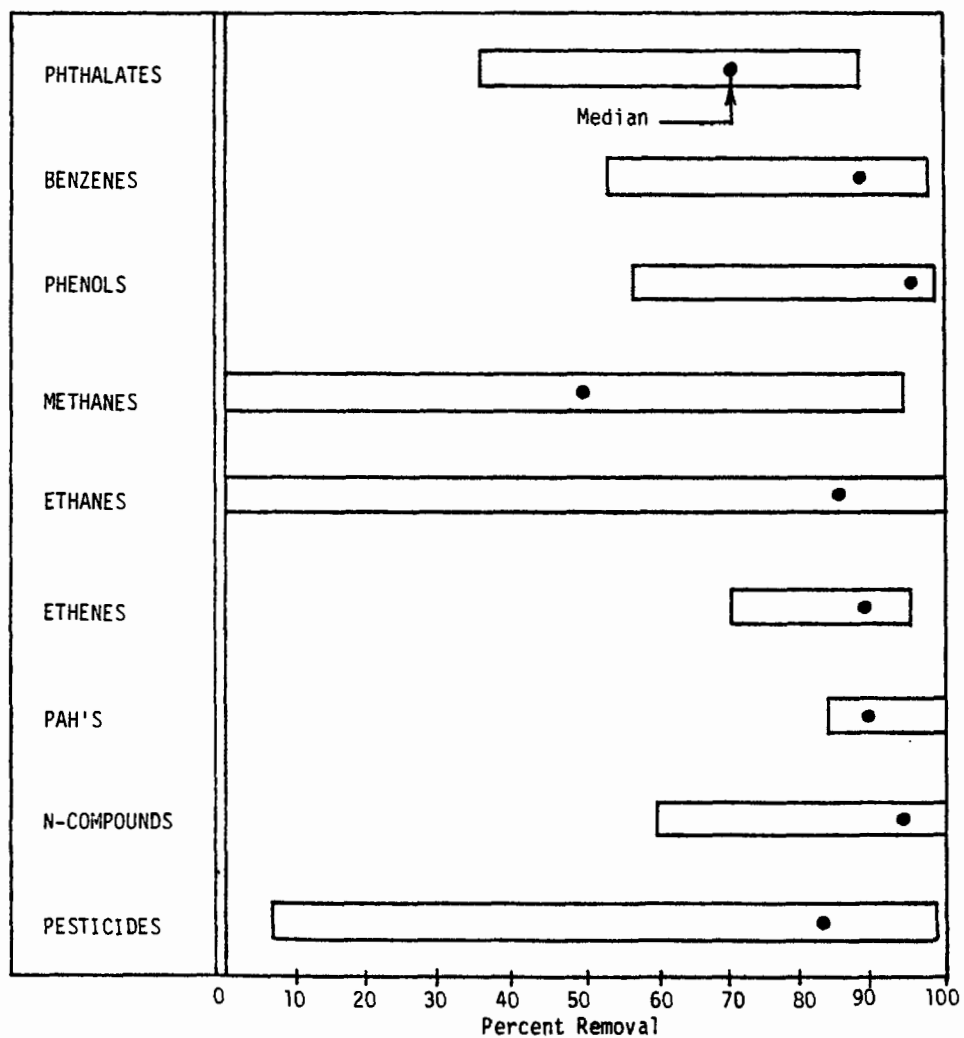


Figure 10. Interquartile Ranges of Percent Removals
for Classes of Priority Organics

The wide range in pesticide removals is most likely due to the low concentrations of pesticides near the analytical detection limits in wastewaters and the attendant analytical errors involved. The use of only 24-hour composites for calculation of percent removals does not necessarily reflect the long-term removals at the surveyed plants and will cause some scatter in the data. Median removals for benzenes, phenols, ethanes, ethenes, PAH's, nitrogen-containing-compounds, and pesticides were all greater than 80 percent. Of these, the ethenes and PAH's have the most consistent range of removals.

Figure 11 was prepared to determine if there was any relationship between the total concentrations of priority organics in the raw wastewaters and in final discharges. Final discharge concentrations were plotted against raw wastewater concentrations for 31 plant visits. The best fit curve through these data was found to be a power relationship with a coefficient of determination of only 0.3. Again, the fact that wastewater samples were only 24-hour composites with very different compositions from plant to plant would not lead one to expect a quantitative relationship between gross concentrations in and out of different POTWs. Several individual compounds were also tested but no relationship could be discerned between influent and effluent concentrations.

Because analysis for specific priority pollutant organics is both time-consuming and expensive, there have been a number of attempts to relate concentrations and removals of priority organics to other more common analytical parameters. Figure 12 shows a plot of percent priority organics remaining after treatment versus the percent COD remaining. As observed from the figure, COD removal is not a good predictor of priority organics removal. The coefficient of determination of the least squares power function fitted to these data was only 0.1.

Paired influent-effluent data from 34 samplings were classified as being from secondary plants (23 events) or from tertiary plants (11 events). The tertiary data were from eight separate plants consisting of either activated sludge, trickling filters, or lagoons followed by either trickling filters (1 plant), gravity filters (2 plants), lagoons (3 plants), RBC (1 plant), or land application (1 plant). On average, the percent reduction in number of compounds detected between the influent and effluent in secondary systems was 28 percent ($\sigma = 10\%$); for tertiary systems, 36 percent ($\sigma = 19\%$). This difference in reduction in numbers of compounds cannot be called significant at either the 5 percent or 10 percent confidence levels. In terms of total concentrations of priority organics, secondary plants had an average of 51 percent removal ($\sigma = 42\%$) while tertiary plants had an average of 67 percent removal ($\sigma = 44\%$). Again, because of the high variability between plants, it was not possible to call this difference in percent removal significant at either the 5 percent or 10 percent level.

Production of Chlorinated Organics

Where chlorination was practiced, effluent samples before and after chlorination were examined for both the total number and total concentration of potential chlorination products. These included 20 chlorinated aliphatics,

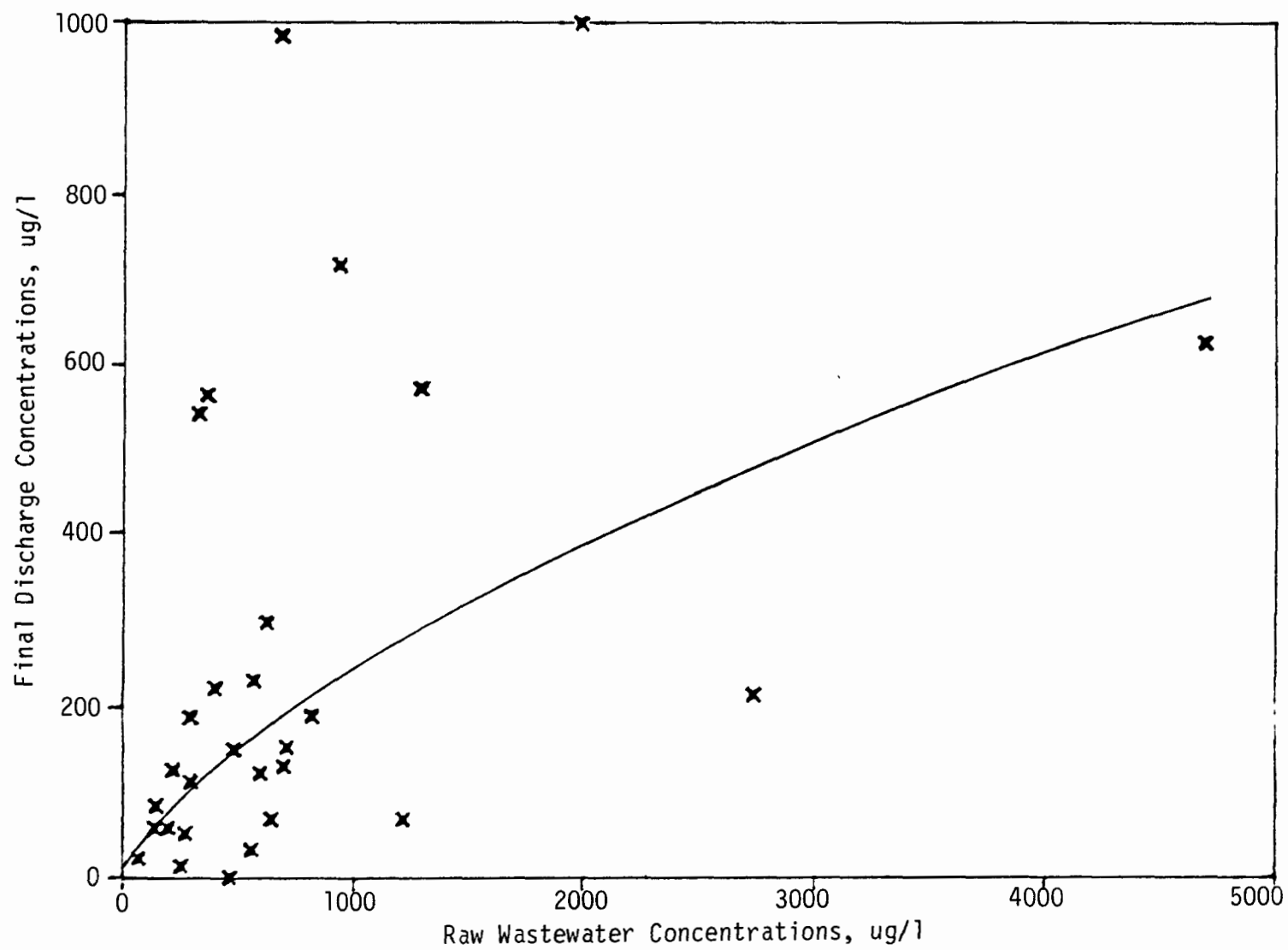


Figure 11. Final Discharge v. Raw Wastewater Concentration of Priority Organics

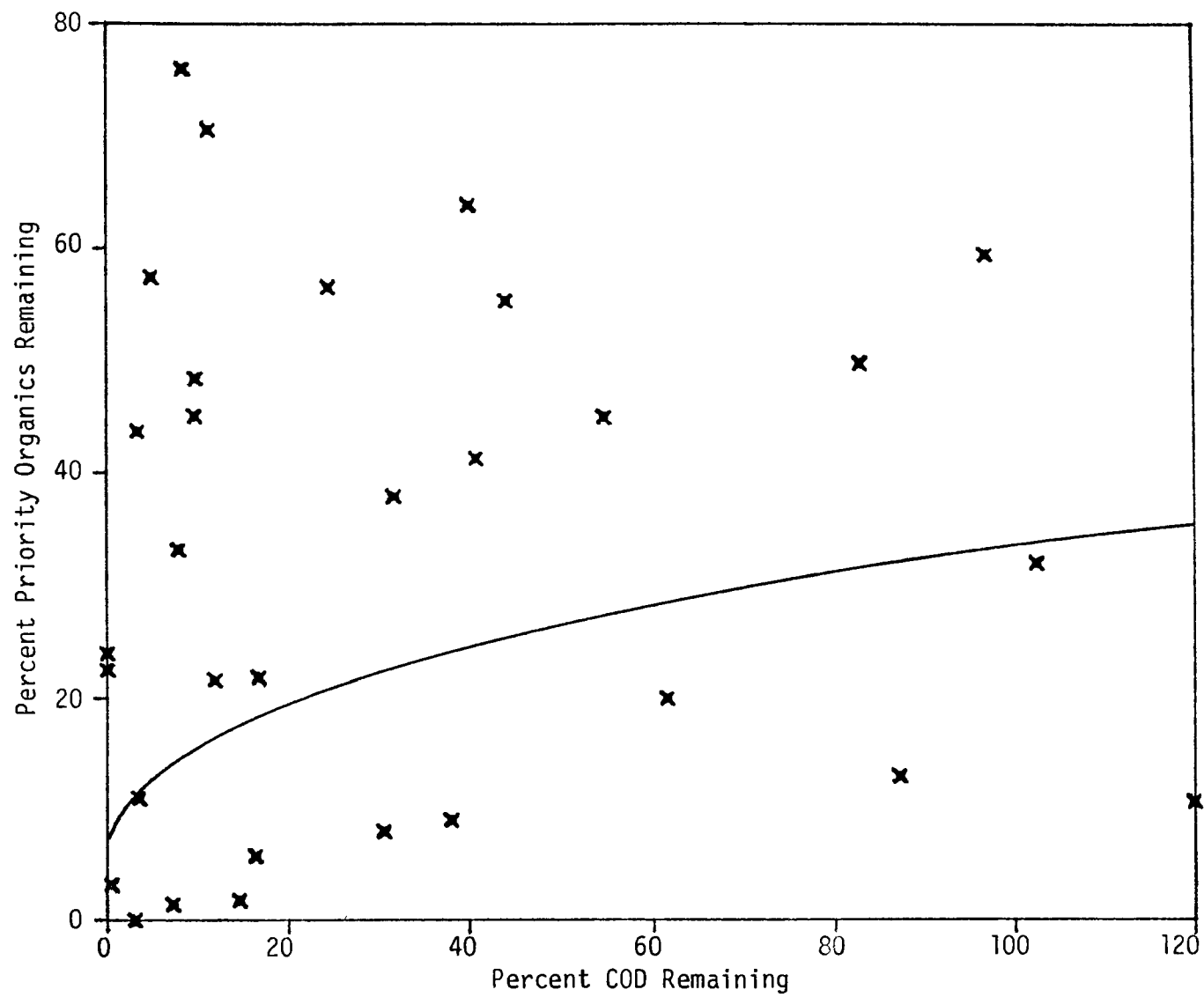


Figure 12. Percent Priority Organics Remaining v. Percent COD Remaining

6 chlorinated benzenes, and 4 chlorinated phenols. On average, 0.73 more compounds were detected after chlorination than before ($\sigma = 3.2$) and total concentrations of the selected compounds increased 68 ug/l ($\sigma = 218$). Given the variability of these results, it was not possible to discern at 5 percent or 10 percent confidence levels that chlorinated organics were being produced by chlorination.

Significance of Priority Organics in Final Discharges

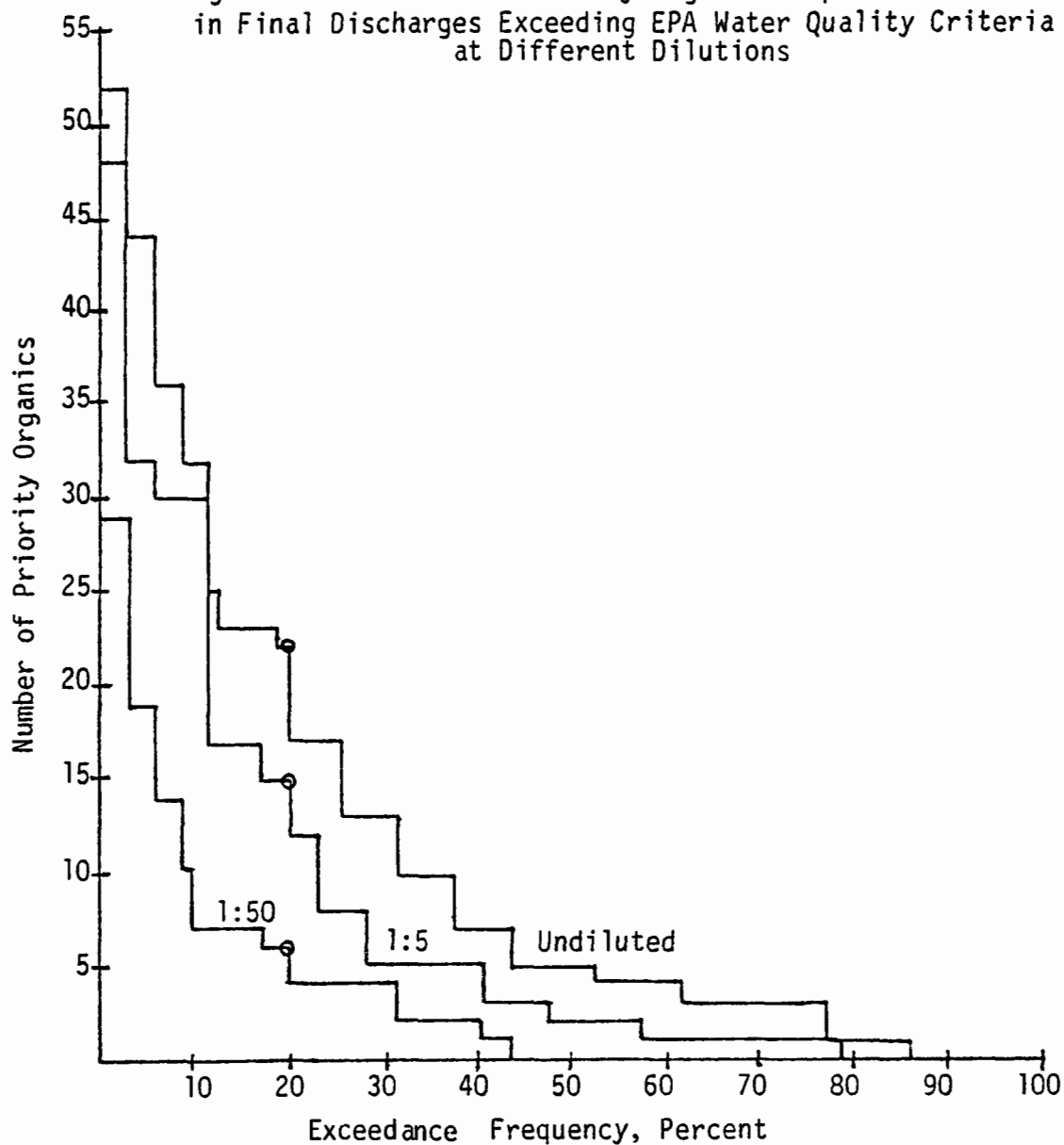
Concentrations of individual priority organics in final discharges were compared with proposed USEPA Water Quality Criteria (12) to evaluate their significance in terms of potential damage to human health or aquatic life. The specific criterion selected for comparison for each compound was the minimum of the:

- (a) 24-hr. maximum aquatic life criterion
- (b) Never-to-be-exceeded aquatic life criterion
- (c) Human health criterion for non-carcinogens
- (d) Human health criterion for carcinogens based on one induced cancer per 1,000,000 people

Comparisons were made with undiluted final discharges and at 1:5 and 1:50 dilutions with receiving waters assumed to contain no priority organics. Figure 13 shows the variation in number of priority organics exceeding water quality criteria a specific percent of the time at the given dilutions. For example, there are five compounds that exceed criteria 50 percent of the time with no dilution of final effluent, two compounds that exceed criteria 50 percent of the time at 1:5 dilution and no compounds that exceed criteria 50 percent of the time at 1:50 dilution.

Note that the number of compounds exceeding a criterion begins to rise sharply only when exceedance frequencies drop below about 20 percent. Using the latter frequency as a cut-off point, the compounds that are in exceedance at each dilution level at 20 percent or greater frequency are listed at the bottom of Figure 13. There were 22 such compounds in undiluted final discharges, 15 at 1:5 dilutions, and 6 at 1:50 dilutions. Compounds included in this list but not in Table 5 as the most frequently detected compounds are dichlorobromomethane, 2,4-dichlorophenol, 1,2 diphenylhydrazine and the pesticides. The pesticides appear here, even though they are present in final discharges at very low concentrations, because of the highly restrictive water quality criteria for those compounds.

Figure 13. Number of Priority Organic Compounds
in Final Discharges Exceeding EPA Water Quality Criteria
at Different Dilutions



Compounds with Exceedance Frequencies of 20% of More

<u>Undiluted</u>	<u>1:5 Dilution</u>	<u>1:50 Dilution</u>
3 Halogenated Methanes	3 Halogenated Methanes	4 Pesticides
2 Chlorinated Ethenes	8 Pesticides	Dichloro-
4 Phthalates	Tetrachloroethene	methane
9 Pesticides	Bis(2-Ethylhexyl)	Phenanthrene
Benzene	Phthalate	
2,4-Dichlorophenol	2,4-Dichlorophenol	
Phenanthrene	Phenanthrene	
1,2-Diphenylhydrazine		

SUMMARY AND CONCLUSIONS

1. Analytical methods have been developed that are capable of detecting priority pollutants in wastewaters and sludges at levels of 1 $\mu\text{g/l}$ (10 ng/l for pesticides). Extensive use of blank and recovery samples is required to produce reliable quantitative data.
2. The majority of plants in the survey had between 20 and 50 priority organics in their raw influents (at total concentrations under 1 mg/l) with only 10 to 30 compounds in their final discharges.
3. Only 15 compounds were found above 1 $\mu\text{g/l}$ (or 1 mg/kg) at least 80% of the time in raw wastewaters and at least 50% of the time in final discharges and sludges.
4. There was only a weak correspondence between the percent industrial flow to a plant and the number of priority organics found in the raw wastewater on a given day.
5. Concentration levels of classes of priority organics varied widely between the plants surveyed. Median percent removals of total benzenes, phenols, ethanes, ethenes, PAH's, nitrogen-containing compounds, and pesticides were all higher than 80%.
6. The influent concentration of a compound or class of compound on any given day was found to be a poor predictor of effluent concentration. Removal of priority organics was not strongly associated with COD removal.
7. The survey results could not statistically validate the hypotheses that chlorination produces more chlorinated priority organics or that tertiary treatment systems obtain higher removals of priority organics than secondary systems.
8. With regard to proposed water quality criteria it appears that some 22 compounds may be problematic in at least 20% of the systems surveyed. The most significant of these are dichloromethane, phenanthrene, and four pesticides.

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CONTROL OF SPECIFIC ORGANIC AND METAL CONTAMINANTS
BY MUNICIPAL WASTEWATER TREATMENT PROCESSES

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ABSTRACT

The U.S. EPA's Municipal Environmental Research Laboratory is assessing the removability of toxic substances from municipal wastewater by conventional wastewater treatment processes. The studies feature pilot-scale primary/ secondary treatment of the raw wastewater spiked with selected priority pollutants (metals and organics). In the studies, the treatment plant performance on spiked wastewater is compared to the performance of identical treatment on the unspiked raw wastewater. The assessment employs costly analyses (GC/MS and atomic absorption methods) for the selected toxic substances in the various process streams and sludges of the conventional treatment plant. A biomonitoring approach to assess health and ecosystem effects is also being evaluated to supplement the specific toxic substance removal data. From the studies to date, conventional treatment is generally effective in removing selected toxic substances, typically achieving better than 90% removal of organics and from 60-80% removal of the metals. A few of the toxic substances, however, pass through into the treatment plant's final effluent in sufficient concentrations which, based upon EPA recommended water quality standards, may present a possible environmental hazard.

INTRODUCTION

With the establishment of the Consent Decree list of priority pollutants (1), the U.S. Environmental Protection Agency (EPA) began assessing the occurrence of priority toxics entering the environment and the removability of these toxics by wastewater treatment systems. The occurrence and removal of the toxics at municipal wastewater treatment plants have been evaluated principally in two National surveys (2, 3). As National surveys with limited repetitive analyses at a given treatment plant, these surveys provide a qualitative perspective on the occurrence and removability of the priority pollutants for a wide range of U.S. municipal wastewaters.

Several factors, however, reduce the quantitative certainty of the survey data for removability and fate of the toxics during municipal treatment. These factors are:

- The highly variable and low concentrations of the specific compounds in the municipal wastewater.
- The inherent analytical variability associated with measurement of parts per billion concentrations of the specific toxic compounds in presence of substantial interfering background.
- Lack of proper experimental control systems in the municipal treatment plant.

The quantitative uncertainty from these factors can be minimized by the addition of spiked amounts of representative specific organic pollutants to the influent of the municipal treatment system, by providing sufficient repetitive measurements of the compounds of interest, and by using a parallel identical treatment system on the same wastewater for experimental control purposes.

The Municipal Environmental Research Laboratory of the EPA in co-operation with the EPA's Newtown Fish Toxicology Station is assessing the removability, the fate and partitioning of specific toxics, and the impact of the toxics during conventional municipal wastewater treatment at the EPA's new Test and Evaluation Facility in Cincinnati. The basic approach features the continuous spiking of individual toxics and groups of selected toxic pollutants into the raw wastewater from the Cincinnati Mill Creek Sewage Treatment Plant. The spiked wastewater enters pilot conventional wastewater treatment systems in which the fate and impact of the added toxics are repetitively monitored as they pass through the treatment processes. Other identical pilot systems, as the controls, treat the unspiked Cincinnati raw wastewater.

The ongoing studies include the evaluation of:

- The fate and removal of selected priority organic pollutants (4, 5).
- The fate and removal of indigenous metals (6).
- The impact and removal of selected spiked metals (7, 8, 9, 10).
- The removal of toxicity by the treatment system as measured by the reduction in acute toxicity (11).
- The occurrence of chronic toxicity to fathead minnow embryos in the treated effluents (12).

In the past, the municipal wastewater treatment plant has been designed and operated for removal of organic (BOD) loading and occasionally for nutrient control. This research provides data for developing design and operating criteria for municipal wastewater treatment to remove toxicity. It provides a perspective on the use of the municipal wastewater treatment plant as a cost-effective centralized alternative to industrial pretreatment for toxics removal.

EXPERIMENTAL SYSTEMS

The removal of specific toxics in municipal wastewater treatment occurs through three chief mechanisms (13):

- Adsorption or precipitation and removal with the solids in the treatment processes.
- Biodegradation
- Stripping of the volatile toxics during aeration

Ideally, it is desirable to assess the removal of toxics in full-scale wastewater treatment plants. However, this approach is not practical with spiking of toxics. In order to reduce the variability and increase the influent priority toxics concentration to levels where reasonable numbers of repetitive measurements produce statistically reliable results, the addition of suitable amounts of appropriate priority toxics, for periods approaching steady-state (three sludge retention times), can be very expensive. Full-scale spiking is also objectionable because of toxic discharges either in the plants effluent, in the plant's sludges or in the air from the plant's aeration processes.

Pilot Systems

As a practical approach for the work to date, we performed the assessment in small scale pilot systems (Figure 1 and Table 1) for the semi-volatile or non-volatile toxics (organics and metals) and in large pilot systems (Figure 2 and Table 1) with representative side water depths for volatile organics. The Cincinnati Millcreek Plant's raw wastewater was used for the studies. In the work, a total of four identical small pilot systems (Figure 1), and two identical large pilot systems (Figure 2) were used. The operation at steady flow of these municipal treatment systems was performed with continuous 24-hour operator supervision. The operating conditions for the various studies and the various spiked mixtures added to experimental treatment trains are described later in the subsequent discussions of the individual studies.

Cincinnati Wastewater

The raw wastewater entering the Cincinnati Mill Creek Sewage Treatment Plant by U.S. standards is a strong municipal wastewater (typical COD ~ 650 mg/l) from a collection system with significant industrial contributions. The substrate concentrations in the wastewater compared to those in the Cincinnati municipal water supply (Table 2) reveals the substantial increases in substrate caused by the municipal/industrial use in Cincinnati. These substrate increases compared to substrate increases in typical U.S. domestic wastewater indicate a strong industrial contribution. Thus, the raw wastewater, by itself, is well suited for assessing the use of the central municipal plant for control of toxics.

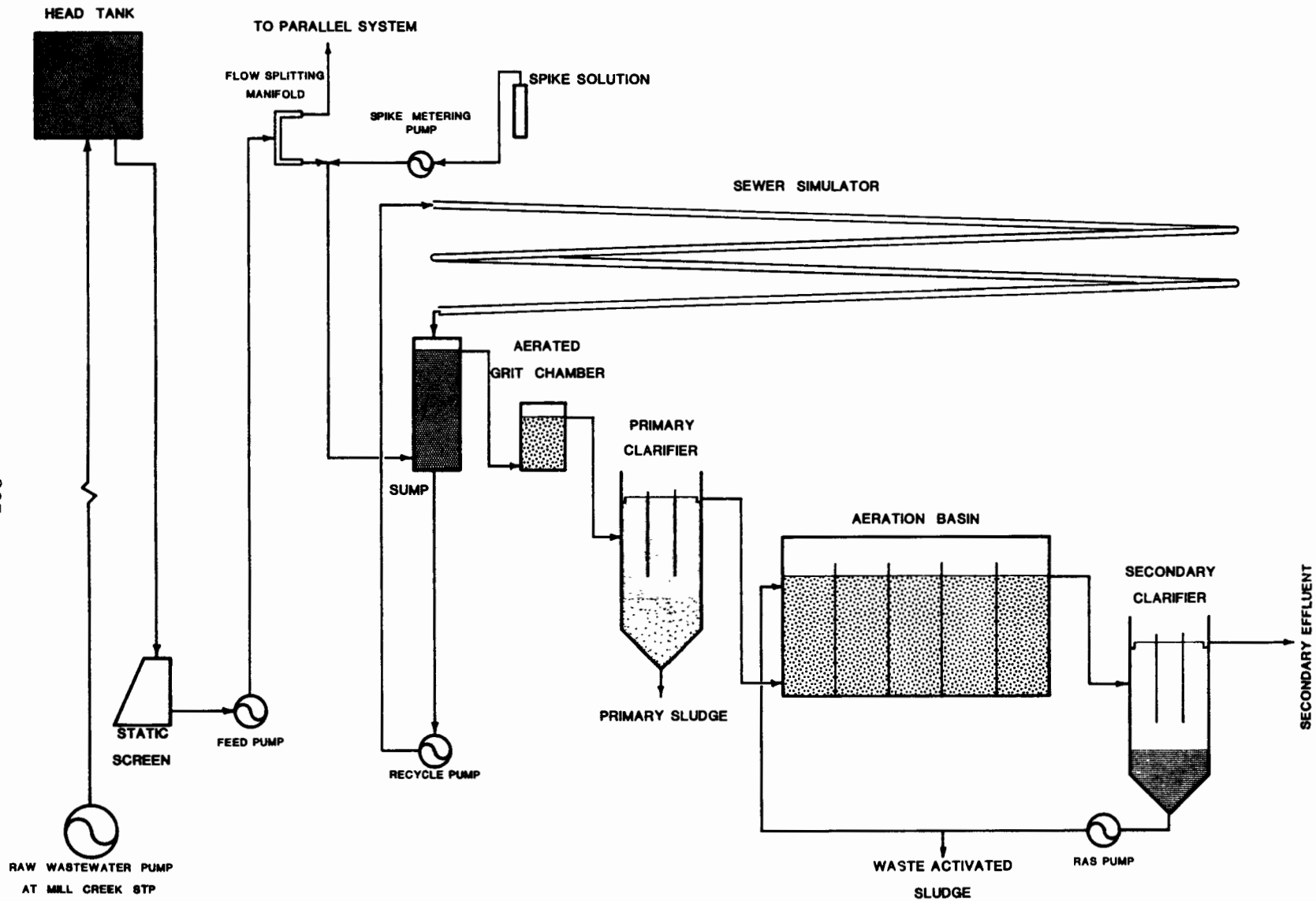


Figure 1. Pilot System for Metals and Semi-Volatile Organic Studies.

Table 1. Pilot System Specifications.

Pilot System	Metal and Semi-Volatiles ^(a)	Volatile Organics
Design flow, L/d	7570	190,700
Aerated grit chambers		-
diameter, m	0.55	-
side water depth, m	0.5	-
air rate, L/S	0.3	-
residence time, min.	20	-
Primary clarifiers		
diameter, m	0.91	2.94
side water depth, m	1.5	3.66
surface area, m ²		6.82
overflow rate, m ³ /m ² /d	12.4	24.2
Aeration basins		
width, m	0.61	3.05
length, m	3.05	5.36
sidewater depth, m	1.37	3.66
residence time, hr	7.5	7.5
Secondary clarifiers		
diameter, m	0.91	3.63
sidewater depth, m	1.5	3.66
surface area, m ²		10.36
overflow rate, m ³ /m ² /d	12.4	15.9

^a System include a sewer simulator (steel pipe) 25.6 meters long, 102 mm in diameter, with 3.15 L/S recycle pump and a 380 L sump.

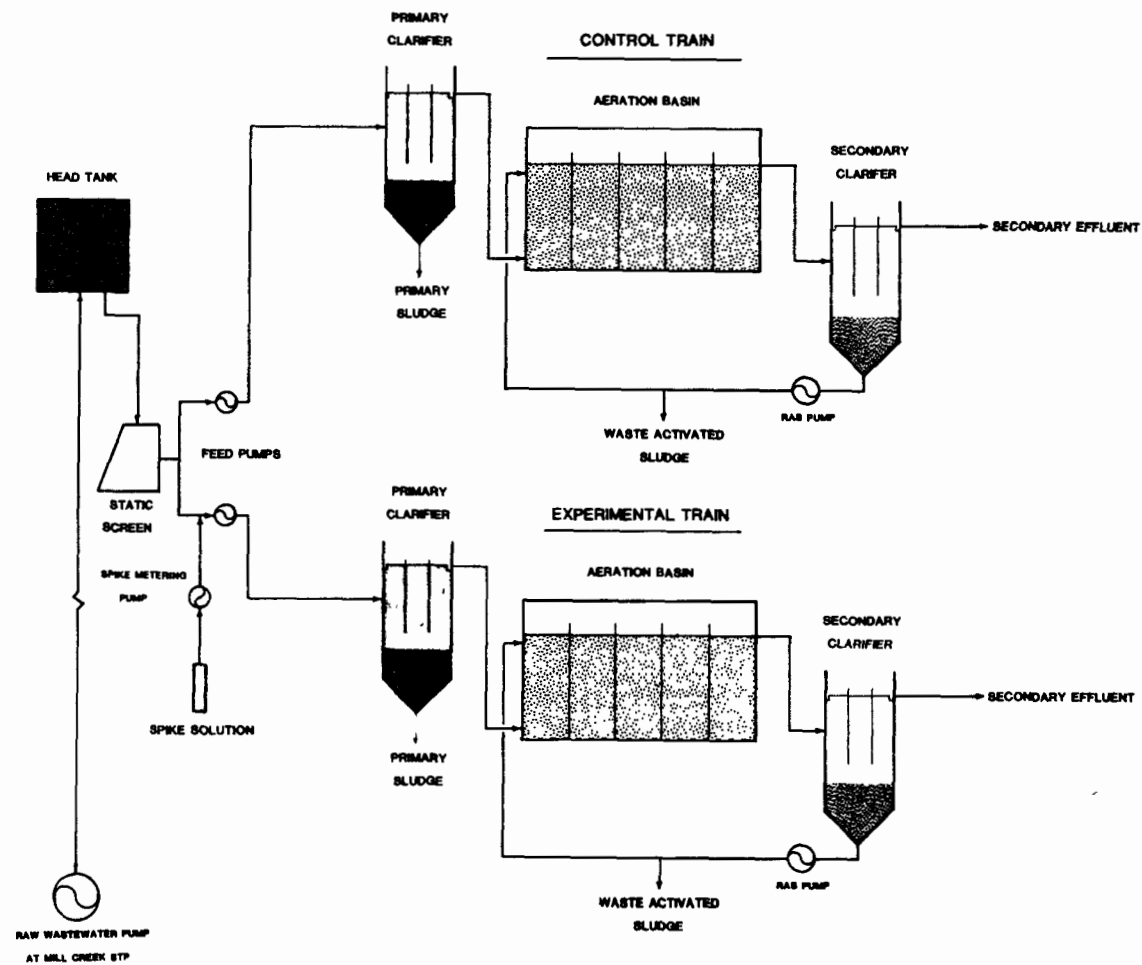


Figure 2. System for Volatile Organics Study.

Table 2. Increase in Substrate Concentration Due to Municipal/Industrial Use in Cincinnati.

Substrate	Influent Wastewater			Drinking Water mg/l	Increase mg/l
	N	Standard Deviation	Arith. Mean mg/l		
Ag ^(a)	37	13.0	8.0	0.00 ^(b)	< 8.0
As ^(a)	37	19.6	20.6	5 ^(b)	15.6
Ca	39	26.0	86.0	45	41
Cd ^(a)	37	14.7	20.9	0.000 ^(b)	20.9
Cr	37	0.47	0.63	0.002 ^(b)	0.63
Cu	37	0.38	0.80	-	-
Fe	37	2.58	4.29	0.30 ^(c)	3.99
Hg ^(a)	37	1.0	< 2.0	0.1 ^(b)	< 2
Mg	39	4.9	17.6	9.2 ^(c)	8.4
Mn	37	0.3	0.65	0.00 ^(c)	0.65
Ni	37	0.75	0.45	-	-
Pb	37	0.43	0.88	0.006 ^(b)	0.87
Zn	37	0.65	1.24	-	-
Cl	39	76.0	269	28 ^(c)	241
F	39	0.2	0.6	0.3 ^(c)	.3
SO ₄	39	63.0	288	83 ^(c)	205
SiO ₂	39	28.9	45.1	4-7 ^(c)	44.6
TDS	39	526	1537	251 ^(c)	1286

^a Micrograms/liter.

^b Average for 1978.

^c Average for 1974-1978.

NOTE: The average COD of the influent wastewater was 650 mg/l for October 1, 1979-August 8, 1980.

Analytical Procedures

During the studies, repetitive sets of appropriate composite (usually 24-hr) samples of the process flows and sludges were simultaneously collected for both the control and spiked treatment systems. The sample sets were analyzed at either the EPA laboratory operated by the Waste Identification and Analysis Section of the Municipal Environmental Research Laboratory or by Battelle-Columbus Laboratories. The EPA Laboratory provided all conventional analyses including metals using EPA methods (14) or Standard Methods (15). The specific organic analyses were shared by the EPA and Battelle Laboratories. In the organic analyses, the Laboratories used modifications of the Agency's standard procedures for wastewater (16) and interim procedures (17, 18) for sludges. The complexities involved in the sample work-up procedures and in GC/MS methodology for the analysis of the organic priority pollutants in municipal wastewaters have been reported by Bishop (19).

Additionally, appropriate grab samples were collected at 4-hour intervals for routine process control and included mixed liquor and return sludge respiration rates, effluent turbidities, settled sludge volumes (30-minute), pH, and alkalinity.

Quality Control Approach

In the earlier phases of the work, the EPA Laboratory employed limited QC procedures in its automated conventional analyses. These consisted of the required standard curves, check standards (5-10%), and blanks. Large numbers of repetitive samples provided the statistical support to the data. As the studies progressed, the Laboratory participated in the development of the Agency's prototype Sample File Control (SFC) system for data management and documentation of quality control. The QC results (precision and accuracy) for the conventional analyses are included in the evaluation report (20) of the prototype SFC system.

Due to the complexities associated with the sample work-up procedures for specific organics as well as the variability in results reported by other investigators, a comprehensive quality control protocol was employed in the studies for the organic priority pollutant analyses. The sample locations were evaluated for matrix effects based on conventional water quality parameters such as COD, TOC, $\text{NH}_3\text{-N}$, etc. The influent, the sewer simulator effluent, and the aerated grit chamber effluent all had essentially identical matrix characteristics for the purposes of the QC program. The four additional matrices identified were the primary clarifier effluent, the activated sludge effluent, the primary sludge, and the return activated sludge.

With each organic sampling set, appropriate samples were collected from the control treatment system for each of the five matrices identified. Two replicate samples were run as duplicate background blanks. Two other replicate samples received a quality control spike containing all of the compounds being studied. This scheme provided duplicates on both the background blanks and the quality control spiked samples for precision and

accuracy determinations in all matrices. All quality control samples were then subjected to the complete sample work-up and analytical protocol. The QC results are or will be summarized in the individual reports (4, 5) on the organic studies.

ORGANICS REMOVAL

Semi-Volatile Organics Study

Pilot Plant Operations

Routine sampling and data acquisition for the semi-volatile organics studies commenced on October 1, 1979, and the project was terminated on August 8, 1980. Table 3 presents a summary of the process operation for the 312-day study period. The influent flows to both systems had a mean variation of only 6.1 percent; the means being 0.083 l/s and 0.088 l/s for the control and spiked systems, respectively. The return activated sludge (RAS) flows averaged 0.035 l/s for the control train and 0.038 l/s for the spiked train, or about 42 percent of the influent flows for each system. The mixed liquor suspended solids and RAS concentrations were approximately 1900 mg/l and 6400 mg/l, respectively.

The systems were operated at a nominal SRT of 7 days, which resulted in a F/M of 0.6 kg COD applied per day per kg MLSS, or a F/M of 0.18 on a TOC basis. The average normalized oxygen uptake rates (OUR) for the mixed liquors and RAS's are shown in Table 3. A comparison of the operating data between the control and spiked treatment sequences indicates that both systems were operated in essentially the same manner.

Median water quality data for both treatment sequences are summarized in Table 4, and these data indicate that the pilot scale systems were providing good treatment. Total suspended solids, COD, and TOC reductions were 95, 89, and 90 percent, respectively. The activated sludge processes produced significant ammonia nitrogen reductions even though they were not operated to consistently nitrify.

Twenty-two semi-volatile organics spiked into the raw wastewater (Table 5) were selected from those organics found in the raw wastewaters of the first 20 of the municipal wastewater treatment plants from the 40 Cities Survey (2). A nominal spiking concentration of 50 µg/l for each single component compound was selected as representative of the concentrations found in the Survey. Since Arochlor 1254 and toxaphene were multicomponent mixtures, a nominal 150 µg/l of each mixture was selected to improve their quantitation by GC/MS.

The purity of each compound was verified by gas chromatography prior to preparation of the spiking solution. The selected semi-volatile organics were dissolved in toluene to provide a spiking solution containing 0.143 percent by weight of each of the single component compounds and 0.429 percent for the toxaphene and Arochlor 1254. As a quality control highlight, the recoveries of spikes of the individual semi-volatile organics into four system matrices are presented in Table 5. The recoveries

Table 3. Process Operation During Semi-Volatile Organic Study;
October 1, 1979 through August 8, 1980.

Parameter	Treatment Sequence	
	Control	Spike
Influent Flow, Q, l/s (gpm)	0.083 (1.31)	0.088 (1.39)
RAS ^(a) Flow, Q _r ; l/s (gpm)	0.035 (0.55)	0.038 (0.60)
Waste Activated Sludge, l/d (gpd)	169 (44.7)	158 (41.8)
Waste Primary Sludge, l/d (gpd)	34 (9.1)	37 (9.7)
MLSS, mg/l	1887	1929
RAS concentration, mg/l	6382	6398
Normalized ML-OUR, ^(b) hr ⁻¹	0.041	0.027
Normalized RAS-OUR, hr ⁻¹	0.018	0.018
SVI, ml/gm	111	66
SRT, days	6.7	7.7
F/M (COD), days ⁻¹	0.61	0.60
F/M (TOC), days ⁻¹	0.19	0.18

^a RAS is return activated sludge.

^b OUR is oxygen uptake rate; it is normalized by dividing the observed rate by the solids concentration in the sample.

Table 4. Plant Performance During Semi-Volatile Organics Study
October 1, 1979 through August 8, 1980

Parameter	Influent (mg/l)	Sewer Simulator Effluent (mg/l)	Grit Chamber Effluent (mg/l)	Primary Clarifier Effluent (mg/l)	Activated Sludge Effluent (mg/l)	Overall Removal (%)
- - - - - Control Sequence - - - - -						
TSS	490	505	408	265	26	95
COD	650	660	700	390	74	89
TOC	180	194	198	122	18	90
T-P	8.3	8.0	8.7	5.6	2.9	65
NH ₃ -N	20	19	19	19	1.5	93
NO ₂ and NO ₃ -N	.1	.1	.1	.1	3.6	-
Alkalinity (as CaCO ₃)	-	-	290	300	188	-
- - - - - Spiked Sequence - - - - -						
TSS	430	505	490	257	26	94
COD	640	640	670	365	76	88
TOC	180	185	190	114	19	90
T-P	8.1	7.9	8.6	5.6	2.7	67
NH ₃ -N	20	17.3	17.3	18.3	.9	96
NO ₂ and NO ₃ -N	.1	.1	.1	.1	5.2	-
Alkalinity (as CaCO ₃)	-	-	290	290	170	-

Table 5. Mean Recovery Values for Various Sample Locations and Classes of Compounds.

	Reported ^(a) by Kleopfer, et al. (21)	Average Recovery ^(b) for Eight Sample Sets			
		Inf.	Act. Sludge Eff.	Pri. Sludge	Return Act. Sludge

<u>PESTICIDES/PCB's</u>					
Arochlor 1254	42	54	74	61	53
Heptachlor	49	71	98	28	68
Lindane	64	60	71	32	72
Toxaphene	83	60	64	66	74
Mean Recovery	60	61	77	47	67
<u>PHENOLS</u>					
2,4-dimethylphenol	72	60	54	44	17
Phenol	54	72	68	28	31
Pentachlorophenol	84	74	120	49	79
Mean Recovery	70	69	81	74	42
<u>PHTHALATES</u>					
Bis(2-ethylhexyl)phthalate	66	86	76	34	50
Butylbenzylphthalate	49	70	58	25	62
Diethylphthalate	65	77	70	69	31
Dimethylphthalate	66	73	62	56	15
Di-n-butylphthalate	58	74	70	57	68
Di-n-octylphthalate	88	65	68	31	62
Mean Recovery	65	74	67	45	48
<u>POLYNUCLEAR AROMATIC HYDROCARBONS</u>					
Acenaphthene	78	79	77	71	58
Anthracene	79	64	71	45	60
Benz(a)anthracene	51	59	55	48	35
Chrysene	77	48	71	63	56
Fluoranthene	63	62	58	64	58
Fluorene	88	79	81	75	59
Naphthalene	89	66	84	25	41
Phenanthrene	79	71	65	67	60
Pyrene	68	60	61	62	52
Mean Recovery	75	65	69	58	53

^a Industrial and municipal wastewaters.

^b Recovery expressed as percent.

in the wastewater matrices are comparable to those reported by Kleopfer (21) as representative of reasonable analytical performance.

The Cincinnati raw wastewater with a high average COD of 650 provides a stronger background of extractable interferences than typical municipal wastewaters. This wastewater produced sludges, especially primary sludges, which were intermittently very difficult to handle, especially during the extraction process. As expected the recoveries in the sludge matrices were generally lower than those encountered in the wastewater matrices.

Semi-Volatile Organics Removal

The results of the analyses for the organic priority pollutants in the control and spiked systems are presented in Tables 6 and 7. The data reported are the arithmetic means for all eight sample sets and all concentrations are in micrograms per liter. Additionally, all concentrations used for computing the mean concentration were corrected for recovery factors determined from the quality control spikes for each sample set.

Most of the selected organics in the unspiked raw wastewater were found at or near the detection limits (1 to 10 $\mu\text{g/l}$) of the analytical methods. Phenol, several phthalates and naphthalene were observed in relatively substantial concentrations in the unspiked raw wastewater. Spiking, substantially increasing the concentrations of the organics in the wastewater, permitted evaluation of the removals across the plant.

In general, the concentrations of the spiked organic priority pollutants found in the effluent of the primary clarifier (Table 7) were slightly higher than the influent concentrations. Even a cursory examination of the data for the primary sludge samples indicates that considerable removal of these materials did occur in the primary clarifier. The inconsistency between the influent and primary effluent data can be understood when one realizes that the standard deviations for both the influent and the primary effluent samples typically ranged from 50 to 100 percent, which is normal for GC/MS quantitation at low concentrations.

A comparison of the influent and activated sludge effluent data of the spiked system indicated that the treatment sequences were generally effective in reducing the concentrations of the organic compounds in the wastewater streams. The spiked system typically produced a more than 97 percent reduction in the concentrations of the compounds being spiked. Most of the residual concentrations of the chemicals were below the detection limits in the activated sludge effluent. Lindane, bis-(2-ethylhexyl)-phthalate, di-n-octylphthalate pentachlorophenol, and phenol were found in analytically significant concentrations (< 4.8 to $25.8 \mu\text{g/l}$) in the secondary effluent.

Substantial concentration increases occurred in both the primary and return activated sludges. Typically a two order of magnitude increase in concentration, based on the influent values, occurred in the primary sludge samples. Concentration increases observed in the return sludge ranged from 0.5 to 1.5 orders of magnitude depending on the specific compound.

Table 6. Mean Concentrations of Semi-Volatile Organics in the Control Treatment System.

	Influent ($\mu\text{g/l}$)	Primary Effluent ($\mu\text{g/l}$)	Activated Sludge Effluent ($\mu\text{g/l}$)	Primary Sludge ($\mu\text{g/l}$)	Activated Sludge ($\mu\text{g/l}$)
<u>PESTICIDES/PCB's</u>					
Arochlor 1254	< 2.9	< 2.9	< 2.6	< 403.0	< 200.0
Heptachlor	< 1.0	< 1.0	< 1.0	< 52.9	< 58.7
Lindane	< 2.0	< 1.0	< 1.0	< 146.0	< 88.2
Toxaphene	< 2.9	< 2.9	< 2.6	< 1,063.0	< 97.9
<u>PHENOLS</u>					
2,4-dimethylphenol	< 17.5	< 13.6	< 0.9	< 57.9	< 197.0
Phenol	111.0	46.0	< 8.2	983.0	< 35.5
Pentachlorophenol	8.9	< 5.7	< 1.8	853.0	< 52.4
<u>PHTHALATES</u>					
Bis(2-ethylhexyl)phthalate	63.1	30.5	6.4	6,384.0	928.0
Butylbenzylphthalate	< 7.2	< 5.6	< 4.4	2,841.0	< 294.0
Diethylphthalate	< 7.1	< 12.3	< 1.0	< 75.8	< 180.0
Dimethylphthalate	< 3.2	< 17.5	< 0.7	< 13.6	< 12.6
Di-n-butylphthalate	16.0	< 11.8	< 2.4	1,255.0	< 190.0
Di-n-octylphthalate	< 6.3	< 3.6	< 2.2	< 770.0	< 13.9
<u>POLYNUCLEAR AROMATIC HYDROCARBONS</u>					
Acenaphthene	< 1.8	< 2.9	< 1.0	< 121.0	< 53.0
Anthracene	< 5.4	< 2.7	< 1.0	< 560.0	< 89.6
Benz(a)anthracene	< 0.8	< 0.8	< 0.8	< 292.0	< 59.1
Chrysene	< 5.1	< 2.1	< 0.8	431.0	< 132.0
Fluoranthene	< 2.1	< 1.3	< 0.6	< 488.0	< 84.1
Fluorene	< 2.4	< 2.1	< 0.7	< 175.0	< 46.8
Napththalene	< 95.4	< 74.7	< 1.0	3,583.0	< 37.5
Phenanthrene	8.8	< 4.5	< 0.9	< 646.0	< 69.0
Pyrene	< 1.8	< 1.3	< 0.7	< 844.0	< 110.0

Table 7. Mean Concentrations of Semi-Volatile Organics in the Spiked Treatment System.

	Influent (µg/l)	Primary Effluent (µg/l)	Activated Sludge Effluent (µg/l)	Primary Sludge (µg/l)	Return Activated Sludge (µg/l)
<u>PESTICIDES/PCB's</u>					
Arochlor 1254	< 33.5	< 114.0	< 2.9	13,500.4	5,403.0
Heptachlor	31.7	< 28.5	< 2.3	< 2,152.0	526.7
Lindane	45.5	< 41.8	25.8	< 1,130.3	< 173.7
Toxaphene	< 47.4	< 87.5	< 2.9	< 8,213.1	< 1,655.4
<u>PHENOLS</u>					
2,4-dimethylphenol	< 82.1	60.9	< 0.9	< 20.7	< 20.0
Phenol	261.3	> 196.2	< 13.5	< 2,348.3	< 92.1
Pentachlorophenol	7.6	13.0	< 6.3	< 410.7	< 20.0
<u>PHTHALATES</u>					
Bis(2-ethylhexyl)phthalate	51.7	52.4	11.3	< 6,713.0	978.0
Butylbenzylphthalate	33.5	37.5	< 1.3	< 8,160.0	< 123.3
Diethylphthalate	46.4	57.7	< 1.2	< 710.3	< 196.7
Dimethylphthalate	< 41.8	< 37.2	< 0.8	< 37.2	< 39.5
Di-n-butylphthalate	43.8	54.4	< 2.7	3,482.4	< 233.8
Di-n-octylphthalate	28.2	< 34.4	< 4.8	< 5,278.0	< 580.7
<u>POLYNUCLEAR AROMATIC HYDROCARBONS</u>					
Acenaphthene	39.8	53.6	< 1.2	3,354.0	< 68.3
Anthracene	34.8	33.9	< 0.9	4,809.8	< 84.6
Benz(a)anthracene	23.8	24.9	< 0.6	< 3,241.5	< 208.9
Chrysene	38.9	36.6	< 1.2	5,982	< 240.9
Fluoranthene	30.6	39.9	< 1.9	5,281.0	< 196.0
Fluorene	37.9	51.6	< 0.7	< 3,921.0	< 57.9
Napththalene	76.7	242.5	< 0.7	< 3,463.0	< 18.3
Phenanthrene	40.4	44.3	< 1.1	< 4,931.0	< 28.4
Pyrene	30.4	39.1	< 2.0	< 6,640.0	< 104.2

Mass distributions (Table 8) were computed for each compound based on the mean concentrations observed in the influent, secondary effluent, primary sludge, and return activated sludge, and the operating parameters presented in Table 3. Since most of the compounds studied are biodegradable to some extent, the probability of accounting for 100 percent of any given chemical was expected to be low; additionally, good accounting of the chemicals in the distribution balances was not anticipated due to the inherent variability in the GC/MS data. The results of the distribution calculations for the spiked system, in general, are much better than anticipated; the total percentages of the different compounds which were accounted for are reasonable.

The pesticides and PCB's partitioned approximately equally between the primary sludge and the waste activated sludge. A substantial portion of lindane, 55 percent, was found in the activated sludge effluent. The three phenols studied were not found to concentrate in either of the sludge streams. These data indicate that two of the phenols studied are relatively biodegradable. Pentachlorophenol substantially passed through the treatment plant.

On a mass flow basis the phthalates were unevenly distributed between the two sludge streams; more of the compounds were found in the primary sludge. The results indicate that diethyl and dimethylphthalate are more biodegradable than the other compounds studied in that class. Since these two compounds have the simplest structure, this finding is not surprising.

The PAH's are the least polar of all the compounds. One would, therefore, assume their preferential adsorption to the solids, which are removed in primary treatment. As a class, the PAH's concentrated in the primary sludge to the greatest degree. In contrast, only low amounts of the PAH's were found in the WAS samples.

This research did demonstrate that a typical POTW, with the processes studied, significantly reduced the concentrations of the 22 organic priority pollutants; however, certain compounds, most notably lindane, bis(2-ethylhexyl)phthalate, phenol, and di-n-octylphthalate were present in the activated sludge effluent in relatively significant concentrations. The impact of these low-level residuals on the aquatic environment would be a function of many site-specific factors, such as ambient water quality and dilution flows. However, based on the potential for bioaccumulation (22), toxicity data reported in the literature (23), suggested water quality criteria (24), and the presence of the materials in the secondary effluent, one can only conclude that the POTW is not a totally effective system for controlling the entry of some compounds into the environment via the wastewater discharge. Furthermore, although some compounds were biodegraded, many of the chemicals studied were present in the sludges in very high concentrations. The fate of these materials in the solids handling processes is not known at this time, and additional research must be conducted to provide answers to this pressing question.

Table 8. Distribution of Semi-Volatile Organics in the Spiked Treatment System.

	Influent (gm/day)	Percent in Primary Sludge	Percent in Waste Activated Sludge	Percent in Final Effluent	Total Mass Recovered (percent)
<u>PESTICIDES/PCB's</u>					
Arochlor 1254	0.977	51	60	2	113
Heptachlor	0.240	33	35	7	75
Lindane	0.345	12	8	55	75
Toxaphene	0.977	31	27	2	60
<u>PHENOLS</u>					
2,4-dimethylphenol	0.622	0	0.5	1.1	1.6
Phenol	1.979	4.4	0.8	5	10.2
Pentachlorophenol	0.058	26	5	81	112
<u>PHTHALATES</u>					
Bis(2-ethylhexyl)phthalate	0.392	63	39	21	123
Butylbenzylphthalate	0.254	119	7	4	130
Diethylphthalate	0.352	7	9	3	19
Dimethylphthalate	0.317	0.3	2	2	4.3
Di-n-butylphthalate	0.332	39	11	6	56
Di-n-octylphthalate	0.214	91	43	17	151
<u>POLYNUCLEAR AROMATIC HYDROCARBONS</u>					
Acenaphthene	0.302	41	4	3	48
Anthracene	0.264	67	5	3	75
Benz(a)anthracene	0.180	67	18	2	87
Chrysene	0.295	75	13	3	91
Fluoranthene	0.232	84	13	6	103
Fluorene	0.287	51	3	2	56
Napththalene	0.581	22	0.5	1	23.5
Phenanthrene	0.306	59	1	3	63
Pyrene	0.230	107	7	6	120

Volatile Organic Study

Pilot Plant Operations

The study on the volatile organics is ongoing. The results presented represents a preliminary assessment of the available data. Quality control refinement has not been applied to the data. The operations of the two pilot systems (Figure 2) for the volatile studies are summarized in Table 9. The two systems exhibited stable and straight-forward operation. The water quality performance for the treatment sequences (Table 10) indicated the pilot systems provided good treatment. The operations data and performance in both control and spiked systems indicate essentially the same operation for both systems.

The 16 volatile organics in the initial phase of the work were nominally spiked into the experimental treatment system at 50 µg/l. Higher spiking concentrations will also be employed. Organic analysis, in addition to the usual wastewater and sludge process streams, will include gas phase analyses of the air streams from the aeration basin.

Volatile Organics Removal

The selected volatile organics (Table 11) are usually present in the Cincinnati raw wastewater in analytically measurable quantities (0.2 µg/l). The spiking substantially increased the concentrations of the organics to improve the evaluation of the removability by the treatment plant.

The initial evaluation of the data (Table 12) reveals excellent removals for most of the purgeable organics with 90 percent or better removals. Two organics, 1,1,2-Trichloroethane and Dibromochloromethane exhibited relatively low removals of approximately 70 percent with substantial residuals in the secondary effluent. Toluene, Ethylbenzene, the combination of tetrachloroethylene and tetrachloroethane, and 1,2-Dichloropropane, while exhibiting nearly 90 percent removals or better, passed through the treatment system into the secondary effluent with concentrations greater than 2 µg/l. The ongoing work will provide further evaluation of the fate and distribution of these organics.

METALS REMOVAL AND IMPACT

The metals studies involved two separate operational periods; an evaluation of the distribution and removal of indigenous metals in the Cincinnati raw wastewater by the control treatment system (6) during the semi-volatile organics studies; and subsequent studies in which four specific metals--Cd (7), Pb (8), Hg (9), and Cr (10)--were spiked into the small pilot systems (Figure 1) in a sequence of increasing concentrations of the individual metal. The studies on the individually spiked metals included an unspiked control system in order to compare the impacts of the increasing metal concentrations with the conventional plant operation and performance.

Table 9. Operations Summary, Volatile Priority
Pollutant Sequence; January-June 1981

	System	
	Control	Spiked
Influent Flow, Q (gpm)	38.4	34.7
Flow, Q_r (gpm)	8.5	8.4
Waste AS, Q_w (gpd)	1,472	1,471
Primary Sludge, Q_p (gpd)	897.0	1,219
OUR ^a - ML (mg-hr/l)	37.8	47.8
OUR - RAS (mg-hr/l)	96.6	87.2
Normalized ML OUR	0.015	0.016
Normalized RAS OUR	0.006	0.005
MLSS (mg/l)	2,694	2,940
RAS (mg/l)	12,954	13,326
ML Cent. Vol. (%)	3.4	3.7
RAS ^b Cent. Vol. (%)	17.3	15.8
SRT (days)	4.6	5.1
SVI (ml/gm)	114.0	111.0

a = Oxygen Uptake Rate (OUR).

b = Return Activated Sludge (RAS).

TABLE 10. Performance of Volatile Priority Pollutant
Treatment Sequences; January-June 1981

Parameter	Influent (mg/l)	Primary Effluent (mg/l)	Removal by Primary Clarifier (%)	Activated Sludge Eff. (mg/l)		Overall Removal (percent)	
				Control	Spike	Control	Spike
TSS	447.0	214.0	52.0	30.0	23.0	93.0	95.0
COD	557.0	317.0	45.0	91.0	87.0	84.0	85.0
Total-P	9.3	6.0	35.0	3.1	2.8	67.0	70.0
TKN	43.5	36.7	16.0	19.4	18.4	55.0	58.0
Organic N	20.4	14.2	30.0	5.7	5.2	72.0	75.0
NH ₃ -N	23.1	22.5	3.0	13.2	13.2	43.0	43.0
NO ₂ & NO ₃ -N	0.2	0.2	-	6.4	6.3	-	-
Total-N	43.7	36.9	16.0	25.8	24.7	41.0	43.0
Turbidity (NTU)	-	-	-	12.0	10.0		
UCOD(a)	683.0	421.0	38.0	152.0	148.0	78.0	78.0

^a UCOD = Ultimate Combined Oxygen Demand = COD + 4.6 (NH₃-N).

Table 11. Mean Concentrations^(a) Observed in Eight Sample Sets:
January 12 through July 28, 1981.

	Control System					Spiked System				
	Inf.	Pri. Eff.	Activ. Sludge Eff.	Return Activ. Sludge	Pri. Sludge	Inf.	Pri. Eff.	Activ. Sludge Eff.	Return Activ. Sludge	Pri. Sludge
Methylene Chloride	47.1	33.2	5.8	2.0	32.3	84.3	76.0	1.0	1.2	111.4
1,1-Dichloroethene	0.3	< 0.2	< 0.2	< 1.0	< 1.0	43.5	9.6	< 0.2	< 1.0	0.7
Chloroform	11.3	7.4	0.5	7.2	3.5	45.8	37.2	1.4	3.6	9.7
Carbon Tetrachloride	3.6	4.0	< 0.2	< 1.0	< 1.0	20.3	6.7	0.3	< 1.0	0.6
1,2-Dichloropropane	< 0.2	< 0.2	< 0.2	1.8	< 1.0	55.6	52.7	2.3	< 1.0	63.2
Trichloroethylene	4.3	1.5	< 0.2	< 1.0	9.2	36.5	32.7	1.2	< 1.0	263.3
1,1,2-Trichloroethane	1.5	1.0	1.2	16.6	2.9	62.0	50.8	22.5	17.4	18.9
Dibromochloromethane	0.7	0.3	< 0.2	1.9	< 1.0	42.0	31.2	12.1	1.3	2.9
Benzene	0.5	0.8	< 0.2	< 1.0	14.4	35.0	16.5	0.2	< 1.0	224.3
1,1,1-Trichloroethane	83.0	32.7	1.7	2.4	41.9	254.3	63.2	1.5	< 1.0	40.3
Bromodichloromethane	< 0.2	< 0.2	< 0.2	< 1.0	< 1.0	19.7	19.8	0.9	< 1.0	0.2
Chlorobenzene	57.0	36.5	< 0.2	1.8	702.5	169.3	126.3	0.3	1.5	736.7
Tetrachloroethylene and Tetrachloroethane	19.3	10.0	< 0.2	6.8	161.6	71.7	74.2	3.8	5.6	570.9
Toluene	114.0	98.6	18.1	122.8	591.3	162.7	158.6	13.7	2.9	627.9
Ethylbenzene	19.5	27.3	< 0.2	26.0	522.6	41.8	39.7	4.9	3.6	570.7

^a All concentrations in µg/l.

Table 12. Distribution of Volatile Priority Pollutants
In the Spiked Treatment Sequence

	Percent Found in Primary Sludge	Removal by Primary Clarifier (percent)	Percent Found in Waste Act. Sludge	Removal by ^a Treatment Sequence
Methylene Chloride	3.2	9.9	0.04	98.9
1,1-Dichloroethene	0.04	77.9	0.07	> 99.5
Chloroform	0.52	18.7	0.23	99.4
Carbon Tetrachloride	0.08	66.9	0.16	98.7
1,2-Dichloropropane	2.76	5.2	0.06	96.1
Trichloroethylene	17.54	10.4	0.09	97.0
1,1,2-Trichloroethane	0.74	8.1	0.83	65.6
Dibromochloromethane	0.16	25.7	0.09	72.7
Benzene	15.63	52.9	0.09	99.4
1,1,1-Trichloroethane	0.39	-	0.01	99.4
Bromodichloromethane	0.02	NR	0.16	95.7
Chlorobenzene	10.62	25.4	0.03	99.8
Tetrachloroethylene and Tetrachloroethane	19.42	NR	0.23	95.0
Toluene	9.42	2.5	0.06	97.1
Ethylbenzene	33.29	5.1	0.25	88.9

a = Removals based on mass balance.

NR = Not removed.

Distribution and Removal of Metals

The pilot plant operating conditions (Table 3) and overall plant performance (Table 4, control system) are the same as those in semi-volatile organics study. The pilot system exhibited very stable operations and very satisfactory plant performance with the background metals in the influent wastewater.

The metals concentrations in the Cincinnati raw wastewater (Table 2) is typical of U.S. cities with substantial industrial contributions. Comparison to the mean influent metals in Washington, D.C. (25) and Dallas, Texas (26) revealed significantly higher metals concentrations. In many cases (Zn, Pb, Mn, Cu, and Cr), the concentration difference exceeded one order of magnitude. The Cincinnati metals concentrations, however, are all less than the maximums reported for twenty municipal treatment plants (2).

The metals concentrations for plant process streams and the primary and secondary sludges, except for the soluble Cu and Mg and Hg (concentration below or near the detection limit), were presented as log normal probability distributions. Summary highlights of the data analyses for this metals overview work are presented in Tables 13, 14, and 15.

Table 13 summarizes the metals removals, computed from the mean concentrations, for the primary clarifiers, activated sludge process, and the overall system. As one would predict, there was no significant removal of either Ca or Mg. The ambient Hg concentrations were too low to permit the quantitation of removal in a proper manner. The total removals for the remainder of the metals range from 19 percent for As to 80 percent for Cu, and are representative of typical plant removals.

Metals concentration factors are presented for the primary and return activated sludges in Table 14. The concentration factor was computed by dividing the mean sludge concentration (mg/l basis) by the mean influent concentration.

Mass balance calculations were performed using the mean contaminant concentrations and the pilot plant operating data presented in Table 4. The results of the mass balances are summarized in Table 15. For the 13 cations reported the average closure on the mass balance was 94.2 percent for the total system, and 94.0 percent on the primary clarifier. Cd and Fe were the two metals with the worst closures. Examination of the Cd data indicates that the concentration in the waste activated sludge is probably low, while the problem with iron appears to be low concentrations in both sludges.

Metals Impact

Soluble salts of the metals Cd, Pb, Hg, and Cr(+6) were added to the small pilot systems (Figure 1) and the spiking concentration periodically increased to ultimately stress the treatment process to failure. A parallel control system without the spike was operated identically during each metal study. The operating conditions at each metal concentration level were maintained relatively constant, but were varied during the course of the

Table 13. Metals Removal in Control Treatment Systems.

Metal	Influent (mg/l)	Pri. Eff. (mg/l)	Removal by Pri. Clar. (percent)	Act Sludge Eff. (mg/l)	Removal by Act. Sludge (percent)	Total Removal (percent)
Ag ^(a)	8.0	9.0	0.0	5.0	44.0	37.5
As ^(a)	20.6	16.0	22.3	16.7	0.0	18.9
Ca	86.0	81.0	5.8	81.0	0.0	5.8
Cd ^(a)	20.9	17.0	18.7	7.9	53.5	62.2
Cr	0.63	0.51	19.0	0.34	33.3	46.0
Cu	0.80	0.57	28.8	0.16	71.9	80.0
Fe	4.29	2.44	43.1	1.01	58.6	76.5
Hg ^(a, b)	2.0	2.0	-	2.0	-	-
Mg	17.6	18.0	0.0	17.9	0.0	0.0
Mn	0.65	0.55	15.4	0.40	27.3	38.5
Ni	0.45	0.27	40.0	0.18	33.3	60.0
Pb	0.88	0.58	34.1	0.11	81.0	87.5
Zn	1.24	1.28	0.0	0.46	64.1	62.9

^a Micrograms/liter.

^b The Hg concentration is near the detection limit for the metal and removals cannot properly be calculated.

Table 14. Metals Concentration Factors into Sludges
in Control Treatment System.

Metal	Inf. (mg/l)	Return Act. Sludge (mg/l)	RAS Conc. Factor	Primary Sludge (mg/l)	P.S. Conc. Factor
Ag ^(a)	8.0	117	14.6	179	22.4
As ^(a)	20.6	156.6	7.6	114	5.5
Ca	86.0	106	1.2	524	6.1
Cd ^(a)	20.9	88	4.2	139	6.7
Cr	0.63	15.8	25.1	20.4	32.4
Cu	0.80	18.0	22.5	31.0	38.8
Fe	4.29	42.6	9.9	51.5	12.0
Hg ^(a)	< 2.0	9.0	> 4.5	18.0	> 9.0
Mg	17.6	24.0	1.4	64.3	3.7
Mn	0.65	10.7	16.5	20.5	31.5
Ni	0.45	4.5	10.0	14.6	32.4
Pb	0.88	15.1	17.2	32.2	36.6
Zn	1.24	19.0	15.3	46.4	37.4

^a Micrograms/liter.

Table 15. Metals Mass Balances in Control Treatment System.

Parameter	Percent of Influent in		Mass Balance Primary Clarifier	Percent of Influent in		Mass Balance Total System
	Primary Effluent	Primary Sludge		A.S. Effluent	Waste A.S.	
Ag	112.3	10.7	123.0	61.1	34.6	106.4
As	97.3	0.3	97.6	78.4	17.9	96.5
Ca	93.7	2.9	96.6	91.5	0.0	94.4
Cd	80.7	3.1	83.9	36.7	9.9	49.8
Cr	80.6	15.4	96.0	52.5	59.1	126.9
Cu	70.9	18.4	89.3	19.4	53.0	90.8
Fe	56.6	5.7	62.3	22.9	23.4	52.0
Hg	< 100.0	< 0.4	< 100.4	< 99.5	< 10.9	100.8
Mg	101.8	1.7	103.5	98.8	3.2	103.8
Mn	84.2	15.0	99.2	59.8	38.6	113.4
Ni	59.8	15.3	76.1	38.8	23.3	77.4
Pb	65.6	17.4	83.0	12.2	40.5	70.1
Zn	102.7	17.7	120.5	36.0	36.1	89.9
Cl	102.5	0.6	103.1	105.1	2.5	108.2
F	116.9	0.1	117.0	113.4	2.0	115.5
SO ₄	96.8	0.1	96.9	97.5	0.3	98.0
SiO ₂	103.1	0.7	103.8	86.2	11.0	97.9
TDS	93.6	0.8	94.4	87.8	4.4	92.9

spiking sequence such that the control system was essential to assess the metal impact performance. While the principal operating control, the sludge retention time (SRT), was varied from below 2 to over 12 days, most of the spiking operations were performed with about 5- to 8-day SRT. The data analyses have been completed for three of the metals. The chromium evaluation is ongoing.

In addition to the conventional operations and water quality parameters (Tables 3 and 4), the spiked and control systems were monitored for effluent turbidity and their mixed liquors microscopically examined using dark field and 645X. A series of photomicrographs were taken to document changes in microbiota as the metal concentration was increased.

The experimental results were used to evaluate the principal effects of the metals addition on the plant operation as a function of increasing metal concentration. The studies also provided process and system metal removals, metal partitioning to the plant's sludges, and passthrough of the metal from the treatment plant; all as functions of the spiked influent metal concentration.

Limited highlights of the results are presented in Tables 16 and 17 and Figures 3-8. Table 16 presents the effects of three of the metals on the activated process. Significant deterioration of the overall plant performance generally requires substantial influent metals concentrations, well above typical background metals concentrations.

The metals concentration correlations, usually as a function of system or process influent concentration, exhibited reasonable correlation coefficients (Table 17) and can be used to predict process or system performance for the removal of metals. The graphical presentations of the metals concentrations in the spiked system's process and sludge streams as a function of the influent metal concentration (Figures 3-5) reveal a breakthrough discontinuity for Cd in the activated sludge effluent but continuously increasing final effluent breakthroughs for Pb and Hg. Representative graphical presentations for the concentration correlations on the spiked metals are provided in Figures 6-8.

TOXICITY REMOVAL

Even with extensive removability data and occurrence concentrations on the individual toxics, health or ecosystem effects from the complex mixture of metals in municipal or industrial wastewaters and treated effluents are very difficult to evaluate. A biomonitoring approach to assess health and ecosystem impacts and to supplement the specific occurrence and removal data is being evaluated in the MERL toxics studies.

The EPA's Newtown Fish Toxicology Station in Cincinnati is assessing removal of toxicity from the spiked and control raw wastewater during conventional treatment of the MERL toxics studies. The Fish Toxicology Station is using fathead minnows, rainbow trout and Daphia magna as testing targets in 96-hour static (LC50) acute toxicity tests for the fish and in 48-hour static (EC50) acute toxicity tests for Daphia magna. The acute toxicity

Table 16. Summary of Effects of Metals Spiking
on the Activated Sludge Process.

Effect Observed	Metal Concentration, mg/l, Entering Activated Sludge Process		
	Cd	Pb	Hg
Breakthrough of metal into secondary effluent	2.0	continuous	continuous
Increase in effluent COD	4.1	-	-
Increase in SVI	8.6	0.76	.16
Inhibition of nitrification	8.6	1.7	12.5
Decrease in respiration rates	10-20	-	3.3
Decrease in colonial stalked ciliates	8.6	30.5	-
Significant turbidity increase	30.5	30.5	3.3
Floc destabilization and complete process failure	-	71	-

Table 17. Removal Correlations for Cd, Pb and Hg.

I. Activated Sludge Effluent -y vs Primary Effluent -x:

Cadmium (mg/l):

$$\log y = 0.731 \log x - 0.964 \quad r = 0.89$$

Lead (mg/l):

$$\log y = 1.060 \log x - 1.070 \quad r = 0.93$$

Mercury (mg/l):

$$\log y = 0.890 \log x - 0.618 \quad r = 0.95$$

II. Primary Sludge -y (Mg/Kg) vs Influent Wastewater -x (mg/l):

Lead:

$$\log y = 0.31 \log x + 2.34 \quad r = 0.86$$

Mercury:

$$\log y = 1.248 \log x - 1.756 \quad r = 0.96$$

III. Waste Activated Sludge -y (mg/kg) vs Primary Effluent -x (mg/l):

Cadmium:

$$\log y = 1.005 \log x + 3.077 \quad r = 0.96$$

Lead:

$$\log y = 1.140 \log x + 2.350 \quad r = 0.97$$

Mercury:

$$\log y = 0.653 \log x + 1.254 \quad r = 0.90$$

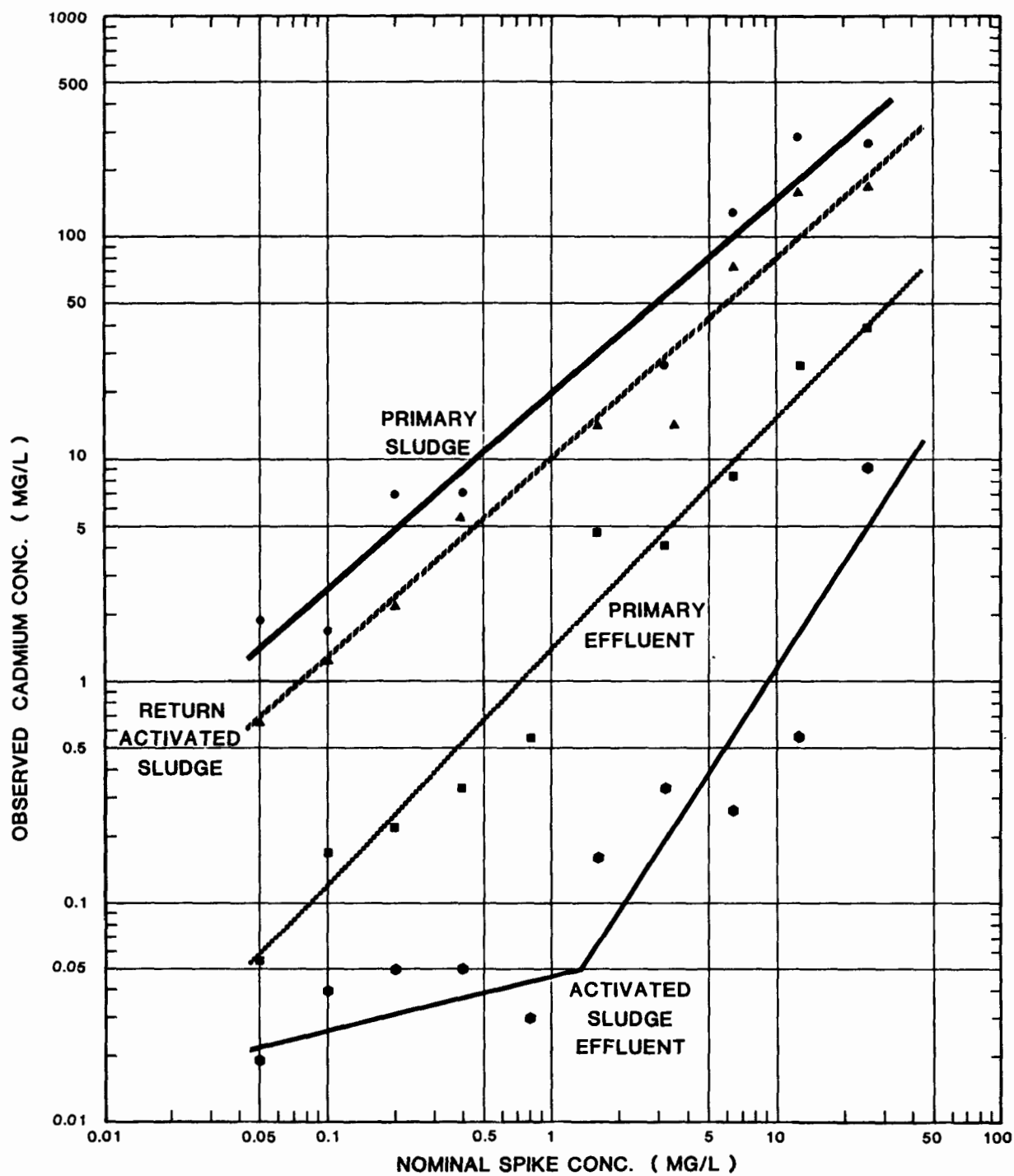


Figure 3. Cadmium Concentrations in Spiked System.

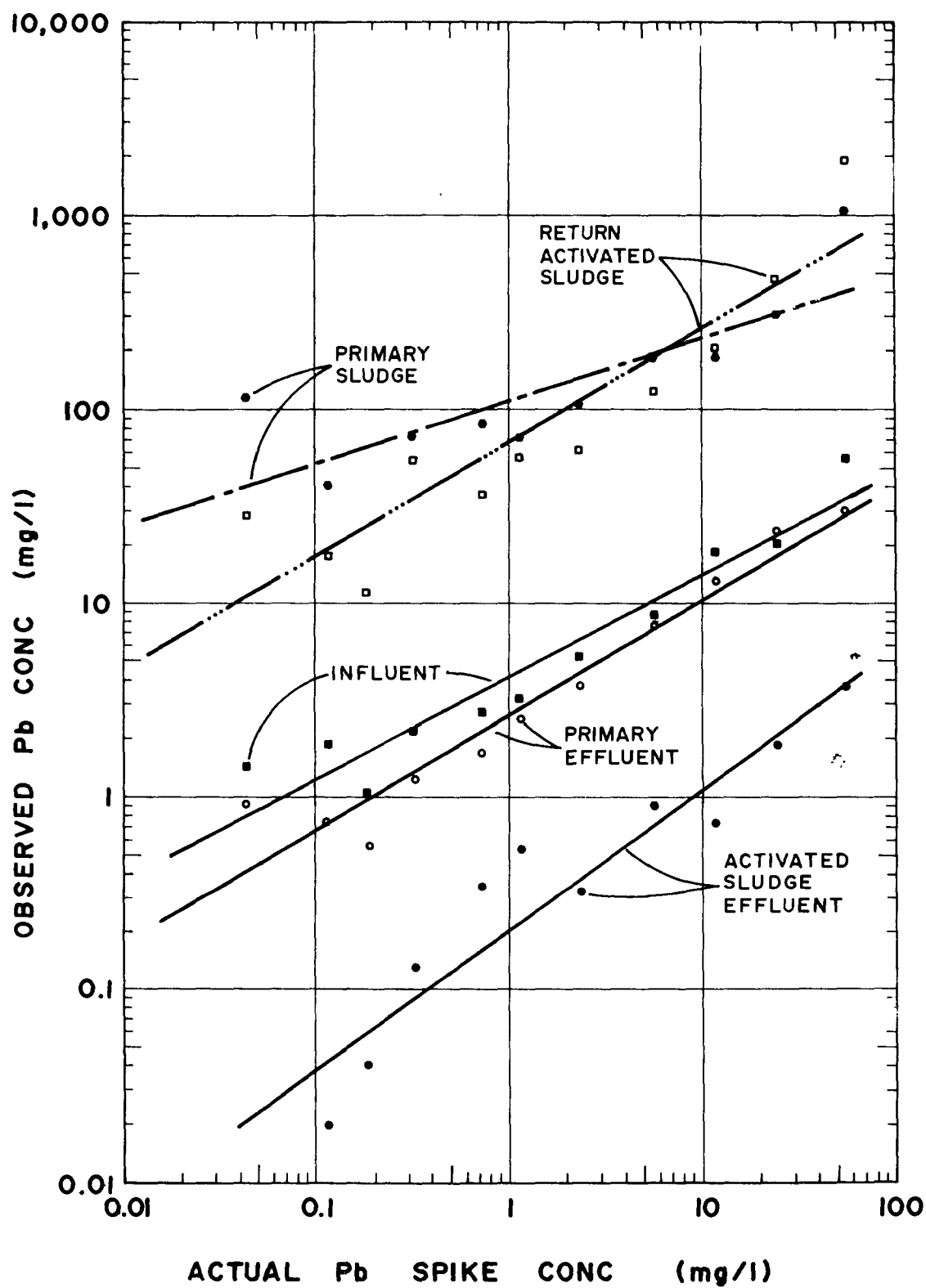


Figure 4. Lead Concentrations in Spiked System.

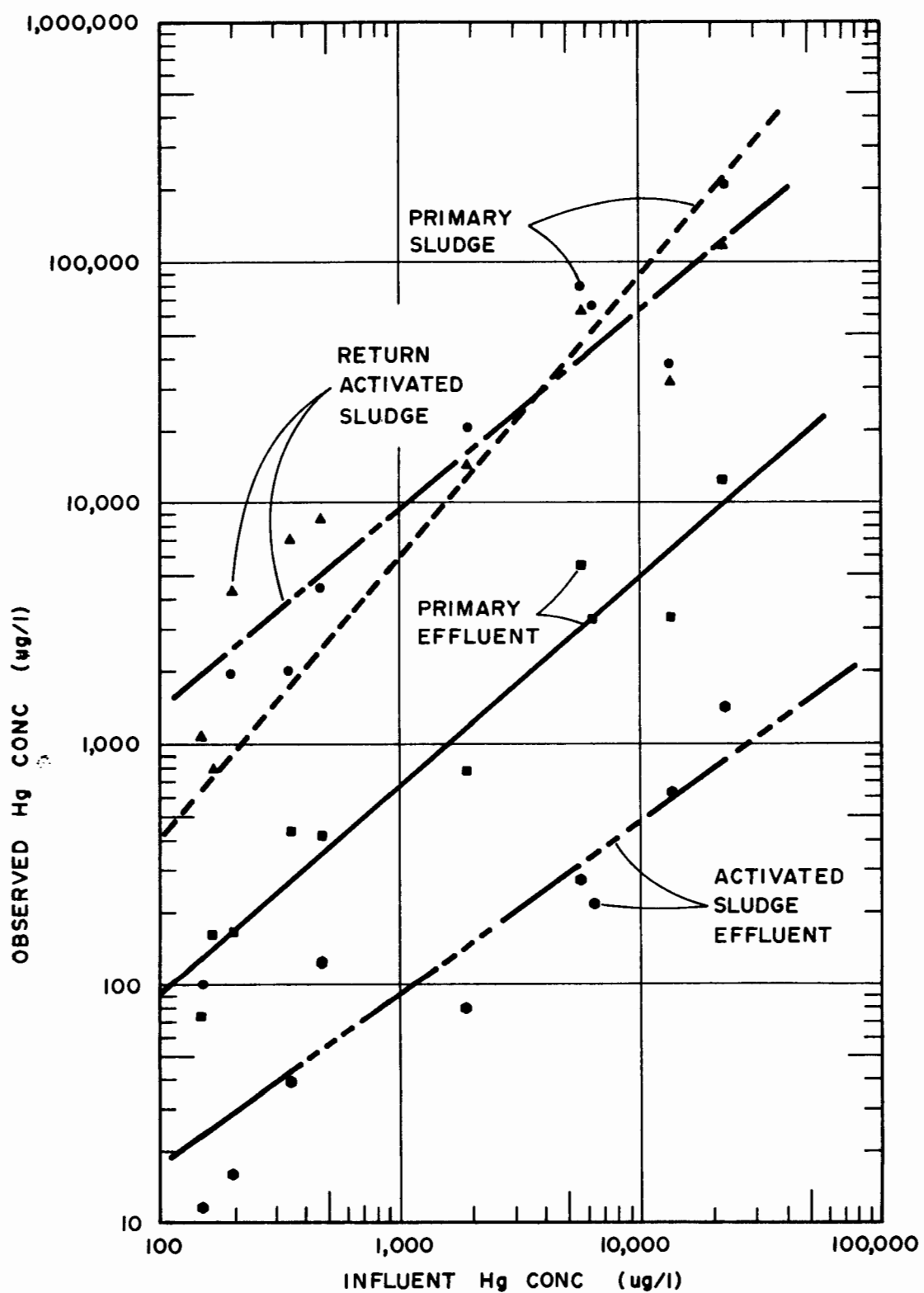


Figure 5. Mercury Concentrations in Spiked System.

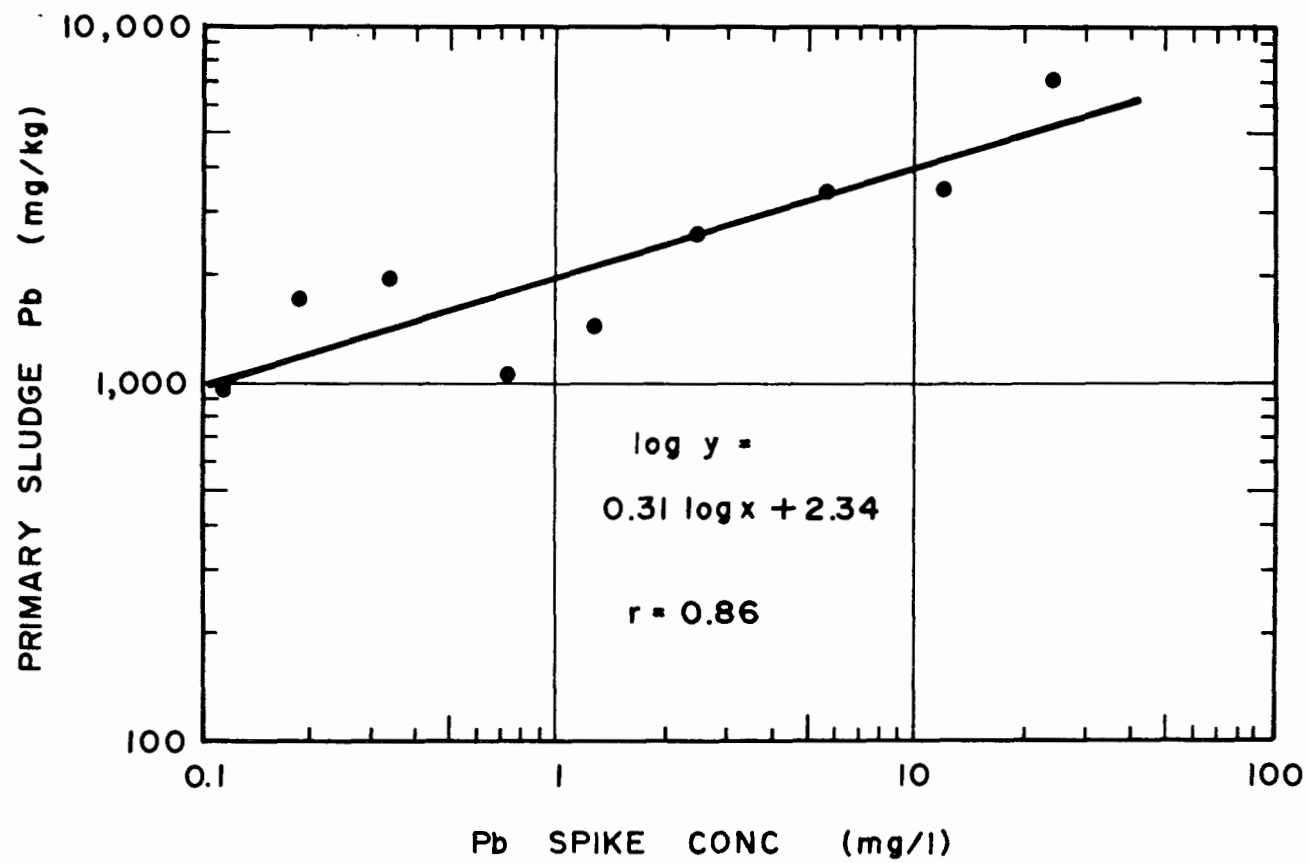


Figure 6. Primary Sludge Pb Concentrations as a Function of the Influent Pb Concentration.

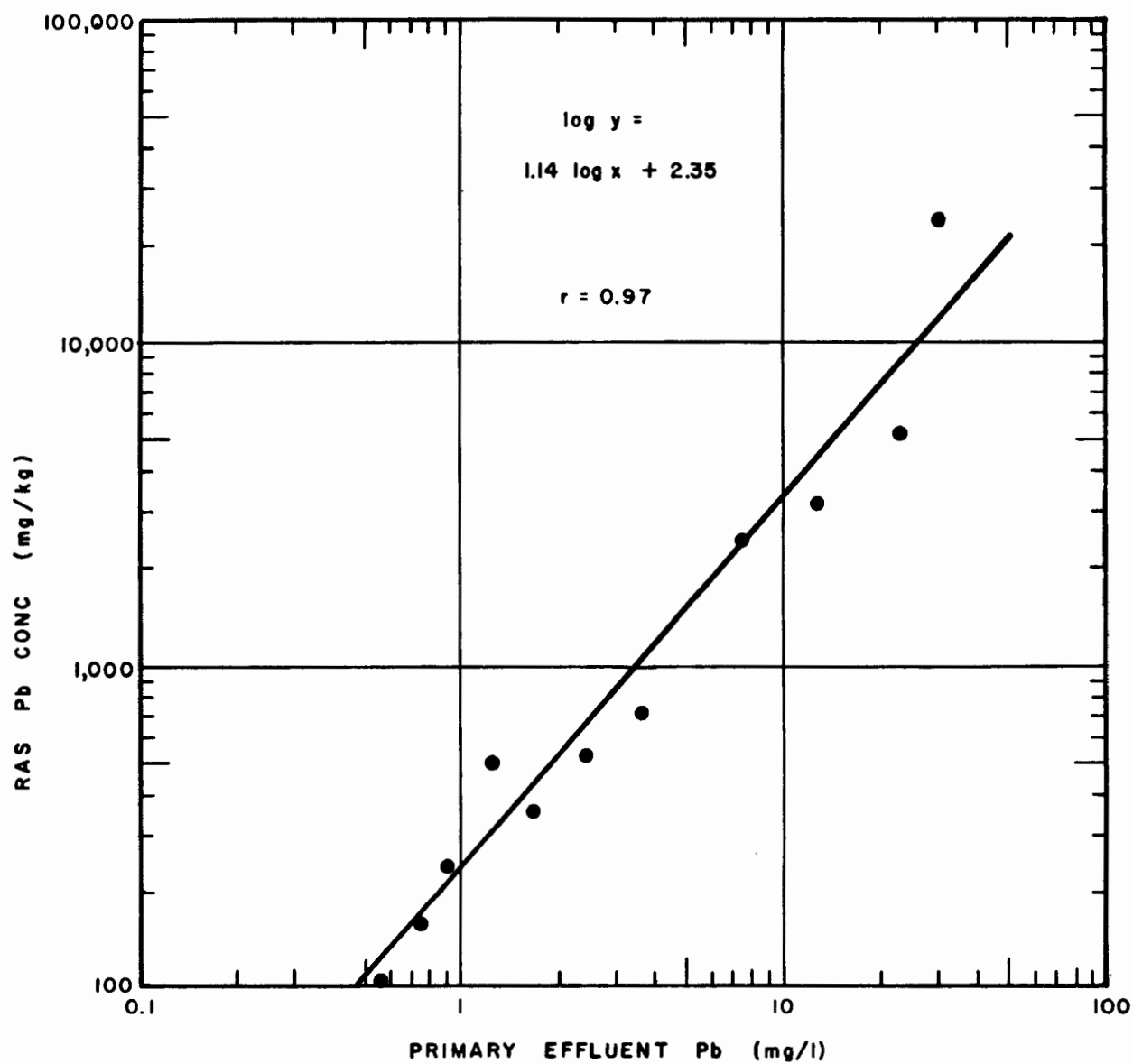


Figure 7. RAS Pb Concentration as a Function of the Primary Effluent Pb Concentration.

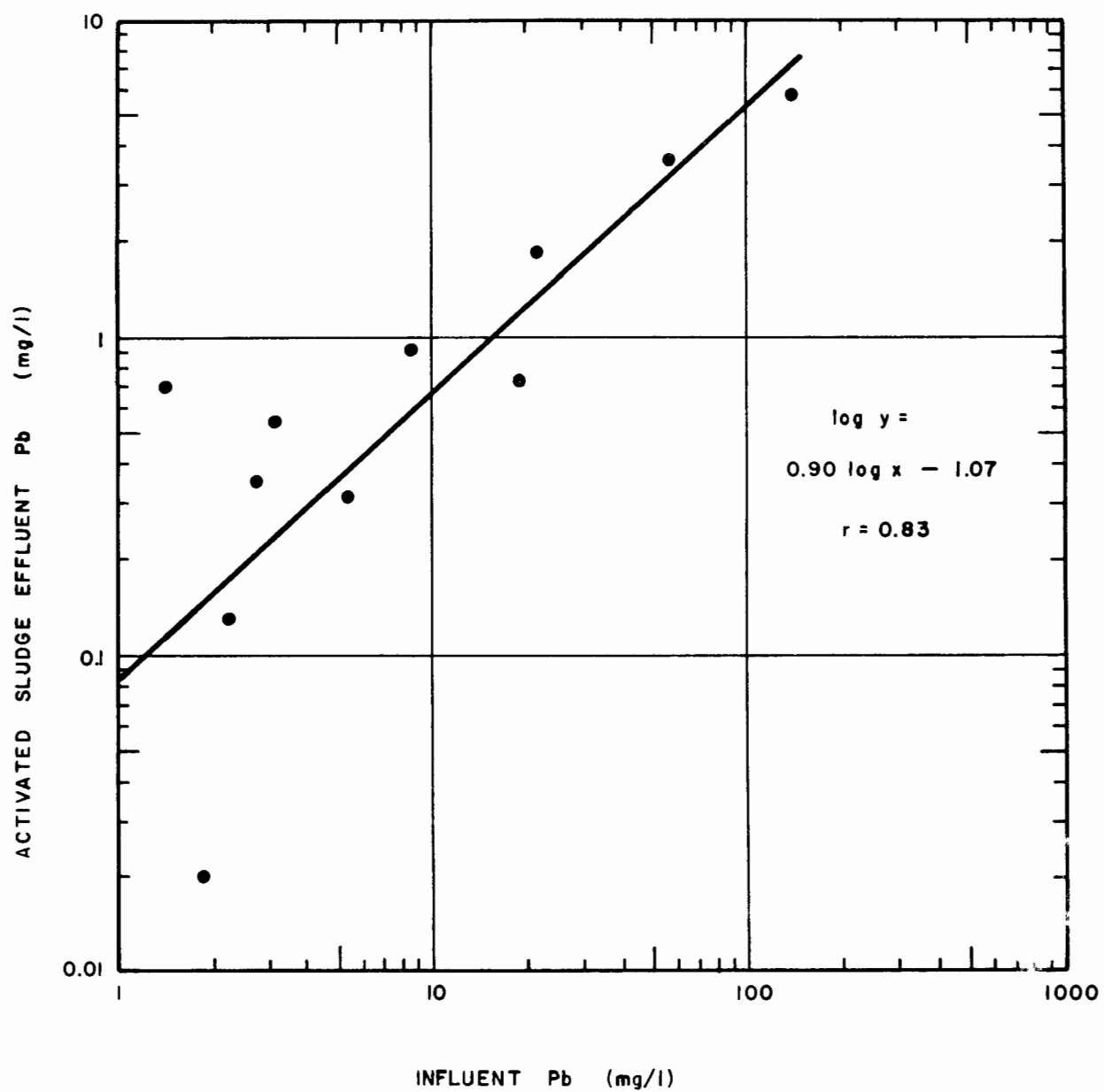


Figure 8. Lead Concentration in the Activated Sludge Effluent as a Function of Influent Pb Concentration.

reductions by the treatment systems are based upon the reduction of lethal units measured in the influent and effluents from the treatment systems or processes. The lethal units are calculated from:

$$\text{LU.} = \frac{100\%}{\text{LC50 or EC50 in percent wastewater}}$$

Chronic toxicity residuals are also determined in the wastewater or effluents using the early-life-stage (ELS) chronic test on fathead minnow embryos. The chronic exposures with various dilutions of wastewater are initiated with eggs less than 24 hours old and continue through 30 days after hatching. The effects on embryo survival and larvae survival and growth are measured to estimate chronic toxicity of the wastewater to the fish embryo-larvae as a representative ecosystem organism.

Finally, an assessment of mutagenicity reduction by the treatment systems has been recently initiated using the Ames Test as the indicator. The work with the Cincinnati Health Effects Research Laboratory involves extracting the raw wastewater, primary effluent, secondary effluent, and chlorinated secondary effluent from an unspiked treatment system with methylene chloride. The extraction separates the bacteria and viral cells in the wastewater from the toxic organics. The organic extracts are solvent transferred to dimethyl sulfoxide (DMSO) and the DMSO extracts are used in the Ames test to assess the reduction in mutagenicity by the treatment processes. Results are not yet available on this work.

Ecosystem Toxicity Removal

The ecosystem toxicity work was performed during the semi-volatile organics studies. The operating conditions, water quality levels in the wastewaters and effluents, and specific toxics concentrations encountered in the semi-volatile studies describe the wastewaters used by the Fish Toxicology Station.

The results to date on the acute toxicity reductions are summarized in Tables 18 and 19. In Table 18, the unspiked raw wastewater exhibited moderate acute toxicity which increased when the priority pollutants were added. The conventional treatment system essentially eliminated the acute toxicity from the wastewater in the control (unspiked) study. Conventional treatment also reduced but did not eliminate the acute toxic effects of the effluent from spiked wastewater system. The initial study also revealed that de-chlorination of chlorinated secondary effluent essentially prevented increased acute toxicity from chlorination (Table 19) of the secondary effluent.

The embryo-larvae chronic testing for five percent and lower concentrations of unchlorinated final effluent from the spiked system (Table 20) revealed no statistical difference in the embryo/larvae survivals or in the growth of the larvae-juveniles compared to the control. The application of chlorination/dechlorination to the final effluent, however, produced (Table 21) a statistically significant reduction in larvae-juvenile survival and in growth rate at the 5 percent concentration level of plant effluent in diluent water.

Table 18. Acute Toxicity (96-hour) of Municipal Wastewater Before and After Conventional Wastewater Treatment: Fathead Minnow - Phase I

Sample Date	LC-50, Percent		Percent Toxicity Reduction	LC-50, Percent		Toxicity Reduction
	Unspiked Influent	Unspiked Effluent		Spiked ^(a) Influent	Spiked ^(a) Effluent	
12-14-79	-	100	-	-	36.1 (28.5-43.4)	-
12-19-79	-	64.4 (59.2-70.5)	-		22.3 (18.7-25.8)	-
1-16-80	30	100	100	4.6 (3.5-16.2)	13.1 (10.0-16.0)	65
1-22-80	11.0 (9.3-12.5)	100	100	2.7 (2.2-3.3)	16.1 (13.6-19.0)	83
4-2-80 ^(b)	9.3 (7.9-11.2)	100	100	9.5 (8.1-11.1)	35.5 (30.3-43.3)	73
4-15-80 ^(b)	30	100	100	4.5 (3.2-5.8)	6.6	32
4-24-80 ^(b)	10.2 (8.5-13.0)	100	100	4.3 (3.3-6.5)	9.4 (7.9-11.5)	55
5-5-80 ^(c)	18.5 (16.5-20.7)	100	100	5.8 (4.8-7.6)	30	81
5-13-80 ^(c)	10.1 (98.3-11.1)	100	100	6.5 (5.6-7.6)	30	78
6-4-80	20.6 (17.8-25.3)	100	100	1.9 (1.0-2.3)	8.0	76

^a Spiked samples - mixtures of 22 organic priority pollutants in pilot treatment system.

^b Slight excess control fish mortality in samples.

^c Samples were 24-hour composites - others were grab samples.

NOTE: Toxicity reduction calculations based on lethal units ($LU = \frac{100}{LC50}$).
Numbers in parentheses are 95% confidence limits.

Table 19. Acute Toxicity of Municipal Wastewaters^(a) With and Without Chlorination.

Sample Date	Test Animals	Influent	Pre-Chlorinated Effluent	Chlorinated/Dechlorinated Effluent	Percent Toxicity Reduction ^(b)
6-30-80	Fathead Minnow	12.1 ^(c) (10.4-14.2) ^(d)	27.9 (24.4-32.0)	32.3 (28.0-38.1)	63
	Rainbow Trout	-(e)	9.7 (7.2-12.8)	14.2 (10.8-20.2)	-
	<u>Daphnia magna</u>	17.0 ^(f) (14.6-22.3)	16.8 (14.3-19.7)	11.2 (9.4-13.5)	+ 51
7-7-80	Fathead Minnow	15.1 (12.8-19.7)	44.7 (40.4-51.2)	60	75
	Rainbow Trout	-	17.2 (14.7-20.8)	17.8 (15.4-21.4)	-
	<u>Daphnia magna</u>	1.6	23.5 (22.0-34.4)	9.8	16.3
7-16-80	Fathead Minnow	12.8 (11.3-14.9)	39.9 (34.6-44.9)	49.0 (42.6-61.1)	74
	Rainbow Trout	-	13.4 (10.2-18.7)	16.1 (13.4-19.7)	-
	<u>Daphnia magna</u>	1.6	6.4 (1.9-11.4)	10.2	84
7-24-80	Fathead Minnow	10.5 (8.6-13.3)	24.5 (19.3-29.9)	23.2 (18.0-28.3)	55
	Rainbow Trout	-	17.2 (14.7-20.8)	17.8 (15.7-21.4)	-
	<u>Daphnia magna</u>	1.9 (1.0-2.6)	21.8 (18.0-25.6)	8.0	76
7-31-80	Fathead Minnow	5.4 (4.1-6.8)	32.3 (26.5-41.2)	42.7 (34.0-52.4)	88
	<u>Daphnia magna</u>	1.6	9.1 (6.7-12.2)	1.6	0
8-5-80	Fathead Minnow ^(g)	1.6	5.0	5.5	71
	<u>Daphnia magna</u>	1.6	22.6 (17.2-27.8)	15.2 (10.8-20.1)	89

^a Influent wastewater continuously spiked with 22 organic priority pollutants.

^b Toxicity reduction based on lethal units of influent and chlorinated/dechlorinated effluent.

^c Fish - 96-hr LC50 percent waste.

^d 95% confidence limits.

^e Indicates no data.

^f Daphnia magna 48-hr EC50 percent waste.

^g Slight excess control fish mortality.

TABLE 20. Survival and Growth of Early-Life-Stages of Fathead Minnows
Exposed to Activated-Sludge Effluent in Spiked System.

Nominal concentration %	Embryo survival %	Larval-juvenile survival (%) at 30 days	Juvenile weight (mg) mean + S.D. at 30 days	Juvenile length (mm) mean + S.D. at 30 days
5.0	88	-	113 + 38	20.6 + 2.4
	82	93	117 + 36	23.5 + 1.5
2.5	92	97	167 + 49	23.1 + 2.4
	86	90	155 + 36	22.9 + 1.7
1.2	86	100	167 + 49	23.3 + 2.5
	88	93	178 + 49	24.1 + 2.2
0.62	88	-	197 + 53	24.5 + 2.8
	88	90	207 + 60	24.7 + 2.7
0.31	88	97	207 + 55	24.5 + 2.3
	90	93	196 + 64	24.4 + 2.7
0.16	86	100	183 + 57	23.9 + 2.1
	92	80	200 + 35	24.6 + 1.5
Control	90	93	196 + 53	24.3 + 2.1
	88	100	188 + 63	24.5 + 2.5

TABLE 21. Survival and Growth of Early-Life-Stages of Fathead Minnows Exposed to Chlorinated/Dechlorinated Effluent from Spiked System.

Nominal concentration %	Embryo survival %	Larval-juvenile survival (%) at 30 days	Juvenile weight (mg) mean \pm S.D. at 30 days	Juvenile length (mm) mean \pm S.D. at 30 days
5.0	86	10 (a)	66 \pm 8 (a)	18.3 \pm 0.58
	80	7 (a)	72 \pm 12 (a)	19.0 \pm 1.4
2.5	90	100	130 \pm 52 (a)	22.3 \pm 2.5
	92	93	129 \pm 47 (a)	21.8 \pm 2.4
1.2	88	80	165 \pm 53	23.4 \pm 2.6
	80	93	162 \pm 53	23.0 \pm 2.4
0.62	94	83 (a)	173 \pm 54	23.4 \pm 2.5
	90	67 (a)	165 \pm 29	23.1 \pm 1.2
0.31	86	83	170 \pm 44	23.9 \pm 1.9
	82	100	203 \pm 49	24.7 \pm 1.6
0.16	78	97	191 \pm 40	24.0 \pm 1.7
	92	97	200 \pm 44	24.6 \pm 1.6
Control	86	100	213 \pm 67	24.8 \pm 2.3
	82	90	186 \pm 46	24.8 \pm 2.2

^a Significantly different (P=0.05) from control.

SUMMARY

The U.S. EPA's Municipal Environmental Research Lab is assessing, at its Test and Evaluation Facility in Cincinnati, the removability of toxic substances from municipal wastewater by conventional wastewater treatment processes. The raw wastewater used at the Test Facility is a mixed domestic/industrial wastewater from the highly industrialized Mill Creek area of Cincinnati. The studies feature pilot-scale primary/secondary treatment of the raw wastewater spiked with selected priority pollutants (metals and organics). In the studies, the treatment plant performance on spiked wastewater is compared to the performance of identical treatment on the unspiked raw wastewater. The assessment employs costly analyses (GC/MS and atomic absorption methods) for the selected toxic substances in the various process streams and sludges of the conventional treatment plant.

From the studies to date, conventional treatment is generally effective in removing selected toxic substances, typically achieving better than 90% removal of organics and from 60-80% removal of the metals. A few of the toxic substances, however, pass through into the treatment plant final effluent in sufficient concentrations which, based upon EPA recommended water quality standards, may present a possible environmental hazard.

Even with extensive removability data on the individual toxic substances, health effects or ecosystem effects from the complex mixtures of materials in municipal or industrial wastewaters and effluents are very difficult to evaluate. A biomonitoring approach to assess health and ecosystem effects is also being evaluated to supplement the specific toxic substance removal data being collected in the EPA studies. The EPA's Newtown Fish Toxicology Station in Cincinnati is assessing the removal of acute toxicity to ecosystems from the spiked and control raw wastewater during various stages of treatment. The fish toxicology station is using fathead minnows, rainbow trout, and Daphnia magna as testing targets in 96hr static (LC₅₀) acute toxicity tests for the fish and in 48 hr static (EC₅₀) acute toxicity tests for the Daphnia. The acute toxicity reductions by the treatment systems are based on the reduction of lethal units (L.U.) measured in the plant's influent and effluent.

Chronic toxicity residuals are also determined in the final effluents from the spiked system using the early-life-stage (ELS) chronic test on fathead minnow embryos. The effects on embryo survival and on larvae survival and growth are measured to estimate chronic toxicity for various dilutions of plant effluent.

From results to date, the unspiked raw wastewater exhibited moderate acute toxicity which increased when the priority pollutants were added. The conventional treatment system essentially eliminated the acute toxicity from

control (unspiked) wastewater. Conventional treatment also reduced, but did not eliminate, the acute toxic effects from the spiked wastewater.

The initial study has also revealed that dechlorination of chlorinated secondary effluents essentially prevented increased acute toxicity from the chlorination. The embryo/larvae chronic testing for 5% and lower concentrations of unchlorinated final effluent from the spiked system revealed no statistical differences in the embryo/larvae survivals or in the growth of the larvae/ juveniles compared to the control. The application of chlorination/ dechlorination to the final effluent, however, produced a statistically significant reduction in larvae/juvenile survival and in the growth rate of the larvae/juveniles at the 5% concentration level of plant effluent in diluent water.

An important overall observation from the above work on toxic substances is that the treatment of the strong domestic/industrial wastewater at the Cincinnati plant has exhibited remarkable stability over a wide range of operating conditions and produced consistent and excellent treatment of the wastewater, even when spiked with large doses of metals or toxic organics. In contrast, earlier experience in treating domestic wastewater over similarly wide ranges of operating conditions has revealed operating areas where the growth of organisms such as Sphaerotilus natans predominate in the activated sludge process and contribute to poor settling characteristics of the sludge. These organisms produce plant effluent deterioration through carryover of solids. One possible explanation to the improved treatment stability in the Cincinnati study is that the background of toxic contaminants in the Cincinnati raw wastewater reduces the biological competitiveness of the exposed filamentous organisms.

In any event, the EPA work indicates that the central municipal wastewater treatment plant potentially represents a cost effective alternative to industrial pretreatment for the control of many toxic substances. Use of the treatment plant for toxics control, however, requires evaluation of the impact of the toxics on the sludge handling and final disposal processes. In addition, the use and practical management of the central municipal treatment plant for toxicity control would greatly benefit from the availability of suitable biomonitoring tests for determining the removal of overall toxicity, both acute and chronic, for health and ecosystem protection.

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AN INDUSTRIAL PERSPECTIVE ON
JOINT MUNICIPAL-INDUSTRIAL WASTEWATER MANAGEMENT

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AN INDUSTRIAL PERSPECTIVE ON JOINT MUNICIPAL-INDUSTRIAL WASTEWATER MANAGEMENT

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Gentlemen from Japan, I welcome this opportunity to discuss with you one subject in your program this afternoon. My talk concerns largely policy matters, the organizational arrangement of wastewater treatment, rather than the technical matters.

I have looked forward to talking to you about this subject. I have enjoyed the papers you have presented during the past two days and I have taken occasion to study the papers from your previous conferences. In high school I experienced the value and enjoyment of working with Japanese people. I had the experience of sharing two years of high school with two boys of Japanese ancestry named James Kowabata and Frank Iwatsuki. I was well acquainted with them because James and Frank plus Kelly Berkeley and myself were the four 11th graders selected to be allowed to attend honor study hall in that high school. I learned to appreciate the ability of Frank and James during the work solving problems in physics and chemistry in that study period. I learned to benefit from their good minds. I had reason to remember them because they established grade averages better than mine. I have lost track of Frank and James over the years but no doubt they are out there making a technical contribution someplace. Not only did I experience this scholarship competition but dealt with Fumico Iwatsuki, the fastest typist in the school, and Shogo Adachi the best wrestler in my weight class.

The subject that I want to talk to you about this afternoon is that of the treatment of the entire wastewaters from a community in one common or shared system. Such a system is one that accepts the wastewaters from the homes, from commercial establishments and from industrial plants and conveys them to a shared treatment plant. In the United States this is often referred to as joint treatment.

The reason I want to talk to you about joint treatment is to be sure you understand its advantages, its problems and its administration.

The topic of my talk to you gentlemen today is an old and frayed one. Yet, it is timely to talk about this subject with you because of recent attention to it in this country by way of emphasis on pretreatment programs. A pretreatment program is a term which the Federal agency people brought forth about 1980. The term speaks to the controls and restraints on use by industry of publicly owned wastewater treatment works. This Federal agency attention is new; that is the only thing new about the subject. The practice has existed from the very first development of wastewater collection systems. The first published reference I have come across is a declaration by the Royal Commission on Pollution Control of London. In 1903, these gentlemen proclaimed "the most economic way to dispose of trade wastes is to discharge them to the city sewers and treat them along with domestic sewage".

This joint treatment practice has been followed in practically every community sewer system in this country. Only in a few instances have municipalities and local industry built separate wastewater treatment plants. Table 1 illustrates the number of major industries using the sewer system in each of five representative cities in this country.

Each of the industrial plants included in these numbers are large enough dischargers that the cities go to the trouble and expense of measuring and sampling their wastewaters. These figures do not include the small industrial and commercial users--there are at least an equal number of them.

The practice is widespread among the various categories of industry. Most and in some cases all of many industrial categories discharge their wastewaters to municipal systems as shown in the Table 2. All soluble coffee plants, all but one major soap and detergent plant, 95 percent of the edible oil refining plants, 15 percent of paper converting plants discharge their wastewaters to municipal systems. At the other extreme only a small percentage of petroleum refining plants discharge process wastewaters to municipal systems. Likewise few if any steel rolling, blast furnace and by-product coking plants discharge to municipal systems. A recent EPA wastewater plant survey indicates on a flow basis industrial wastes constitute an average of 16 percent of municipal wastewater treatment plant inflow.

The practice is so common and widespread one might well ask if there is any need for discussion of the subject by way of relating information to persuade people to continue or institute the practice. My answer is yes there is. There is a need for several reasons. One is that there have been a couple of problems throughout the history of the practice which have never been completely settled. One of these is the degree of control most appropriate to discharge of wastewaters containing fat, oil or grease. A second problem area is the control needed for stormwater runoff from industrial plants. Still another is control of slug loads--short-time abnormal flows or masses of treatable pollutants.

Then there is a perennial question about fair sharing of the costs among industrial, commercial and residential users.

The most current topic of joint treatment is the control of toxics that may be present in some industrial wastewaters. Toxics can interfere with treatment or pass through without sufficient removal.

The Federal EPA beginning about June of 1978, launched a significant part of their huge resources on a mission to control what they chose to call indirect dischargers. Indirect industrial dischargers are dischargers to publicly owned treatment works. The EPA had concentrated their early efforts on control of industry that treated their wastewaters in their own facilities and discharged them directly to public waterways. This newly initiated EPA control effort directed at indirect dis-

chargers was a part of an interest in control of toxic pollutants. The EPA had started the program of establishing limitations on toxics for direct dischargers at about this time and it was natural to implement at the same time control of indirect dischargers.

This attention to control of toxics by requiring pretreatment programs of communities has caused an aura of alarm and a certain confusion in minds of non-specialists about joint treatment. This confusion and excitation has caused a questioning, a certain skepticism, a negativism about joint treatment. For example cities have entertained or pressed for unreasonable and unrewarding control of ordinary compatible everyday treatable wastes--wastes that are perfectly compatible with transport and treatment in publicly owned treatment works. Wastes that are adequately characterized by conventional pollutant parameters such as BOD and suspended solids. Cities have begun to question and propose that no capacity be provided for industrial growth when treatment facilities are enlarged. Municipalities have said to their industrial members we do not have sufficient capacity for the loads of treatable wastes you are discharging; therefore we are going to limit each of you to a specified maximum daily load.

Some of the language in the Federal regulations on limiting excessive short time loads of compatible wastes seems to encourage municipalities to limit the industrial use for compatible wastes. When considering a limit on a high load from industry, a consideration of the maximum capacity of the treatment plant likely is involved. Thus the control of excessive loads to prevent pass through of above limit concentrations of BOD may very well limit industrial use. The need for a slug load limit is recognized for any parameter. We are concerned and have serious objections to restrictions that will prevent ordinary day-to-day use.

Joint treatment is threatened in the matter of eligibility of the share of treatment facilities built with Federal grant funds for the treatment of the wastes from industries of the community. The way that industrial users of municipal systems are dealt with in the Federal grant program is complex. Too complex to go into here and there is still hope for change. Industry believes generally that they should be fully eligible to use the grant supported public works just like other members of the community are on the basis that industry has provided a significant part of the Federal grant funds via corporate profits tax.

My introduction of the subject to you has been long. I wanted to be sure you had opportunity to understand the problems and to learn the solutions we advocate. My talk is not going to be nearly so technical in terms of processes or equipment as have been the excellent discussions you have previously heard. I hope my dwelling on administration or policy will be of interest.

I would like to sell you on joint treatment. I would like to point out the best road to follow in certain problem areas in its practice. I

would like to provide assurance in the turmoil, quiet your concerns, point out how to overcome true problems and send you back to your home as evangelists for this cause.

Let us look now at the advantages of joint treatment.

The dominant one is economic. A larger plant is simply cheaper to construct and operate than a number of smaller ones. This is brought home in real terms by this graph which shows the capital costs per million gallons per day of capacity for biological treatment plants of a range of sizes, Figure 3. This data is from an EPA report. The line of best fit to the data exhibits a marked decline in the costs per million gallon capacity as the size increases. Figure 4 shows the operating, maintenance and replacement costs per 1000 gallons of normal domestic sewage from biological waste treatment plants of a range of sizes. There is a marked decline in the costs as the size increases.

Most emphatically this information proves joint treatment is lower in costs than separate treatment. A fair wastewater service charge system furthermore will result in all users benefitting from lower costs, homeowners as well as industrial users.

I must call attention to the possibility that in some special situations an industrial plant may be able to treat their own wastewaters more economically. For example, for wastewaters that are highly seasonal and where land treatment during the warm season is practical. Another example would be an industrial plant at a location so remote that conveyance costs were prohibitive. A third situation would be an industry with adequate and low cost land and effluent quality limitations such that an aerated lagoon system would suffice. But for the vast majority of situations I feel sure joint treatment is the most economic for industry and saves the homeowner money too.

Some lesser but important advantages include a savings in land space devoted to treatment plants. A single large plant takes up less space than would a number of smaller ones.

Some industrial wastes are lacking in the nutrients--nitrogen and phosphorous needed. Domestic sewage has an excess of these. Thus joint treatment saves the cost of chemical addition.

Finally, the large staff of specialists in wastewater treatment that would be found in a large joint treatment plant will do a superior job of providing a good uninterrupted effluent quality. A number of smaller plants would much more likely suffer upsets and occasional poor quality because of part-time and non-specialist operators.

Having been convinced of the advantages of joint treatment, I would like to lead you through some problem areas.

The Fat, Oil and Grease Misunderstanding

The wastewater treatment technologists have suffered a great handicap in control of oil-bearing wastewaters being discharged to municipal systems. Early information and recommendations were incompletely presented and easily misunderstood. Only recently has an adequate control strategy been developed. Substantial information on treatability and removal of oil in municipal wastewater treatment plants is just currently appearing in the literature. The equipment and processes used in municipal wastewater treatment have changed so that many old concerns have disappeared. Most concerns have been moderated or eliminated. Controls other than limitations on concentration have been established as best solutions. Let me tell you the details of this subject and lay out for you where I believe the practice should be in control of oil in wastewaters in industrial wastewaters discharged to municipal systems. Pretreatment programs have been saddled with precedents and recommendations that are unnecessarily harsh, and restrictive. They have caused economic waste.

One cause of the problem is the analysis for oil measures two kinds of oil and some miscellaneous other compounds. The two kinds of oil are different in their significance and control needs. Their treatability and therefore the restrictions that are appropriate for one are inappropriate for the other. The two kinds of substances that are generally involved are oils of animal and vegetable origin and oils of petroleum origin. The chemical structure of these kinds is very different. The material of animal and vegetable origin is in its original form a structure known as a triglyceride. A triglyceride is the three carbon molecule to which is attached three fatty acids. The fatty acids are detached from the glyceride base in the making of soap and other derivatives. The fatty acids themselves measure as oil as are the soap compounds made from them. Chemists call this whole family of animal and vegetable oil compounds lipids. Remember that term as I am going to use it frequently. The petroleum kind of oil can include a huge number of structures the predominance of which in the usual situation are a straight chain of carbon and hydrogen atoms. Petroleum can also contain the ring form of linked carbon atoms known as aryl and cyclic compounds.

The lipids are natural constituents of food and are virtually everywhere in nature. They constitute a significant portion of the diet of people in the United States, in Japan and other places. Estimates have been made that such oils constitute 25 to 40 percent of the calories in the average diet in this country. They are a major constituent in meat, poultry, nuts, many baked goods, milk, salad dressings, etc. They are used as a heat transfer liquid in frying and as an anti-sticking coating in baking. Compounds of lipid origin are a major functional part of much of the household soaps and in some laundry products. They are excreted on the skin and hair. It is little wonder then that they constitute a large percent of the organic matter in domestic sewage from households. From at least fifteen to twenty-five percent of the organic matter in

sewage is estimated to be lipids. The human feces itself has been found to contain five percent or more of oil on a dry-weight basis. So the lipid type of oil is a natural and universal major constituent of domestic sewage. Therefore, a municipal wastewater treatment plant that cannot adequately transport and treat wastewaters containing lipids is not appropriate to use. A satisfactory plant has to be able to handle wastewaters containing major amounts of lipids. A look at the data available on the concentration of oil in municipal wastes indicates a range of about 30 to 50 mg/l.

The lipid type of oil is biologically degradable at rates comparable to other organics such as carbohydrates and proteins. The latest researcher to make observations of this is Professor Hrudey of the University of Edmonton in Canada. He published a landmark paper on this in June of 1981. Preceding him there were noteworthy contributions by Dr. James Young of Iowa State University, Dr. Perry McCarty of Stanford University, Messrs. Pico and Watson of Kraft Foods, Dr. Loehr of Cornell University, and the consulting firm of AWARE with which Professor Eckenfelder of Vanderbilt University is associated.

Portions of the lipid in the wastewater inflow to a municipal system will be separated as scum in the primaries, will be a significant constituent in the primary sludge, and will be present at a low percentage in the waste biological sludge. The lipids in these sludges is readily converted to methane gas in anaerobic sludge digestors. The kinetics of this conversion is such that the lipids will be converted at higher rates than carbohydrate and protein fractions in the sludges. Lipids are the major contributor of methane in such sludges. These facts have been established in research described in articles by Dr. O'Rourke, Dr. McCarthy and others.

Please understand that such bacteriological degradation will take place rapidly for lipids sufficiently dispersed. The exposed surface of large particles is too low relative to the quantity of lipid to permit rapid degradation. Fortunately domestic sewage contains dispersing agents which cause oil to disperse and sustain it in suspension. Primary settling will remove lipids in floatable size particles. The oil reaching the biological treatment process will be, therefore, sufficiently dispersed for rapid biodegradation.

As for anaerobic process, the modern digestors provide enough mixing that sufficient dispersion of the lipids occurs. The former problems of solid oily scum forming have been practically eliminated.

We have, therefore, arrived at the point in this discussion to deal with the heart of the matter, the decision that you may be involved in, namely, what are the appropriate controls for the lipids in industrial wastewaters. The evidence I have presented established that dispersed lipids (animal or vegetable oil or fats) are perfectly compatible for

transport and treatment in municipal systems. Dispersed lipids are BOD and suspended matter which biological treatment is capable of removal and degradation. Our position is that no limit is appropriate for dispersed lipids.

Wastewaters of many industrial categories contain significant concentration of dispersed lipids, such as

- Edible oil refining
- Margarine manufacture
- Fish oil processing
- Milk processing
- Cheese making
- Meat processing
- Poultry processing
- Candy manufacture
- Rendering plants
- Soap manufacture
- Certain food plants

This pretreatment capital costs for thousands of industrial plants to meet some arbitrary and unrewarding limit could amount to great amounts of money. The primary process for removal of the dispersed lipids is dissolved air flotation enhanced by the use of chemical coagulants. In order for the recovered material to have value it must be treated to remove the coagulating chemicals. The costs are such that there seldom is a payout in the value of lipid recovered. We all are conservationists to a degree and do not like to condone waste. However, in this situation in case after case the dispersed lipid is not economically recovered. The float of lipid and chemical simply becomes a waste material. Our recommendation is do not force this pretreatment process on industrial users. The treatment of these dispersed lipids in the municipal treatment is perfectly feasible and is the most economic solution.

Lipids in a floatable form may cause distribution and fouling problems. By floatable form is meant oil or fat in a droplet or scum form of sufficient bulk that the droplets will tend to rise to the surface under quiescent conditions. In other words, oil or fat which will float to the surface in a gravity settling system. The settling system in mind would be one which was designed according to primary settling basin criteria.

The floatable oil is removable by simple equipment and can be made use of for animal feed or other use. Recovery of floatable oil where significant quantities are involved can have a return. So the removal of floatable oil by the provision of gravity settling systems is a logical pretreatment regulation. The simple regulation needed is simply a provision that floatable oil, fat and grease be removed. Floatable oil must be defined in the ordinance as oil that is removed by gravity settling in a facility meeting design guidelines of the district. The only decision the agency must make is whether or not to require an industrial user to

install a gravity settling system for floatable oil removal. In most cases the decision will be obvious and undisputed. In a rare case some testing may be necessary to learn of the presence or absence of significant quantities of floatable oil in an effluent. A simple bench scale test can be designed to establish this. There is no need to become concerned over a small quantity of floatable oil because after all there is considerable floatable lipids in household wastes.

The wastewaters from restaurant and food preparation establishments are a frequent target of control agencies in control of incidence of obstruction of sewers. One major city has pursued a policy for such control which has worked well. After finding a sewer obstruction in the sewer serving such establishments the owner is required to install a suitable size gravity separation system. These are called fat traps in the restaurant business. In no case has an obstruction incident occurred after such a facility was installed in this one city's experience.

There is no need to try to relate an oil concentration to likelihood of obstruction of sewers. Ordinances use general descriptive language prohibiting materials that cause obstructions. This control suffices. It is just as easy to police good operation of a fat trap as it is to police for a concentration violation.

You may have noted my preceding remarks were directed at the lipids--their treatability and their logical control. I earlier mentioned another kind of oil--petroleum oil. Petroleum oil is referred to also as hydrocarbons.

Hydrocarbons are not nearly as biodegradable in aerobic systems as are lipids. They are not degradable at all anaerobically. These facts do not mean that they are not removable in a municipal treatment plant. Some will be degraded as it becomes associated with suspended biological matrix and remains under treatment for long periods. Other such oil may find an outlet through being included in the residues disposed of by incineration or otherwise.

However, investigations have shown that there is a limit to the concentration of hydrocarbons which can be allowed to reach the biological floc. Data I have seen indicate that when the concentration of hydrocarbons in the feed is greater than about 25 mg/l some of the floc particles will have a specific gravity the same as water and will not settle in the final clarifiers.

In the anaerobic part of the process, anaerobic sludge digestion, petroleum oil in the skimmings feed to the digester will rise into the digester supernatant. When this supernatant is returned to the raw sewage the oil can become again part of the skimmings. Thus there is created a recirculating oil-laden stream within the treatment plant.

Therefore, a limit on hydrocarbons in the discharge of users to the system is a logical element of a pretreatment program. Since about

25 mg/l in the total mix of community wastes received at the treatment plant is no problem, a higher concentration in individual plants effluent can be allowed. Such a concentration can be selected appropriate to the particular wastewater system. An analytical method for hydrocarbons has been available in standard methods since the Fifteenth Edition.

As a final point, I want to mention that in early literature, in manuals of practice of the Water Pollution Control Federation, in early textbooks on treatment technology, a limit of 100 mg/l on oil--total fats, oil and grease that is--was recommended. This was a mistake, was used in a way not originally intended, and is no longer a recommendation of the Water Pollution Control Federation or the United States Environmental Protection Agency. In spite of the fact that city after city has this limit in their ordinance it is not needed and is inappropriate. This is witnessed by the fact that very few cities enforce it. So my advice to you is not to follow that precedent.

Rainfall Runoff From Certain Areas of Industrial Plants

There is another rather minor appearing problem that I would like to call to your attention. The managers of municipal systems will in many cases do their very best to minimize the flow of rainfall runoff into the sewers leading to their treatment plant. Regulations will typically provide that no stormwater is allowed to be discharged to the municipal system. Campaigns to find roof drain connections and leaky manhole covers and the like are commonly conducted. Yet many industries will find themselves with a rainfall runoff from limited areas of the industrial plant which is polluted with leaks, spills, dusty materials, etc. The areas involved are those along the railroad siding or the truck stations where bulk materials are unloaded. Typically there will be pumps at these places. The locations I am speaking of will not be inside of buildings or under roof. Consequently the rain will fall on these areas, wash off any leakage or spill that is there and produce some polluted wastewater. A strict sewer use code will not allow this to be dumped into the sanitary sewer. Segregation and storage of this water is very costly and troublesome. We believe that the runoff from these limited spill prone areas should be considered a legitimate industrial waste and be allowed to be discharged with the other wastes to the sewer. The volume is small and will not stress the usual system. The industries generally provide spill protection such as curbs or retention tanks for these areas so that a large spill may be kept from the sewer. Sometimes discharge of rainfall can be delayed somewhat.

Control of Peak Loads From Industry

Another hard-to-deal-with subject that I mentioned in the introduction is control of industrial loads. The management of a municipal system must be supplied information on the discharge schedule, average daily volume, and average daily load of the significant industrial users. The information is needed for design, for operation, and for revenue program

planning. Likewise the peak load and flows expected must be known. Many industries have a difficult time maintaining a constant load for extensive periods of time. Equipment fails, raw materials change, people fail and some accommodation to variation on loading is a necessity. So there can be a problem with high short-duration volumes or mass discharge of pollutants--a high day or high week. A short period, high load--for say an hour or two is usually meaningless because the dispersion of the load in other flow in the system in the sewers and at the treatment facility will average out the peak so that the effect is the same as a daily or half day peak.

Most treatment facilities have a built-in ability to accept and adequately handle a peak daily load. Domestic sewage has a daily variation in flow and strength which allows off peak hours for catch up after unusual industrial loads are received. Activated sludge has a remarkable ability to handle a great increase without a marked impact on effluent quality.

A real potential problem with an excessive peak load would be when the effluent quality is significantly adversely affected. In most receiving water situations the damages from excursions out of the ideal water quality standards for short times is not serious. The criteria are based on long-term exposure. Short term modest excesses could not be considered serious.

A good way to regulate industrial peak loads via a general easily-interpreted meaningful ordinance has not been invented.

Limitations on concentrations is one approach. It is okay for toxics. For compatible pollutants it is too simplistic to warrant attention. The mass of pollutants is the critical issue. Any such concentration limit is anti-conservation of water. A concentration limit on compatible (easily treated) pollutants is not a good way to control loads.

Equalization requirements is another approach. Cities have been so extreme as to suggest 24-hour retention basins for evening out effluents to their system. Equalization basins require aeration to keep odors from developing. They need mixers to keep solids from accumulating. Equalization basins remove nothing. Generally they are not cost effective. The money would better be spent on more treatment plant capacity.

A definition of an unlawful load is a load the discharger knows or has reason to know will cause interference with the treatment facilities. This is an illusion as a practical solution in my opinion. The provision is an invitation for debate, for endless talk and controversy, and ill-will and external frustration.

What then can be recommended in the face of absence of precedence of practical control systems? My suggestion is to obtain needed control by addressing this subject in each industrial permit. The peak volumes and loads could be specified for those industrial dischargers large enough to

be of concern. Each specification could be tailored to specific circumstances. Real problems only could be addressed and most economic control assured.

Fair Charges to Users

Arriving at fair charges for wastewater services for all users---commercial, residential, and industrial---is a complicated matter. A rational and detailed analysis can produce a revenue program that can be defended as fair. Technical societies such as the American Society of Civil Engineers, the Water Pollution Control Federation, and the American Public Works Association have jointly sponsored manuals setting for the principles for a fair revenue program.

The basic philosophy of a fair charge system is that the users and beneficiaries of the system be made to pay in proportion to the costs of providing the use and benefit each received. Key to application of this philosophy is the proper recognition of the cost causative agents and the uses and benefits of the system. For instance in distributing capital costs each element of the system--the pipes, pumps, treatment tanks--is assigned to the appropriate cost-causing agent or agents. Pipes or sewers for example are sized to carry certain peak flows therefore the costs should be distributed according to the peak flow of each user. Another example would be the cost-causing agent assigned the air blowers for the activated sludge system. The air blowers are for the ultimate purpose of removing BOD and are sized to deliver enough air for the BOD load. Therefore, the capital cost of the blowers is assigned to BOD of each user.

A necessity to fairness of application of this principle is that all the uses must be recognized and all the cost-causing agents in the system recognized. The uses that are neglected in defective cost distribution systems can be the use for the conveyance and treatment of stormwater, the use for conveyance of infiltration, and the capacity for future users. In every system in spite of trying to exclude stormwater by careful construction there will be experienced significantly higher flows after storms. Likewise practically every system in this country experiences a great deal of leakage or infiltration. A representative case will have fifty percent of the annual flow attributable to infiltration. Logical planning indicates that considerable capacity for growth ought to be included in any conveyance and treatment system. These three uses and perhaps others are referred to as community costs or public costs by some rate engineers. The insinuation is that these uses cannot be identified with any particular user of the system. For example most of the stormwater will originate in the streets which are of course publicly owned. Infiltration comes into the sewers owned by the district in the streets largely. The revenue program for these public use related costs must be carefully thought out. The recommendation in the guidance referred to above is that these costs be considered a general obligation of the city and therefore should be assigned according to ownership of property in the city, in other words from property taxes. In many cases in this

country it is not practical to levy new taxes. Property benefit charges may be possible. An alternative is to distribute costs equally per customer or equally per customer in each class.

The point I wish to leave with you is that the simple concept of sharing costs proportionate to use and benefit is complex in its application. If public use is not recognized the large deliberate users such as manufacturing industry are assigned an unfairly large portion of the public use costs. For example an industry that discharges 25 percent of the flow would in a simple apportionment receive 25 percent of the costs for stormwater, infiltration, and future capacity in the system. This is obviously not fair. So I advise you to get the help of experts in designing your revenue program. There are engineering firms who have the expertise and experience.

Toxic Pollutants

The portion of this subject which I have left for last is the part of the subject of use by industry of a municipal wastewater system that has caused so much recent concern and is probably the first that comes in your mind when contemplating the subject. The topic is the discharge of toxics to the municipal system. In earlier years of control of industrial use of these systems the effort was directed largely at the metals such as copper, nickel, and zinc. Concerns were 1) interfering with the biological processes and 2) harmful effects on fish. In recent years the interests have broadened the meaning of a great range of effects such as causing birth defects, mutagens, cancer, etc., harm to any organism in the environment, bio-concentration, etc. The Federal law has focused attention on over 100 organic compounds with the potential of exerting some such environmental effect in wastewaters. Research on health and environmental effects being conducted will likely bring attention to other compounds in time. Experience to date indicate that a monitoring program will not be easy, it will not be cheap. A practical problem in analysis may make it very difficult to learn whether the material is present or not in the mix of biological residues in a treated effluent. The control scheme proposed by the Federal EPA could be so complicated that municipal administrators will find it too much administrative trouble to use. This could harm the cause of joint treatment severely. Even though the vast majority of such organic compounds will be subject to biological degradation and will, with good biological treatment, not be present in the effluent at harmful concentrations.

I do not mean to make light of the need for attention to control of toxics. Of course the public recreational waters, the desirable array of fish and other aquatic organisms, and particularly public water supplies, must be protected. We only ask for addressing only real problems and management of the significant risks.

We believe it is possible to control and manage the use of municipal systems for the treatment of many of these organic compounds through a variety of monitoring and control systems that would be developed for

each individual system. Practically speaking, each system is unique, the treatment may differ slightly, the mix of other wastes will differ, the dilution with other wastes will differ, the most economic way to limit the toxic may differ, and the concentration of concerns in the effluent may well differ from location to location. We feel the system that will work best and which the municipal managers will find most manageable and economic will be for them to first determine the concentration of the candidate toxics present in the wastes as discharged from the municipal treatment plant. Readily biodegradable toxics will not be found and the concern for them can be minimal. Any compound found above a concentration of concern can be controlled by such means as the management of the facility may choose. The choice may be to limit it severely at the source or limit the toxic just sufficient to equal the capacity of the treatment plant to satisfactorily remove or some other means.

Gentlemen, I do not wish to make light of a real problem. We want to be sure there is a real problem. We must use the most practical control system taking into account effectiveness, economics, including the utility of the product involved and manageability. I do not believe we will have trouble working such out in a cooperative effort.

Close

I hope you have become more aware of the value of joint treatment and industrial interest in it. Many industrial plant executives recommend their plant managers view the municipal wastewater treatment plant that serves them as an extension of their own manufacturing facility. The same regard is recommended to be held for its proper functioning and its quality of effluent as there would be for the functioning of the product manufacturing equipment and product quality.

The concern is illustrated by the story of one paper plant manager who received a call from the local sewage treatment plant manager. This industry made a great deal of colored paper. The treatment plant person said, "You have dyed my whole treatment plant blue, the primaries are solid blue and there is blue all over, even in the final tanks". Of course the industrial plant manager was concerned. He was anxious to do the best thing to correct the situation to the satisfaction of the treatment colleague. So he thought of the most accommodating thing he could do. So he said, "We have a variety of colors out here, what color would like the plant to be?"

That is a sick, sick joke of course. I tell it to emphasize that we are aware of the need to cooperate, to value, to nurture our colleagues in the waste end of the business. Please call on us if you think we could be of any help.

TABLE 1

NUMBER OF INDUSTRIAL PLANTS USING COMMUNITY SEWER SYSTEMS

<u>CITY</u>	<u>TOTAL USERS</u>	<u>SIGNIFICANT INDUSTRIAL USERS</u>	<u>PERCENTAGE OF TOTAL FLOW</u>	<u>TOTAL BOD</u>
Atlanta	100,000	66	15	--
Chicago	2,000,000	350	6	19
Dallas	230,000	119	10	50
Salem	50,000	18	9	39
South San Francisco	20,000	24	13	47

TABLE 2

USE OF COMMUNITY WASTEWATER SYSTEMS BY INDUSTRIAL CATEGORIES

<u>INDUSTRY CATEGORY</u>	<u>PERCENT OF INDUSTRIAL PLANTS PRACTICING JOINT TREATMENT</u>
Laundry detergents	95
Bar soap	99
Coffee	100
Edible oil refining	92
Fruit and vegetable canning	
Paper converting	
Steel rolling	0
Blast furnace	0
By-product cooling	0
Paper pulping	5

TABLE 3

CAPITAL COST OF SEDIMENTATION FOR VARIOUS PLANT CAPACITIES*

<u>CAPACITY OF PLANT mgd</u>	<u>CAPITAL COST OF SEDIMENTATION</u>		
	<u>TOTAL \$</u>	<u>PER mgd CAPACITY \$</u>	<u>PERCENT OF 1 mgd CAPACITY %</u>
1	\$ 42,000	42,000	--
10	\$160,000	16,000	38
100	\$920,000	9,200	22

*1971 costs

TABLE 4

CAPITAL COST OF DIFFUSED AIR SYSTEMS FOR ACTIVATED SLUDGE PLANTS

CAPACITY OF PLANT mgd	CAPITAL COST OF AIR SYSTEM		
	TOTAL \$	PER mgd CAPACITY \$	PERCENT OF 1 mgd CAPACITY %
1	65,000	65,000	--
10	320,000	32,000	49
100	1,820,000	18,200	28

*1971 costs

TABLE 5

CAPITAL COST OF TOTAL ACTIVATED SLUDGE PLANT*

CAPACITY OF PLANT mgd	CAPITAL COST		PERCENT OF 1 mgd CAPACITY %
	TOTAL \$	PER mgd CAPACITY \$	
1	550,000	550,000	--
10	3,200,000	320,000	58
50	11,000,000	220,000	40

*1967 costs

TABLE 6

LABOR COSTS FOR DIFFUSED AIR SYSTEM OPERATION

CAPACITY OF PLANT mgd	LABOR MAN HOURS/YEAR		PERCENT OF 1 mgd CAPACITY %
	TOTAL \$	PER mgd CAPACITY \$	
1	1,480	1,480	--
10	4,400	440	30
100	16,100	161	11

COST-EFFECTIVENESS AND WATER QUALITY
JUSTIFICATION FOR ADVANCED WASTEWATER
TREATMENT (AWT) FACILITIES

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I. INTRODUCTION

In 1977, the "Vertex" draft report⁽¹⁾ prepared by an EPA consultant concluded that many advanced wastewater treatment (AWT) projects funded by the U.S. Environmental Protection Agency (EPA) were too costly and resulted in few, if any, water quality benefits. AWT was basically defined as any treatment beyond secondary, which is the minimum required by law. The Vertex report recommended that all further funding of AWT projects be stopped until questions concerning the accuracy of water quality analyses used to justify these projects and the high project costs could be resolved.

Spurred by this report, and by a growing concern about high cost wastewater treatment projects, the Appropriations Conference Committee of the U.S. Congress issued a directive in October 1978 which required that EPA grant funding for AWT projects with incremental capital costs for treatment beyond secondary of greater than \$1.0 million may be provided only if the EPA Administrator "personally" determines that the project "will definitely result in significant water quality and public health improvement."^{*} The House and Senate Appropriations Committees raised the incremental cost to \$3.0 million for projects reviewed during fiscal year 1980. For projects with lower marginal costs, AWT approval is given by Regional EPA Administrators.

To implement the requirements of this directive, EPA issued Program Requirements Memorandum (PRM) 79-7, which outlined the criteria for review of AWT projects. PRM 79-7, which requires that effluent limitations for AWT projects must be fully justified by technically sound water quality analyses.^{**} This will ensure that expenditures for AWT processes will result in significant water quality benefits; however, this evaluation does not involve a cost-benefit analysis in which the "worth" of the benefits is weighed against the cost. Rather, given a specified water use goal, this analysis seeks to assure that this goal will be achieved at minimum cost.

*There have been various arguments that this directive is inconsistent with the Clean Water Act, and Section 510 of the Act in particular which allows States to set water quality standards more stringent than the Federal minimum if they choose. The main question is whether the Act requires EPA to fund projects to meet these standards.

**These criteria include evaluation of water quality modeling, appropriateness of beneficial use classifications and water quality criteria, and a review of cost-effectiveness. The State of Illinois (IEPA) sued EPA in December 1979, claiming that the AWT review violated the Clean Water Act. IEPA and EPA reached an out-of court settlement on the case, and EPA agreed to simplify some of the review requirements. As a result of this settlement, a final revised PRM will be issued and specify the review criteria to be used in future AWT reviews nationwide.

An EPA Headquarters AWT Task Force has reviewed 68 AWT projects, with capital costs for AWT of over \$500 million. As of this writing, these reviews have resulted in deferral of EPA funding for over \$114 million worth of unjustified AWT processes. The findings in these reviews have created intense controversy among affected cities, states, EPA regional offices, and EPA Headquarters.

This paper explains the rationale for the review criteria and approach used in the AWT review at EPA Headquarters, provides an assessment of removal capabilities and marginal costs of various AWT processes, and describes case studies to indicate the results and implications of the reviews conducted to date. This paper also analyzes the accuracy of water quality models used to justify AWT processes.

II. REVIEW CRITERIA AND APPROACH

A. General Approach

In typical water quality management planning studies, water quality analysts established permit effluent limitations based on results of water quality modeling studies, but without regard for the costs of unit processes required by those limitations. Engineers were then required to design facilities to meet these limitations, regardless of the uncertainties in the water quality analysis, or the costs of the unit processes that were required. The efforts of the water quality analyst and the design engineer stopped at opposite ends of the pipe, with neither venturing into the sphere of the other.

Now this may be changing. The justification for AWT projects reviewed in Headquarters has basically involved balancing the uncertainties in the water quality analyses against the marginal costs of the unit processes being considered. For unit processes with relatively high marginal costs, more rigorous water quality analyses have been required than for unit processes with lower marginal costs. This approach allows more flexibility in establishing permit limitations.*

*This approach in effect results in deferral of EPA funding in cases where it is not conclusively shown that these processes are needed. It has been argued that this approach is inconsistent with the Clean Water Act, which requires that a "margin of safety" must be provided to compensate for uncertainties in the water quality analysis. This may be a valid argument, although the definition of a "margin of safety" is subject to interpretation. Nevertheless, EPA made a policy decision to carry out the Congressional directive in this manner.

B. Evaluation of AWT Processes

Nitrification and Tertiary Filtration Systems

A summary of the types of advanced unit processes proposed and approved by the Headquarters AWT Task Force appears on Table 1. As shown on this table, the most commonly proposed AWT processes were various nitrification and tertiary filtration systems. In nearly all cases where filtration was proposed, it was an add-on to nitrification rather than following secondary treatment.

Table 1 shows that nitrification was proposed in 55 cases and approved in 51 cases. On the other hand, tertiary filtration was proposed in 48 cases and approved in only 21 cases. Moreover, in many cases where filtration was approved, approval was based on a consideration of the inflationary impacts that would result from delays for redesign following elimination of filtration, rather than on technically sound water quality analyses. Thus, the number of cases where there was adequate water quality analyses to justify filtration was less than the approvals indicated by Table 1.

The high approval rate for nitrification, and low approval rate for filtration, resulted from two major reasons. First, nitrification has a relatively low marginal cost per unit of ultimate oxygen demand (UOD)* removed, and removes a large percentage of UOD. Filtration, on the other hand, has a high marginal cost per unit of UOD removal, and removes only a small percentage of UOD.**

The second reason involves the predictive accuracy of water quality models. Generally, it is relatively easy to develop water quality models accurate enough to determine whether the level of UOD removal provided by nitrification is needed. However, since filtration following nitrification removes only a small percentage of UOD, inherent uncertainties in water quality modeling make it more difficult to accurately predict whether filtration is needed. These issues are discussed in more detail in the following sections.

*UOD is defined as the total carbonaceous and nitrogenous oxygen demand. This may be estimated as follows:

$$\text{UOD, mg/l} = 1.5 \times \text{CBOD}_5, \text{ mg/l} + 4.57 \times \text{NH}_3\text{-N, mg/l}$$

where CBOD_5 = total carbonaceous BOD_5 which includes SS.

**Filtration should obviously not be used to remove UOD after nitrification since little UOD remains in the wastewater. Filtration could be used for suspended solids removal, total phosphorus removal, or for disinfection.

Cost Effectiveness

A schematic diagram showing the removal capabilities of a typical AWT facility providing nitrification followed by tertiary filtration appears on Figure 1. This figure shows that nitrification will reduce ammonia concentration from about 20 mg/l (following secondary treatment) to an average of about 1 mg/l, and reduce CBOD₅ concentrations from 30 mg/l to about 8 mg/l. Nitrification thus increases UOD removal from 67 percent to 96 percent. In addition, nitrification has the added benefit of reducing ammonia toxicity in the receiving water.

Graphical presentations showing the marginal present worth cost per mg/l UOD removed for secondary, nitrification and tertiary filtration appear on Figure 2.* The figure shows that the marginal cost per mg/l UOD removed by nitrification is significantly less than for tertiary filtration, and even less than the marginal cost per unit of UOD removed for secondary treatment. Specifically, Figure 2 shows that the marginal present worth cost per mg/l UOD removed by nitrification in a typical 10 MGD two stage activated sludge system is about \$24,000, as compared to a marginal cost for secondary treatment of about \$42,000 per mg/l UOD removed. The marginal cost for tertiary filtration following nitrification is over \$1.0 million per mg/l UOD removed. Thus, the high approval rate for nitrification and low rate for filtration in part resulted from differences in the marginal costs and removal capabilities of these processes.

Predictive Accuracy of Water Quality Models

These findings have significant implications for the water quality analyses used to justify nitrification and tertiary filtration. Since nitrification has relatively low marginal costs and provides significant reduction in UOD, simplified water quality analyses were often adequate to justify this process.** On the other hand, since filtration has a high marginal cost for UOD removal and removes only a small amount of UOD, more detailed water quality analyses, often involving a calibrated and/or verified water quality model, were generally required. More accurate water quality models were also generally required to justify filtration because it is more difficult to predict in-stream responses to the relatively small UOD removals provided by filtration than for the relatively large UOD removals provided by nitrification.

*These estimates are based on the "typical" values shown on Figure 1, and would vary from plant to plant. UOD removal efficiencies assume CBOD₅ tests are used.

**New guidance for using simplified DO modeling techniques for justifying filtration was recently issued by EPA. These techniques require sensitivity analyses to justify filtration, and would require gathering calibration and/or verification data if the results of the sensitivity analysis do not conclusively establish that filtration is needed.

The adequacy of simplified DO models and calibrated/verified DO models for justifying AWT processes depended on the technical accuracy of the model, and the marginal cost and removal capabilities of the process in question. In general, simplified approaches were adequate to justify nitrification, but calibration and/or verification data were generally required to justify tertiary filtration. The case studies presented in Section III provide several examples where simplified and calibrated DO models were used to justify AWT processes, and explain the rationale for accepting or rejecting these justifications.

The most significant parameters affecting the predictive accuracy of water quality DO models are the deoxygenation rates (K_d day⁻¹) (i.e., including both day⁻¹ carbonaceous and nitrogenous deoxygenation), and the reaeration rates (K_a). In simplified models, these rates are determined based on literature values for similar water quality scenarios, from data from nearby similar receiving streams, or, for K_a 's, from empirical formulas. Since the rate constants used in these simplified approaches is not based on site specific data, the accuracy of these models is limited.

For calibrated and verified model, rate constants are determined based on site specific in-stream measurements. For example, carbonaceous BOD (CBOD) decay rates would be determined by measuring CBOD decay in-stream. However, the accuracy of these rates to predict future water quality impacts of treatment levels may be questionable, since K_d values are generally lower at higher treatment levels, and the exact amount of the reduction cannot be estimated with certainty. This concern can be reduced if the water quality analyst performs adequate sensitivity analyses within the range of typically expected K_d values.

Nitrogenous oxygen demand (NOD) decay may be determined by measuring ammonia decay. However, this approach may have to be modified in cases where high algal populations exist, since ammonia depletion may actually result in large part due to algal uptake, rather than oxidation. In these cases, the rate of nitrate increase may be a more accurate measure of ammonia oxidation.

The greatest amount of variability in most DO models involves K_a . There are several empirical formulas available for estimating K_a (e.g. O'Connor - Dobbins⁽²⁾, Tsivoglou, et al.⁽³⁾, Owens, et al.⁽⁴⁾), and suitability of each varies depending on the characteristics of the receiving water. For a typical low flow stream, K_a values may vary anywhere from 1.5 to 7.0 per day, base e. It is not unusual to find K_a values as high as 20 per day in some models. On the other hand, typical K_d values resulting from discharge of a well treated secondary or nitrified effluent into a low flow stream generally range from about 0.3 to 0.6 per day, base e. Thus, the possible range for K_a is much greater than for K_d .

K_a values in most water quality models reviewed in EPA Headquarters were determined from empirical formulas. These K_a values were usually adjusted to fit observed DO data, where available, which is the generally

correct procedure for model calibration. However, the accuracy of this approach is often uncertain since the initial difference between observed and predicted DO may be due to factors other than inaccuracies in the estimate of K_a .

The most accurate means for determining K_a is use of the inert gas tracer⁽³⁾ technique. This approach basically involves actual measurement of gas transfer in the receiving water in question. However, even this approach has inherent uncertainties, because of differences in conditions at the time of the gas tracer measurement and the prediction conditions, and because of uncertainties in measurement techniques. Thus, K_a rates estimated by gas tracer techniques may also have to be adjusted during calibration.

There are also several other variables that may introduce additional uncertainty into water quality models. These include measurement and prediction of sediment oxygen demand, effects of algal activity, and background DO concentrations. These factors are not addressed in detail in this paper, but should be considered in determining the accuracy of DO models, and the expected improvements from AWT.

Despite these uncertainties, even simplified models are often adequate to show whether nitrification is needed, since the DO impacts of the UOD removed by nitrification are generally much greater than the uncertainties in DO models. However, in the case of tertiary filtration following nitrification, the confidence limits of DO models (even with calibrated and verified models) often exceeds the predicted incremental DO benefits resulting from providing tertiary filtration following nitrification. Therefore, water quality models are often not accurate enough to show whether tertiary filtration is definitely required following nitrification to meet a given DO criteria.

The effects of the uncertainties in DO models on the ability to determine the need for filtration following nitrification are illustrated on Figures 3, 4, and 5. Figure 3 shows the expected DO improvement resulting from providing filtration after nitrification, as a function of varying K_a to K_d ratios, assuming a stream to effluent dilution ratio of 1:1. For example, for $K_a/K_d = 6$, filtration following nitrification would improve DO by about 0.25 mg/l.

Figure 4 shows estimated confidence bands for the DO deficit resulting from discharge of a nitrified effluent. These confidence limits were estimated by assuming that K_a has been determined with certainty (i.e., either via calibration or using gas tracers), and that K_d could vary by 25 percent to 75 percent. Uncertainties in estimating loadings sediment oxygen demand (SOD), algal effects, etc., were not considered. Thus, this Figure represents a minimum amount of uncertainty associated with a calibrated DO model. However, even given these conditions, Figure 5 shows that for a $K_a/K_d = 6$, the DO deficit resulting from discharge of a nitrified effluent would be 1.0 mg/l, but that it could range anywhere from 0.7 mg/l to 1/3 mg/l due to modeling uncertainties (i.e. uncertainties in K_d).

To illustrate the effects of these uncertainties on determining the need for tertiary filtration, Figure 5 shows that the DO improvement resulting from providing tertiary filtration falls within the range of confidence limits associated with the effects of a nitrified effluent.

An example illustrates the effect of this problem. Assuming a $K_a/K_d = 6$ and a background DO of 5.8 mg/l, Figure 5 shows that discharge of a nitrified effluent would depress DO by about 1.0 mg/l to 4.8 mg/l. This is below the DO criteria for warm water fisheries. If filtration were provided following nitrification (and K_a/K_d again = 6), the DO would be depressed by about 0.7 mg/l, to about 5.1 mg/l. This is above the DO criteria. However, if K_d is reduced to 0.3 per day, which is well within the acceptable range for K_d 's, DO resulting from discharge of a nitrified effluent would only be depressed by about 0.6 mg/l, to about 5.2 mg/l. This also is above the DO criteria. Thus, it would not be possible to determine whether tertiary filtration following nitrification would definitely be needed to achieve DO criteria, or whether nitrification alone would be adequate to achieve this DO level. If other modeling uncertainties are introduced, the ability to determine the need for tertiary filtration would be limited even further. These uncertainties warrant particular attention in view of the high marginal costs associated with tertiary filtration for UOD control.

There are two possible options for resolving this issue. One is to construct only the nitrification facilities, and then to monitor water quality to determine whether more treatment is needed. This approach has the advantage of providing precise site-specific data for determining treatment needs; the disadvantage is that delays while data is monitored and analyzed would increase costs for additional treatment if needed, and could prolong discharge of inadequately treated wastewater.

The other option is to use post-audit water quality data from similar nitrification facilities to refine modeling accuracy and more precisely determine whether tertiary filtration is needed. However, while this approach could reduce modeling uncertainties, and preclude project delays, it would still be limited in its predictive accuracy.

C. Phosphorus Controls

Phosphorus removal was proposed for 24 projects and approved for 16 (see Table 1). In most cases where phosphorus removal was approved, the projects discharged either into the Great Lakes or the Chesapeake Bay, where there had been extensive studies of the potential impacts of phosphorus on the trophic state of the receiving waters.* For dischargers into other water bodies, adequate water quality analyses existed in only a few cases. However, where phosphorus removal was not justified, it was recommended that the plant be built to account for the possibility that phosphorus removal may be required in the future, in order to avoid possible retrofit problems.

*These studies have resulted in an international agreement between the U.S. and Canada which limits phosphorus discharges into the Great Lakes and their tributaries to 1.0 mg/l for all municipal discharges greater than 1 MGD.

The primary reason for the lack of adequate water quality justification for phosphorus removal is the inherent complexity of accurately modeling phosphorus-chlorophyll relationships. Modeling the effects of phosphorus requires an assessment of non-point loadings, and bio-assay studies to identify the limiting nutrient. Other complicating factors, such as predicting reaction rates, bio-feedback, turbidity, light penetration, etc., also make this analysis relatively complex.

Because of the complexities mentioned above, EPA Headquarters has re-evaluated the criteria for approval of phosphorus removal. This re-evaluation considered the following issues:

- ° modeling of phosphorus-chlorophyll relationships is difficult, and could not be done in many cases due to limitations in man-power, technical expertise, etc.
- ° potential adverse impacts from not controlling phosphorus could be severe and difficult to reverse
- ° capital costs for phosphorus control are relatively low, at least down to about the 1.0 mg/l level. (Clarification would generally not be capable of meeting effluent limits below 1.0 mg/l. Thus, more stringent limits would require some type of filtration system or other costly processes)

Based on these considerations, simplified procedures are being developed for justifying phosphorus removal.* Criteria for justification using simplified procedures include:

- ° demonstrating that there is an existing or potential (marginal) phosphorus-related water quality problem
- ° if the water quality problem is only marginal, demonstrating (supporting data) that phosphorus loadings will increase significantly
- ° demonstrating that phosphorus is the limiting nutrient
- ° demonstrating that point sources contribute a significant portion of total phosphorus loading
- ° demonstrating that phosphorus controls will not result in significant cost impacts (both capital and operating costs)

These criteria will be applied to future projects where phosphorus removal is proposed, and may also be applied to projects where EPA funding for phosphorus removal has previously been deferred.

*The decision to allow simplified approaches to evaluate phosphorus removal evolved during the review process. Specific criteria and procedures to be used in this regard will be included in the revised PRM, to be issued shortly, and other subsequent water quality guidance.

D. Separate Stage Denitrification

Separate stage denitrification was proposed in only two cases, and was not justified in either case. Like phosphorus, the impact of nitrates on a receiving waters' trophic state is relatively difficult to model. Even predicting future nitrate concentrations may be difficult since denitrification could occur in-stream. However, unlike phosphorus removal, both capital and operating costs for separate stage denitrification with methanol are high.* Therefore, justification for nitrogen control would have to be more rigorous than for phosphorus controls. This justification may involve gathering field data to calibrate the results of model predicted in-stream reactions, as well as nutrient limiting studies, and an assessment of non-point sources.

III. CASE HISTORIES

Manasquan, New Jersey

The Manasquan project provides a good example of the use of simplified modeling procedures for justifying effluent limitations. The proposed Manasquan project is a new 8.1 MGD oxidation ditch unit, using the Carrousel system. The project involved regionalizing several small package plants located further upstream. The project had originally included tertiary filtration following the Carrousel unit, but the Regional EPA Office in New York determined that the filters were not justified.

The facility would discharge to the Manasquan River Estuary, several miles upstream from the Atlantic Ocean. The river is designated for contact recreation, fishing and shellfish harvesting. Shellfish harvesting had been discontinued due to pollution from upstream package plants. The DO standard for the river is 5.0 mg/l.

The effluent limitations included a 10 mg/l limitation for CBOD⁵, and a 2.0 mg/l seasonal limitation for ammonia, which applied from May through October. A simplified desk-top water quality (estuary) model was used by the EPA Regional Office to verify the need for these effluent limitations. Since no discharge existed near the proposed outfall, the Kd rate constants used in the model had to be based on literature values. To account for inherent inaccuracies in using these literature values, the Region performed sensitivity analyses to indicate the in-stream DO responses under various possible scenarios. Results of this analysis showed that the allowable UOD loading ranged between 400 lbs/day and 2,000 lbs/day, depending on which rate constants were used. The Regions' "best" estimate of allowable UOD loading was 1,200 lbs/day. If secondary treatment were provided, the effluent UOD loading would be 12,700 lbs/day, or more than six times the maximum estimated allowable concentration. If tertiary filters and nitrification were provided, effluent UOD loading would be 750 lbs/day. If the Carrousel system were used without filtration, the UOD loading would be about 1,600 lbs/day.

*Where methanol is not used for denitrification, partial denitrification (~ 80% removal) in single sludge in extended aeration or Carrousel systems would not be high.

The marginal present worth costs per mg/l UOD removal for secondary, nitrification and tertiary filtration for this project are shown in Figure 6. Figure 6 shows that the marginal costs per mg/l UOD removed for secondary and nitrification are about \$76,000 and \$13,000, respectively. The total cost for nitrification was estimated to be only about \$1.0 million. In contrast, the marginal cost per mg/l UOD removed for filtration would be over \$1.0 million. The capital cost for filtration would be about \$3.0 million. Because the simplified analysis showed it is very likely that treatment beyond secondary is needed, and because the marginal cost for nitrification is relatively small, the Region concluded that the Carrousel unit is justified. Although the UOD loading from the Carrousel process without filtration would be slightly greater than the "best estimate" allowable UOD loading, the Region concluded that the filters were not justified because of their relatively high marginal cost, and because of the inaccuracies inherent in the simplified mode. The Headquarters AWT Task Force concurred with these conclusions.

Rochester, New Hampshire

The Rochester project provides an example where both nitrification and filtration were justified.

The proposed project involves construction of a new 4 MGD facility, using two-stage nitrification, followed by dual media tertiary filtration. The city currently discharges raw sewage. The nitrification system would include a roughing trickling filter, followed by separate stage activated sludge.

The receiving water is the Cocheco River. The designated uses (recently upgraded) of the river include swimming and cold water fishery. Large oyster beds exist downstream, but these have been closed due largely to pollution from Rochester. Thus, one potential benefit from the project would be to re-open the oyster beds. In addition, the State is implementing plans, including installing fish ladders, to establish salmon spawning.

The critical design flow (7 day once in 10 year low-flow) in the river is about 2.2 cubic feet per second (CFS). The river is currently highly polluted due to the Rochester discharge. The water quality standards require a DO of 6.0 mg/l or 75 percent of saturation, whichever is greater.

The design effluent CBOD₅ and ammonia limitations were 5.0 mg/l and 1.0 mg/l, respectively. A calibrated (but not verified) DO model was used to determine these limitations.

In its original final AWT report, the Headquarters AWT Task Force concluded that the water quality analyses did not justify the need for the dual media filters. This conclusion was reached because the proposed tertiary filtration system had a cost of about \$1.3 million, and would provide only slight additional reduction in UOD. Similarly to other projects, the marginal cost per mg/l UOD removed for filtration vis-a-vis secondary treat-

ment and nitrification was very high. In addition, since the city currently discharges raw sewage, there were uncertainties in the water quality analyses because of inherent difficulties in estimating rate coefficients under future conditions with highly treated effluents.

Following a review of this report, the State submitted new information supporting the need for tertiary filtration. This information provided lower cost estimates which showed that there would be negligible savings if the filters were eliminated. These lower costs resulted from changing the filtration system from dual media to sand, which reduced the capital cost of the filters from about \$1.3 million to about \$550,000.

In addition, the State noted that additional chlorine would be required without filters because of higher suspended solids concentrations. This would necessitate dechlorination, further reducing potential savings from eliminating the filters. Using these assumptions, the present worth cost of retaining the filters exceeded the cost without filters by only about \$95,000.

The Task Force also considered the possibility that further water quality analyses would justify filtration. Despite some questions and shortcomings concerning the existing water quality modeling efforts, the overall work was reasonable. The water quality model was properly calibrated, and showed that filtration is needed. Thus, it was not unlikely that verification of the existing model might also show that filtration is required.

Finally, more specific evidence was provided showing steps being taken by the State to establish a salmon fishery, including spawning grounds, below the discharge point. The State also planned to re-open oyster beds that are now closed due to pollution from Rochester. Although not demonstrated (and difficult to prove), the reduced suspended solids loading with the filters might enhance these uses. In view of these considerations, it was considered likely that filters would prove to be justified if additional water quality studies were conducted.

Based on the above considerations, the Task Force concluded that sand filters were justified for this project.

IV. CONCLUSIONS

A rigorous review of water quality related effluent limitations should occur prior to facility planning. Such review should provide for flexibility in establishing effluent limitations by allowing the marginal costs for AWT processes and uncertainties in water quality analyses to be weighed before final effluent limits are set. Generally, water quality analyses for AWT processes with higher marginal costs should be more rigorous than analyses for processes with lower marginal costs.

Nitrification and tertiary filtration were the most commonly proposed AWT processes. Nitrification was justified in most cases because it has a relatively low marginal cost per unit of UOD removed, and because water quality models are usually accurate enough to determine whether this level of treatment is needed. Tertiary filtration was not justified in most cases because it has a relatively high marginal cost per unit of UOD removed and because inherent uncertainties in water quality modeling make it difficult to accurately predict whether the level of treatment provided by filtration following nitrification is needed to meet water quality standards.

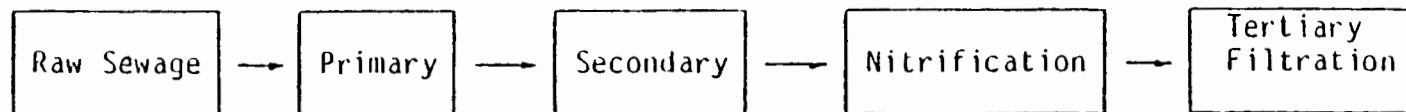
Water quality analyses for justifying phosphorus or nitrogen removal are complex. Since phosphorus removal is generally relatively inexpensive, simplified water quality analyses are often adequate. Since nitrogen removal is more costly to remove, more sophisticated analyses, possibly involving a calibrated model, may be required.

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FIGURE 1

Two Stage Nitrification -- Typical Removal Efficiencies



	Raw Sewage	Primary	Secondary	Nitrification	Tertiary Filtration
CBOD ₅ (mg/l)	200	130	30	8	4
(% Removal)		35%	85%	96%	98%
NH ₃ -N (mg/l)	25	25	20	1	1
(% Removal)		0%	20%	96%	96%
UOD (mg/l)*	414	309	136	17	12
(% Removal)		25%	67%	96%	97%
Cost (\$ million)**			11.7	2.87	3.5

* UOD = 1.5 (CBOD₅) + 4.57 (NH₃-N)

** Based on 10 MGD facility

FIGURE 2
MARGINAL COSTS FOR TYPICAL
TWO-STAGE 10 MGD NITRIFICATION FACILITY

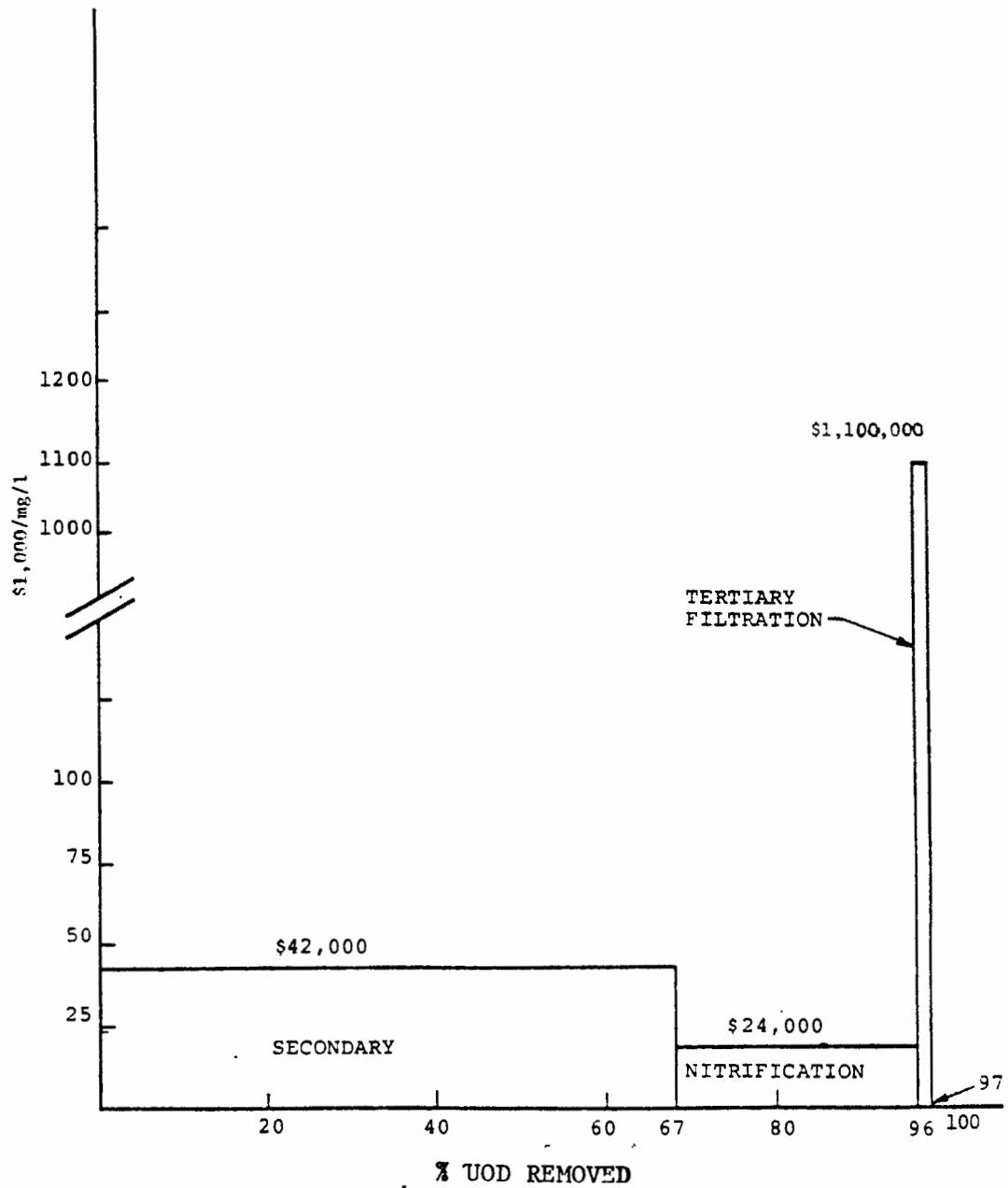


FIGURE 3

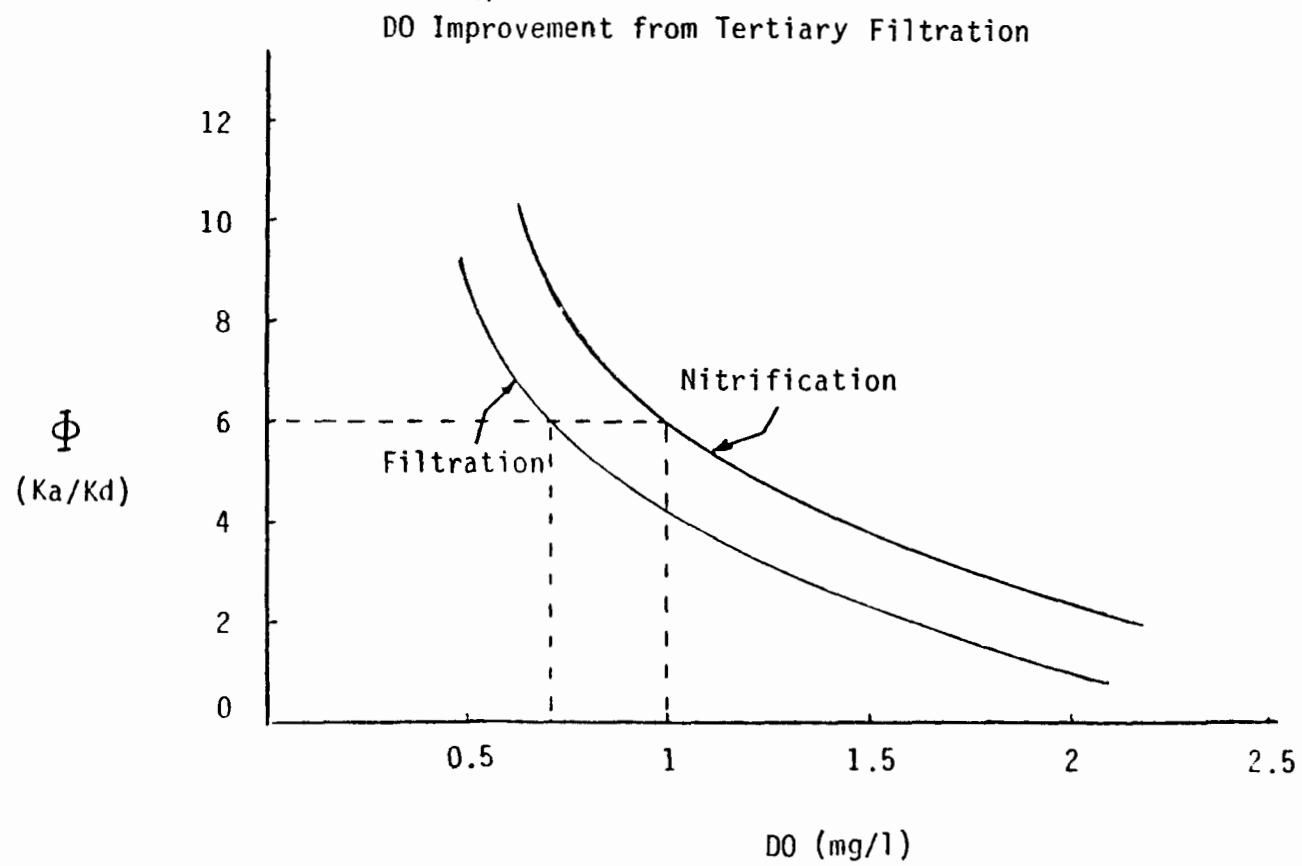


FIGURE 4

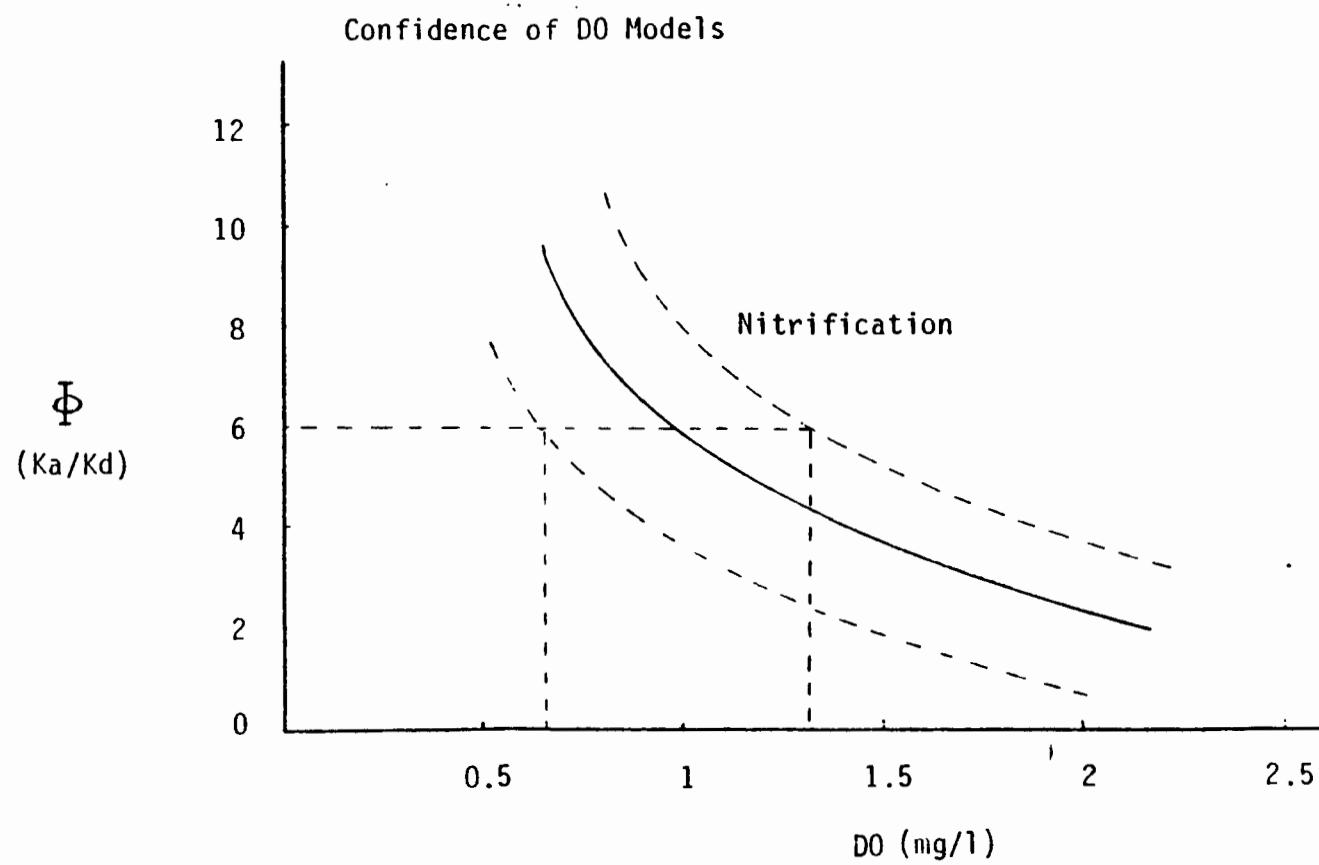


FIGURE 5

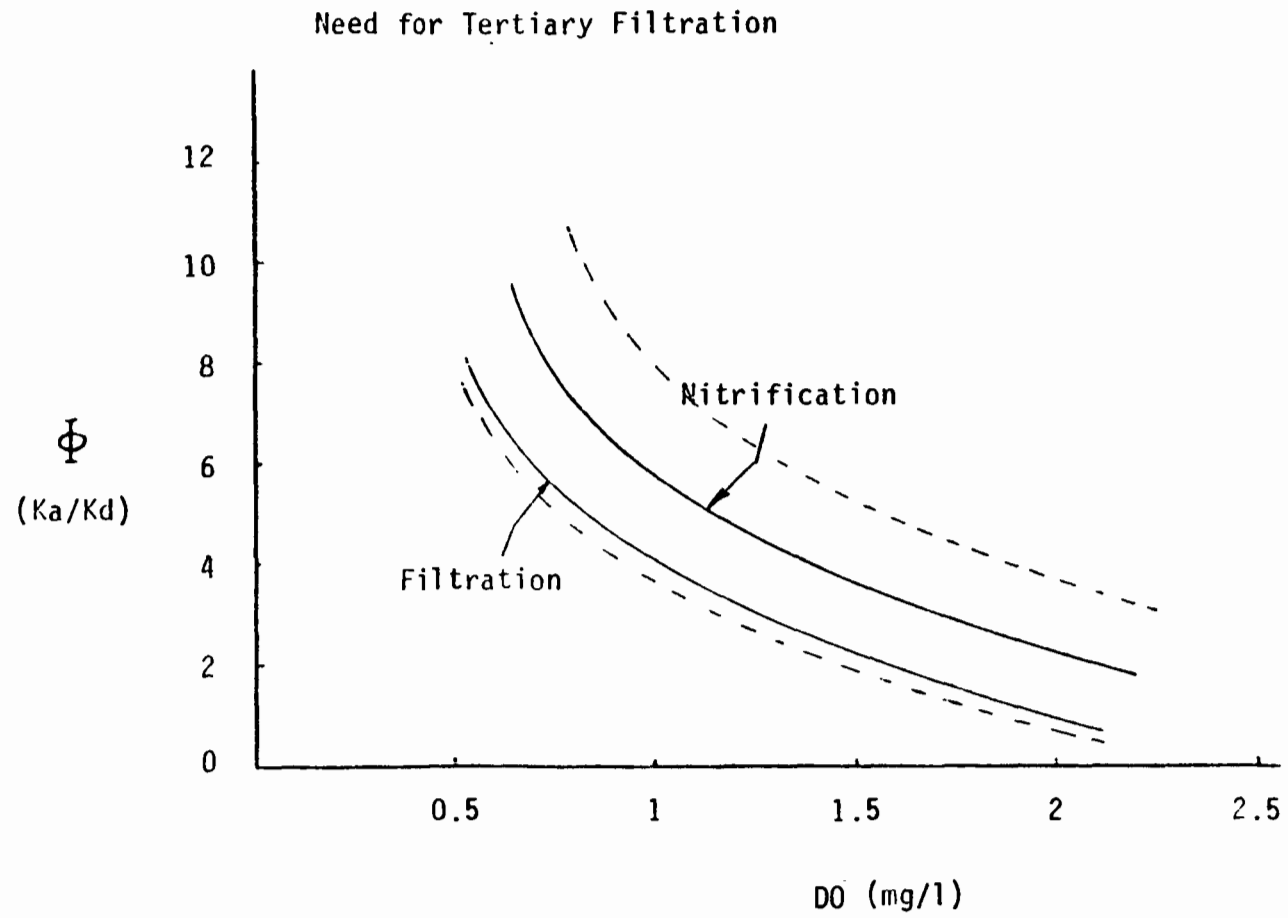


FIGURE 6
 OXIDATION DITCH - MANASQUAN, N. J.
 8.0 MGD - REMOVAL COSTS

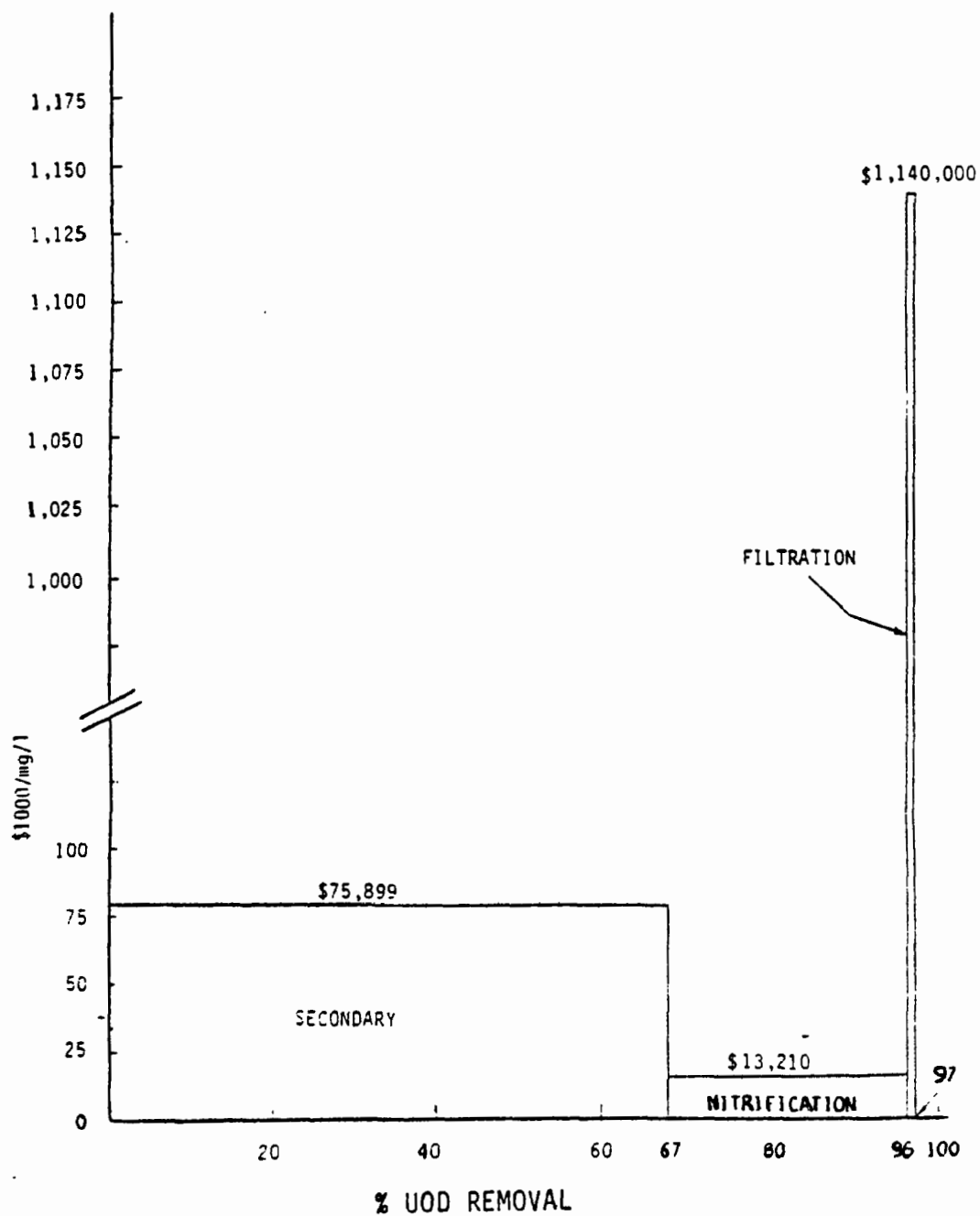


TABLE I

ADVANCED TREATMENT PROCESSESPROPOSED AND APPROVED

<u>PROCESS</u>	<u>NUMBER PROPOSED</u>	<u>NUMBER APPROVED</u>
Nitrification	55	51
Tertiary Filtration	48	21
Phosphorus Removal	22	14
Denitrification	2	0

EFFECTS OF MULTIPLE DIGESTION ON SLUDGE

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ABSTRACT

This paper presents the development and application of the mesophilic-thermophilic process that has been pioneered by the City of New York at their Rockaway wastewater treatment plant. This was accomplished by the use of a two-stage digestion system, consisting of a mesophilic stage followed by a thermophilic stage. A part of the thermophilically digested sludge was also recycled through the aeration tanks to obtain additional destruction of organic solids. The advantages of the thermophilic process are retained without the disadvantages. Results indicate that the physical characteristics of meso-thermo digested sludge are changed to the extent that the economics of dewatering are significantly improved. Moreover, has met the time-temperature requirements for pathogen destruction.

INTRODUCTION

Coastal cities, including the City of New York, being under Federal mandate to cease ocean dumping of sludge derived from the treatment of wastewater, have been engaged in studies of land-based disposal alternatives for the past couple of years. Many of these studies were aimed at determining the optimal methods of dewatering digested sludge as well as the subsequent steps for ultimate land disposal.

In studying the work performed by Kraus in 1946(1), it was noted that exposing volatile solids to both anaerobic and aerobic environments resulted in improved destruction of volatile solids. This led to the idea that alternate exposure of volatile solids to different environments could substantially reduce the quantity of sludge for ultimate disposal. The work of Kraus when considered in conjunction with the work of Buhr and Andrews(2) on the thermophilic digestion process led to the concept of the new process proposed herein. The idea was advanced that, as a fundamental and first priority part of the management program, present plant facilities should be tested for use in reducing to a minimum the rate of sludge production from an activated sludge plant, both as to volume and volatile matter. The rationale would be based on the exploitation of biochemical mechanisms; namely, that improved destruction of volatile solids could be

obtained by exposing the mesophilically digested solids to the enzyme systems of thermophilic digestion and activated sludge. Advantage gained in this investigation would be reflected commensurately in the economics of all the sludge dewatering and post-dewatering processes that were previously studied.

Present Practice of Thermophilic Anaerobic Digestion

Thermophilic anaerobic digestion is very similar to mesophilic anaerobic digestion except the temperature at which it operates is 120-130⁰ F instead of 90-100⁰ F. It thus takes advantage of the fact that biochemical reaction rates can be increased by increasing temperature. It is only natural, therefore, that conversion of existing mesophilic digesters to thermophilic operation should be considered as a low-cost technique for increasing the sludge processing capability of wastewater treatment plants. Full-scale studies by the Metropolitan Sanitary District of Greater Chicago, (3), the Ontario Ministry of the Environment, Canada, (4) and in Moscow, U.S.S.R. (5) have indicated that the sludge processed per unit volume of digester capacity could be doubled by converting from mesophilic to thermophilic operation.

Besides its increased sludge processing capability, thermophilic operation also offers two other significant advantages over mesophilic operation: improved sludge dewatering characteristics and increased destruction of pathogens.

Garber's work on the vacuum filtration of thermophilic sludge at the Hyperion plant in Los Angeles provides an example of how sludge dewatering can be improved by the thermophilic digestion.(6) He reported a 270 percent increase in vacuum filter yields with a 48 percent decrease in coagulant dosage for thermophilic, compared to mesophilic sludge. Improved solids-liquid separation is important in land application of sludge by decreasing the quantity of wet sludge for disposal and thus lowering transportation costs.

An example of the increased destruction of pathogens by thermophilic digestion is given by Popova and Bolotina (5) in their report of the practice of thermophilic digestion in Moscow, U.S.S.R. They state: "The most essential advantage of this process is the sanitary quality of the thermophilic sludge. According to the sanitary officials of the health department, viable eggs of helminths are absent from such a sludge." This improvement in sanitary quality is of special significance in light of the current trend toward land disposal of digested sludge.

Development of the Mesophilic-Thermophilic Process

The Rockaway wastewater treatment plant, having a connected population of 100,000, was chosen for a full-scale test. The plant employs conventional facilities for the activated sludge process, and the sludge generated undergoes mixed primary and secondary sludge thickening prior to mesophilic

digestion. The digested sludge is transported to sea. As presently operated, the primary tanks provide a detention of about 2 hours; the aeration tanks provide 3.3 hours, with step feed provisions; and the final tanks provide 3 to 5 hours of settling depending on the number in use. Two 45-ft. diameter thickening tanks are used for mixed sludge thickening and the mesophilic digestion is accomplished in a 1 cu.ft./capita tank.

For purposes of this test, the following steps were taken: (1) an additional 1 cu.ft./capita digestion tank was placed in service to receive the overflow sludge from the mesophilic digester and its contents heated up to 120-122° F, the lower limit of the thermophilic digestion range; (2) piping was installed to carry a portion of the overflow from the thermophilic digester directly to a single 45-ft. diameter tank to be employed as a rethickening and elutriating tank; (3) piping was installed to conduct the remainder of the flow from the thermophilic digester into the primary effluent and thereby directly into the aerator of the secondary treatment system, and (4) city water was conducted to the elutriation tank. The elements of this new method of sludge processing therefore involved: (1) subjecting the mesophilically digested sludge to subsequent thermophilic digestion; (2) recirculating part of the sludge leaving the thermophilic digester directly to and through the secondary treatment system; and (3) subjecting the other part of the sludge leaving the thermophilic digester to a rethickening and elutriation step.

The thermophilic digester was placed in operation in September 1979. On January 15, 1980, the necessary piping additions were completed and on that date the recirculation and rethickening elements of the new method were brought into service.

Effect of Recirculation on Process Performance

At the time the full-scale test was started, the activated sludge had a rather low sludge density index of 0.6 to 0.7. Microscopic examination revealed a significant population of bacterial filaments along with colonies of stalk ciliates and some rotifers. After the digested sludge recirculation was in practice for only a few days, the sludge density index was found to have risen to 1.0 and the bacterial filaments were found to have diminished substantially. During the entire course of the following test, the sludge density index continued to lie in the stable range of 1.0 to 1.4.

Since the flow received at the plant approximates 200 gals/capita/day, the influent wastewater strength is low, averaging about 100 mg/l each of suspended solids and BOD₅. Prior to the test, the suspended solids and BOD₅ in the effluent averaged about 12 and 12 mg/l, respectively. The monthly treatment results for the prior period July to December 1979 are presented in Table I, as well as the treatment results during the course of this test run from January 15 through May 29, 1980, for comparative purposes. It can be seen from these data that no significant effect on treatment efficiency was experienced as a result of the continuous recirculation of digested

Table I

Rockaway Wastewater Treatment Plant (WTP) Treatment Efficiency - July 1979 to May 1980

Month	Flow (M.G.D.)	Influent Wastewater		Final Effluent	
		SS (mg/l)	BOD ₅	SS (mg/l)	BOD ₅
July	22	88	111	12	13
August	22	83	117	16	12
September	23	93	90	12	10
October	25	116	103	18	12
November	22	140	112	13	11
December	21	125	124	10	11
Average Pre-Test		107	109	14	12
January (15-31)	21	86	91	8	8
February	19	86	85	9	8
March	23	106	57	13	8
April	27	94	49	15	6
May	29	94	43	15	6
Average Test		95	65	12	7

sludge through the aeration system, at least during the first three months. In the latter two months, suspended solids in the effluent did increase by about 3 mg/l with a substantial increase in flow rate from about 20 M.G.D. to 27-29 M.G.D.

Nutrient Removal

In order to further evaluate whether the recirculation of digested sludge through the secondary system had an adverse effect on effluent quality, the data pertaining to the parameters nitrogen and phosphorus shown in Table II. The effluent values are of special interest since the raw wastewater samples did not contain the recirculating flow. Inspection of the data for the two periods, pre-test and test, shows that the total average inorganic N was 8.2 ppm vs. 9.6 ppm, respectively. Although the individual months vary based on the period averages, inorganic nitrogen shows an increase of 1.4 ppm in the effluent during the test period over the pre-test period. On the other hand, the organic nitrogen showed a decrease of 4.3 ppm. Phosphorous concentrations in the effluent remained essentially unaffected when comparing the two periods. It would appear that the digested sludge recirculation had a rather minor effect on the effluent quality with respect to the nutrients nitrogen and phosphorus.

Heavy Metals Removal

The results of the monthly metal analyses of composite influent and effluent samples for the pre-test period July to December 1979, and for the test period January to May 1980, are presented in Table III. Comparing the overall averages of these test periods and focusing on the two metals that have been demonstrated to be able to exert a major effect on human physiology, namely cadmium and mercury, there does not seem to be a significant difference between the removals. In fact, the activated sludge process does not appear capable of reducing appreciably the very low concentration of either of these metals. It should also be pointed out that mass balance studies of the metal data, except cadmium and mercury, have been generally good. Because of the low concentrations of the metals cadmium and mercury, and the sensitivity of the testing procedure, the mass balances were not good. As to the other heavy metals, comparative inspection of the data presented indicates some variable effects of treatment during individual months with the overall averages not significantly changed for the subject periods.

Effect of Recirculation on Oxygen Requirements

As to the influence of digested sludge recirculation on the dissolved oxygen requirements, there was no change in air compressor output over the course of the test. Unfortunately, the air compressor was operating at a level to produce more than adequate dissolved oxygen and its rate could not be lowered before or during the test to specifically attempt to evaluate any demand changes. A calculated estimate, based on the fact that meso-digestion (without the benefit of subsequent thermo-digestion) destroys 90%

Table II

Rockaway WTP Nutrient Concentrations Influent & Effluent

Rockaway Wt. Nutrient Concentrations Influent & Effluent			N (mg/l)				P (mg/l)		
Month			NH ₃ -N	Org.-N	NO ₃	NO ₂	Total Inorg. N	Total Ortho	
1961	July 78	Infl.	13.0	5.8	0	0	13.0	2.3	1.7
		Effl.	1.6	2.9	7.0	2.0	10.6	1.8	1.8
	August	Infl.	7.8	8.4	2.3	.5	10.1	2.5	1.8
		Effl.	1.8	3.0	5.9	.9	9.7	2.1	1.4
	September	Infl.	- *	10.5	- *	- *	-	2.5	1.1
		Effl.	- *	9.6	- *	- *	-	2.1	1.7
	October	Infl.	7.8	8.4	0	0	7.8	2.0	1.4
		Effl.	0.6	3.0	5.2	0.2	6.0	1.2	1.2
	November	Infl.	12.5	15.1	0.1	0.2	12.8	2.7	1.9
		Effl.	9.4	8.6	0.4	0.5	10.3	1.6	1.2
	December	Infl.	10.4	16.0	0.3	0	10.7	1.8	3.5
		Effl.	9.2	14.8	0.2	0	9.4	1.6	3.0
Average									
Pre-Test	Infl.	10.3	10.8	0.5	0	10.8	2.4	1.9	
	Effl.	4.5	7.0	3.7	0.5	8.2	1.7	1.7	
January 80	Infl.	9.4	9.8	0.2	0.1	9.6	2.7	1.6 Transition	
	Effl.	0.6	3.0	6.3	0	6.9	1.8	1.5 Month	
February	Infl.	11.6	10.6	0.3	0	11.9	2.8	1.8	
	Effl.	2.6	2.2	8.2	0	10.8	1.7	1.4	
March	Infl.	9.6	15.6	0.8	0	10.4	3.9	1.9	
	Effl.	3.0	4.2	10.0	0.2	13.2	2.0	0.7	
April	Infl.	7.0	8.6	0.3	0	7.3	2.9	1.2	
	Effl.	1.0	3.2	5.6	0	9.8	2.4	1.1	
May	Infl.	9.4	9.2	0.3	0	9.7	2.5	1.3	
	Effl.	2.4	1.2	5.4	0	7.8	2.0	1.6	
Aver. Test									
Feb. to May		Infl.	9.4	11.0	0.4	0	9.8	3.0	1.6
		Effl.	2.3	2.7	7.3	0	9.6	2.0	1.2

* Analytical results deleted

Table III
Rockaway WTP Metal Data in (mg/l)

		1979						Average
		July	August	September	October	November	December	
Cu	Inf.	.11	.11	.13	.13	.16	.11	.12
	Eff.	.07	.05	--	.03	.06	.05	.05
Cr	Inf.	.001	.020	.012	.009	.011	.034	.014
	Eff.	.006	.015	.008	.002	--	.007	.008
Ni	Inf.	.07	.02	.02	.01	.02	.01	.03
	Eff.	.04	.02	.01	.01	.02	.02	.02
Zn	Inf.	.11	.26	.23	.09	.12	.08	.15
	Eff.	.14	.35	.16	.17	.15	.08	.17
Pb	Inf.	.017	.024	.110	.014	.027	.023	.036
	Eff.	.006	.006	.006	.008	.020	.007	.009
Fe	Inf.	.8	.9	1.5	1.5	1.6	1.1	1.2
	Eff.	.2	1.0	.7	1.2	2.0	.1	.9
Cd	Inf.	.0001	.0001	.0004	.0001	.0010	.0008	.0004
	Eff.	.0001	.0001	.0002	.0001	.0015	.0006	.0004
Ca	Inf.	41.0	40.0	41.0	29.0	19.0	13.0	30.0
	Eff.	36.0	48.0	32.0	41.0	20.0	15.0	32.0
Mg	Inf.	94.0	107.0	102.0	94.0	85.0	76.0	93.0
	Eff.	98.0	112.0	105.0	101.0	88.0	77.0	97.0
Hg	Inf.	.0010	.0007	.0007	.0005	.0026	.0005	.0010
	Eff.	.0006	.0006	.0002	.0005	.0028	.0009	.0009

Table III (Continued)
Rockaway WTP Metal Data in (mg/l)

		January	February	1980 March	April	May	Average February to May	
893	Cu	Inf.	.095	.080	.110	---	.075	.088
		Eff.	.22	.0035	.0400	---	.0380	.027
	Cr	Inf.	.0007	.0012	.0038	.010	.0038	.0047
		Eff.	.0012	.001	.003	.005	.001	.002
	Ni	Inf.	.015	.009	.0042	.0068	.0086	.0072
		Eff.	.018	.021	.0024	.014	.011	.012
	Zn	Inf.	.066	.093	.090	.21	.085	.12
		Eff.	.070	.086	.065	.079	.10	.082
	Pb	Inf.	.014	.049	.0089	.0088	.010	.018
		Eff.	.030	.0024	.0016	.0034	.0064	.0035
	Fe	Inf.	.57	.65	.55	.84	.83	.72
		Eff.	1.00	.16	.21	.13	.46	.24
	Cd	Inf.	.0018	.0005	.0011	.0046	.0011	.0018
		Eff.	.0005	.0004	.0009	.0029	.0017	.0015
	Ca	Inf.	14.0	16.0	14.0	13.0	23.0	16.0
		Eff.	14.0	17.0	15.0	14.0	21.0	17.0
	Mg	Inf.	70.0	60.0	58.0	54.0	59.0	58.0
		Eff.	72.0	64.0	59.0	55.0	62.0	60.0
	Hg	Inf.	.0009	.0005	.0009	.0003	.0003	.0005
		Eff.	.0011	.0003	.0002	.0005	.0004	.0004

of BOD₅, indicates that the BOD₅ of the part of the digested sludge continuously recirculated would add less than 5% to the oxygen demand of the primary effluent.

Operating Results

As was pointed out previously, a 1 cu. ft./capita tank was placed in service as a thermo-digester at about 121° F. Its contents overflowed by gravity to both the primary effluent and to a 45-ft. diameter rethickening and elutriating tank where about 3:1 of city water was added to the influent sludge. The rethickened underflow sludge was pumped by a duplex plunger pump, actuated by time clock, to spare empty digesters where its volumetric rate was measured by filling the tanks during the months of March and April. Very importantly, such procedures did not involve the use of any manpower except for periodically blowing back clogged lines.

The data obtained are presented in Table IV. Here it can be seen that the volatile matter leaving the meso-digester averaged 9,000 lbs./day, thus effecting a reduction of $16,200 - 9,000 = 7,200$ lbs./day. The thermo-digester accounted for a further reduction of $9,000 - 7,200 = 1,800$ lbs./day. It should be pointed out that such reductions were being effected on the combination of raw primary solids, activated sludge solids and the recirculating solids that had been previously subjected to meso- and thermo-digestion.

The conventional activated sludge treatment units are represented in Figure 1. Since the economics of sludge disposal is fundamentally a function of the amount of volatile material to be disposed of, only the rates of production of volatile matter (V.M.) are discussed. In an overall sense, it can be seen that the reduction of volatile matter by the meso-digester of 7,200 lbs./day, added to 1,800 lbs./day by the thermo-digester, results in a total of 9,000 lbs./day. Additionally the aerator destroyed 2,000 lbs. V.M./day for a total reduction of 11,000 lbs. V.M./day. Since the treatment system was removing a total of 12,900 lbs. V.M./day, the net amount requiring disposal was reduced to 1,900 lbs. V.M./day. Previous data show that the average amount of volatile matter carried to sea in the period just prior to this work (thermo-digestion was being started in August and September 1979) was 5,700 lbs. Thus, the amount of volatile solids was reduced by $\frac{5,700 - 1,900}{5,700} = 2/3$. Volume reduction was in the same proportion;

that is, 4,800 cu. ft./day to 1,650 cu. ft./day, or about 2/3.

The daily amount of gas generated during the entire course of the thermo-digestion is shown in Table V. Based on the averages for the period February to May, the meso-digester accounted for an 83,900 cu.ft./day rate, slightly less than the comparable preceding period without digested sludge recirculation through the aerator. The gas generated by the thermo-digester increased from an average of 7,000 cu.ft./day to 14,000 cu.ft./day

Table IV
Rockaway WTP Amount and Concentration of Solids Passing Through System¹

Month 1980	Flow MGD	#VSS Capt. @ 75% V.M.	Raw Thick Pump Cu.ft./Day	% Conc. V.M.			#V.M. From Thick.	#V.M./Day Leaving			
				Raw Thick	Meso Dig.	Thermo Dig.		Meso Dig.	Thermo Dig.	Rethickener & Elutriator Under Over Flow Flow	
Jan.	21	10400	5900	3.9	1.6	1.1	14500	6000	4100	-	-
Feb.	19	9500	7300	3.6	1.8	1.3	16500	8200	5900	-	-
Mar.	23	14800	8400	3.3	1.7	1.5	17500	8900	7800	1800 ²	500
April	27	13400	6500	3.5	1.9	1.5	14500	8000	6400	2000 ²	400
May	29	14200	8500	3.0	2.0	1.6	16200	10800	8600	-	600
<hr/>											
Average Feb. to 25 May		12900	7700	3.4	1.9	1.5	16200	9000	7200	1900	500

Note - (1) Calc. of V.M. inventory in digesters after February show the inventory change does not significantly influence the data.

(2) Measured volume March 1600 cu. ft/day x 63 x 1.8% = 1800 # V.M./day
April 1700 cu. ft/day x 63 x 1.9% = 2000 # V.M./day

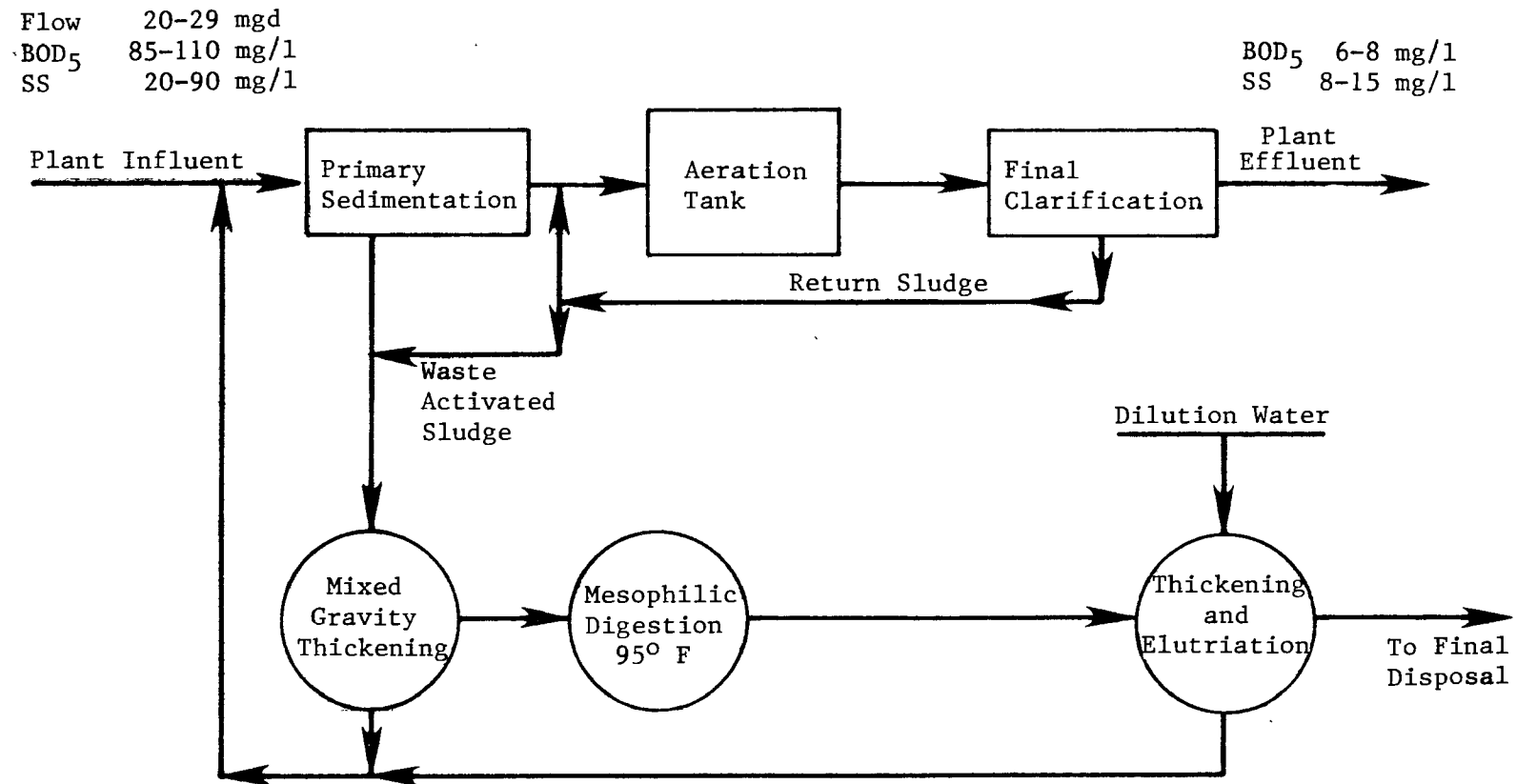


Figure 1
 Rockaway Wastewater Treatment Plant
 Before Thermophilic Digestion Addition

Table V
Rockaway WTP Daily Gas Production

Month	Mesophilic Digester cu. ft./day	Thermophilic Digester cu. ft./day
September 1979	79200	5300
October	93800	8200
November	90300	6900
December	77200	7500
Average No Recirculation	87600	7000
January 1980	79200	8500
February	88000	12700
March	86800	13600
April	76400	11900
May	84300	17700
Average With Recirculation Feb. to May 1980	83900	14000

during recirculation. The gas mixers in both digesters were found to be causing the formation of large solids masses in the digesters with consequent clogging of the overflow; it was found necessary to operate the mixers only a few minutes per day to alleviate the condition.

Garber (6) in Los Angeles had determined that the thermo-digested sludge required half the dose of iron coagulant and produced almost four times the yield on a vacuum filter as meso-digested sludge. Accordingly, to obtain some estimate of the improvement on coagulability achieved by the use of the thermo-digestion in this instance, the meso- and thermo-digested sludges were subjected to polymer treatment. It was found, on a laboratory scale, that using a high-molecular-weight, low-charge polymer #2535CH (as manufactured by American Cynamid), the coagulability improved radically. Specifically, dosages of up to 4,000 ppm on meso-digested sludge did not produce an end point, although some flocculation was observed. In contrast, the thermo-digested sludge released 73% of the water in 30 minutes in gravity settling at a dose of 2,500 ppm. Thermo-digested sludge, after a 3:1 elutriation, required a lesser comparative dose of 1,650 ppm of the same polymer to release 64% of the water within 30 minutes in gravity settling.

Destruction of Pathogens

An effective way for the destruction of pathogenic organisms in sludge is exposure to high temperature for an adequate period of time. Many researchers have shown that the effectiveness of disinfection increases with temperature or time. For example, Rudolfs et al. (1951)(7), using Ascaris suum, found that:

at 45° C, 2 hours:	had no effect
at 50° C, 30 minutes:	retarded development
at 50° C, 2 hours:	killed all ova
at 55° C, 10 minutes:	killed all ova

Work by many others show similar findings with heat death of ascaris eggs (8). Table VI also shows the effect of temperature and time on other pathogenic organisms (11).

For the past several years, Garber and co-workers at the Hyperion Treatment Plant in the City of Los Angeles have been operating a full-size digester in the thermophilic temperature range of about 49° C, in parallel with other digesters operating in the mesophilic temperature range of about 35° C (9). For approximately two years, in a cooperative program between the Hyperion plant staff and the Municipal Environmental Research Laboratory (MERL), grab samples of raw, mesophilic and thermophilic sludges were forwarded to MERL for pathogenic organisms analyses. A summary of the analyses of bacteria is shown in Table VII (10). Thermophilic digester treatment consistently reduced the Salmonella densities to below the detectable limits of the analytical procedure, whereas Salmonella were consistently detected after mesophilic digestion. The density of the indicator organisms was reduced 2 to 3 logs more than was the case for

Table VI

Temperature and Time for Pathogen Destruction in Sludges

Microorganisms	Exposure Time (Minutes) for Destruction at Various Temperatures (°C)				
	50° C	55° C	60° C	65° C	70° C
Cysts of <u>Entamoeba histolytica</u>	5				
Eggs of <u>Ascaris lumbricoides</u>	60	7			
<u>Brucella abortus</u>		60		3	
<u>Corynebacterium diphtheriae</u>		45			4
<u>Salmonella typhi</u>			30		4
<u>Escherichia coli</u>			60		5
<u>Micrococcus pyogenes var. aureus</u>					20
<u>Mycobacterium tuberculosis var.</u>					20
Viruses					25

Table VII
Reduction in Bacterial Densities in Mesophilic and Thermophilic Anaerobic
Digestion (20-day detention)

	Bacterial Densities (number/100 ml)*		
	Raw Sludge Feed	Mesophilic Digestion (36° C)	Thermophilic Digestion (50° C)
Fecal Streptococcus	2.7×10^7	2.0×10^6	3.7×10^4
Fecal Coliform	3.6×10^8	5.5×10^6	2.9×10^4
Total Coliform	5.2×10^9	7.0×10^7	6.4×10^4
Salmonella	7530	62	BDL

NOTE: BDL - Below detection limits (< 3/100 ml)

*Average of measurements taken over 2-year period

mesophilic digestion by thermophilic digestion. Mesophilic digestion reduced the density of Salmonella by 2 logs and indicator organisms 1 to 2 logs, as compared to raw sludge.

Twelve sets of animal enteric virus analyses were conducted over the 2-year period (10). Test results are as follows:

<u>TYPE OF SLUDGE</u>	<u>PFU/GRAM OF LIQUID SLUDGE (2 to 5% SOLIDS)</u>
Raw	25.40
Mesophilic Digested	2.10
Thermophilic Digested	0.03

Thermophilic digestion produced a 2-log improvement over mesophilic digestion. In fact, viruses were not detected in 6 out of the 12 samples analyzed. Based on the limited data, there was essentially no effect on Ascaris lumbricoides concentrations in either type of digester treatment. The findings are in conflict with the reported reasons for using the thermophilic process in the U.S.S.R. Popova and Bolotina (5), in their report on the practice of thermo-digestion in Moscow, state "The most essential advantage of this process is the sanitary quality of the thermophilic sludge. According to the sanitary officials of the health department, viable eggs of helminths are absent from such sludge."

The Moscow and Los Angeles data agree on the degree of viral and bacterial destruction but differ in the effect on helminths. Note, however, that the Los Angeles digester was operated at 49°C and Rudolph's results indicate that temperature is very important in helminth destruction. It appears then that operation of the thermophilic stage slightly in excess of 50°C would produce a sludge that is hygienically safe for disposal.

Potential Process Applications

Since the 1980 EPA Municipal Wastewater Facilities Construction Need Survey showed that there will be over 4,200 municipal treatment plants utilizing anaerobic sludge digestion by 1986, the EPA Office of Research and Development initiated a separate study (12) that investigated the feasibility of applying meso-thermophilic digestion to a major treatment facility. The District of Columbia Blue Plains treatment plant was selected because it had anaerobic digesters in operation and the sludge management methodology needed upgrading for operating and economic reasons.

Based on review of the anaerobic sludge digestion options and how they could be adapted to the existing facilities, the study recommends that the thermophilic anaerobic digestion process be implemented on a full-scale basis. This recommendation is based on a thorough review of the present state of practice in the United States and other countries.

Although the meso-thermophilic digestion process could be the optimum solution for other plants, the thermophilic process is recommended for Blue Plains because it could be implemented with a minimum of time and money. Other significant advantages are: (1) increased sludge processing capability; (2) improved sludge dewatering as to coagulant demand and yield; and (3) increased destruction of pathogens, all of which are pertinent to the needs of the Blue Plains plant.

It is especially important to check the structural competency of the existing digesters and piping at the thermophilic temperatures, as well as the temperature control system prior to start-up.

A carefully formulated transition plan should be prepared so that the transition can be carried out effectively and with minimum interference with plant operations.

DISCUSSION

It has been noted in the literature that the thermophilic digestion process, by itself, presents a problem in that an excessively long period of 6 to 12 months may be required to achieve a satisfactory operating performance. This required long period of adaptation of the biological species to the hostile high temperature environment interferes with plant operation and is costly in economic terms. Therefore, to reduce substantially this time period, it becomes mandatory to effectively seed thermophilic digesters. In the case of meso followed by thermo, it is less critical that the thermophilic digester promptly achieve satisfactory performance. Consequently, it is elective to seed the thermo digester to expedite the operating performance at a satisfactory level.

The reader is cautioned that each plant will have a maximum upper limit of the amount of digested sludge that can be recirculated. It should be pointed out that the proportion of digested sludge recirculated continuously through the secondary treatment system should lie in the range of 30% - 60%. Moreover, operation should generally be conducted in the lower part of this range at wastewater temperatures near 75° F, and in the higher part of this range at temperatures near 55° F.

SUMMARY

A full plant-scale test was conducted at the Rockaway Plant in New York City with a connected population of 100,000 for a period of 5 months to evaluate a new method of reducing the amount and volume of sludge produced from the activated sludge process. This method involved the novel use of: (1) high stability thermophilic digestion following mesophilic digestion and (2) the recirculation of a portion of such thermo-digested sludge directly to and through the secondary system

of the activated sludge process while the remainder was conducted to a rethickening and elutriation step. Operating results have demonstrated that the volatile matter normally transported to sea after meso-digestion was reduced by 2/3. Moreover, the volume of sludge produced was lowered by 2/3 without chemical or mechanical aids. It was determined on a laboratory scale that the residual solids exhibited improved coagulability having undergone thermo-digestion, which change would improve the economics of all subsequent dewatering processes. The treatment process performed without significant adverse effect on any accepted parameter due to the continuing recirculation of digested sludge through the activated sludge process.

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RESEARCH SUPPORTED BY THE NATIONAL SCIENCE FOUNDATION RELATING TO TREATMENT
OF WASTEWATER AND MANAGEMENT OF RESIDUAL SLUDGES

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ABSTRACT

Scientific research has played an important role in development of our present understanding of all matters relating to management of water. This knowledge has been applied by engineers in solving problems of availability, quality, treatment, and use of water and in treatment of wastewater for reuse or discharge to minimize adverse environmental impacts. The roles of science and engineering as they relate to management of water are becoming progressively more important as pressures mount for its more intensive use.

Since its establishment in 1950 as an independent agency of the Executive Branch of the Federal Government, the National Science Foundation (NSF) has provided support for research to broaden the base of understanding on topics directly and indirectly relating to management of wastewater. Between 1973 and 1981, NSF supplied substantial support for research on innovations in management of sludges and in using wetlands to provide a degree of treatment equivalent to that obtained through capital and energy-intensive, physical, and chemical advanced (tertiary) treatment processes.

Wetlands have been shown to be potentially capable of absorbing the nutrient load from conventional secondary treatment processes without adverse, short-term effects. Full-scale use of a wetland for placement of a secondary effluent is currently in its fourth year of operation and evaluation at Houghton Lake, Michigan.

The combined capacity of two installations for disinfection of sludges using energized electrons in the United States, will be 300,000 gallons per day with scheduled completion of the unit at Miami, Florida, in 1981. The concept of combining disinfection of sludges by use of energized electrons, pipeline transport, and direct injection into topsoil on land dedicated to use for stabilization of sludges appears to be a promising new approach to management of sludges.

Dr. Edward H. Bryan is Program Director, Water Resources and Environmental Engineering in the Engineering Directorate's Division of Civil and Environmental Engineering. His prior program management responsibilities since joining the National Science Foundation in 1972 have included Regional Environmental Systems, Systems Integration and Analysis, Regional Environmental Management, Community Water Management and Appropriate Technology.

INTRODUCTION

In 1973, the National Science Foundation (NSF) began supporting research on problems that had regional significance with regard to their potential adverse environmental impact. One concerned the pollutional impact of effluents from secondary wastewater treatment plants on receiving waters from nutrients remaining in the effluent. Another concerned the currently large and rapidly growing problem of managing sludges produced during treatment of wastewater. A common factor linking these two interrelated problems was a desire to find solutions that were less capital and energy intensive than conventional physical, chemical, and biological methods.

INNOVATION IN SLUDGE MANAGEMENT

The context within which NSF's support of research on sludge management started in 1974 was the projected increase in the amounts of sludge resulting from implementation of new water pollution control legislation and imminent foreclosure of ocean placement and incineration as options for dealing with sludges. NSF's program sought a better understanding of the basic elements that comprise all systems for processing sludges as a step toward a new concept that would be more efficient and acceptable than simple refinement of current practices (1).

The approach that NSF's program took was strongly influenced by results from the initial project (2). Investigators at the University of Texas found that during conventional treatment of wastewater, most of the viruses were concentrated in sludges where they remained viable during subsequent processing. When these sludges were incorporated into soil, viruses were adsorbed on soil particles, remained viable for long periods of time, and were capable of being released under conditions simulating rainfall. These findings suggested that disinfection of sludges might become an essential pretreatment step for infected sludges that would be managed by placement on land, and the need for a transport and placement method that would minimize the risk and nuisance associated with processing, transporting, and application of sludge to land.

Since 1974, NSF's allocation in support of research directly relating to sludge management totalled about \$4 million. The interdisciplinary nature of this problem and the diversity of issues that were addressed is evident from the summaries in Tables 1 and 2. Research personnel from more than 20 public and private institutions and organizations participated in this effort and over 150 reviewers were consulted in evaluating the unsolicited proposals that led to actions necessary to sustain this effort.

The cost of sludge management is directly proportional to the amount produced. An initial logical step to minimize cost of sludge management would be to select treatment processes for wastewater that minimize production of sludges, consistent with other treatment objectives. The quality of sludges also affects the cost of additional processing and management. Regulation of industrial discharges into the collection system to limit or prohibit entry of heavy metals and toxic organic compounds is likely to be more

TABLE 1. RESEARCH SUPPORTED BY THE NATIONAL SCIENCE FOUNDATION APPLIED TO MANAGEMENT OF SLUDGES DERIVED FROM TREATMENT OF MUNICIPAL WASTEWATER

TOPICS, INSTITUTIONS, INVESTIGATORS AND THEIR DISCIPLINES

NO.	INVESTIGATOR	DISCIPLINE	INSTITUTION	SUBJECT OF RESEARCH - TITLE OF PROJECT
1.	BERNARD P. SAGIK	MICROBIOLOGY	UNIVERSITY OF TEXAS SAN ANTONIO, TEXAS	POTENTIAL HEALTH RISKS ASSOCIATED WITH INJECTION OF DOMESTIC WASTEWATER TREATMENT PLANT SLUDGES INTO SOIL
2.	JAMES L. SMITH	AGRICULTURAL ENGINEERING	COLORADO STATE UNIVERSITY FORT COLLINS, COLORADO	MANAGEMENT OF SUBSURFACE INJECTION OF WASTEWATER TREATMENT PLANT SLUDGES INTO TOPSOIL
3.	JOHN G. TRUMP EDWARD G. MERRILL ANTHONY SINSKEY	ELECTRICAL ENGINEERING CHEMICAL ENGINEERING NUTRITION/FOOD SCIENCE	MASSACHUSETTS INSTITUTE OF TECHNOLOGY CAMBRIDGE, MASSACHUSETTS	DISINFECTION OF MUNICIPAL WASTEWATER TREATMENT PLANT SLUDGES BY USE OF HIGH ENERGY ELECTRONS
4.	THEODORE G. METCALF	MICROBIOLOGY	UNIVERSITY OF NEW HAMPSHIRE DURHAM, NEW HAMPSHIRE	INACTIVATION OF ENTERIC VIRUSES IN MUNICIPAL WASTEWATER SLUDGES BY USE OF ENERGIZED ELECTRONS
5.	RICHARD I. DICK	CIVIL ENGINEERING	UNIVERSITY OF DELAWARE INITIALLY, THEN CORNELL UNIVERSITY, ITHACA, NEW YORK	PROCESS INTEGRATION FOR OPTIMAL MANAGEMENT OF SLUDGES DERIVED FROM TREATMENT OF MUNICIPAL WASTEWATER
6.	ROY HARTENSTEIN	INVERTEBRATE ZOOLOGY	STATE UNIVERSITY OF NEW YORK SYRACUSE, NEW YORK	STABILIZATION OF DOMESTIC WASTEWATER TREATMENT PLANT SLUDGES BY SOIL INVERTEBRATES
7.	MARY BETH KIRKHAM WILLIAM J. MANNING	AGRONOMY PLANT PATHOLOGY	OKLAHOMA STATE UNIVERSITY, STILLWATER UNIVERSITY OF MASSACHUSETTS, AMHERST (WALTHAM FIELD STATION)	AGRICULTURAL VALUE OF MUNICIPAL WASTEWATER TREATMENT PLANT SLUDGES IRRADIATED WITH ENERGIZED ELECTRONS
8.	P.C. CHEO	PLANT PATHOLOGY	CALIFORNIA ARBORETUM FOUNDATION, LOS ANGELES ARBORETUM, ARCADIA, CALIFORNIA	MECHANISMS OF PLANT VIRUS INACTIVATION IN SOILS INJECTED WITH MUNICIPAL WASTEWATER AND SLUDGES
9.	C. FRED GURNHAM	CHEMICAL ENGINEERING	GURNHAM AND ASSOCIATES, INC. CHICAGO, ILLINOIS	SOURCES AND CONTROL OF HEAVY METALS IN MUNICIPAL WASTEWATER TREATMENT PLANT SLUDGES
10.	JACK E. COLLIER	INDUSTRIAL ENGINEERING	COLLIER EARTHWORM COMPOSTING SYSTEMS, INC., SANTA CLARA, CALIFORNIA	CONVERSION OF MUNICIPAL WASTEWATER TREATMENT PLANT SLUDGES INTO EARTHWORM CASTINGS FOR AMENDMENT OF SOIL
11.	STEPHEN C. HAVLICEK ROBERT S. INGOLS	ORGANIC CHEMISTRY BIOLOGY	GEORGIA INSTITUTE OF TECHNOLOGY ATLANTA, GEORGIA	EFFECT OF INFRARED RADIATION ON COMPACTION OF MUNICIPAL WASTEWATER SLUDGES
12.	GEORGE D. WARD	CIVIL ENGINEERING	GEORGE D. WARD AND ASSOCIATES PORTLAND, OREGON	CONTROLLED SOIL MICROBIAL DETOXIFICATION OF PHENOXY HERBICIDE RESIDUES
13.	CHARLES FINANCE	FILM PRODUCTION	MEDIA FOUR PRODUCTIONS, INC. HOLLYWOOD, CALIFORNIA	SYNTHESIS OF A SYSTEM FOR MANAGEMENT OF MUNICIPAL WASTEWATER TREATMENT PLANT SLUDGES, A 16 MM FILM BASED ON RESEARCH IN ABOVE-LISTED PROJECTS (1) - (6), INCLUSIVE
14.	ROGER BLOBAUM	COMMUNICATIONS	ROGER BLOBAUM AND ASSOCIATES DES MOINES, IOWA	AN ASSESSMENT OF THE POTENTIAL FOR APPLYING URBAN WASTES TO AGRICULTURAL LAND
15.	STEPHEN J. MARCUS	ENGINEERING	ENERGY RESOURCES COMPANY, INC. CAMBRIDGE, MASSACHUSETTS	PUBLIC HEALTH AND NUISANCE ASPECTS OF COMMUNITY WASTEWATER SLUDGE MANAGEMENT
16.	LEON W. WEINBERGER	SANITARY ENGINEERING	ENVIRONMENTAL QUALITY SYSTEMS, INC. ROCKVILLE, MARYLAND	PREDICTION AND CONTROL OF HEAVY METALS AND TOXIC ORGANIC SUBSTANCES IN MUNICIPAL SLUDGES
17.	ROBERT W. KAUFMAN	POLITICAL SCIENCE	WESTERN MICHIGAN UNIVERSITY KALAMAZOO, MICHIGAN	WORKSHOP ON THE ROLE OF EARTHWORMS IN STABILIZATION OF ORGANIC RESIDUALS FROM DOMESTIC AND INDUSTRIAL SOURCES
18.	CLARENCE GOLUEKE	ENVIRONMENTAL ENGINEERING	CAL RECOVERY SYSTEMS, INC. RICHMOND, CALIFORNIA	BENEFITS AND PROBLEMS OF COMPOSTING MIXTURES OF MUNICIPAL SLUDGES AND SOLID WASTES
19.	ROGER HAAG	CIVIL ENGINEERING	RICKEL MANUFACTURING CORPORATION SALINA, KANSAS	AGRICULTURAL UTILIZATION OF SLUDGES DERIVED FROM TREATMENT OF COMMUNITY WASTEWATER
20.	GEORGE D. WARD	CIVIL ENGINEERING	GEORGE D. WARD AND ASSOCIATES PORTLAND, OREGON	SUSCEPTIBILITY OF MT. ST. HELEN'S VOLCANIC ASH TO STABILIZATION BY USE OF ORGANIC SLUDGES
21.	GEORGE D. WARD	CIVIL ENGINEERING	GEORGE D. WARD AND ASSOCIATES PORTLAND, OREGON	ELIMINATION OF SEPTIC TANK SLUDGE TRANSPORT BY MANAGEMENT ON SITE OF ITS PRODUCTION
22.	ROY HARTENSTEIN	INVERTEBRATE ZOOLOGY	STATE UNIVERSITY OF NEW YORK SYRACUSE, NEW YORK	EARTHWORM-MICROBIAL INTERACTIONS DURING STABILIZATION OF ORGANIC WASTES FOR RECOVERY OF THEIR RESOURCE VALUES
23.	JAMES E. ALLEMAN	CIVIL ENGINEERING	UNIVERSITY OF MARYLAND COLLEGE PARK, MARYLAND	BENEFICIAL USE OF SLUDGES IN PRODUCTION OF BUILDING COMPONENTS
24.	MARY BETH KIRKHAM	AGRONOMY	KANSAS STATE UNIVERSITY MANHATTAN, KANSAS	PRODUCTIVITY OF LAND AND QUALITY OF WHEAT GROWN USING SLUDGES AS ORGANIC SOURCES OF PLANT NUTRIENTS
25.	RAYMOND C. LOEHR EDWARD F. NEUHAUSER	CIVIL/SANITARY ENGINEERING SOIL BIOLOGY	CORNELL UNIVERSITY ITHACA, NEW YORK	STABILIZATION OF ORGANIC RESIDUES DERIVED FROM TREATMENT OF SELECTED INDUSTRIAL AND MUNICIPAL WASTES

TABLE 2. SUMMARY OF AWARDS BY THE NATIONAL SCIENCE FOUNDATION IN SUPPORT OF RESEARCH ON SLUDGE MANAGEMENT, FISCAL YEARS 1972 - 1981¹

No.	INSTITUTION (PRINCIPAL INVESTIGATOR)	FISCAL YEAR OF AWARDS - AMOUNTS ARE IN THOUSANDS OF DOLLARS									
		1972/73	1974	1975	1976	1976-77	1977	1978	1979	1980	1981
1.	UNIVERSITY OF TEXAS (BERNARD P. SAGIK)	263.0	-	58.9	72.4	-	89.2	87.5	-	-	-
2.	COLORADO STATE UNIVERSITY (JAMES L. SMITH)	-	51.0	68.9	86.1	-	15.4	-	-	-	-
3.	MASSACHUSETTS INSTITUTE OF TECHNOLOGY (JOHN G. TRUMP)	-	113.7	198.0	200.0	70.0 ²	285.0	27.2	90.0	-	-
4.	UNIVERSITY OF NEW HAMPSHIRE (THEODORE G. METCALF)	-	-	43.0	40.7	-	35.0	17.0	-	-	-
5.	UNIVERSITY OF DELAWARE AND CORNELL UNIVERSITY (RICHARD I. DICK)	-	-	59.1	65.0	-	-	77.2	77.8	-	-
6.	STATE UNIVERSITY OF NEW YORK - SYRACUSE (ROY HARTENSTEIN)	-	-	-	88.5	-	111.7	150.7	90.0	-	-
7.	OKLAHOMA STATE UNIVERSITY (MARY BETH KIRKHAM) UNIVERSITY OF MASSACHUSETTS (WILLIAM J. MANNING)	-	-	-	-	-	87.8	-	-	-	-
8.	LOS ANGELES ARBORETUM FOUNDATION (P.C. CHEO)	-	-	-	-	-	39.0	37.8	-	-	-
9.	GURNHAM & ASSOCIATES, INC. (C. FRED GURNHAM)	-	-	-	-	-	110.9	-	-	-	-
10.	COLLIER EARTHWORM COMPOSTING SYSTEMS, INC. (JACK E. COLLIER)	-	-	-	-	-	9.7	15.6	-	-	-
11.	GEORGIA INSTITUTE OF TECHNOLOGY (STEPHEN C. HAVLICEK AND ROBERT S. INGOLS)	-	-	-	-	-	21.9	-	-	-	-
12.	GEORGE D. WARD & ASSOCIATES (GEORGE D. WARD)	-	-	-	-	-	25.0	185.4	-	-	-
13.	MEDIA FOUR PRODUCTIONS (CHARLES FINANCE)	-	-	-	-	-	49.6	2.8	-	-	-
14.	ROGER BLOBAUM & ASSOCIATES (ROGER BLOBAUM)	-	-	-	-	-	92.1 ³	-	-	-	-
15.	ENERGY RESOURCES COMPANY, INC. (STEVEN MARCUS)	-	-	-	-	-	-	126.4	-	-	-
16.	ENVIRONMENTAL QUALITY SYSTEMS, INC. (LEON W. WEINBERGER)	-	-	-	-	-	-	201.4	123.2	-	-
17.	WESTERN MICHIGAN UNIVERSITY (ROBERT KAUFMAN)	-	-	-	-	-	-	-	43.9 ⁴	4.4 ⁵	-
18.	CAL RECOVERY SYSTEMS, INC. (CLARENCE GOLUEKE)	-	-	-	-	-	-	-	19.6	-	-
19.	RICKEL MANUFACTURING CORPORATION (ROGER HAAG)	-	-	-	-	-	-	-	25.0	-	-
20.	GEORGE D. WARD & ASSOCIATES (GEORGE D. WARD)	-	-	-	-	-	-	-	-	11.7 ⁵	-
21.	GEORGE D. WARD & ASSOCIATES (GEORGE D. WARD)	-	-	-	-	-	-	-	-	24.7 ⁵	-
22.	STATE UNIVERSITY OF NEW YORK - SYRACUSE (ROY HARTENSTEIN)	-	-	-	-	-	-	-	-	73.8 ⁵	-
23.	UNIVERSITY OF MARYLAND (JAMES E. ALLEMAN)	-	-	-	-	-	-	-	-	41.1 ⁵	-
24.	KANSAS STATE UNIVERSITY (MARY BETH KIRKHAM)	-	-	-	-	-	-	-	-	63.8 ⁵	-
25.	CORNELL UNIVERSITY (RAYMOND C. LOEHR AND EDWARD F. NEUHAUSER)	-	-	-	-	-	-	-	-	-	180.0 ⁵
TOTALS		263.0	164.7	427.9	552.7	70.0	972.3	927.0	469.5	219.5	180.0

NOTES:

¹ALL AWARDS LISTED WERE MADE FROM THE WASTE MANAGEMENT STRATEGIES AND RESIDUALS MANAGEMENT ELEMENTS OF PROGRAMS IN REGIONAL ENVIRONMENTAL SYSTEMS/MANAGEMENT AND COMMUNITY WATER MANAGEMENT EXCEPT AS NOTED IN ITEMS 2, 3, 4 AND 5, BELOW.

²INTERAGENCY TRANSFER OF FUNDS FROM THE U.S. ENVIRONMENTAL PROTECTION AGENCY'S MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY, CINCINNATI, OHIO.

³RESOURCE SYSTEMS PROGRAM, DIVISION OF ADVANCED ENERGY AND RESOURCES RESEARCH AND TECHNOLOGY, NATIONAL SCIENCE FOUNDATION.

⁴OFFICE OF PROBLEM ANALYSIS, DIRECTORATE FOR ENGINEERING AND APPLIED SCIENCE IN SUPPORT OF PLANNING FOR A PROGRAM IN APPROPRIATE TECHNOLOGY.

⁵PROGRAM IN APPROPRIATE TECHNOLOGY, NATIONAL SCIENCE FOUNDATION.

efficient in improving the quality of sludges in that respect than subsequent detoxification. However, the presence of unstabilized organic matter and pathogenic bacteria, protozoa, viruses and intestinal parasites is an inherent characteristic of all wastewaters derived from or associated with human contacts. All sludge processing and management systems address common issues of disinfection, detoxification, stabilization, transport and final placement. The most efficient total system is the one that minimizes costs of achieving acceptable resolution of these issues, consistent with the effect on treatment operations with which it must be integrated.

The concept that emerged from the NSF program pointed toward disinfection by use of energized electrons, pipeline-transport to a suitable land application site where the sludge would be injected into topsoil for stabilization under carefully controlled conditions as having the attribute of minimum total cost. The advantages of this concept beyond those directly associated with elimination of costly dewatering processes and other capital and energy-intensive manipulations include:

- Retention of the nutrient values that are lost during processing by digestion, composting and other treatment procedures.
- Elimination of conditioning agents and their residuals as potential contaminants of the final product.
- Elimination of construction and operating costs associated with plant capacity no longer needed to accommodate strong and process-disruptive return-flows from thickeners, digesters, elutriation devices, ash pits, drying beds, centrifuges, vacuum filters and similar equipment.

The concept of coupling electron-beam disinfection with direct injection of sludges into topsoil was portrayed in a brief 16mm film entitled: "New Concepts in Sludge Management" (3). A more comprehensive film was also produced for use in briefing potential participants in a planned large-scale experiment to study those attributes of the integrated concept that were necessary to understand sufficiently to permit their use in engineering design of full-scale systems (4). In addition, a preliminary step was taken to design the experiment itself as a basis for estimating its potential cost (5).

Research to determine the role of soil invertebrates in stabilizing sludge led to new insights into the nature of stability as the concept is used to characterize sludges (6). Studies initiated to provide background for determining the effect of high-energy electrons on chlorinated hydrocarbons in sludges led to the observation of the complete destruction of a herbicide (monuron) and two polychlorinated biphenyls (3,4,2'-trichlorobiphenyl and 4-chlorobiphenyl) in water solutions (7).

Two recent publications summarized progress toward implementation of large-scale disinfection of sludges by use of energized electrons (8) (9). The 170,000 gallon per day unit currently under construction at the Miami-Dade Water and Sewer Authority's Wastewater Treatment Plant on Virginia Key in Florida is expected to be operational during 1981, providing disinfection

capability for one-fourth of the plant's production of sludge. The original unit used for the NSF-supported research at the Metropolitan District Commission's Deer Island Wastewater Treatment Plant in Boston was recently modified by the High Voltage Engineering Corporation, expanding its nominal capacity from the original 100,000 gallons per day to 170,000. This unit is currently operational and was used to refine the engineering design for the Florida installation.

WETLANDS FOR WASTEWATER MANAGEMENT

Complementary to research on management of sludges from primary and secondary treatment processes for wastewater, the National Science Foundation has been supporting research to better understand the potential role that wetlands can play in managing both the water and nutrients contained in effluents from conventional secondary treatment processes. While placement of a wastewater that had been freed of its demand for oxygen but which was rich in nutrients into a wetland appeared to have only desirable consequences of increasing the wetland's productivity, questions remained as to potential negative effects of this practice on the wetland ecosystem (10).

Ecosystem models constructed during the initial two years of a study involving potential application of a secondary effluent to a peat wetland near Houghton Lake, Michigan (11) (12), led to a two-year pilot-scale evaluation of the concept (13). Results of that study were sufficiently encouraging to initiate the full-scale placement of effluents from the Houghton Lake community's secondary oxidation pond of the 2000-acre wetland. The wetland is so large in comparison to the load imposed on it that nutrient-removal has been observed to be virtually complete within 100 meters from the line of entry. A 20-year monitoring program was established by the Houghton Lake community to determine any changes in the biota of the wetland attributable to its use for wastewater management. Initial results (14) (15) (16) (17) have been utilized to guide formulation of engineering design criteria for further application of this concept (18). A wetland that emerged upon failure of a land-application system to fully absorb wastewater placed on it has also been studied to provide guidance for potential use of the "wetland-concept" by communities lacking nearby natural wetlands (19).

In Florida, extensive studies have been conducted on cypress dome wetlands near Gainesville (20) (21) (22) and on a cypress stand wetland near Jasper (23) to characterize both seepage and flow-through type wetlands for their potential role in conserving nutrients and renovating wastewater. Over a six-year period, a test dome received the effluent from a small, activated sludge treatment plant serving a trailer park. Studies included effects on local groundwater quality, tree-growth rate, seedling germination, mosquito population and the survival characteristics and mobility of viruses. Hydrological characteristics, concepts of site management and characterization of the wetland's metabolism were studied and related to potential general use of this concept in Florida. Studies at the Jasper site, which has received wastewater varying in degrees of prior treatment over a period of 60 years, are expected to provide insights into long-range effects of using wetlands for treatment of wastewater.

TABLE 3. SUMMARY OF AWARDS MADE TO SUPPORT RESEARCH THE PROGRAM DESCRIBED IN THE FILM: "WETLANDS - OUR NATURAL PARTNERS IN WASTEWATER MANAGEMENT"

FISCAL YEAR	INSTITUTION AND PRINCIPAL INVESTIGATOR (GRANT NUMBER)	AMOUNT OF AWARD AND DURATION	TITLE OF THE PROJECT	NSF PROGRAM AND PROGRAM MANAGER
1971/72	UNIVERSITY OF MICHIGAN, JOHN A. KADLEC (G1 34812)	\$133,550 FOR 12 MONTHS	"THE EFFECTS OF SEWAGE EFFLUENT ON WETLAND ECOSYSTEMS"	REGIONAL ENVIRONMENTAL SYSTEMS WASTE MANAGEMENT STRATEGIES JEROME S. DAEN
1973	UNIVERSITY OF FLORIDA, HOWARD T. ODUM (G1 37821)	\$324,700 FOR 24 MONTHS	"CYPRESS WETLANDS FOR WATER MANAGEMENT, RECYCLING AND CONSERVATION"	REGIONAL ENVIRONMENTAL SYSTEMS WASTE MANAGEMENT STRATEGIES RICHARD C. KOLF
1973	UNIVERSITY OF MICHIGAN, JOHN A. KADLEC AND ROBERT H. KADLEC (G1 34812)	\$128,700 FOR 12 MONTHS	"THE EFFECTS OF SEWAGE EFFLUENT ON WETLAND ECOSYSTEMS"	REGIONAL ENVIRONMENTAL SYSTEMS WASTE MANAGEMENT STRATEGIES EDWARD H. BRYAN
1974	UNIVERSITY OF MICHIGAN, ROBERT H. KADLEC (G1 34812)	\$131,800 FOR 12 MONTHS	"THE EFFECTS OF SEWAGE EFFLUENT ON WETLAND ECOSYSTEMS"	REGIONAL ENVIRONMENTAL SYSTEMS URBAN/RURAL ENVIRONMENTS EDWARD H. BRYAN
1975	UNIVERSITY OF FLORIDA, HOWARD T. ODUM (ENV 73-07823)	\$223,000 FOR 12 MONTHS	"FEASIBILITY OF UTILIZING CYPRESS WETLANDS FOR CONSERVATION OF WATER AND NUTRIENTS IN EFFLUENT FROM MUNICIPAL WASTEWATER TREATMENT PLANTS"	REGIONAL ENVIRONMENTAL MANAGEMENT RESIDUALS MANAGEMENT EDWARD H. BRYAN
1975	UNIVERSITY OF MICHIGAN, ROBERT H. KADLEC (ENV 75-08855)	\$140,000 FOR 12 MONTHS	"FEASIBILITY OF UTILIZATION OF WETLAND ECOSYSTEMS FOR NUTRIENT REMOVAL FROM SECONDARY MUNICIPAL WASTEWATER TREATMENT PLANT EFFLUENTS"	" "
1976	WILLIAMS & WORKS, JEFFREY C. SUTHERLAND (ENV 76-20812)	\$31,200 FOR 7 MONTHS	"USE OF WETLANDS FOR MANAGEMENT OF POND-STABILIZED DOMESTIC WASTEWATER"	" "
1976	UNIVERSITY OF MICHIGAN, ROBERT H. KADLEC (ENV 75-08855)	\$129,900 FOR 16 MONTHS	"FEASIBILITY OF UTILIZATION OF WETLAND ECOSYSTEMS FOR NUTRIENT REMOVAL FROM SECONDARY MUNICIPAL WASTEWATER TREATMENT PLANT EFFLUENTS"	" "
1976	UNIVERSITY OF FLORIDA, HOWARD T. ODUM (ENV 73-07823)	\$223,600 FOR 12 MONTHS	"FEASIBILITY OF UTILIZING CYPRESS WETLANDS FOR CONSERVATION OF WATER AND NUTRIENTS IN EFFLUENT FROM MUNICIPAL WASTEWATER TREATMENT PLANT EFFLUENTS"	" "
1976 TO	BOYLE ENGINEERING CO., WALTER R. FRITZ (ENV 76-23276)	\$43,700 FOR 12 MONTHS	"TERTIARY TREATMENT OF MUNICIPAL WASTEWATER USING CYPRESS WETLANDS"	" "
1976 TO	UNIVERSITY OF MICHIGAN, ROBERT H. KADLEC (ENV 75-08855)	\$43,500 FOR 4 MONTHS	"FEASIBILITY OF UTILIZATION OF WETLAND ECOSYSTEMS FOR NUTRIENT REMOVAL FROM SECONDARY MUNICIPAL WASTEWATER TREATMENT PLANT EFFLUENTS"	" "
1977	UNIVERSITY OF FLORIDA, HOWARD T. ODUM (ENV 77-06013)	\$91,500 FOR 24 MONTHS	"UTILIZATION OF CYPRESS WETLANDS FOR MANAGEMENT OF MUNICIPAL WASTEWATER TREATMENT PLANT EFFLUENTS"	" "
1977	WILLIAMS & WORKS, JEFFREY C. SUTHERLAND (ENV 76-20812)	\$6,400 FOR 4 MONTHS	"USE OF WETLANDS FOR MANAGEMENT OF POND-STABILIZED DOMESTIC WASTEWATER"	" "
1978	BOYLE ENGINEERING CO., WALTER R. FRITZ (PFR 78-19199)	\$163,759 FOR 24 MONTHS	"ADVANCED TREATMENT OF COMMUNITY WASTEWATER BY FLOW-THROUGH CYPRESS STRAND WETLANDS"	REGIONAL ENVIRONMENTAL MANAGEMENT COMMUNITY WATER MANAGEMENT EDWARD H. BRYAN
1978	UNIVERSITY OF MICHIGAN, ROBERT H. KADLEC (ENV 77-23868)	\$141,744 FOR 12 MONTHS	"WETLAND UTILIZATION FOR MANAGEMENT OF COMMUNITY WASTEWATER"	" "
1978	UNIVERSITY OF FLORIDA, HOWARD T. ODUM (PFR 77-06013)	\$20,800 FOR 0 MONTHS	"UTILIZATION OF CYPRESS WETLANDS FOR MANAGEMENT OF MUNICIPAL WASTEWATER TREATMENT PLANT EFFLUENTS"	" "
1978	WILLIAMS & WORKS, JEFFREY C. SUTHERLAND (PFR 77-20273)	\$85,103 FOR 19 MONTHS	"UTILIZATION OF WETLANDS FOR MANAGEMENT OF POND-STABILIZED DOMESTIC WASTEWATER"	" "
1978	FLORIDA, STATE DEPARTMENT OF HEALTH AND REHABILITATION, FLORA MAE WELLINGS (PFR 77-26849)	\$48,072 FOR 12 MONTHS	"MOBILITY AND SURVIVAL OF VIRUSES IN CYPRESS DOME WETLANDS"	" "
1979	UNIVERSITY OF MICHIGAN, ROBERT H. KADLEC (ENV 77-23868)	\$152,275 FOR 12 MONTHS	"WETLAND UTILIZATION FOR MANAGEMENT OF COMMUNITY WASTEWATER"	COMMUNITY WATER MANAGEMENT EDWARD H. BRYAN
1979	FORUM, LTD., RONALD G. CAPALACES (PFR 79-19067)	\$59,384 FOR 8 MONTHS	"UTILIZATION OF WETLANDS FOR WASTEWATER MANAGEMENT," TREATMENT AND PRODUCTION OF A FILM	COMMUNITY WATER MANAGEMENT EDWARD H. BRYAN
1979	IMAGE ASSOCIATES, CLAYTON EDWARDS (PFR 79-19066)	\$3,000 FOR 1 MONTH	"UTILIZATION OF WETLANDS FOR WASTEWATER MANAGEMENT," TREATMENT CONCEPT ONLY	GOVERNMENT AND PUBLIC PROGRAMS SUSAN BARTLETT
1980	UNIVERSITY OF MICHIGAN, ROBERT H. KADLEC (ISP 80-14690)	\$37,528 FOR 12 MONTHS	"SOLIDS MOVEMENT IN WETLANDS"	APPROPRIATE TECHNOLOGY EDWARD H. BRYAN
1980	UNIVERSITY OF FLORIDA, HOWARD T. ODUM (ISP 80-14973)	\$27,299 FOR 12 MONTHS	"APPROPRIATE ENVIRONMENTAL SYSTEMS FOR WASTE MANAGEMENT"	" "
1980	FORUM, LT., RONALD G. CAPALACES (PFR 79-19067)	\$14,500 FOR 1 MONTH	"UTILIZATION OF WETLANDS FOR WASTEWATER MANAGEMENT" (SUPPLEMENTAL AWARD)	APPROPRIATE TECHNOLOGY EDWARD H. BRYAN

The wetlands projects in Michigan and Florida are the subject of a documentary film produced in 1980 to summarize progress and to assist in bringing the availability of the results of this research to the attention of potential users (23). A summary of awards made to support this research program is contained in Table 3.

SUMMARY AND CONCLUSIONS

New concepts for management of sludges produced during treatment of wastewater and to manage effluents from treatment processes for conservation of their nutrient and water content are needed which meet acceptable standards of public health and environmental quality and which also conserve capital, material and energy resources. Wetlands appear to provide that potential for effluents that have received **primary** and secondary treatment by conventional physical and biological processing. This concept is especially compatible with the first principle of good sludge management, the introduction of a tertiary step that in contrast to other physical, chemical or biological processes does not produce sludge. Direct injection of sludges into topsoil is already in actual use in many locations in the United States. The concept of applying sludge to land that is dedicated to "receiving sludge in perpetuity" was recently described as underway at the Reno-Sparks Joint Water Pollution Control Facility at Reno, Nevada (25).

The combined capacity of the Miami-Dade Virginia Key facility and that at the Deer Island plant in Boston will total in excess of 300,000 gallons per day for electron-beam disinfection of sludges by the end of this year. The concept of combining disinfection, pipe-line transport and direct injection of sludges into topsoil on land dedicated to function as a stabilization bed remains as a promising concept for assessment of its acceptability with regard to risk, technical and economic feasibility, and environmental compatibility.

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REMOTE SENSING OF SEPTIC SYSTEM PERFORMANCE
USING COLOR INFRARED AERIAL PHOTOGRAPHY

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ABSTRACT

Failed septic leach fields resulting in surfacing of partially treated wastewater to the ground surface can frequently be detected by remote sensing. The surfacing nutrients may increase the growth of vegetation which shows as a brighter red on color infrared (CIR) aerial photographs. Effluent continually ponded on the surface will eventually kill the vegetation by suffocating the roots. Thus, depending upon the severity of the failure, CIR photographs may reveal red stripes that delineate the tile field, bright red plumes in a downslope direction, brown spots where vegetation has died, and dark blue spots denoting standing surfaced effluent.

INTRODUCTION

As a result of the Federal Water Pollution Control Act (P.L. 92-500) and the 1977 Clean Water Act (P.L. 95-217), the Environmental Protection Agency (EPA) was given the authority to grant funds for the construction of sewage collection systems. Under the eligibility requirements for the construction grants program, Federal rules and regulations clearly state that the need for wastewater treatment facilities be proven by documenting the number of septic field failures within the existing target area, and assessing their effect upon water quality and public health in general (1).

"New collector sewers should be funded only when the systems in use (e.g., septic tanks or raw discharges from homes) for the disposal of wastes from the existing population are creating a public health problem, contaminating groundwater, or violating the point source discharge requirements of the Act. Specific documentation of the nature and extent of health, groundwater and discharge problems must be provided in the facility plan. Where site characteristics are considered to restrict the use of on-site systems, such characteristics

(e.g., groundwater levels, soil permeability, topography, geology, etc.) must be documented by soil maps, historical data, and other pertinent information. The facility plan must also document the nature, number and location of existing disposal systems (e.g., septic tanks) which are malfunctioning. A community survey of individual disposal systems is recommended for this purpose, and is grant eligible."

Originally, the only way to satisfy this program requirement was the door-to-door survey. This, however, required large commitments of personnel, time, money, and technical assistance. Also, a question of validity often arose because of local controversy sometimes surrounding sewer projects. Clearly, an alternative survey method was needed.

Surface failure of septic leach fields is usually caused by one or more of the following:

- The soil is too compacted causing very slow percolation rates.
- There is a close, underlying, impervious layer below the drainage field.
- The water table is close to the surface during the wet season.
- Breakage or mechanical malfunctions exist.
- The septic tank itself is overdue for a cleaning. This allows the loss of normally removed materials to coat and seal the sides and bottom of the percolation trenches.

Only those malfunctions which are noticeable on the surface can be detected on aerial imagery. Failures related to sewage backing up into the home, or too rapid transport through the soil into the groundwater, cannot be detected via remote sensing. In instances where the latter is occurring, the groundwater monitoring studies may be necessary to determine the existence of a problem.

HISTORY

The first known documentation of septic field problems using remote sensing was in Greensboro, North Carolina, in 1974. Although the results of this initial survey were not definitive, it did show promise that a specialized technique for septic system analysis was feasible (2). By employing stereo pairs of "false-color" infrared and conventional color photography, an analytical technique was developed in 1977 at the EPA-Environmental Photographic Interpreta-

tation Center (EPIC) that has since been shown to be reasonably successful depending upon the climatic and soil conditions at the time of over flight. EPIC produced several photo interpretation "keys" on septic field analysis and initially tested them on seven communities in EPA, Region V. This technique was touted to have \$36 million over conventional techniques (3). In early 1978, EPIC's technique was tested again in Hawkins, Greene and Union Counties in Tennessee. These communities were chosen because of their geologic structure, soil and topographic conditions, and their pressing need for a disposal system. The photographic interpretation was field checked, and out of 55 suspected failures, 52 were confirmed - an accuracy of 94.5 percent. This aerial survey reinforced the suspicions of Tennessee public health officials that current septic tank systems were not satisfactory for disposal of wastes within the study area.

The EPIC and other remote sensing techniques for septic field analysis have been used often as a part of the 201 Construction Grants Process. The primary document describing the photo-interpretative keys is still being reviewed within EPA and will be published by the Agency as a separate document.

METHODOLOGY

The technique currently uses both color (Kodak Ektachrome 2448) and color infrared (Kodak Ektachrome 2443) photography. Color infrared is the primary tool; standard color photographs are also used for orientation purposes and sometimes for verification. A 60 percent end lap of each photograph is required for stereo viewing to obtain topographic information. The photo interpreter scrutinizes the lot of each house for signatures of septic tank leachate such as vegetative distress or enhanced growth, and excessive soil moisture. A signature is an identifiable pattern characteristic of a certain specific object or situation. The signature key for septic tank failure developed by EPIC is summarized as follows (2).

Surface Failure

The obvious, blatant manifestations of septic system failure on CIR photographs are characterized by a deep red color and one or more dark gray or black spots where the actual septic effluent has surfaced and killed the surrounding vegetation (see Figure 1). Often, if the failure is severe, the effluent will break out into the driveway or street and run into storm sewers or surface waters. This type of failure may represent a health problem, especially if the effluent is standing stationary or occurs many times in a given area.

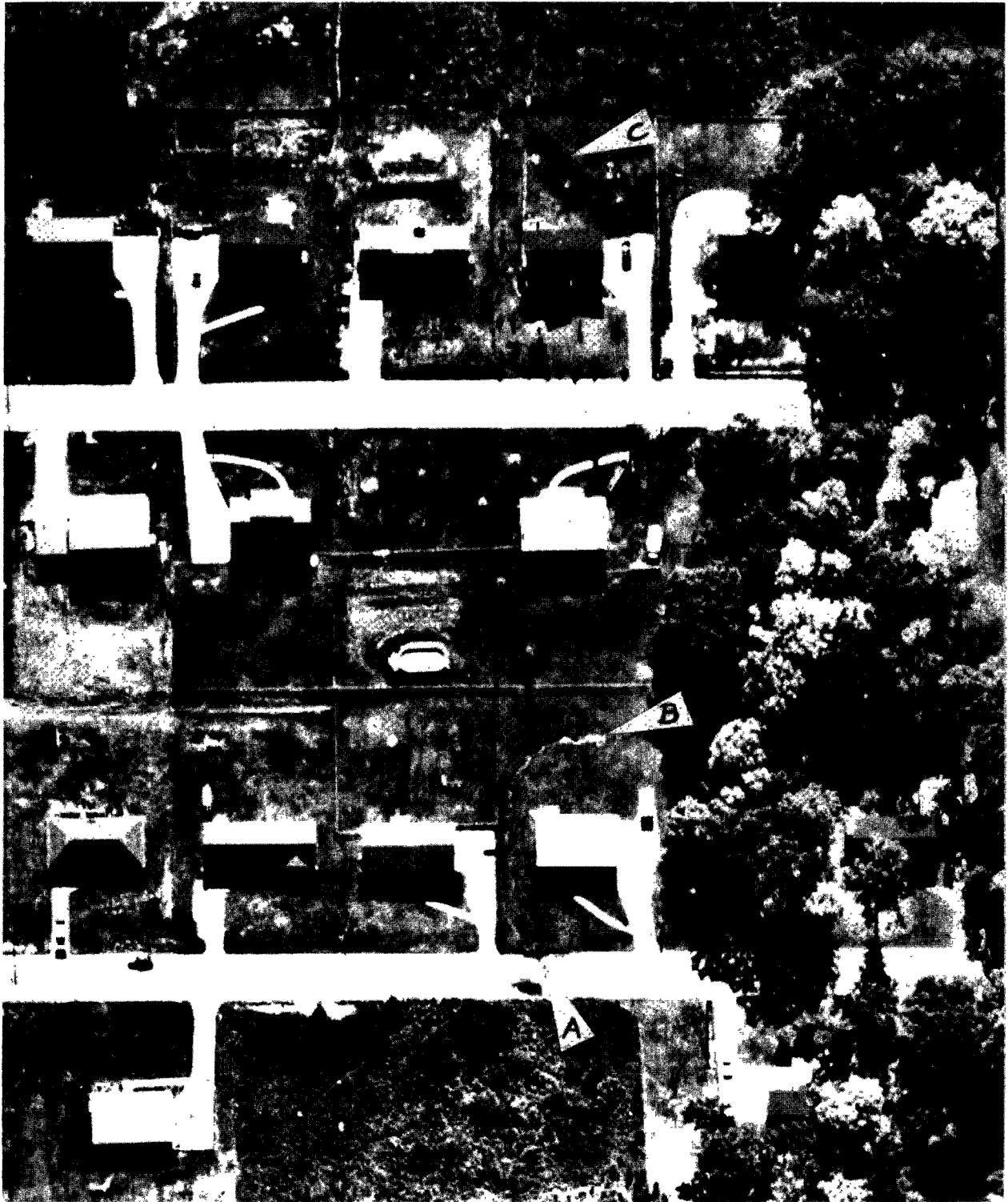


Figure 1. Black and White Copy of a Color Infrared Photograph.
Typical Signatures of Septic Field Failures Are:
(A) Overflow Into Street,
(B) Dead Grass,
(C) Lateral Lines Defined by Lush Grass.

Seasonal Failure

This signature is less definitive than the surface failure but nevertheless is readily identifiable with a high degree of accuracy. The seasonal failure may not show surfacing effluent when the photograph was taken but there are signs that either all or part of the system has failed in the past, or will probably fail in the future when seasonal conditions, such as excessive rain or a high water table, will strain the system. This signature is characterized by unusually lush growth caused by excessive surface moisture. In many cases, all or part of the disposal system will be well defined by the lush growth directly above it. Clear delineation of the lateral lines is usually cause for subsequent ground verification, even though such systems may not be failing. Also, evidence of past failure on the surface, such as dead vegetation in the form of a plume over all or part of the septic leachate field, is similarly classified.

Seasonal Stress

This signature is the least definitive of all the septic signatures but is still very important from a planning viewpoint. Seasonal stress signatures depict excessive moisture at or near the ground surface that may be related to septic system problems. Seasonal stress signatures are characterized by faint or partial definition of the lateral lines, excessive growth of vegetation over the probable location of the leach field, or any general sign that there is moisture near the surface.

Failure signatures are not always obvious and training is required to produce a proficient photo-interpreter. Similar signatures can be caused by common occurrences such as uneven spreading of lawn fertilizer, manure piles, compost heaps and animal droppings. For these reasons, field checking a percentage of the area is always recommended. In some cases, depending upon the soils of the particular area, the outline of the drainage line(s) of a properly functioning septic system can be distinguished on aerial photography. This peculiarity points up the need for tailoring "photo interpretation keys" to specific geographic areas (4).

EXAMPLES

Using the above "signatures" as photo interpretation keys, potential septic system failures have been identified in several study areas. The following examples are chosen from the Southeastern United States (5):

- Louisville (Jefferson County), KY
(Flown in November 1979)
Very extensive failure was noted.
A field check in January 1980 of 70 percent of the

area showed 323 surface failures and 565 seasonal failures. During rainy weather, drainage from these failed systems would wash into the combined sewers which serve as a direct conduit to the Ohio River.

- Maryville (Blount County), TN

(Flown in October 1980)

This study showed the greatest number of failures or problems yet recorded in a single study:

259 surface failures
1,445 seasonal failures
1,095 seasonal stress

2,799 total problems

- Orlando (Orange County), FL

(Flown November and December 1980)

This was a pilot study that determined the success of these techniques when applied to the unique climate and sandy soils of central Florida. Results in the test area showed:

47 surface failures
232 seasonal failures
167 seasonal stress

446 total problems

- Apalachicola area (Gulf, Franklin, and Wakulla Counties), FL (Flown January and February 1981)

These detection techniques for this coastal area are currently being studied as a research project. Normally, remote sensing is not suitable for use along beaches or other areas of unconsolidated sand. However, this area is part of the "Piney Woods Flatlands" which is underlain by an extensive hardpan. The hardpan may make use of this technique possible. This project was undertaken to determine possible sources of cholera organisms which are reaching production shellfish beds in Apalachicola Bay.

Nationwide, septic field failure surveys using this remote sensing technology have been conducted in the following locations including those detailed above) during the fiscal years shown:

1978

	USER	
Lake Geneva, WI	U.S. EPA	Region V
Crystal Lake, MI	"	V
Silver Lake, WI	"	V
Otter Trail Lake, MN	"	V

Crooked & Pickerel Lakes, MI	"	V
Nettle Lake, OH	"	V
Steuben Lakes, IN	"	V
Green Lake, MN	"	V
Spearfish, SD	"	VIII
Smith Mountain Lake, VA	"	III and
	Virginia State Water Pollution Control Board	
Hatboro/Horsham, PA	U.S. EPA Region	III
Upper Nazareth, Bushkill & Plainfield, PA	"	III
Surgoinsville, Baileytown & Luttrell, TN	"	IV
Stanley Co., NC	Stanley Co.	
	Dept. of Health	

1979

Topeka, KS	U.S. EPA Region	VII
Jefferson Co., KY	"	IV
Chalfont, New Britain & Doylestown, PA	"	III

1980

Lower Moreland, Abington & Bryn Athyn, PA	"	III
Blount Co., TN	"	IV
Seattle, WA	"	X

1981

Delaware Co., OH	"	V
Clermont Co., OH	"	V
Orlando, FL	"	IV
Lewes/Rehoboth, DE	"	III

CONCLUSIONS

Based upon the results obtained thus far, the manifestations or photo signatures of failed septic leach fields are best distinguished on normal color or color infrared photographs at scales of 1:10,000 or larger depending upon the quality of the film and camera system.

Some limitations on the use of remote sensing for septic tank system failure analysis have been encountered. Two of the most significant limitations are related to soil/vegetation "homogeneity" and tree cover. Failing systems situated in soils which exhibit a wide range of photo signatures, such as varying soil color/tone and "patchy" vegetative cover (e.g., some sandy soils around lakes), are sometimes difficult to distinguish from naturally occurring phenomena. In areas with a large percentage of tree cover, failing septic systems may be obscured by foliage and/or shadows. These conditions can be minimized by flying at specific times of the day or year. This type of technique optimization is continuing to further reduce the problem of "false negatives," i.e., systems which are actually failing but are not identified

by the photographs. This aspect of the technique is critical to the ultimate acceptance of procedure, since "false positives," i.e., the apparent failure of well-operating on-site systems, merely cause an increase in the ground verification effort, while a significant number of false negatives can obviate the utility of the whole procedure.

The big advantage of this technique is cost-savings. The cost may be as little as 10 percent of the cost of a door-to-door survey. In fact, Region V estimated a savings of \$51 million during 1980, attributed, in significant part, to this technique. For this a team of seven employees was awarded the Excalibur Award for Excellence in Government Service.

This technique is an excellent example of how color infrared aerial photography can be put to practical use in saving millions of taxpayer dollars.

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TWO-PHASE ANAEROBIC DIGESTION OF ORGANIC WASTES

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ABSTRACT

Anaerobic digestion is a multi-step biochemical process mediated by several microbial groups (phases) having significantly different physiology, nutritional requirements, growth kinetic and metabolic characteristics, environmental optima, and sensitivity to environmental stresses. Conventional engineering application of this process provides for concurrent enrichment of the various microbial phases under an identical environment; this leads to slow overall process kinetics, higher capital costs, low net energy production efficiency, and other disadvantages. A multi-stage advanced digestion process in which the microbial phases are enriched in separate optimized environments, and the substrate is stabilized by sequential acidogenic and methanogenic fermentations is discussed. The process, known as two-phase digestion, is a generic system, and could consist of two or more continuous stirred-tank, plug-flow, packed-bed, or fluidized-bed fermentors. The energetic, kinetic, and economic advantages of the two-phase process are discussed with reference to its application to several soluble and solid organic wastes. The status of development of the two-phase process, potential problems, and research needs are discussed.

INTRODUCTION

Anaerobic digestion is a multi-step biochemical process which is mediated by several symbiotic microbial groups or phases. As indicated in Figure 1, the overall digestion process consists of the following major coupled reaction steps:

- Enzymatic hydrolysis of particulate and high-molecular-weight substrates to simple monomers
- Conversion of the monomers to higher fatty acids, carbon oxides (mainly CO_2), hydrogen, and acetate
- Degradation of the higher fatty acids to acetate and CO_2
- Cleavage of acetate and/or reduction of CO_2 to form methane.

The first two steps outlined above are carried out by a group of acidogenic bacteria. It is believed that the third reaction step is conducted by the so-called "acetogenic" organisms which derive energy by oxidizing the higher fatty acids to acetate, hydrogen, and CO_2 (1). Little information exists on the physiology, kinetic properties, and nutritional and metabolic characteristics of the acetogens. Cleavage of acetic acid, which is believed to be the major substrate for methane formers in digestion of wastes, is the

slowest and the least energy-yielding of the two methane-forming reactions (2).

In view of the above, anaerobic digestion of organic wastes may be viewed as a two-phase process in which acid-forming organisms convert the process feed to acetic acid, which is next transformed to methane and CO_2 by the methane-forming bacteria. The acid-forming phase could be controlled by any one of the steps of hydrolysis, conversion of the hydrolytic products to acetate, or formation of acetate from higher fatty acids.

There is ample information in the literature to indicate that the dominant digesting populations differ significantly from each other with respect to physiology, nutritional requirements, metabolic characteristics, growth kinetic capability, environmental optima, and sensitivity to environmental stresses (3,4).

Two approaches could be employed in engineering application of the multi-phase digestion process to stabilization and gasification of organic wastes:

- Coculturing of the several microbial groups in a single fermentor (digester) under identical operating and environmental conditions
- Enrichment culturing of the microbial groups under optimized environments in separate digesters.

ENGINEERING APPLICATION -- CONVENTIONAL DIGESTION

In traditional engineering application, anaerobic stabilization of concentrated organic feeds is provided by one of the following process configurations (3):

- Standard-rate digestion
- High-rate digestion
- Stage digestion
- Anaerobic contact process.

These processes provide for the coculturing of the acid-forming and methane-forming populations in slurry-phase digesters under the same physical and chemical environments. The design and operation of these processes are dictated by the sensitivity and kinetic limitations of the slow-growing methane formers. Because the generation time of methane organisms has been estimated to be between 2 and 11 days for waste digestion conditions (4), a minimum retention time between 3 and 16 days is required to prevent washout of the methanogenic organisms. In actual practice, a digester retention time of 10 to 30 days is provided, depending on waste properties, degree of mixing, etc., for reliable process performance (5).

There are serious limitations as to the retention time as well as organic loading that can be applied on conventional mixed-phase digestion to obtain stable process performance, and acceptable gasification and stabilization

efficiencies. This point is illustrated in Figure 2, which depicts the results of mesophilic (35°C) conventional high-rate digestion of a high-chemical oxygen demand (COD)(12,000-26,000 mg/l) soft drink-bottling waste at various loadings and retention times (6). The data show that as the loading was increased from a low value of 0.04 lb VS/ft³-day to a modest level of 0.125 lb VS/ft³-day, the feed, because of its high biodegradability, was rapidly metabolized by the acid formers to volatile fatty acids which accumulated to high levels inhibitory to methanogenic activity. The underlying reason for the resulting digester upset was the kinetic imbalance between the rates of production and utilization of volatile acids, which in this case ensued when a retention time of 10 days and a feed volatile solids (VS) concentration of only 20 g/l were applied. The degree of this imbalance increased when attempts were made to operate the high-rate digester at still shorter retention times and higher loading rates (Figure 3). Digester upsets like this which arise due to unbalanced activities of acid and methane formers are difficult to prevent or correct because it is not possible to control and manipulate the activity of either group of organisms without affecting the activity of the other. In view of the above, the primary disadvantages of conventional mixed-phase anaerobic digestion are:

- Long retention times
- Low loading rates
- Occurrence of unbalanced digestion.

Long retention times and low loading rates in turn lead to the following additional disadvantages:

- Large digestion tanks and large land area requirement
- High capital investment for the installation of the large digestion tanks and associated equipment
- Difficulty of mixing in large tanks -- up to 60% of the volume of conventional digestion tanks is occupied by scum, sludge deposits, incrustations, or dead space
- Low overall stabilization and gasification rate
- Maintenance of the acid formers in the stationary growth phase, and consequent retardation of the hydrolysis and acidification reactions.

Also, as indicated above, the conventional digestion process is vulnerable to varying loading rates and retention times which could easily lead to unbalanced digestion, process instability, and unreliable performance.

Last, but not least important is the fact that conventional anaerobic digestion processes could easily have a negative energy balance; that is, the total external energy input (excluding the energy content of the waste organics) could exceed plant methane energy output, when dilute organic slurries are digested at low loading rates. This is illustrated in Figures 4 and 5 which

depict the net energy production ratio (NEPR) [defined as the ratio of the energy value of the useable energy product (methane), E_p , and the sum of all other energy inputs, E_i , excepting that of the feed] as functions of feed consistency and loading rate. Net energy production ratios are less than one -- this indicates that the process has a negative energy balance and is a net energy consumer -- when the feed slurries are so dilute that the sludge heating requirement is excessive, or when the loading rate is so low that unduly large digesters are required and excessive heat inputs are needed to compensate for heat losses from these digesters.

ADVANCED DIGESTION -- THE TWO-PHASE CONFIGURATION

Advanced digestion utilizes process configurations that could overcome the aforementioned limitations of conventional digestion, it also permits process operation at much higher loading rates and shorter hydraulic retention times (HRT's) than those of the conventional process. As depicted in Figures 2 and 3, digester operation at increased loadings and reduced HRT's leads to the enrichment of an acid-forming culture precluding the establishment of a stable methane-fermentation phase. Since the natural response of an anaerobic digester to high-loading short-HRT operation is separation of the acid-forming phase, it appears reasonable to assist this process and develop a staged system in which conversion of the feed to fatty acids is optimized in the first stage. Because conditions promoting optimum substrate-to-acids conversion are not conducive to stable and efficient acid-to-methane conversion, acidic effluents from the first-stage acid digester must be methanated in a separate methane-phase digester operated in tandem with the first-stage acid digester. Thus, a multi-stage two-phase process, as first suggested by Babbitt and Baumann (7) and later developed by Pohland and Ghosh (3,8), Ghosh *et al.* (9), and others, evolves naturally when anaerobic digestion is conducted at high loading rates and short retention times in the interest of enhanced substrate conversion rate, reduced plant capital cost, and increased net energy production efficiency. Thus, two-phase digestion is an advanced generic multi-stage process in which the acid-forming and the methane-forming bacterial phases are optimized in separate reactors (stages) to substantially enhance the overall process kinetics and NEPR, and reduce plant capital cost.

Reactor Designs

The simplest two-phase system consists of two separate digesters operated in series. If CSTR reactors are used, then the acid digester is usually much smaller than the methane-phase digester. Anaerobic settlers can be used in tandem with each digester to permit densification and recycling of settled effluent solids to increase microbial and substrate solids retention times (SRT's) (8, 10). Depending on the feed properties and operating modes, other reactor designs including plug-flow, packed-bed (anaerobic filter), or fluidized-bed (also referred to as expanded bed and upflow sludge blanket) could be used. For low suspended-solids (SS) feed, an upflow anaerobic filter appears more attractive for methane-phase fermentation, since it allows process operation at a substantially lower HRT. Finally, it should be noted that a two-phase digestion process could conceivably consist of more than two digesters with more than one reactor design used to optimize each digestion phase.

Culture Enrichment Techniques

Several techniques have been proposed for selective enrichment and optimization of the acid-forming and methane-forming phases. Among them are:

- Selective inhibition of methane formers by chloroform, carbon tetrachloride, limited oxygenation, adjustment of redox potential, etc. in the acid digester (11)
- Dialysis separation of the acid- and methane-forming cultures (12, 13)
- Kinetic control of nonmethanogenic and methanogenic organism growth by adjustment of HRT, reactor loading rate, and microbial and substrate SRT's by effluent recycling around each digester of a two-phase system (3, 6, 8, 9, 10). Of the above techniques, phase separation by kinetic control is expected to be superior because:
 - a) It does not have the operating problems of membrane separation.
 - b) It is free from the uncertainties of inhibitor action on both groups of digester organisms.
 - c) The technique is successfully applied to two-phase digestion of soluble and solid substrates (3,9).

TWO-PHASE DIGESTION OF SOLUBLE SUBSTRATES

The application of kinetic control to separate the acid-forming and methane-forming phases of anaerobic digestion of soluble substrates was first demonstrated by Pohland and Ghosh (3, 8), and later by Pohland and Massey (14), Ghosh et al. (9), Ghosh and Klass (15, 16), Heertjes and van der Meer (17), Smith et al. (18), Cohen et al. (19), and Ghosh and Henry (6).

Pohland and Ghosh (3), and Ghosh and Klass (15) studied the following reaction systems to study the kinetic characteristics of acid-forming and methane-forming organisms derived from a digested sewage sludge inoculum and enriched in separate CSTR digesters:

- A. Nutrients + glucose $\xrightarrow[\text{acidification}]{\text{batch-digester}}$ volatile acids + CO₂
- B. Nutrients + glucose $\xrightarrow[\text{acidification}]{\text{continuous-digester}}$ volatile acids + CO₂
- C. Nutrients + volatile acids from B $\xrightarrow[\text{methanation}]{\text{continuous-digester}}$ CH₄ + CO₂
- D. Nutrients + acetate $\xrightarrow[\text{methanation}]{\text{continuous-digester}}$ CH₄ + CO₂

Kinetic constants derived from these digestion data are presented in Table 1. The kinetic information is used to project the performance characteristics of the digestion phases as functions of such control variables as detention time, feed substrate concentration, and loading rate. The parameter selected for evaluation of process performance is substrate conversion rate per unit culture volume, R, given by Equation 1 below:

Table 1. Kinetic Constants* for Mesophilic (37°C) Acidogenic and Methanogenic Cultures Developed on Soluble Substrates.

Kinetic Constants	Acid Formers on Glucose		Methane Formers	
	Batch	Continuous	Mixed Volatile Acid Substrate From Glucose (Continuous)	Acetate (Semicontinuous)
$\hat{\mu}$, day ⁻¹	7.2	30.0	3.4	0.49
K, mg/l	400 (glu)	23 (glu)	600 (acetate)	4200 (acetate)
Y	0.15	0.17	--	0.28

* Kinetic constants shown here are the maximum specific growth rate, $\hat{\mu}$, the saturation constant, K, and the growth yield, Y.

$$R = \frac{S_o (\hat{\mu} \theta - 1) - K}{\theta (\hat{\mu} \theta - 1)} \quad [1]$$

where

S_o = feed substrate concentration,

θ = HRT.

Figure 6 shows that detention times for maximized acidification of glucose and methanation of acetate differ greatly from each other, and optimized high-rate glucose digestion is not expected in a single-stage mixed-phase conventional digester. However, a two-phase system in which CSTR-type acid and methane digesters are operated at HRT's of 3.6 hours and 3 days can, in theory, result in a maximum glucose-to-methane conversion rate. Substantial reduction in these HRT's could be obtained by utilizing reactor designs that provide for maintenance of long SRT's (10).

Various reactor designs, differing from the CSTR digesters initially employed by Pohland and Ghosh (3) and Ghosh and Klass (15), were studied by other researchers. Cohen et al. (19) experimented with a two-phase system consisting of a CSTR acid-phase reactor and a plug-flow type upflow methane digester with a built-in settler to conduct anaerobic digestion of glucose. The cell yield coefficient for acid-phase digestion of glucose at 30°C was 0.11, compared with a yield coefficient of 0.17 at 37°C reported by Ghosh (20) and Ghosh and Pohland (21). Ethanol, acetate, propionate, butyrate, formate, lactate, carbon dioxide, and hydrogen were the main products of acidogenesis. Butyrate was produced in the largest concentrations, followed by acetate. The acidogenic reaction products were gasified in the upflow methane digester to produce head gases having 84.3 mol % methane and 15.7 mol % CO₂.

Ghosh (22), and Ghosh and Henry (6) operated a CSTR acid-phase and an upflow packed-bed methane digester with real soft-drink bottling waste, and demonstrated that a two-phase digestion process could be operated at about 7 times the loading rate and one-half the HRT of the conventional process and still obtain the same methane production as and a slightly higher COD reduction than the conventional process (Table 2). An important advantage of the

Table 2. High-Rate and Two-Phase Mesophilic (35°C)
Digestion of Soft-Drink Bottling Waste.

	Conventional High Rate	Two Phase		
		Acid Phase	Methane Phase	Overall
Loading, lb VS/ft ³ -day	0.04	1.0	0.4*	0.3
HRT, days	15	2.2	5.2*	7.4
Gas Yield, SCF/lb VS added	10.3	1.02	9.44	9.76
Methane Content, %	61.1	0.2	70.5	63.1
Gas Production Rate, vol/vol-day	0.4	1.03	3.68	2.90
Digestion Efficiencies, %				
VS Reduction	72	--	--	64
COD Reduction	84	--	--	96
Digester Volume for 20,000 lb/day TS Load, 1000 ft ³	198	22	44	66
Net Energy Production				
10 ⁶ Btu/day	46,3	--	--	80
Percent of Total Production	37	--	--	64

* Loading and HRT of the upflow filter were calculated on the basis of the gross volume of the packed bed.

two-phase process was that gases from the methane phase had a significantly higher methane content than those of the conventional digester. Two-phase operation allowed the total digester volume (and associated capital and operating costs) to be reduced by 67% and the net energy production to be increased by more than 73% relative to those of the conventional process. Also, while the conventional high-rate digester failed at an HRT of 10 days and a feed COD concentration of 26,000 mg/l, the two-phase process exhibited stable and efficient performance at a system HRT of 7.4 days and a feed COD concentration up to 45,000 mg/l.

Heertjes and van der Meer (17) also conducted two-phase digestion of saccharose and sodium acetate in an upflow digester with an internal settler built at the top (effluent end) of this digester. High conversion efficiencies were obtained at 3- to 6-hour residence time and a relatively low loading (0.12 lb TOC/ft³-day). A two-reactor two-phase system exhibited increased stability at higher loadings up to 0.74 lb TOC/ft³-day.

Smith et al. (18) operated a packed-bed mesophilic (37°C) upflow methane digester ("anaerobic filter") with solids-free acidic substrates derived from animal wastes. Satisfactory acid-phase digestion could not be developed with this waste. Methane digester gas production rates from 0.24 to a high value of 2.77 volume/digester volume-day were observed at hydraulic retention times of 40.5 to 1.1 days.

In a recent study, Pipyn et al. (23) investigated anaerobic digestion of distillery wastewaters (~10,000 mg/l COD) in a two-phase pilot plant consisting of a 36-m³ CSTR acid-phase digester and a 5-m³ upflow methane-phase digester. The acid-phase was operated at 42°±2°C at an HRT of 16 to 72 hours, while the methane-phase digester was maintained at 39°±2°C and an HRT of 14 hours. Overall COD and BOD (biochemical oxygen demand) reductions of 84 percent and 92 percent were obtained. The methane digester gases had a methane content of 75±3 mol percent.

TWO-PHASE DIGESTION OF SOLID SUBSTRATES

Ghosh et al. (9), and Ghosh and Klass (15) first demonstrated the feasibility of separating the acid and methane phases of anaerobic digestion of a particulate feed (activated sludge) by kinetic control. Satisfactory acid-phase digestion occurred at detention times of 10 to 24 hours and high loadings of 2 to 5 lb VS/ft³-day. Acidogenesis occurred at an oxidation-reduction potential (E_c) of -240 mV and a pH of 5.7, compared to -400 mV and 7.0 for methane formers. Kinetic constants determined for acidogenesis of activated sludge and biomethanation of acetate, which was the primary substrate for methanogens, are reported in Table 3.

Table 3. Kinetic Constants for Mesophilic (37°C)
Two-Phase Digestion of Chicago Activated Sludge.

<u>Kinetic Constants</u>	<u>Acidogenesis of Sludge</u>	<u>Biomethanation of Acetate</u>
$\hat{\mu}$, day ⁻¹	3.84	0.49
K, g/l	4.3 (as VS)	4.2 (as acetate)
Y	0.4	0.28

Figure 7, developed from the above kinetic constants, shows that with a concentrated (5 wt percent VS) sludge feed, maximum acidogenesis and methanation rates occurred at HRT's of 0.75 and 3 days, respectively, indicating that high-rate conversion of sludge to methane could be achieved in a two-phase system having an overall HRT of about 4 days. These figures also indicate

that the substrate conversion rate decreases and the HRT for maximum conversion rate increases substantially as the system sludge feed concentration decreases; this means that the HRT of the overall two-phase system increases significantly as the system feed becomes more and more dilute. Thus, for dilute feeds, a two-phase process may not be as superior to the conventional process as it is for concentrated feeds.

Two-phase mesophilic digestion of 1.7 to 2.5 weight percent VS Chicago sludge at an overall HRT of 6.9 to 7.7 days exhibited an average methane yield of 4.3 SCF/lb VS added and a VS reduction of 40 percent (9) compared with 3.5 SCF/lb VS added and 34 percent observed during conventional digestion of this sludge at an HRT of 14 days. The methane content of the conventional digester gases was 60 mol percent compared with 70 percent in the head gases of the methane-phase digester.

Eastman and Ferguson (24) conducted acid-phase digestion of primary sewage sludge at detention times of 9 to 72 hours, and concluded that hydrolysis of the solid sludge particles was the rate-limiting step of the overall acidogenic phase. Lipids were not biodegraded, and 50 percent of the non-lipid COD of primary sludge was solubilized. Acidogenic sludge was difficult to settle. Hydrogen evolution occurred at the minimum detention time of 9 hours. Volatile acid production and distribution of acid species in the effluent appeared to be influenced by the reactor pH. Brown (25) indicated that hydrolysis of particulate substrate was favored at an acidic pH (pH 6), and methane fermentation of the acid-digestion products was better at an alkaline pH (pH 7.5). Detailed investigation of the pH effect was not conducted to delineate the pH optima, however. The methane digester gases contained 80 mol percent methane.

Norrman and Frostell (26) conducted mesophilic (33°C) two-phase digestion of a semisolid synthetic feed (blended dog food) in a laboratory system comprised of a completely mixed acid-phase digester and a packed-bed upflow methane digester. The acid digester was followed by a 500-ml gravity settler, the supernatant from which was fed to the packed-bed methane digester. Acid digester pH was low (pH 4). Solid-liquid separation was a problem with the acid-digester effluent. The overall system was operated at detention times of 2.7 to 12.1 days and low loadings of 0.026 to 0.14 lb VS/ft³-day. A long start-up time was required for the anaerobic filter. The methane digester gases contained 65 to 80 mol percent methane. Like Norrman and Frostell, Therkelsen and Carlson (27) also investigated the two-phase digestion characteristics of dog food, but at a thermophilic temperature of 50°C. The performances of completely mixed and plug flow acid digesters were compared. Surprisingly, lactate was the major acidic product. The pH of the acid digester dropped to 4. Grease and organic nitrogen were not reduced significantly. One interesting observation was that acid production in a plug-flow acid digester was much higher than that in the complete-mix reactor. At the test loadings (0.37 to 0.62 lb VS/ft³-day) and detention times (4.3 to 7.5 days), two-phase thermophilic digestion of dog food was slightly better than thermophilic conventional digestion.

Keenan (28) conducted two-phase digestion of simulated solid waste (Purina Dog Chow) at 22° and 48°C. The acid-phase digester had relatively

long detention times of 4.5 and 6 days; the methane digester had a detention time of 10 days. Acid digester gases contained mainly CO₂ and a small amount of hydrogen. Gases from the methane digester had 80 mol percent methane. The acid digester effluent had 13,000 to 14,000 mg/l of volatile acids. There was no significant difference in acid conversion efficiencies at 22°C and 48°C. The two-phase process provided more stability than the conventional mixed-phase high-rate process.

In contrast to the two-reactor systems studied by most researchers, Johnson (29) found evidence of separation of the acidogenic and methanogenic phases during anaerobic fermentation of pig excrement and biomass leachate in a four-stage system. The two-phase multi-stage process was superior to conventional high-rate digestion.

BENEFITS OF TWO-PHASE DIGESTION

Analysis of the laboratory and pilot plant research data presented above shows that a multi-stage two-phase process evolves naturally when anaerobic digestion is conducted at high loadings and short HRT's in the interest of enhanced substrate conversion and gasification rates, reduced plant capital cost, and increased net energy production efficiency. Investigators of the two-phase digestion process presented ample experimental evidence to indicate that this advanced digestion process is potentially far superior to the conventional "high-rate" digestion process.

The feasibility of phase separation by kinetic control has been demonstrated for both soluble and solid substrates by several authors. Acid-phase digestion can be conducted at residence times as low as 3 to 6 hours for soluble organics, and 9 to 24 hours for particulate organic material. With proper process design, the overall two-phase system could be operated at residence times of 2 to 5 days, a substantial improvement over conventional high-rate digestion conducted at residence times of about 12 to 20 days.

In summary, the two-phase process has the following demonstrated and potential benefits:

- Capability to optimize the environment and operating conditions for each digestion phase
- Maximization of the overall substrate conversion rate per unit culture volume without sacrificing conversion efficiency
- Decreased digester volume, and plant capital and operating costs
- Improved mixing in low-residence time digesters
- Higher methane content (up to 85 mol percent) of the final product gas
- Enhanced net energy production efficiency
- Reduced nitrogen content in the final product gas owing to increased

denitrification of the feed in the acid digester

- Increased process reliability owing to separation of the sensitive methane bacteria and their protection from environmental shocks of sudden bursts of acid production, pH drops, and direct exposure to inhibitors.

PROBLEMS AND RESEARCH NEEDS

A careful consideration of the work of various investigators indicates that several potential problems including inefficient acetate formation, substrate inhibition in methane-phase digestion, retarded digestion of such substrate components as lipids and certain nitrogenous compounds could arise during two-phase digestion. Considerable fundamental research should be undertaken to alleviate these problems and to develop an understanding of the behavior of each microbial digestion phase in response to manipulation of important fermentation parameters, operating modes, and reactor design.

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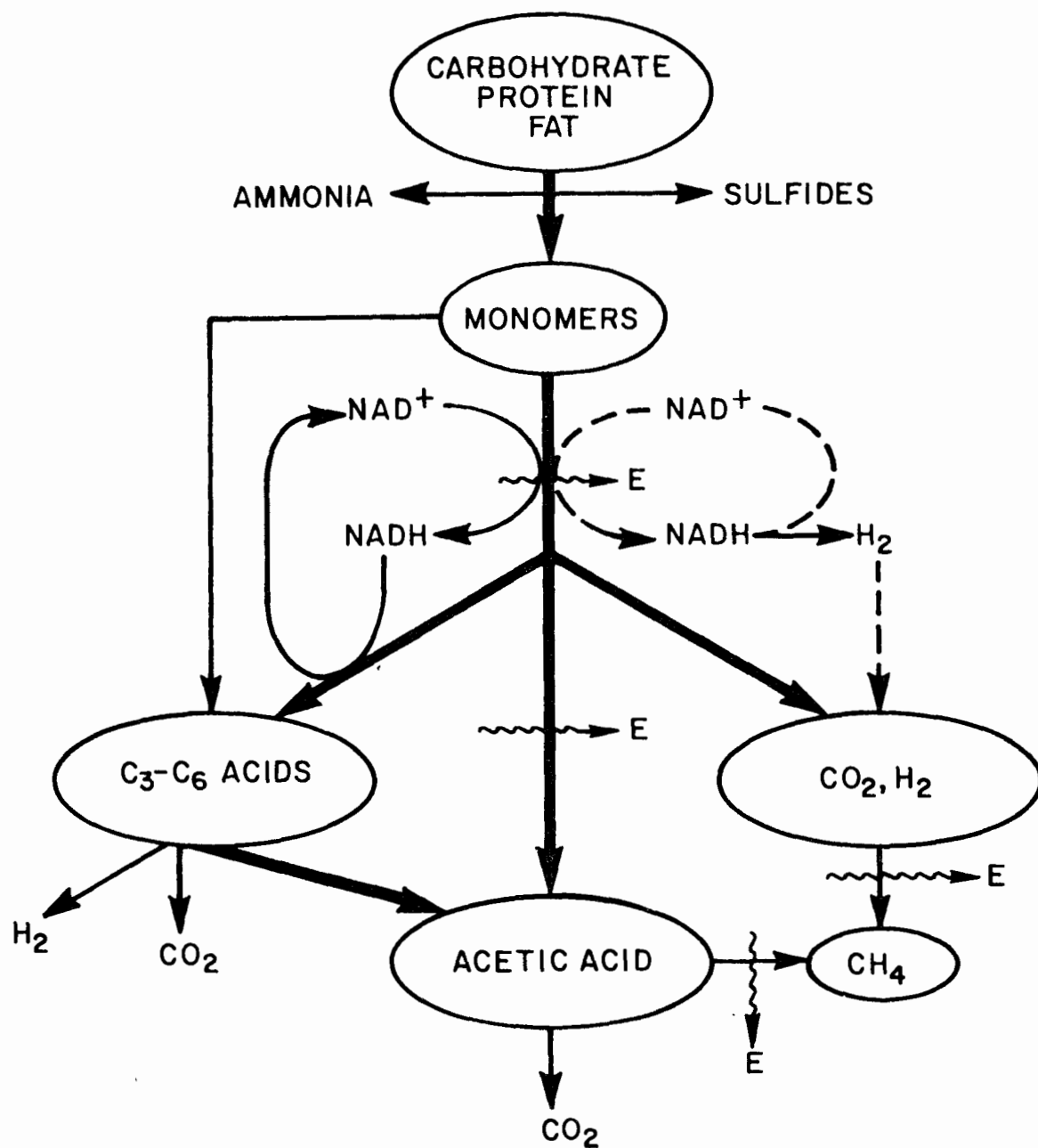
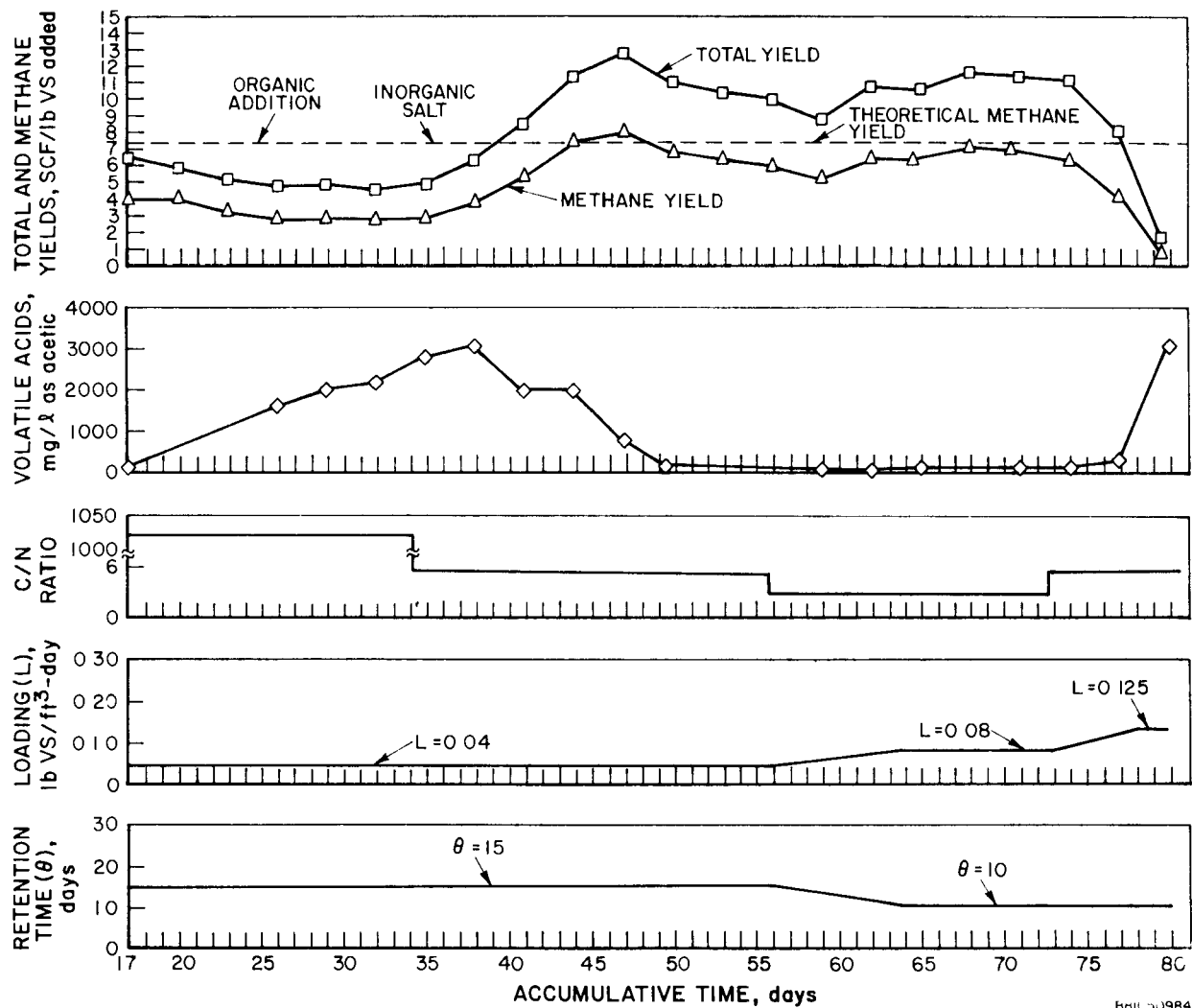


Figure 1. Reaction Steps in Anaerobic Digestion.



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Figure 2. Mesophilic (35°C) Conventional High-Rate Digestion of Soft-Drink Waste.

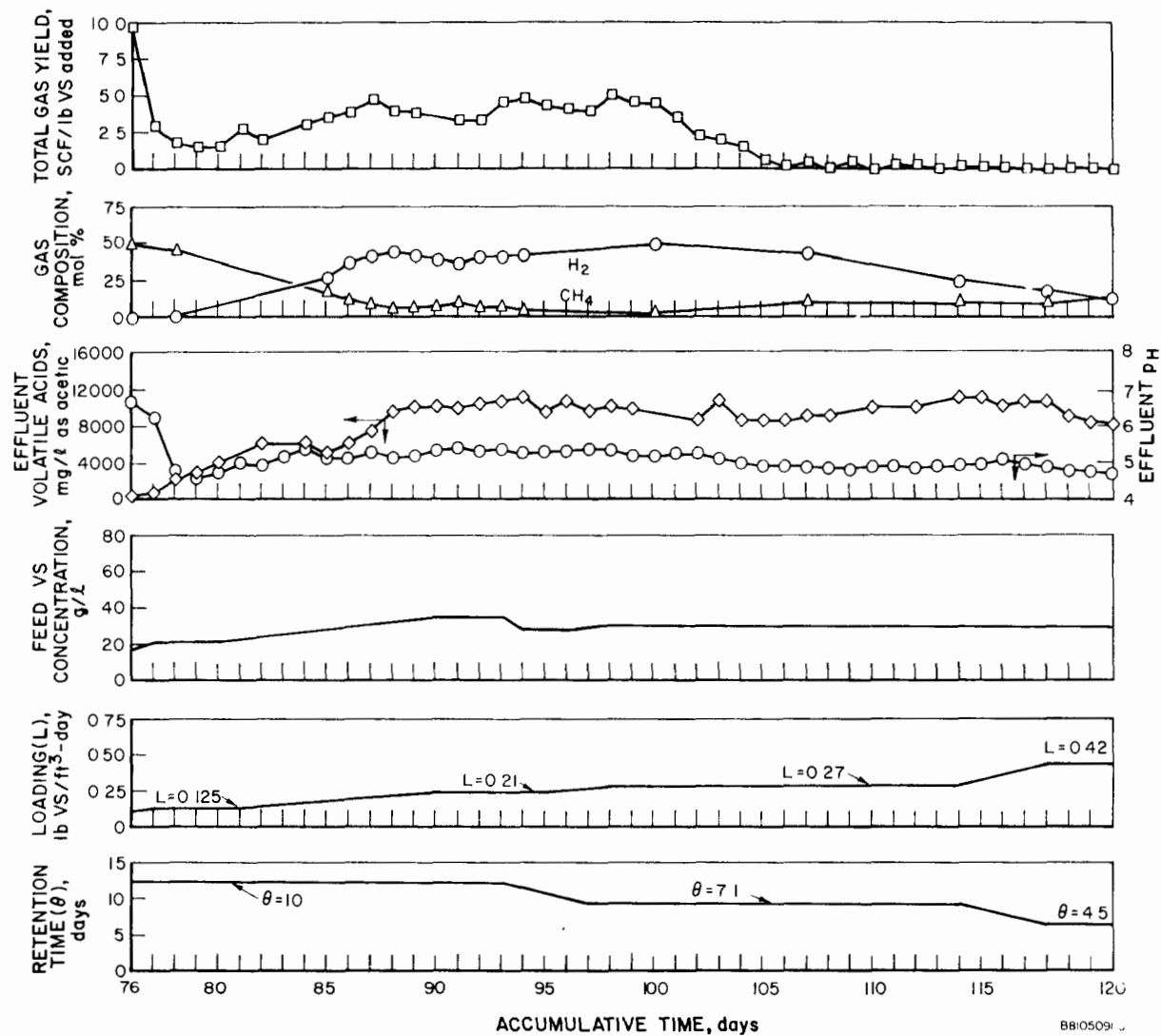


Figure 3. Response of a Continuous-Flow High-Rate Soft-Drink Waste Digester to Increased Loadings and Reduced Detention Times: Development of an Acid-Phase Culture.

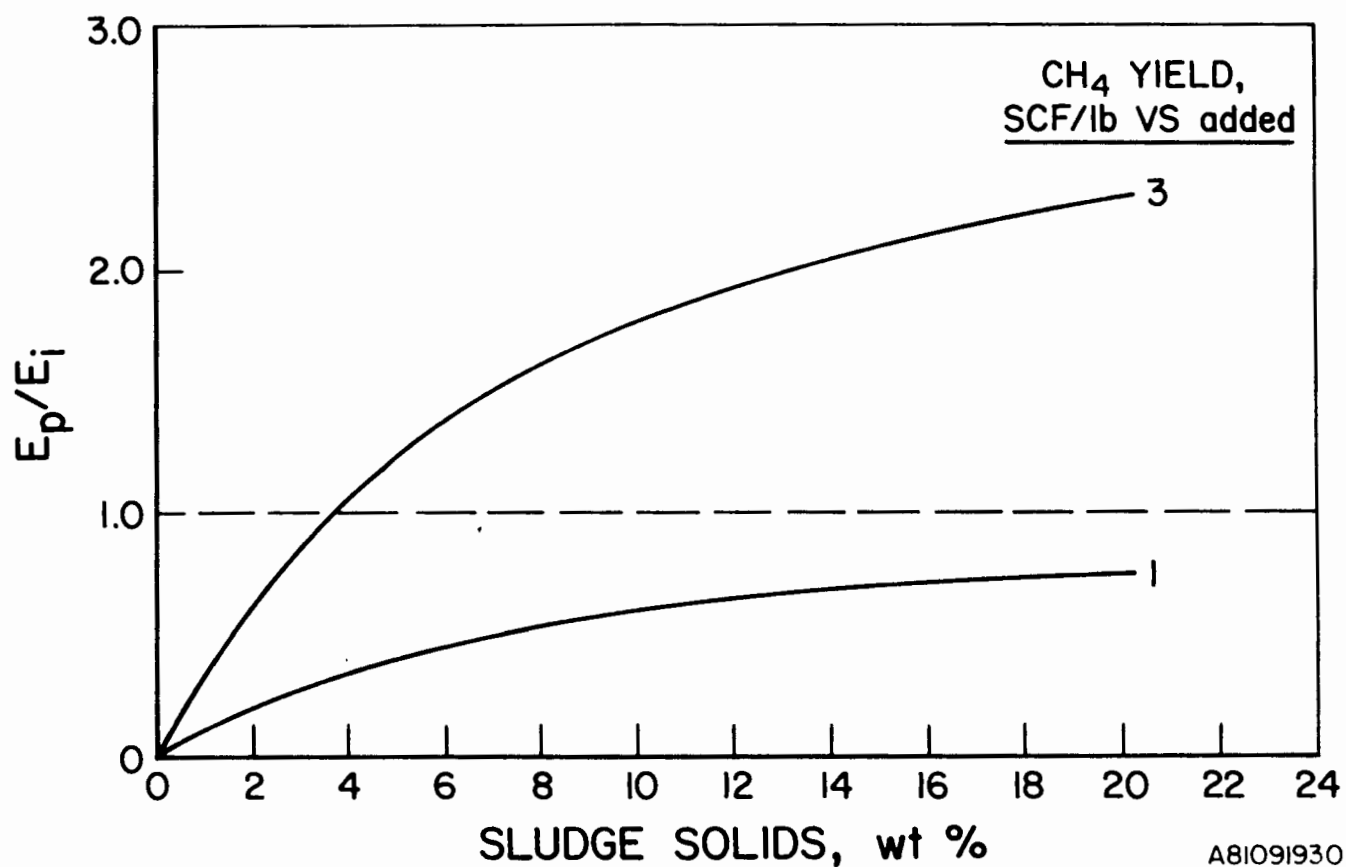


Figure 4. Dependence of the Net Energy Production Ratio on the Feed Sludge Solids Concentration for Mesophilic (35°C) Digestion of a 60-wt Percent VS-Content Feed at a Loading Rate of 0.1 lb VS/ft³-day (The feed slurry is assumed to have a temperature of 15°C).

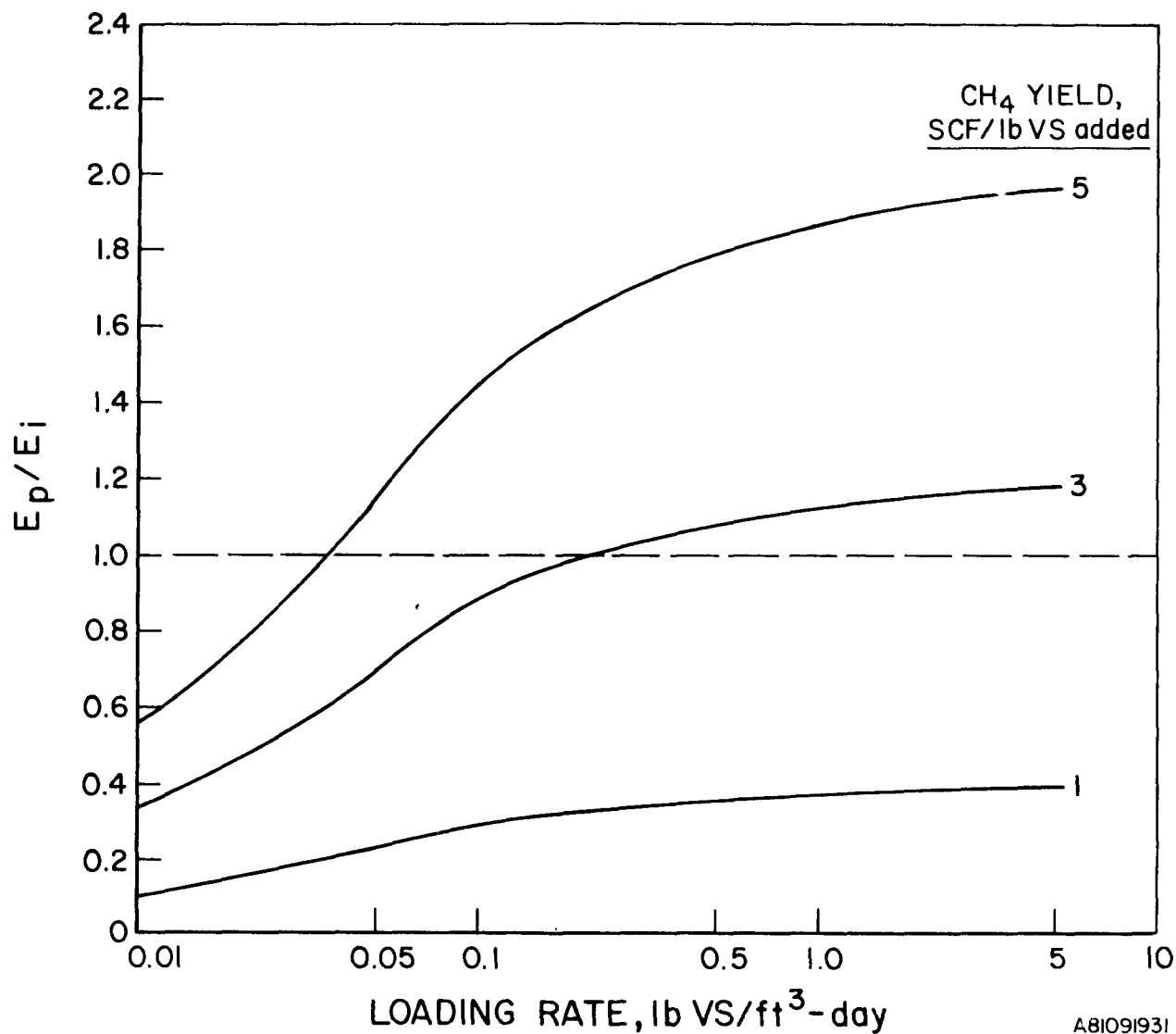


Figure 5. Dependence of the Net Energy Production Ratio on the Digester Loading Rate for Mesophilic (35°C) Digestion of a 60-wt Percent VS-Content Feed (The feed slurry has a 3-wt percent solids concentration and a temperature of 15°C).

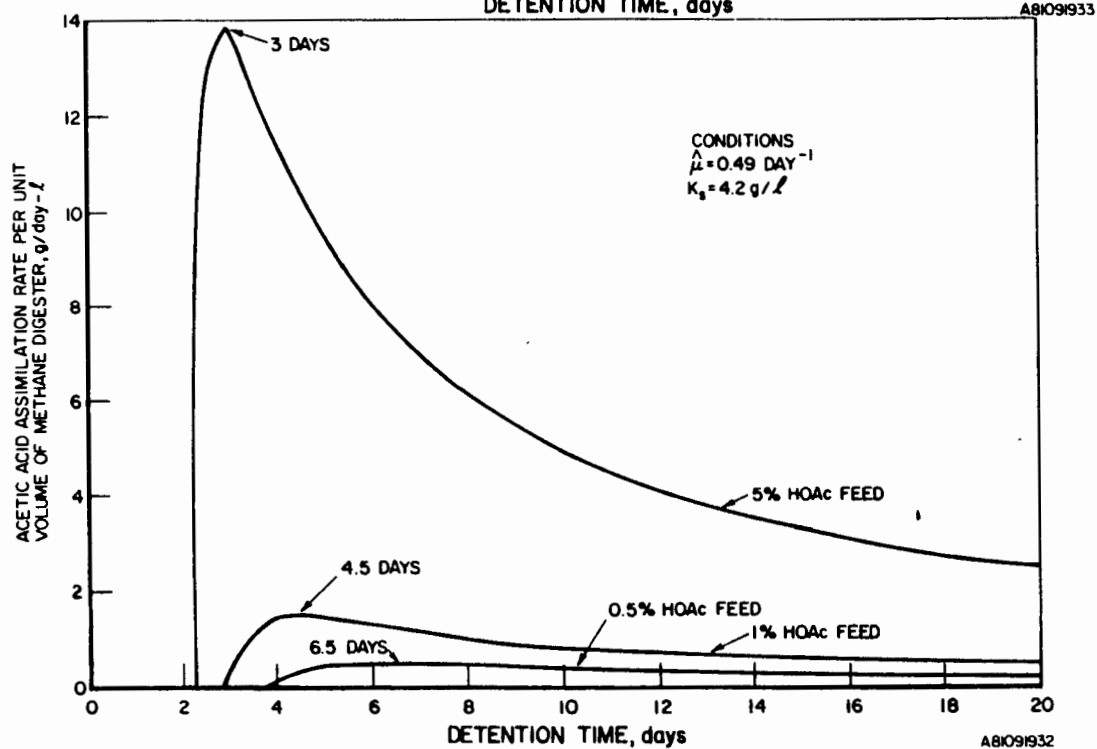
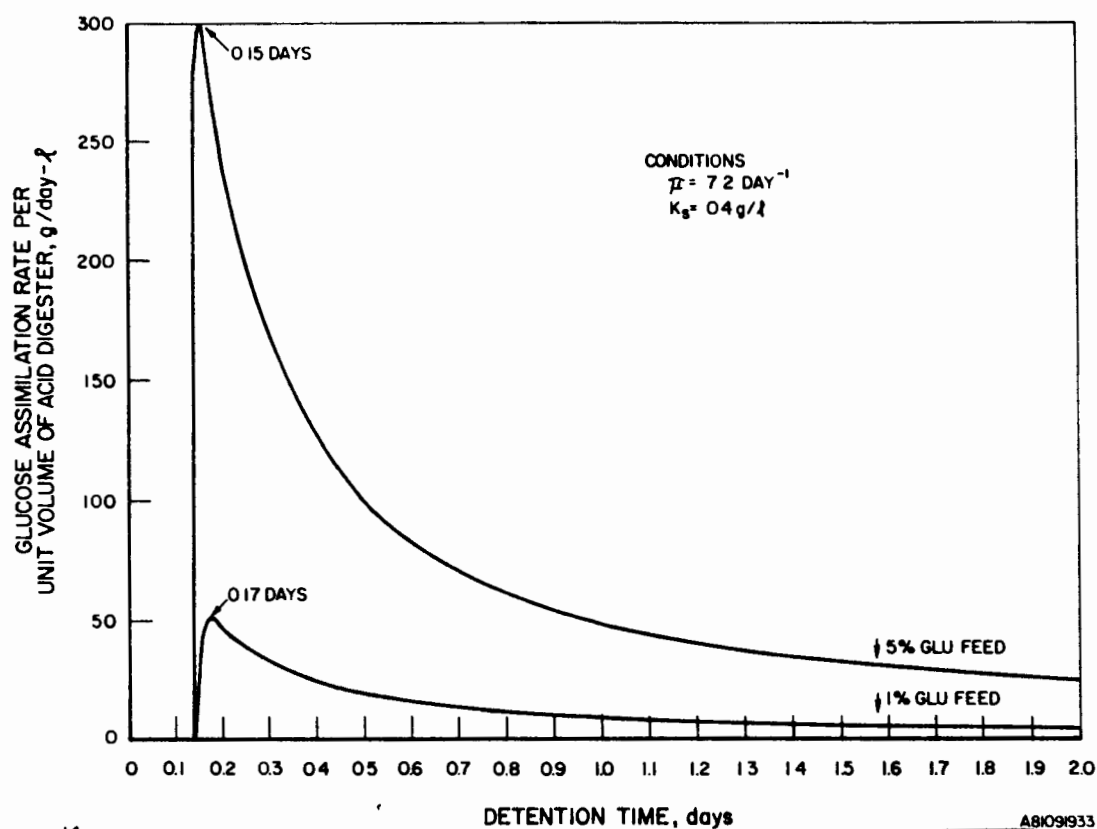
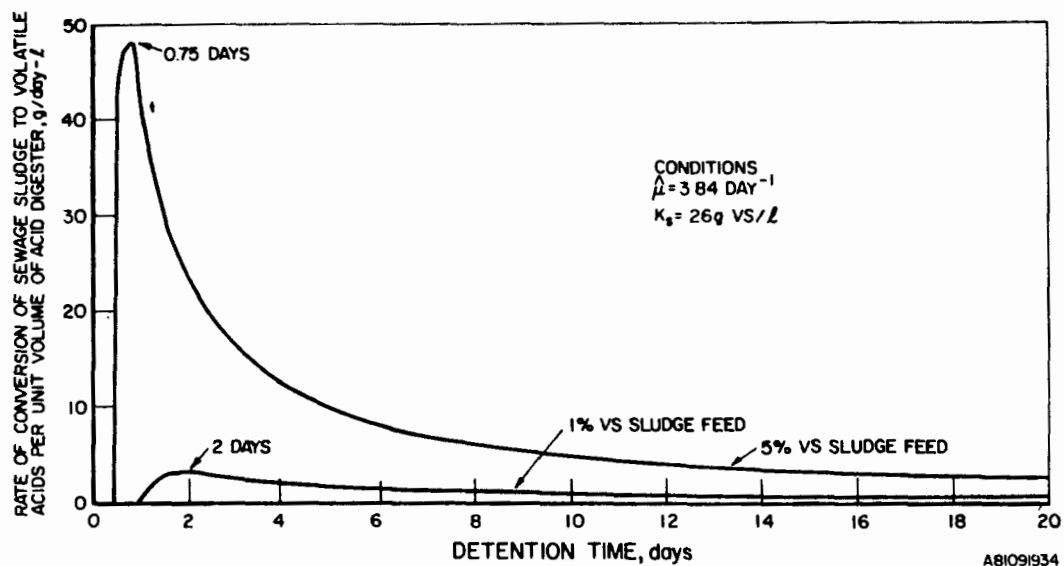
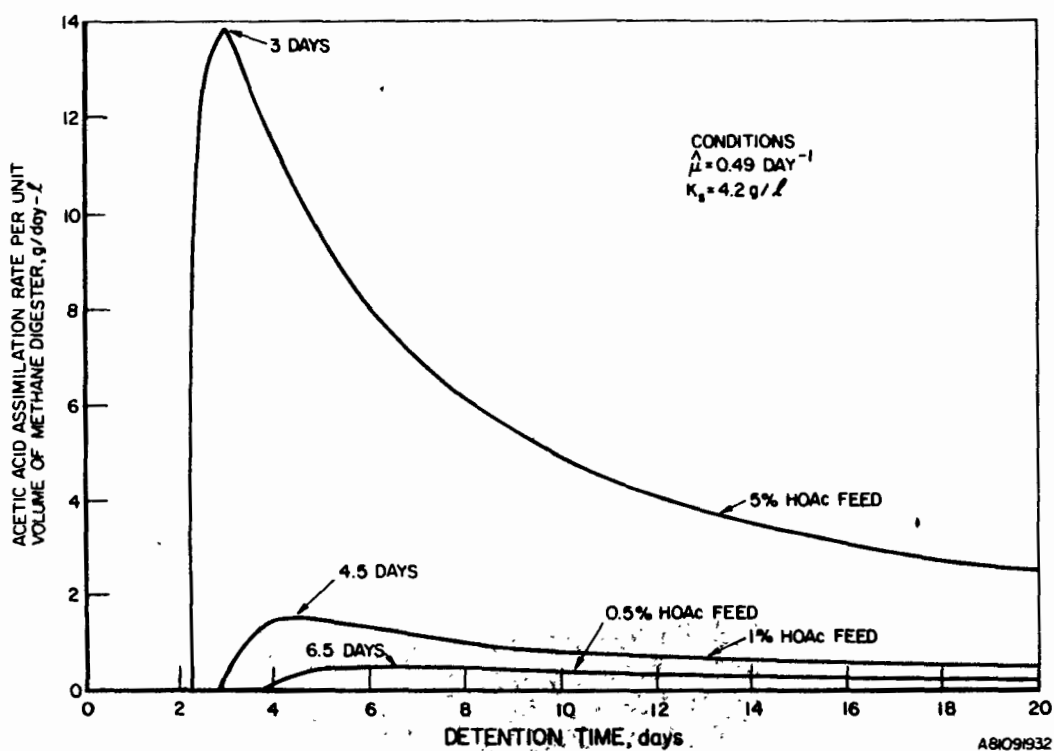


Figure 6. Glucose Acidification and Acetic Acid Methanation Rates Per Unit Acid Digester Volume as Functions of Detention Time and Substrate Concentration Under Mesophilic (37°C) Conditions.



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Figure 7. Sewage Sludge Acidification and Acetic Acid Methanation Rates Per Unit Acid Digester Volume as Functions of Detention Time and Substrate Concentration Under Mesophilic (37°C) Conditions.