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PROCEEDINGS

THE THIRD JAPANESE-GERMAN WORKSHOP ON WASTE WATER AND SLUDGE TREATMENT

Date:
September 29 and 30, 1986

Place:
Public Works Research Institute
Ministry of Construction
Tsukuba Science City, Ibaraki Prefecture, Japan

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

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Foreword

The Third Japanese-German Workshop on Wastewater and Sludge Treatment was held in Japan on September 29 and 30, 1986, and the technical information exchange was deepen between Japan and F.R. Germany.

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

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



Itaru Nakamoto
Director General,
Department of Sewerage and Sewage Purification
Ministry of Construction

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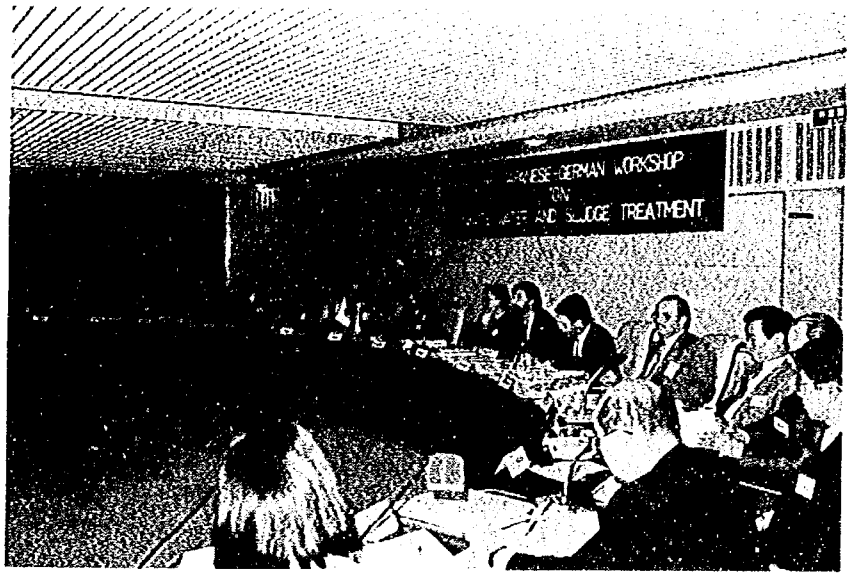
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German and Japanese Participants to the Workshop, Public Works Research Institute, Ministry of Construction.



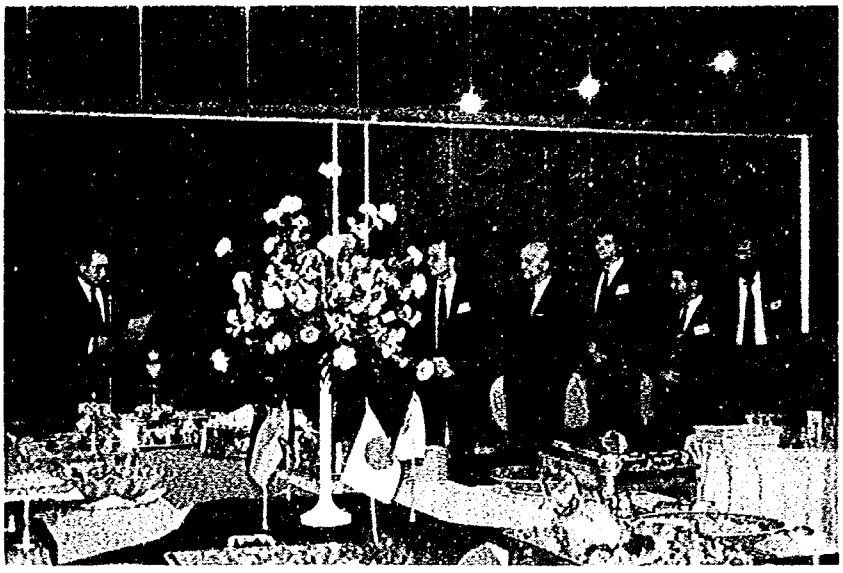
German and Japanese Participants to the Workshop, Public Works Research Institute, Ministry of Construction.



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



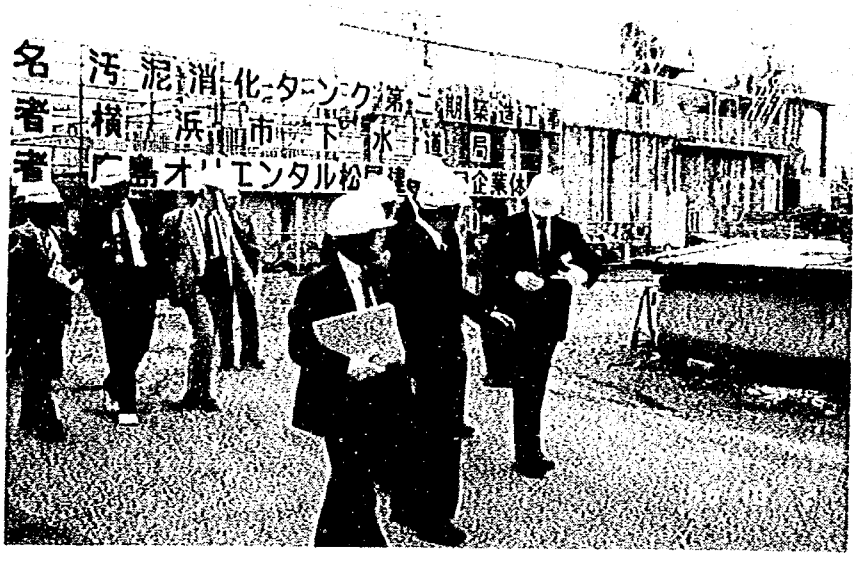
Snap Shot of the Workshop at the International Conference Room of PWRI.



Dr. Tominaga, Director General of PWRI greets at the reception held in Tsukuba Science City.



Study Tour at the Water Recycling Center in Tokyo Metropolitan.



Study Tour at Hokubu II Sewage Treatment Plant in Yokohama City



Study Tour at Hirano Sewage Treatment Plant in Osaka City

SCHEDULE OF THE WORKSHOP AND ATTACHED PROGRAM

September 29 – 30, 1986	Workshop • Public Works Research Institute, Ministry of Construction (Tsukuba, Ibaraki Prefecture)
October 1,	Study Tour • Water Recycling Center (Tokyo Metropolitan)
October 2,	Study Tour • Hokubu II Sewage Treatment Plant (Yokohama City)
October 3,	Study Tour • Ishida Sewage Treatment Plant (Kyoto City)
October 4,	Study Tour • Hirano Sewage Treatment Plant (Osaka City)

JOINT COMMUNIQUE

The Third German-Japanese Workshop on Waste Water and Sludge Treatment
– Municipal Waste Water Field –
Tsukuba, Ibaraki Prefecture, September 30, 1986

1. The third Japanese-German Workshop on Waste Water and Sludge Treatment was held in Public Works Research Institute of Ministry of Construction in Tsukuba-Science City, Ibaraki Prefecture on September 29 and 30, 1986.

2. Delegation from the Federal Republic of Germany headed by Dr. W. Röbel, Deputy Director, Water Technology Agency, Nuclear Research Center Karlsruhe was composed of two representatives acting on behalf of the German Federal Ministry for Research and Technology, six representatives from universities and one representative from a private company.

3. The Japanese Delegation headed by Mr. Itaru Nakamoto, Director General, Department of Sewerage and Sewage Purification, Ministry of Construction was composed of four from Ministry of Construction, one from Environmental Agency, one from Ministry of Health and Welfare, three from Japan Sewage Works Agency, one each from Cities of Tokyo, Yokohama, Kyoto, Osaka and Fukuoka, and one each from Universities of Tokyo and Kyushu. In addition to these members, many experts from Ministry of Construction, Environmental Agency, Japan Sewage Works Agency, municipalities, universities and other organizations joined the Workshop.

4. The chairmanship of the Workshop was shared by Dr. W. Robel and Mr. Itaru Nakamoto.

5. Principal topics of the Workshop were nitrogen and phosphorus removal, administration and finance of sewerage, sludge treatment, small-scale wastewater treatment, sludge disposal and utilization and upgrading of wastewater treatment. The discussion which followed the presentations were also useful to both countries.

6. Study tour is planned from October 1 to 4 after these two day session at a wastewater reuse facility of Tokyo Metropolitan Government in Shinjuku, a centralized sludge treatment plant of Yokohama City, a co-incineration plant of sludge with refuse of Kyoto City, a three storied final sedimentation tank of Osaka City, etc.

7. Both sides agreed that this exchange of experiences is very fruitful. Activities regarding next Workshop should be determined by the time of the German-Japanese Panel for Research and Development on New Environment Protection Technology held in Tokyo in 1987.

SESSION 1

NITROGEN AND PHOSPHORUS REMOVAL

"Nitrification and Denitrification in the Two-stage Sewage Treatment Plant of the City of Krefeld, Designed for a Total Number of Inhabitants and Population Equvalents of 800,000"
F.B. Frechen 3
"Demonstration Experiment on Nitrification-Denitrification Process using Endogenous Respiration for Denitrification"
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SECTION 1

METHODOLOGY AND PHOSPHORUS REMOVAL

NITRIFICATION AND DENITRIFICATION AT THE TWO-STAGE SEWAGE TREATMENT PLANT OF THE CITY OF KREFELD, DESIGNED FOR A TOTAL NUMBER OF INHABITANTS AND POPULATION EQUIVALENTS OF 800,000

Dipl.-Ing. Franz-Bernd Frechen
Institute for Sanitary Engineering
Aachen University of Technology



Nitrification and Denitrification at the Two-stage Sewage Treatment Plant of the City of Krefeld, Designed for a Total Number of Inhabitants and Population Equivalents of 800,000

Dipl.-Ing. F.-B. Frechen

Institute for Sanitary Engineering
Aachen University of Technology

1. Presentation of the sewage plant

Krefeld has about 230,000 inhabitants, small trade and some important industrial enterprises, e.g. of the branches food, fertilizer, chemical, textile, metal and brewery. The sewage discharge is done in a combined sewer system, the river Rhine is the receiving water.

More than 60% of the waste water is of industrial origin. Several studies in the seventies have shown extreme concentration fluctuations of the parameters BOD_5, COD, NH_4^+-N, NO_3^--N, oPO_4-P, conductivity and pH-value. Since the problem of the wastewater was known it was difficult to find a qualified biological sewage treatment system which should replace resp. upgrade the old mechanical plant. Several tests had already been carried out without success when in 1976 the studies of the Institute for Sanitary Engineering (ISWW) of the Aachen University of Technology showed satisfactory results using the Adsorption-Bio-aeration process (AB-process). So the sewage treatment plant of Krefeld was designed according to the AB-process-technology and started operation in 1981.

The AB-process is a two stage activated-sludge-process with a maximum loaded first stage (A-stage) and a normal low-rate bio-aeration in the second stage (B-stage). Characteristics are the strict separation of the sludge cycles of the two stages and the high sludge load of the A-stage. This high sludge load is

achieved by a small volume of the A-stage and a low MLSS-content obtained by taking out a sufficient amount of excess sludge. The sludge load of the A-stage should usually range between 3 and 6 kg BOD_5/(kg MLSS.d), higher loads are definitely possible.

The basic data for the plant design were:

total number of inhabitants and population equivalent	:	800,000 p.e.
specific BOD_5-load	:	60 gBOD_5/(p.e. d)
specific sewage flow	:	180 l/(p.e. d)
BOD_5-load	:	48,000 kg BOD_5/d
daily sewage flow	:	143,000 m^3/d
hourly sewage flow	:	10,200 m^3/h

Figure 1 shows the site plan of the sewage treatment plant. The essential data of the plant and the design are:

A-stage:

aeration tank :			
volume	:	2 x 2,550 m^3 =	5,100 m^3
retention time:			0.5 h
sludge load	:		5 kg BOD_5/(kg MLSS.d)
intermediate clarifier:			
volume	:	4 x 2,550 m^3 =	10,200 m^3
retention time:			1.0 h

B-stage:

aeration tank :			
volume	:	2 x 15,360 m^3 =	30,720 m^3
retention time:			3.01 h
sludge load	:		0.15 kg BOD_5/(kg MLSS.d)
final clarifier:			
volume	:	8 x 5,100 m^3 =	40,800 m^3
retention time:			4.0 h

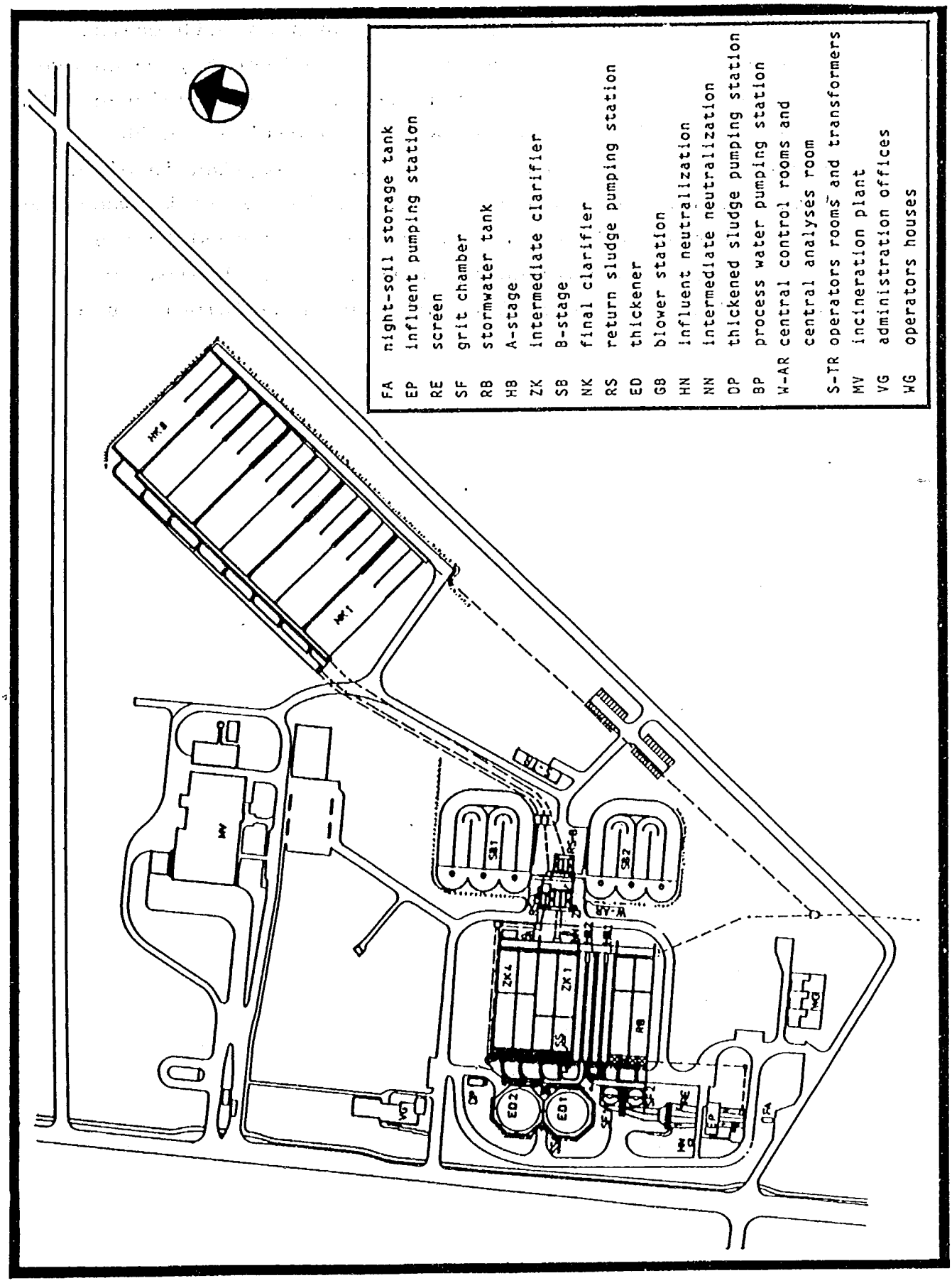


Fig. 1: Site plan of the sewage treatment plant of Krefeld

The two aeration tanks of the A-stage are equipped with coarse-bubble aeration, the two aeration tanks of the B-stage, which are designed as circulation tanks ("carousel-tanks") are equipped with three centrifugal surface aerators each. In addition, in the B-stage there are two times three propellers installed in either tank that maintain the necessary circulation in the tanks when the aerators are switched off. The excess sludge is pumped to the incineration plant after thickening where it is mechanically dewatered, dried and incinerated together with municipal waste.

2. Conditions of the operation results presented

Like every other big plant the sewage treatment plant in Krefeld had to deal with some initial problems.

- The separation efficiency of the grit chambers of the old plant was insufficient. To prevent sand settling in the A-stage, only one of the two tanks was operated and aerated with the full blower capacity.
  In the meantime a grit collector has been installed in the A-stage. Thus, since the middle of 1986 the A-stage also serves as an additional grit chamber.

- The outlets of the B-stage tanks were too low and too wide. The level of two of the three aerators installed in each tank is not adjustable. Hence, an insufficient depth of immersion and a quite low rate of oxygen transfer followed. In the middle of the year 1985 the level of the outlets was raised.

- At the 8 rectangular final clarifiers with suction collectors problems emerged with the rail mounting. Therefore on the average only half of the the tanks were in operation. In the middle of 1984 a reconstruction was started which was finished in September 1985.

- The combustible amount of sludge depends on the amount of municipal waste. Moreover the incineration plant itself has to deal with defects from time to time, so it is not always possible to incinerate a sufficient amount of sludge. Thus, sometimes the sludge has to be stored in the sludge thickener. If no sludge can be incinerated for a longer period, the thickeners can overflow whereby the MLSS content in the aeration tanks of the A-stage and the quantity of sludge in the intermediate clarifier increase. Possibly that may even have an effect on the sludge quantitity in the B-stage.

- Moreover the plant was extremly high loaded in the period between december 1984 and march 1985, in connection with foaming. This effects will be considered later on.

The analyses results presented in the following are the results of the self-supervision by the operator. In the operator's laboratory grab samples as well as 24-h-average samples are analysed.

It is to be mentioned that the COD is analysed with dichromate as oxidizing agent.

3. Load of the plant and results concerning BOD_5 und COD

The average annual load of the plant expressed in the population equivalents can be seen in Figure 2. The population equivalent is based on:

1 p.e.$_{BOD5}$	= 60 g BOD_5/d	(homogenized)
1 p.e.$_{COD}$	= 120 g COD/d	(homogenized)
1 p.e.$_{Hydr.}$	= 180 l/d	

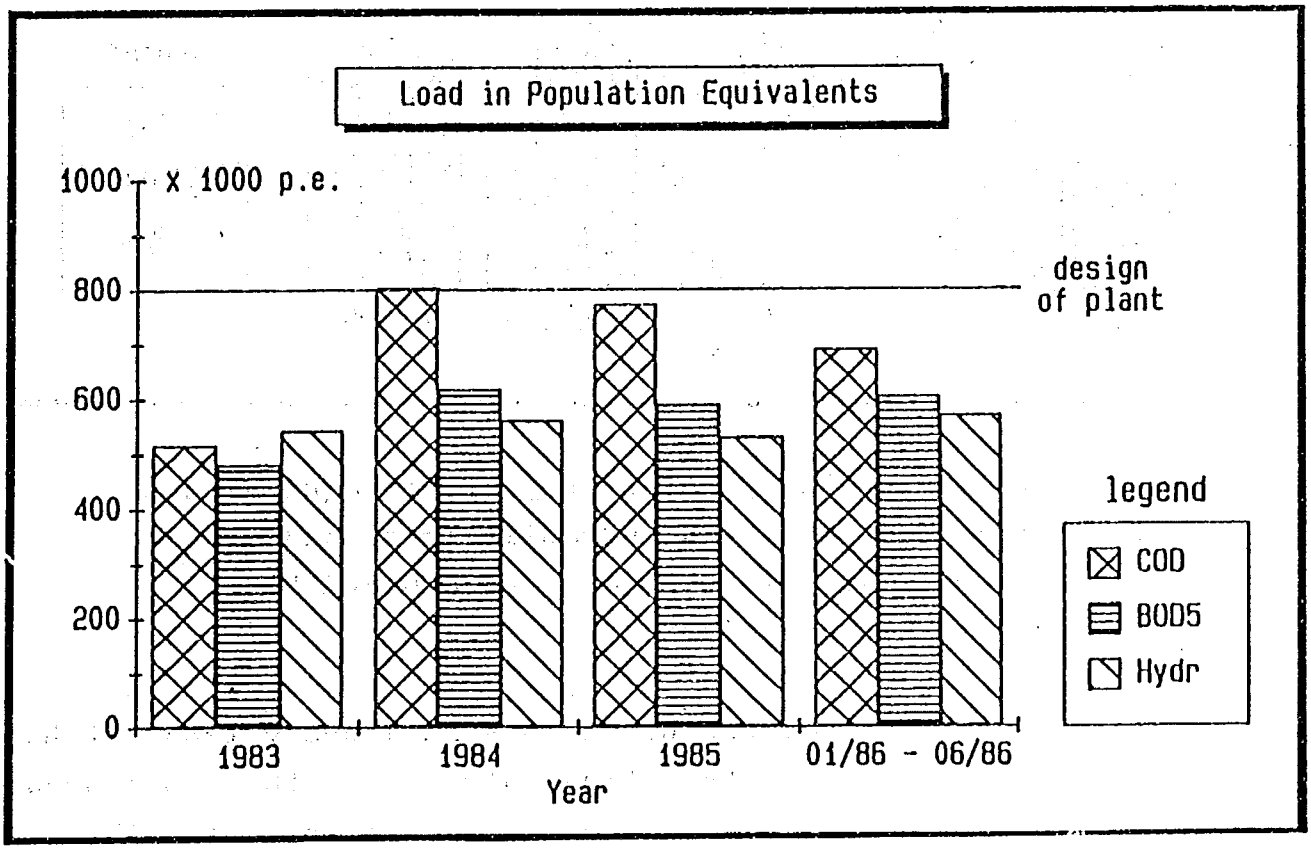


Fig. 2: Average load of the plant in population equivalents concerning COD, BOD_5 and hydraulic load, of the years 1983, 1984, 1985 and the first half year 1986

Of course, short term maximum loads do not appear in the annual average value. As already hinted a very high load had been noticed between the end of 1984 and the beginning year 1985. This is proved in Figure 3 showing the monthly average values of that period.

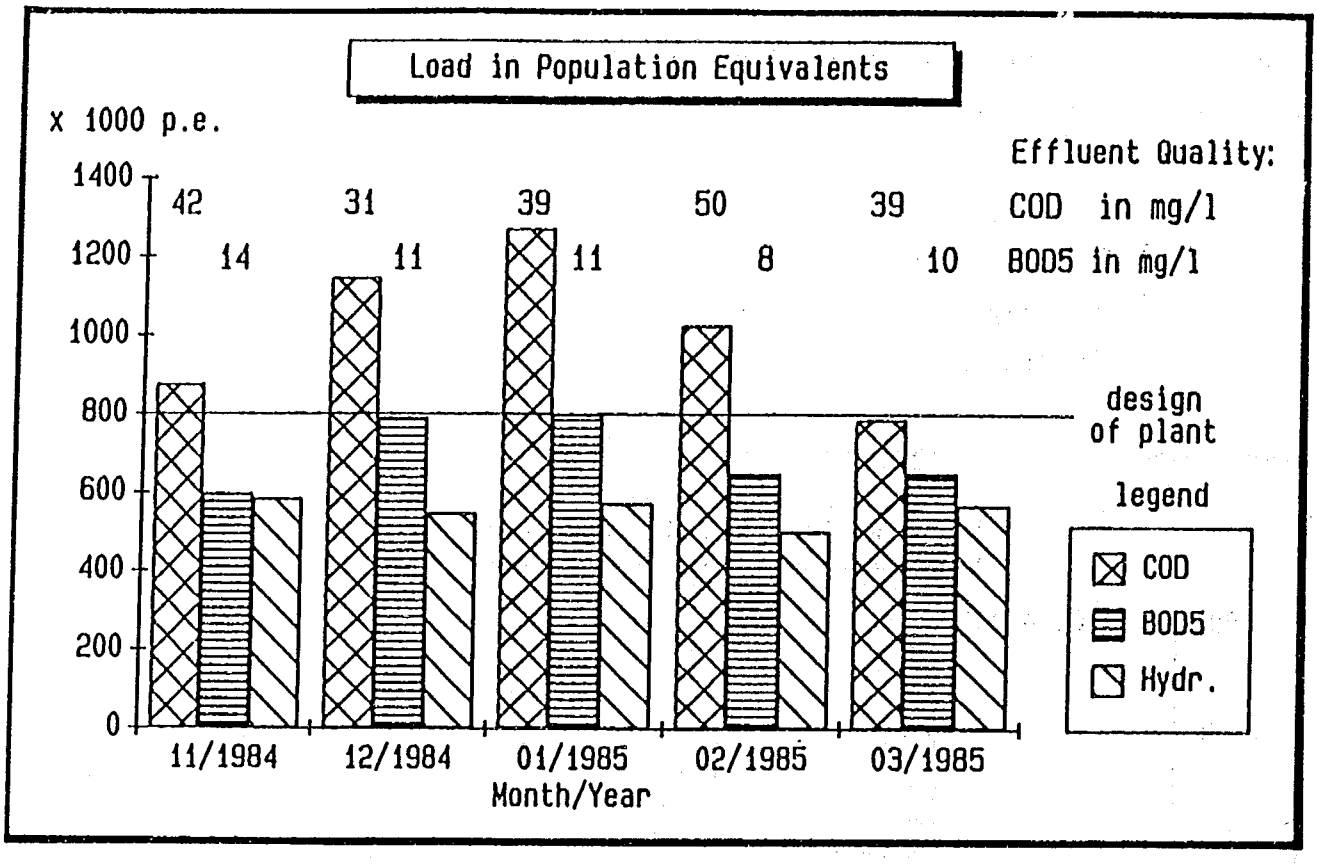


Fig. 3: Average load of the plant in population equivalents concerning COD, BOD_5 and hydraulic load, and average effluent concentrations concerning COD and BOD_5, months 11/1984 to 03/1985

The overload concerning COD ran up to 55 % in january 1985. It has to be considered that during that time only one of the two A-stage aeration tanks was in operation. With respect to this fact the effluent quality concerning BOD_5 and COD also shown in Figure 3 can be regarded as excellent.

Figure 4 presents the results of the first six months of 1986 concerning the COD-concentration in the influent (homogenized), in the effluent of the intermediate clarifier (homogenized) and in the effluent of the final clarifier (sedimentated).

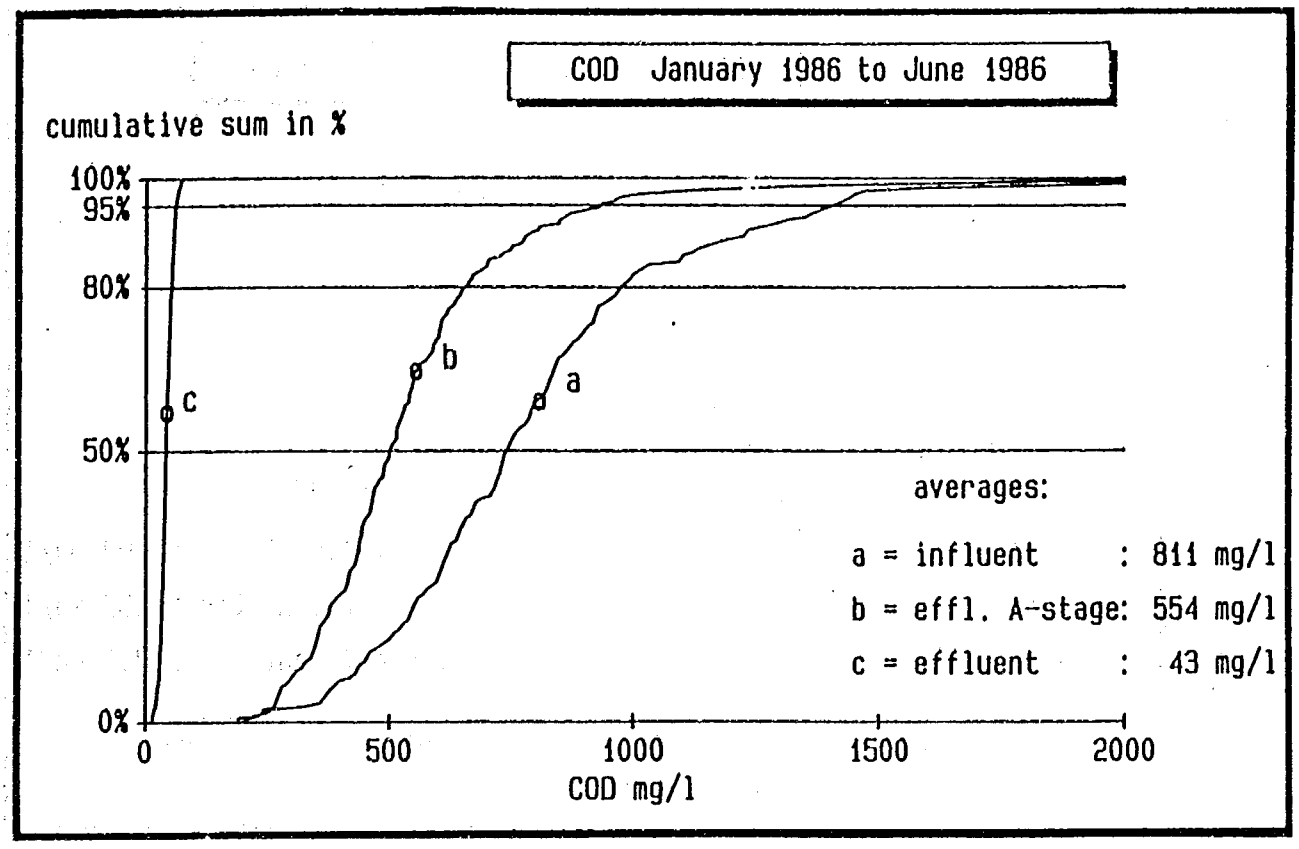


Fig. 4: Cumulative frequencies of the COD-concentration in the influent (hom.), in the effluent of the intermediate clarifier (hom.) and in the effluent of the final clarifier (sed.) from 01/86 to 06/86

Figure 5 shows the results of the same period concerning the BOD_5 in the influent (homogenized), in the effluent of the intermediate clarifier (sedimentated) and in the effluent of the final clarifier (sedimentated).

On the contrary to the past years in this period the efficiency of the A-stage concerning BOD_5 was definitely higher than that concerning COD. Investigations on this effect will be performed.

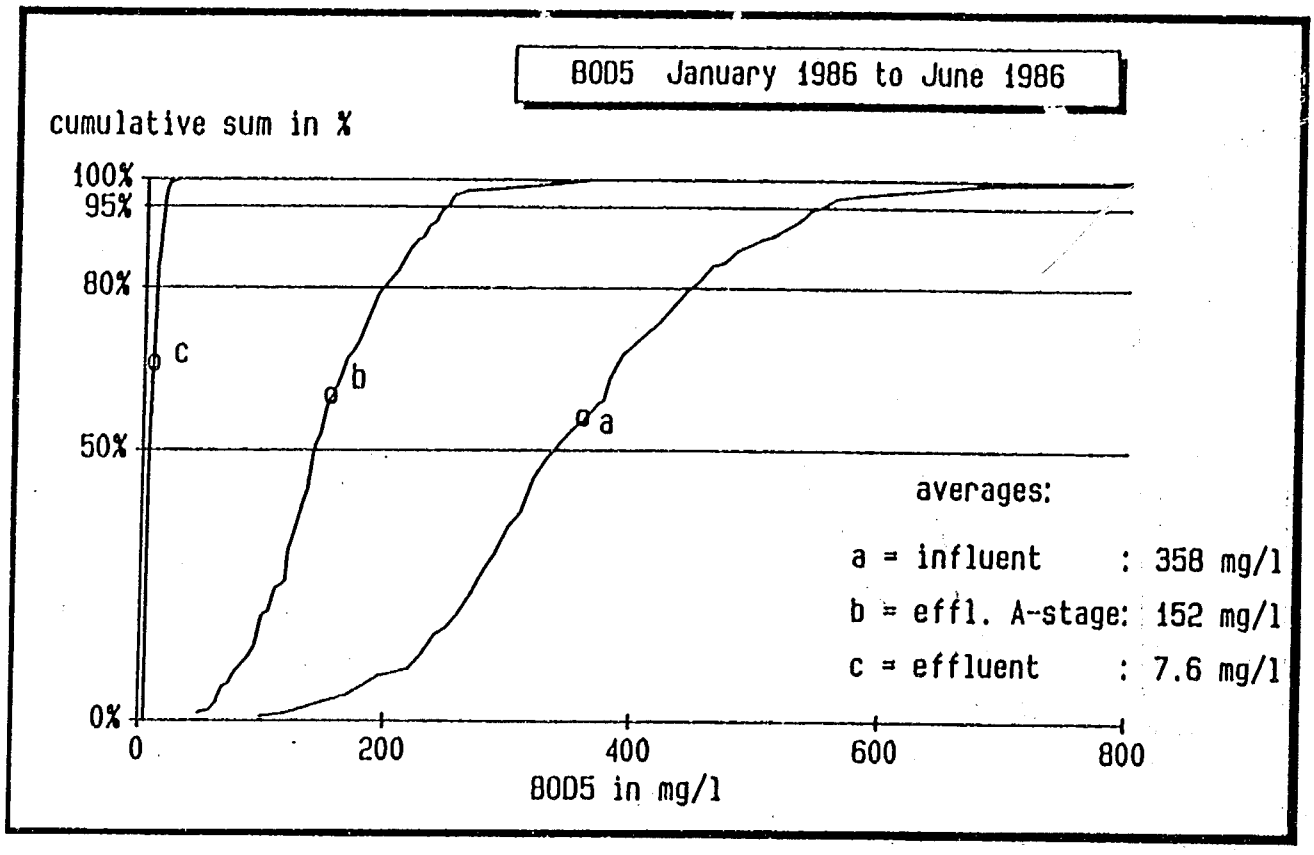


Fig. 5: Cumulative frequencies of the BOD_5-concentration in the influent (hom.), in the effluent of the intermediate clarifier (sed.) and in the effluent of the final clarifier (sed.) from 01/86 to 06/86

4. Comments on the nitrogen analyses

Unfortunately only the parameters ammonium, nitrite and nitrate are determined from the average and grab samples in the course of the self supervision. Therefore direct conclusions on the development of nitrogen during the wastewater treatment process are not possible.

During 36 days an external laboratory carried out analyses of all nitrogen compounds simultaneously with the self supervision. For that reason it was tried to find a correlation between the content of organic nitrogen and one of the compounds analysed within the self supervision. As was expected, no correlation was found between organic nitrogen and the nitrite- and nitrate-concentration. Correlation with ammonium gave the following results:

$N_{org.}$ to NH_4^+-N, influent	: weak correlation
$N_{org.}$ to NH_4^+-N, eff. intermediate clarifier	: good correlation
$N_{org.}$ to NH_4^+-N, eff. final clarifier	: no correlation

Consequently the content of organic nitrogen in the influent and in the effluent of the intermediate clarifier was computed according to the results of the correlation. In the effluent of the final clarifier the content of $N_{org.}$ was constantly estimated to be 2.26 mgN/l as no correlation to the results of measurement could be found.

It should be noticed:

1. This estimation is <u>not</u> done to create real data where no data exist. It is an attempt to <u>estimate</u> the course of the $N_{org.}$ and $N_{tot.}$ concentrations during the sewage treatment process <u>fairly realistical</u>.

2. In the course of the following text estimated values are clearly marked.

5. Conversion of the nitrogen compounds in the course of the biological sewage treatment

Many publications already deal with the possible paths of nitrogen compounds in the course of a biological sewage treatment. PAINTER /3/ gives a good summary and BÖHNKE and PINNEKAMP /1/ have compiled the existing studies. Hence on this occasion only some fundamental remarks should be made.

Figure 6 shows the most important paths of nitrogen in a biological stage. The biological stage comprises aeration tank and settling tank.

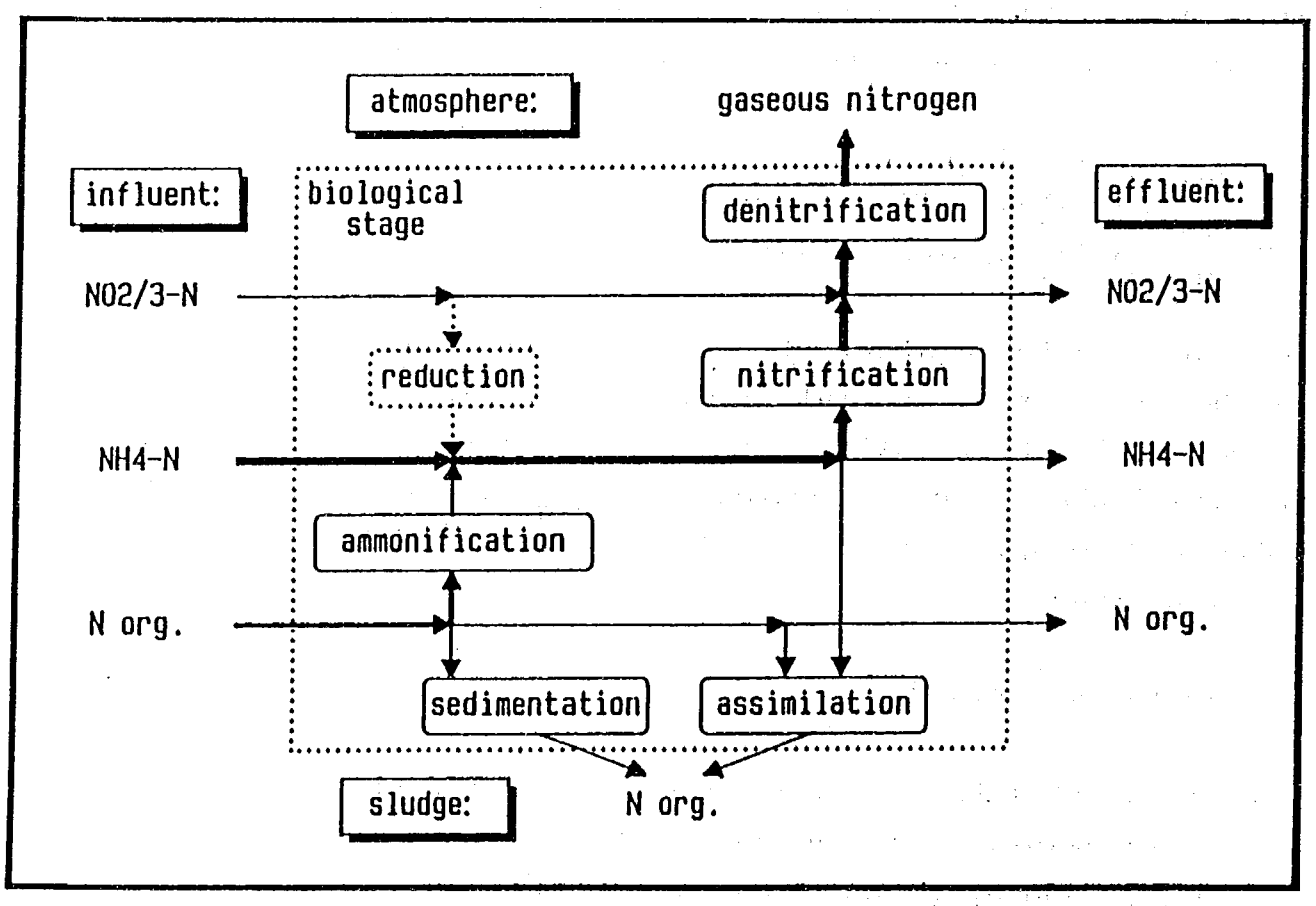


Fig. 6: Conversion and remains of nitrogen compounds in a biologicl stage

Undoubtedly not all paths are shown here which are theoretical possible. Also the relevance of the paths listed here concerning

their quantity is different. Some of the processes could already be confirmed by biological studies carried out in the laboratory of the ISWW with the sludges of the sewage treatment plant of Krefeld.

Table 1 ennumerates the paths resulting in an elimination of nitrogen.

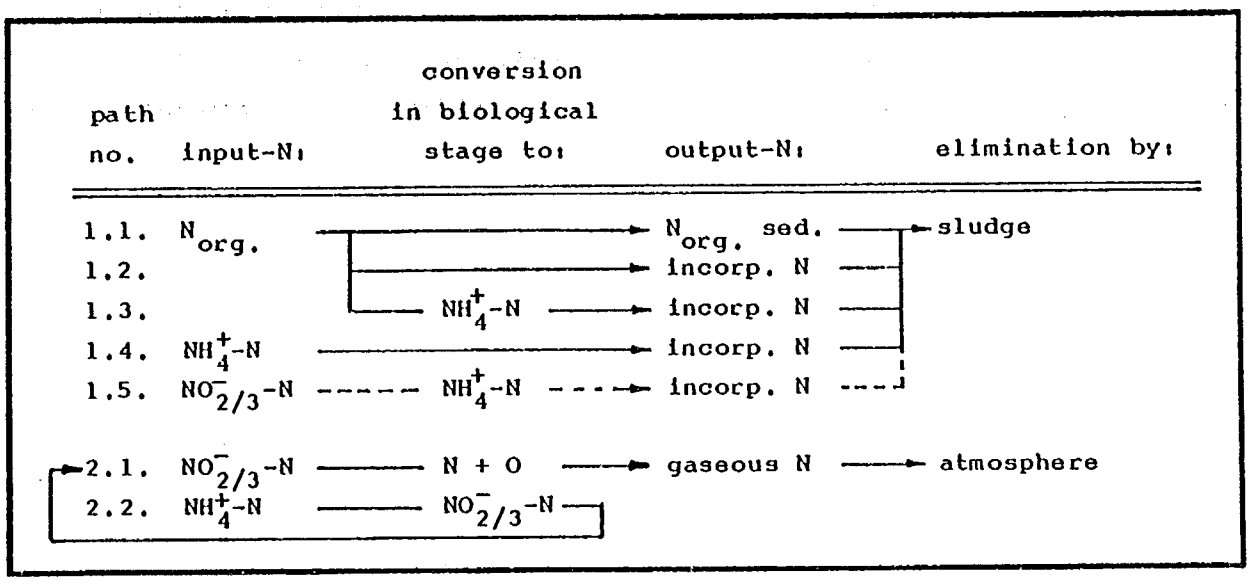


Table 1: Paths to a nitrogen elimination at the biological sewage treatment

A quantitative removal of $N_{org.}$ with the sludge by sedimentation is only possible in the first clarifier. Thus, in a biological stage it is only possible if the clarifier belonging to this stage is the first clarifier in the plant.

Surely the most interesting point is which kinds of nitrogen can be incorporated into cellular material. According to SCHLEGEL /6/ and PÖPEL /5/ only NH_4^+-N can be incorporated in cells. A quantification of this processes seems not yet possible. With regard to quantity, path 1.5. certainly plays a minor role.

E.g., KAPP /2/ quotes the nitrogen portion removed with the sludge to be

- 9 % of the organic matter of the biological excess sludge
- 5 % of the organic matter of the primary sludge.

Path 2.2. in table 1, nitrification, does not lead directly to a nitrogen elimination. Only with a subsequent denitrification, path 2.1., nitrogen is eliminated. If the wastewater contains nitrate not formed by nitrification, nitrogen can be removed through denitrification only. Naturally a nitrification can only be achieved by proper conditions, as especially an sufficient sludge age.

6. Reaction of the A-stage concerning nitrogen compounds

Municipal waste water normally contains only small amounts of nitrate. Because of the industries the sewage water in Krefeld includes a considerable nitrate amount.

Thus, nitrogen elimination in the A-stage is basically possible in all the paths shown in table 1, except for 2.2. + 2.1. due to the short sludge age in the A-stage of about some hours which is not sufficient for nitrification.

Table 2 indicates the fluctuations of the nitrogen parameters in the A-stage in March 1985.

compound	A-stage influent mg/l	A-stage effluent mg/l	decrease mg/l	remarks
NH_4-N	24.24	22.37	1.87	measured
NO_3-N	8.46	1.33	7.13	
NO_2-N	1.87	0.21	1.66	
$N_{org.}$	22.26	14.77	7.49	estimated
tot. N	56.83	38.68	18.15	

Table 2: Change of the nitrogen concentrations in the A-stage, average values of the 24-h-average samples, March 1985

The result is typical for the A-stage. Only the results of the daily 24-h-average samples and the values $N_{org.}$ and $N_{tot.}$, estimated from these samples, were used.

According to the table the elimination of total nitrogen amounts to about 18 mg N/l. Supposing that the nitrogen portion in the excess sludge of the A-stage is about 5 % of the organic matter,

the quantity reduced with the sludge runs up to about 12.74 mg N/l. This means a denitrification of about 5.4 mg N/l.

Hence follows a denitrification rate of

$$v_{DN} = 5.0 \text{ g } NO_3^-\text{-N/(kg MLVSS.h)}$$

This has to be considered as an estimation. The high denitrification rate is not astonishing, since raw sewage is the substrate.

The next figure shows the nitrate concentrations in influent and effluent of the A-stage in March 1985. The results presented are from the grab samples.

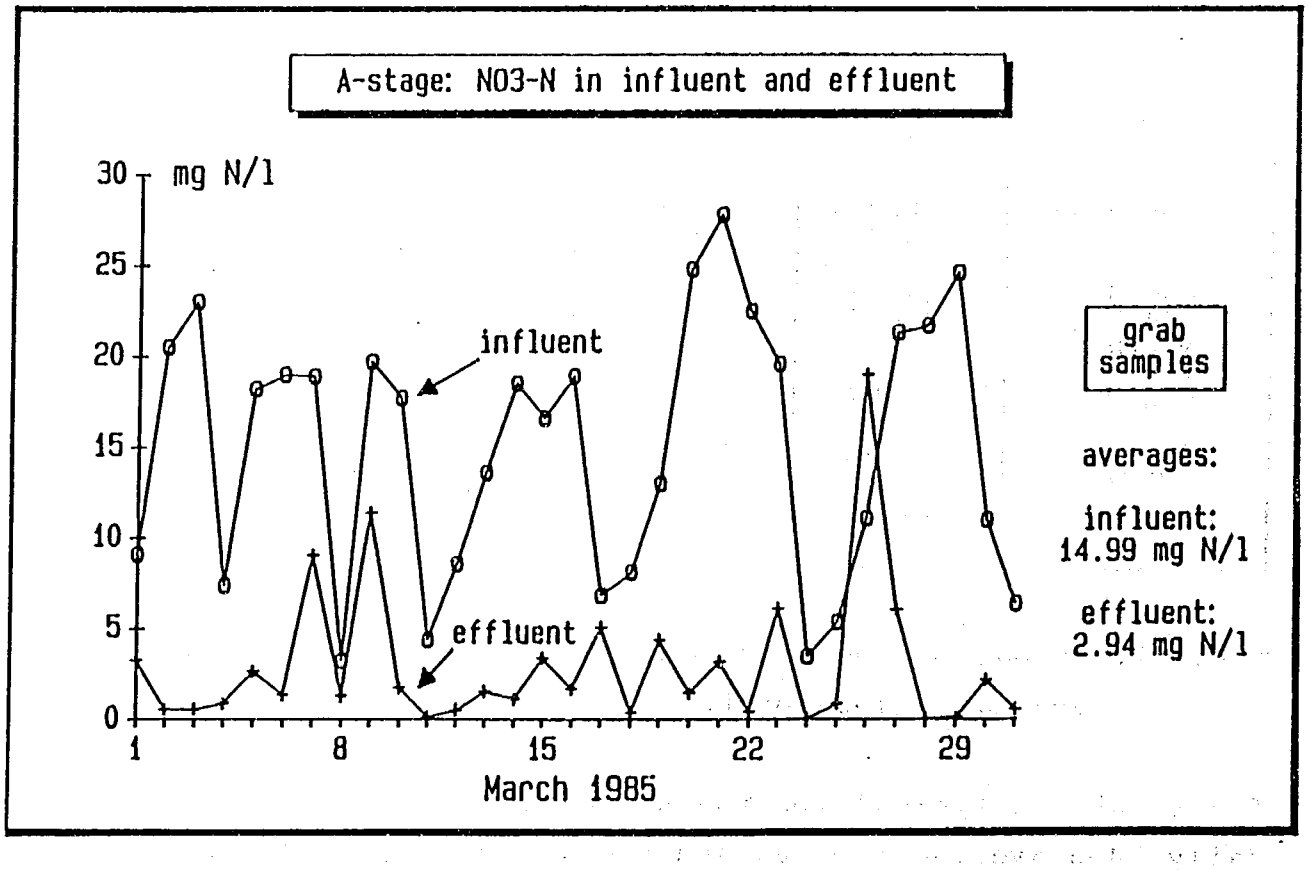


Fig. 7: NO_3^--N-concentrations in influent and effluent of the A-stage, grab samples, March 1985

This figure reveals the evident nitrate reduction in the A-stage obviously mainly caused by denitrification. By way of calculation on the base of these grab samples the nitrate-elimination rate would amount to about 11 g NO_3^--N/(kg MLVSS.h).

Regarding only the 24-h-samples, the elimination of the first stage concerning anorganic nitrogen runs up to 30% and concerning total nitrogen the elimination rate can be estimated to be 32%.

7. Reaction of the B-stage concerning nitrogen compounds

Regarding table 1 the nitrogen elimination in the B-stage is mainly possible with the paths

1.2.	incorporation of $N_{org.}$ in the sludge
1.3.	incorporation of NH_4^+-N in the sludge
2.1.	denitrification of the inflowing $NO_{2/3}^-$-N
2.2. + 2.1.	nitrification and denitrification of NH_4^+-N.

A nitrite/nitrate reduction to ammonium, with or without incorporation of the formed ammonium nitrogen, as occasionally observed at extremly low loaded plants, can be excluded in this case.

It has already been mentioned that the plant was not operated especially for nitrogen elimination. Also the difficulties mentioned above in chapter 2. had negative effects on the efficiency concerning the nitrogen elimination. From this it is obvious that a good nitrogen elimination is not reached at every time, especially due to a missing optimization in plant operation.

But here the question should be answered if the AB-technology basically admits a good nitrogen elimination. In order to answer this question, the time between 18.5.84 and 27.6.84, a period of nearly seven weeks, is considered here in detail. During that period the average load of the B-stage was:

BOD_5-	sludge load :	0,20	kg/(kg MLSS.d)
BOD_5- org.	sludge load :	0.27	kg/(kg MLVSS.d)
NH_4-N-	sludge load :	0.025	kg/(kg MLSS.d)
NH_4-N-org.	sludge load :	0.033	kg/(kg MLVSS.d).

Figure 8 shows the results of the 24-h-average samples in influent and effluent of the B-stage concerning NH_4^+-N in that period.

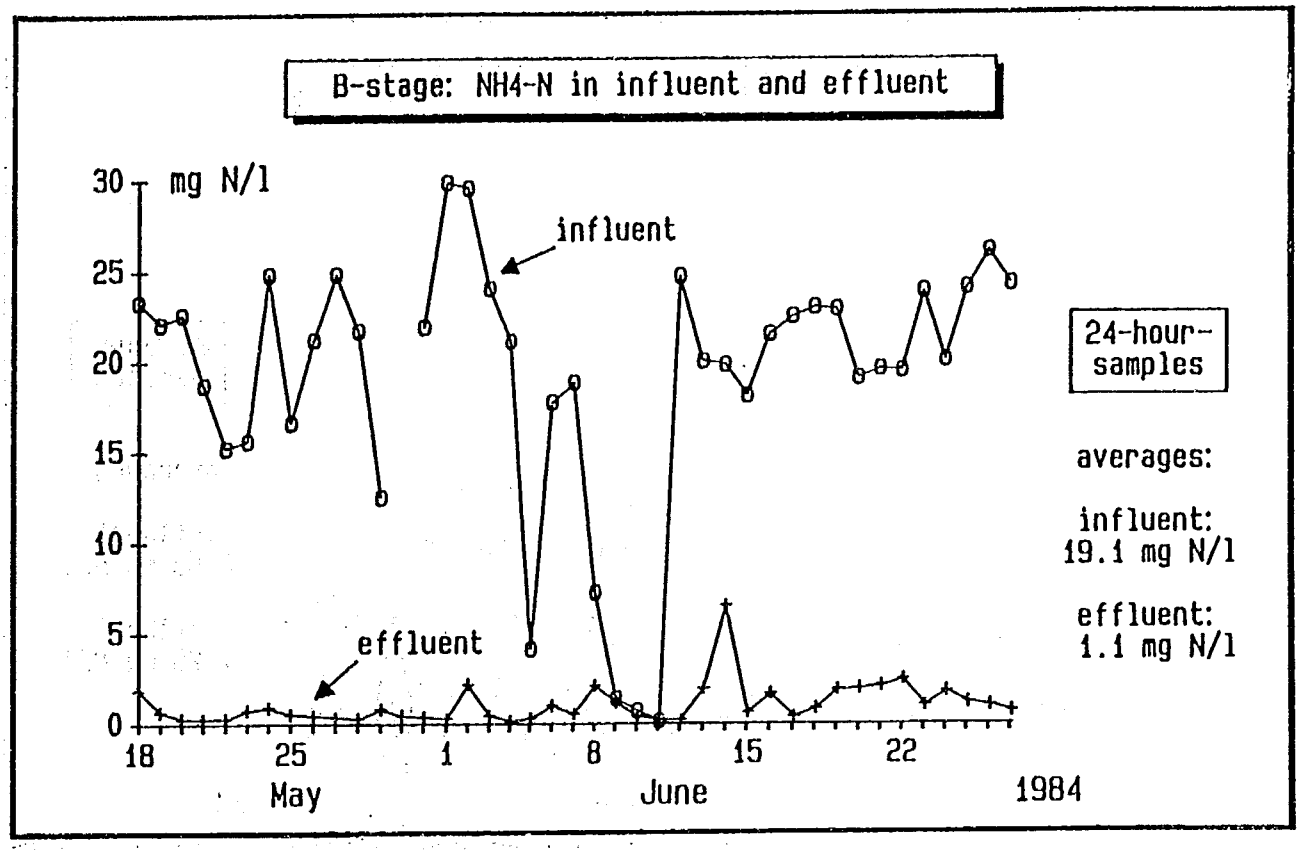


Fig. 8: NH_4^+-N concentrations in influent and effluent of the B-stage, 24-h average samples, 18.5.84 - 27.6.84

As figure 8 illustrates, a far-reaching steady nitrification took place during the whole period.

The appropriate nitrate concentrations can be seen in Figure 9.

Figure 9 denotes that the nitrate concentrations in the effluent of the B-stage were on an average 4.7 mg NO_3^--N/l higher than in the influent. The figures prove that in the B-stage a denitrification had taken place simultaneously, although it was not complete.

Considering as well the organic nitrogen concentrations estimated as quoted, the average values shown in table 3 have relevance in this period.

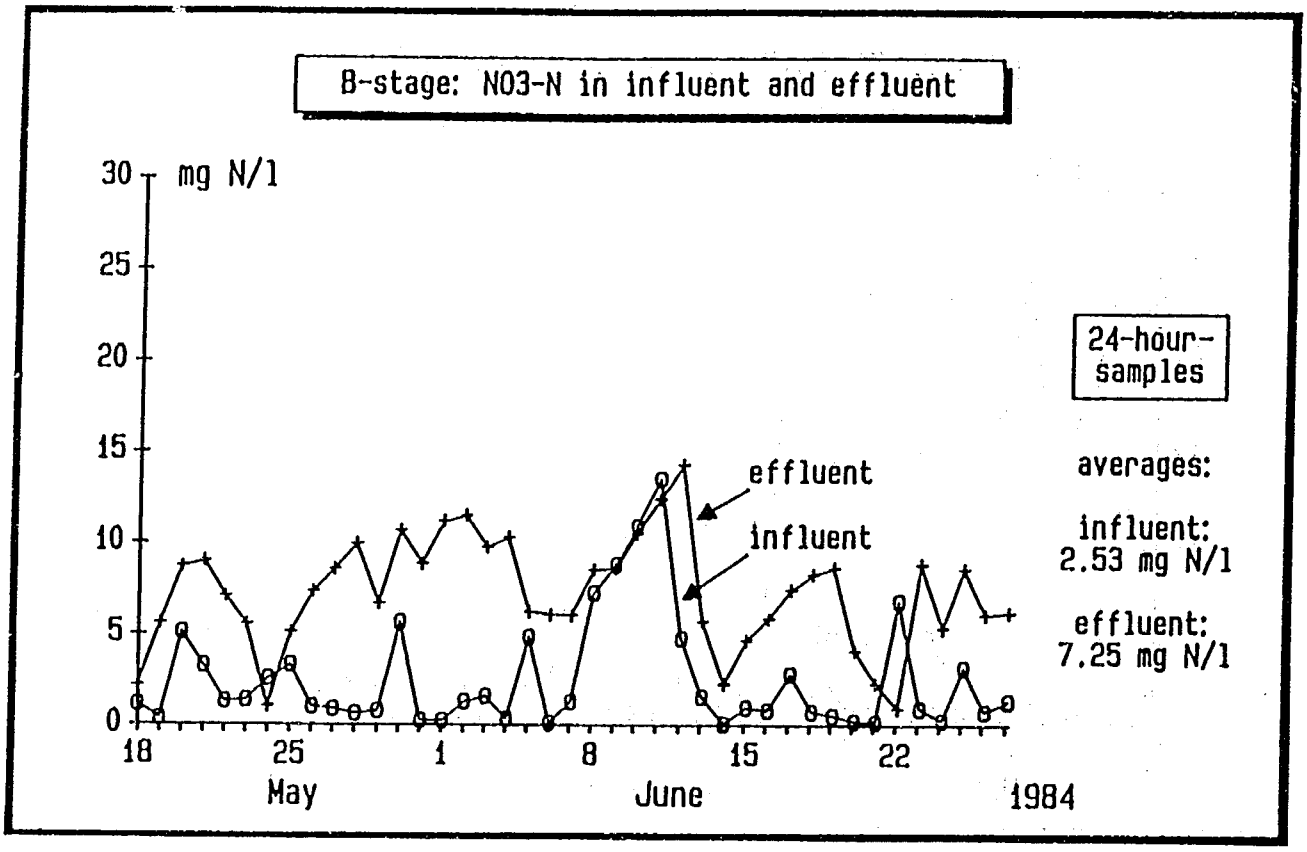


Fig. 9: NO_3^--N concentrations in influent and effluent of the B-stage, 24-h average samples, 18.5.84 - 27.6.84

compound	B-stage influent mgN/l	B-stage effluent mgN/l	remarks
NH_4^+-N	19.1	1.07	measured
NO_3^--N	2.53	7.25	
NO_2^--N	0.45	0.80	
$N_{org.}$	12.9	2.26	estimated
tot. N	35.0	11.4	

Tab. 3: Average values of the nitrogen parameters in influent and effluent of the B-stage, 24-h average samples, measured respectively estimated, 18.5.84 - 27.6.84

During this period the BOD_5 concentration in the B-stage was reduced from an average of 160 mg/l to 10 mg/l. Proceeding on the assumption that in the excess sludge the nitrogen portion is-9% of the organic dried matter content, on an average 5.35 mg N/l were removed with the excess sludge.

With regard to the measured and estimated values shown some conclusions about the processes in the B-stage can be drawn. This conclusions are theoretical as some values have only been estimated. But as these estimations seem to be rather realistic the conclusions shoud be presented as at least a basic approach for further discussion.

As table 1 illustrates the organic nitrogen reduced in the B-stage can only be incorporated in the sludge or transformed to ammonium nitrogen. Hence the increase of NH_4^+-N can be calculated to

$$\begin{aligned} \Delta C_{NH4-N} &= C_{Norg,inf} - C_{Norg,eff} - C_{N,excess\ sludge} \\ &= 12.9 - 2.26 - 5.35 \\ &= 5.29 \text{ mg N/l.} \end{aligned}$$

This effect of an internal increase of the ammonium concentration can usually not be observed because mostly the portion of the incorporated nitrogen is higher than the content of organic nitrogen. Probably an increase is only possible with high concentrations of organic nitrogen or within plants with a low absolute excess sludge production such as second biological stages.

In the case discussed here we have an internal ammonium increase of about 28% from 19.1 to about 24.4 mg NH_4^+-N/l. The nitrification efficiency rate which must consider the internal ammonium increase can hence be calculated to about 96 %.

From this follows that the concentration of oxidized and therefore denitrificationable nitrogen calculates to

$$C_{NO3-N} = C_{NO3-N,inf} + C_{NH4-N,inf} + \Delta C_{NH4-N} - C_{NH4-N,eff}$$
$$= 2.53 + 19.1 + 5.29 - 1.07$$
$$= 25.85 \text{ mg } NO_3^- \text{-N/l}$$

The denitrification in the B-stage then can be calculated as

$$C_{N,den} = C_{NO3-N} - C_{NO3-N,eff}$$
$$= 25.85 - 7.25$$
$$= 18.6 \text{ mg } NO_3^- \text{-N/l}$$

So the efficiency of the simultaneously proceeding denitrification can be quoted to about 72 %.

The nitrification- and denitrification-rate in the B-stage can only be determined if the volume of the aerobic and anoxic part of the tanks is known. As there are oxidation ditches with centrifugal aerators, the system is a plug-flow reactor concerning the oxygen concentration but with a high inner recirculation rate of about 10. Since the effluent still contains nitrate it can be assumed that the tanks have no anaerobic but only an aerobic and an anoxic part.

The volume of both parts can be estimated by the oxygen respiration. A lot of data and calculation formula are known about how many kg oxygen are necessary for a respiration of 1 kg BOD_5. For example, KAPP /2/ mentions a factor of 1 kg O_2/kg BOD_5 and PÖPEL /4/ one of 1.7 kg O_2/kg BOD_5. The calculation according to the ATV-Handbook /7, pp. 395/ as the common design guideline in Germany should be used here as an example. The calculation will be done relating to concentrations. In the period mentioned here, the relevant average values of the B-stage were 2.93 g MLSS/l, 7.4 hours hydraulic retention time, 150 mg/l BOD_5-respiration and 15.7 ^oC water temperature. From this, the total oxygen demand for BOD_5-respiration, endogenous respiration and nitrification can be

calculated as

$$OD = 0.5 \times C_{BOD5,el.} + 0.11 \times MLSS \times \frac{1}{HRT} + 4.6 \times C_{NH4\text{-}N,nitr.}$$

$$= 0.5 \times 150 + 0.11 \times 2.93 \times \frac{1}{7.4} + 4.6 \times 23.3$$

$$= 226 \text{ mg } O_2/l.$$

18.6 mg NO_3^--N/l have been denitrificated and 1 mg NO_3^--N makes 2.9 mg O_2 available for respiration. Thus, denitrification satisfied an oxygen demand of

$$OD_{den} = 2.9 \times C_{N,den}$$
$$= 2.9 \times 18.6$$
$$= 54 \text{ mg } O_2/l$$

Accordingly the division of the tanks into an aerobic and anoxic part can be estimated to

$$\frac{OD - OD_{den}}{OD_{den}} \times \alpha = \frac{226 - 54}{54} \times 0.75 = \frac{2.39}{1} = \frac{70}{30}$$

The factor α is introduced because the use of NO_3^--N - oxygen proceeds slower than that of dissolved oxygen. In publications an average value of 75% can be found so α has the value 0,75. With the data of the period of seven weeks considered here the division results to be 70% aerobic and 30% anoxic. Considering an average dried organic matter content of 2.24 g MLVSS/l, the nitrification rate and the denitrification rate can be calculated as

$$\text{nitrification rate: } \frac{23.3}{2.24 \times 0.70 \times 7.4} = 2.01 \text{ g } NH_4^+\text{-N/(kg MLVSS.h)}$$

$$\text{denitrification rate: } \frac{18.6}{2.24 \times 0.30 \times 7.4} = 3.74 \text{ g } NO_3^-\text{-N/(kg MLVSS.h)}$$

The aerobic sludge age that is relevant for nitrification can be calculated to 9.7 days.

The estimated total nitrogen content in influent and effluent of the B-stage is shown in figure 10.

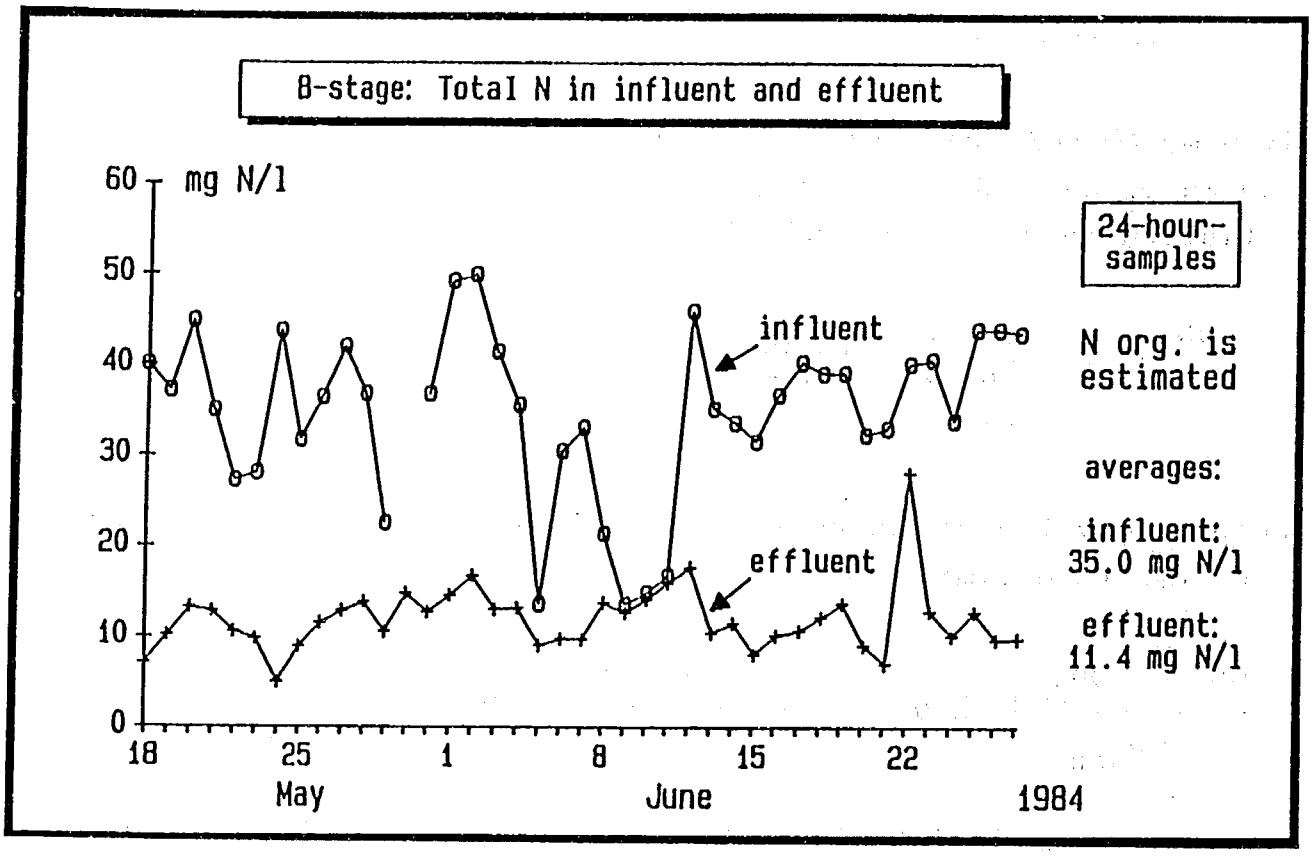


Fig. 10: total nitrogen concentrations in influent and effluent of the B-stage, estimated values of the 24-h-average samples, 18.5.84 - 27.6.84.

The efficiency of the nitrogen elimination runs up to 67% in the B-stage and 77% in the plant in total.

With respect to the circumstances this surely can be considered as a good elimination rate.

8. Conclusions

It had been accentuated that part of the presented values had been only estimated through missing measurement.

In addition

- the operation of the plant was not optimized for nitrification and denitrification and
- some technical problems, e.g. with the aeration capacity of the aerators in the B-stage, emerged.

Nevertheless it seems to be possible to draw the following conclusions from the preceeding presentation.

1. By the great amount of excess sludge in the A-stage already a decisive elimination of nitrogen can be achieved here.

2. In the A-stage, an extensive denitrification is possible in case of high nitrate concentrations in the influent and anoxic operation of the A-stage. The denitrification rate is high.

3. Since the absolute amount of excess sludge in the B-stage is smaller than in an one-stage-plant showing the same sludge age, an internal increase of ammonium is more likely, provided that the organic nitrogen concentration exceeds a certain value. This internal ammonium increase has to be considered when calculating the nitrification and denitrification efficiency.

4. Presumably the nitrification rate calculated in the period from 18.5.84 to 27.6.84 could have been higher. This conclusion can be drawn from the throughout low effluent ammonium concentrations that suggest the ammonium-substrate supply to be the limiting factor.

5. The denitrification rate calculated in the period from 18.5.84 to 27.6.84 is quite high. Since the effluent of the B-stage always contained nitrate, the "substrate" supply certainly was not the limiting factor.

6. The MLSS content determined in the period from 18.5.84 to 27.6.84 was rather low. Presumably a higher MLSS content as well as an enlargement of the anoxic part of the B-stage could have caused better results concerning denitrification. Increasing the MLSS content leads to a higher sludge age and therefore increases the nitrification capacity, thus a reduction of the aerobic part of the tank will not affect nitrification. Besides, the nitrification capacity was not exhausted. The conclusions of BÖHNKE and PINNEKAMP /1/ suggest that the size of the aerobic and the anoxic part of the B-stage should be just the other way than estimated here in order to achieve a good nitrification as well as a far-reaching denitrification. The results presented here confirm the conclusions of BÖHNKE and PINNEKAMP.

Finally, irrespective of the restrictions mentioned above, it can be established that with the AB-process it is possible to obtain an extensive nitrification and denitrification.

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DEMONSTRATION EXPERIMENT ON NITRIFICATION-DENITRIFICATION PROCESS USING ENDOGENOUS RESPIRATION FOR DENITRIFICATION

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ABSTRACT

AT the Kotoh wastewater treatment ptant in Hamamatsu City, a demonstration was made for the high grade nitrogen removal process of biological nitrification-denitrification using endogeneous respiration.

In this demonstration experiment,more than 80% of the nitrogen removal efficiency was obtained except in the winter time,for a total detention time of 21 hours in the aeration tank. However,in the winter time,anmonium and nitrate nitrogen concentration remained in the effluent to as much as 10 mg/l,which exceeds the recommended effluent standard of 5mg/l T-N.

The required total detention time to overcome this situation was theoretically calculated on the basis of the nitrification and denitrification rate and the recommended modification of this process was discussed.

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1. Nutrient Removal Conditions for Municipal Wastewater Treatment Plants in Japan

In Japan, in 1983, 548 municipal wastewater treatment plants were being operated and about 21 Mil.m^3 of municipal sewage per day was purified by mainly the biological treatment process. As shown in Table 1, most of the wastewater treatment plants apply the activated sludge process as their secondary treatment. Recently, the Environment Agency announced environmental water quality standards for nitrogen and phosphones in lakes and reservoirs, and then amended Water Pollution Control Law in which N and P were newly added as water quality indices of effluent standards. In 1983, nine WTPs discharged their effluents to sweet water lakes. These WTPs are generally claimed to remove P and N from the effluent in order to meet the effluent standards. Table 2 shows the list of WTPs which apply N, P removal processes. As shown above only a limited number of WTPs have N, P removal processes at present.

However, in case that nutrients removal measures are extended to the closed sea areas from the point of excess eutrification control, the number of WTPs which are claimed for nutrients removal should increase greatly, because most of the big cities in Japan are on bays or inland sea areas.

Adding to this, in the case that the WTP effluent is dischanged to an irrigation water course for paddy rice, the removal of nitrogen in the effluent is often required from the point of excess nutrient charge to the paddy fields. The number of these cases is now increasing, because of the many small-sized WPT under construction.

As shown in Table 2, the biological nitrification-denitrification process with nitrified liquor recycle (BNDR) is commonly used for nitrogen removal. This process can be accomplished by slight modification of a usual aeration tank where an anoxic zone is arranged by mixer installation and a mixed liquor recirculation line is added.[1)]

The aeration tank of the BNDR including anoxic tank is designed with a total detention time of 12-16 hours. The nitrogen removal efficiency is

Table 1 Number of wastewater treatment plants for different treatment process in Japan

(1983)

Treatment process	Design flow capacity (1000m³ /day)						
	<5	5-10	10-50	50-100	100-500	500 <	Total
PRIMARY:							
Sedimentation	1	1	2	-	-	-	4
HIGH RATE BIOLOGICAL TREATMENT:							
High rate trickling filter	2	4	5	1	-	-	12
Aero - accelater type	3	-	13	2	1	-	19
FULL BIOLOGICAL TREATMENT:							
Conventional activated sludge	38	44	135	71	96	12	396
Step aeration	4	5	18	18	22	7	74
Extended aeration	12	1	2	-	-	-	15
Contact stabilization	2	-	-	-	-	-	2
Pure oxygen activated sludge	-	1	1	1	2	-	5
Oxidation ditch	6	1	-	-	-	-	7
Rotating biological contactor	7	3	3	1	-	-	14
Total	75	74	167	93	120	19	548

Table 2 List of WTPs where N-P removal process is applied

WTP' s name	Effluent quality (mg /ℓ)	Design flow capacity (m³ /day)	Treatment mode	Operation start year
Biwako R.S. Kosei	BOD 5 COD 10 SS 6 T-N 10 T-P 0.5	500	BNDR Coagulant addition Filtration	1984
Biwako R.S. Konanchubu	BOD 5 COD 10 SS 6 T-N 10 T-P 0.5	14000	BNDR Coagulant addition Filtration	1983
Kasumigaura-Kohoku R.S. Kasumigaura	BOD 6 COD 11 SS 6 T-N 15 T-P 0.5	30000	BNDR Coagulant addition Filtration	1983
Kasumigaura-Suigoh R.S. Itako	T-N 20 T-P 1	5400	BNDR Coagulant addition	1985
Nikkoh City Yumoto	T-N 10 T-P 1	3250	Oxidation ditch Coagulant addition	1985
Hamamatsu City Hitomigaoka	T-N 5 T-P 1	1620	Modified A_2O process Filtration	1983
Hamamatsu City Kotoh	T-N 5 T-P 1	1980	BNDE Coagulant addition Filtration	1982

R.S.:Regional Sewege
BNDR:Biological Nitrification-Denitrification Process with nitrified liquor recycle
BNDE:Biological Nitrification-Denitrification Process using Endogeneous Respiration

in the range of 60-70% and this efficiency is hardly increased because there exists a theoritical maximum removal efficiency (Mn) which is determined by the recirculation ratio (R).

$$Mn=\frac{R}{1+R}$$

$$R=\frac{\text{(Mixed Liquor Recirculation Rate)+(Return Sludge Rate)}}{\text{(Flow Rate)}}$$

Practically however, the value of R has an upper limit of about 2, because nitrification and denitrification rate decreases in the bigger recirculation ratio. Thus in the BNDR, we cannot expect more than 70% of nitrogen removal rate. In Japan, the T-N concentration of the primary effluent is approximately 30 mg/l, so the BNDR cannot be applied in case that the effluent standard of nitrogen is more strict than 10 mg/l.

2. Demonstration Experiments of Biological Nitrification-Denitrification Process using Endogeneous Respiration

2-1 Nutrient Removal in Hamamatsu City

Hamamatsu City, which is located east of Nagoya, is on Lake Hamana. Lake Hamana is a brackish lake and the activity of inshore fisheries is very high. Domestic wastewater originating from the residential area in the catchment basin of Lake Hamana is treated in two municipal WTPs, Hitomigaoka and Kotoh WTPs. The final effluent from both WTPs must satisfy the strict water quality standards shown in Table 3, which were contracted with the fishermen's union in order to prevent the pollution and eutrophication of Lake Hamana.

Both plants were designed with an extended aeration mode of activated sludge process without primary sedimentation tank followed by coagulation tank, separate stage nitrification denitrification tank, rapid sand filter and activated carbon. However, a simpler treatment system is being sought to cut down the maintenance and operation costs.

A demonstration field survey had been conducted in both treatment

Table 3 Recommended effluent standard to Lake Hamana

BOD	5	mg/ℓ
COD	10	mg/ℓ
SS	3	mg/ℓ
T-N	5	mg/ℓ
T-P	1	mg/ℓ

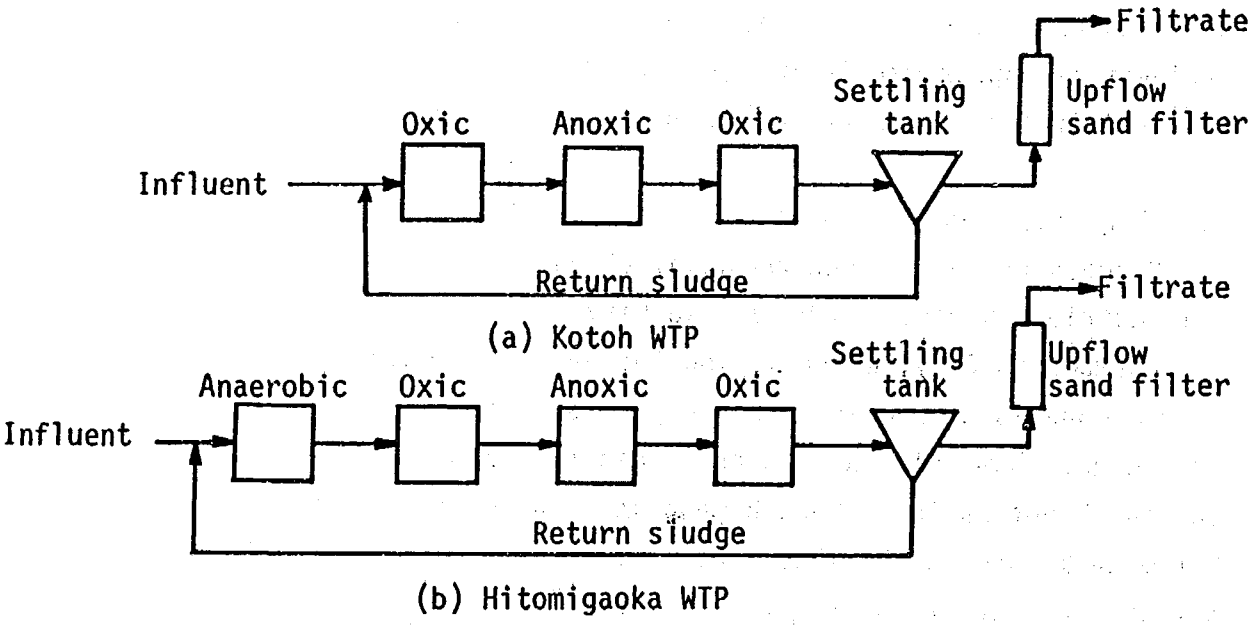


Fig.1 Flow diagrams of Kotoh and Hitomigaoka WTP.

plants where the aeration tanks were modified to the combined carbon oxidation-nitrification-denitrification system as illustraterd in Fig.1. In Hitomigaoka WTP the biological phosphorus removal was also taken in consideration. However, stable biological phosphorus removal has not yet been accomplished by this process. In Kotoh WTP, the phosphorus is removed by adding aluminium sulfate to the end of aeration tank.

2-2 Demonstration Experiment on Nitrogen Removal in Kotoh WTP

As mentioned above the nitrogen removal rate of the nitrified liquor recycled nitrification-denitrification process has a limmit of 60-70% at maximum. Based on this fact, the other biological nitrification-denitrification process using endogeneous denitrification zone was demonstrated in Kotoh WTP to meet the recommended effluent standard of 5 mg/l T-N. The specification of major facilities of Kotoh WTP is shown in Table 4. This plant treats only domestic sewage of 4000 population and average daily flow was about 1100 m^3/day in the year of 1985. The aeration tank is divided to four series where each series has five cells and where two series form one unit line which is combined to the same final sedimentation tank. This activated sludge treatment system is classified as the extended aeration mode and coarse bubble diffuser is installed in this aeration tank. In this plant the diurnal inflow variation is equalized by incorporating a flow equalization tank.

The demonstration experiment using endogenous respiration as denitrification has been carried out since January 1985. At the first stage, two lines of aeration tanks with a detention time of about 40 hours were used for this experiment, but they had some problem in distributing return sludge to two lines and could not have enough stable operation condition (Run 1). So from June 1985, it was decided to use only one line of aeration tank with two anoxic cell for each series (Run 2). In the time it was observed that the anmonium nitrogen concentration increased remarkably probably by shortening the aeration time. Therefore after two weeks operation, the number of oxic cells were increased reducing the number of anoxic cells from two to one. Changing this mode the operation condition became rather stable and the investigation on combined carbon

Table 4 Specification of major facilities and design flow of Kotoh WTP

Facility	Specification
GRIT CHAMBER	W 0.6m × L 4.0m × D 0.3m -2
FLOW EQUALIZATION TANK	W 2.25m × L 10.0m × D 4.1m -2 W 2.25m × L 7.3m × D 4.0m -6 effective volume 580 m^3
BIOLOGICAL REACTOR	W 4.8m × L 25.0m × D 4.0m -4 480m^3 /basin, 96 m^3 /cell
SETTLING TANK	ϕ9.0m × D 2.5m -2 effective volume 160 m^3 /basin surface area 64m^2 /basin
UPFLOW SAND FILTER	filter surface area 5.8m^2 /basin -3 sand 2-3mm 8.1 m^3 8-12mm 1.1 m^3 40-50mm 1.4 m^3 depth 1.90m
DESIGN DAILY MAX. FLOW	1980 m^3 /day
(PRESENT DAILY FLOW)	(1090 m^3 /day)
PLANNED POPURATION	4300 persons
(PRESENT POPURATION)	(3440 persons)

Table 5 Experimental conditions of Kotoh WTP

Parameter	RUN−1	RUN−2	RUN−3
PERIOD	1985.4.1-6.3	1985.6.4-6.19	6.20-1986.3.14
FLOW RATE (m^3 /day)	1064	1131	1080
RETURN RATIO (%)	113	85	81
DETENTION TIME OF A.T.(hrs)	43.3	20.4	21.3
SETTLING TIME (hrs)	7.2	3.4	3.6
AERATION/FLOW RATE	16.4	13.4	14.9
WASTE SLUDGE (m^3 /day)	20.5	16.3	18.8
RSSS (mg/ℓ)	6810	5940	6710
MLSS (mg/ℓ)	4000	2700	3300
MLVSS (mg/ℓ)	3130	2070	2630
MLVSS/MLSS (%)	78.3	76.8	79.7
SVI	238	304	255
SRT (day)	46.8	19.9	23.0
Al ADDED (mg/ℓ)	1.9	1.8	1.9

oxidation-nitrification-denitrification without nitrified liquor recycle was carried out with this condition for nine months (Run 3). Fig. 2 illustrates the treatment flow of each run and the detailed experimental conditions are shown in Table 5.

2-3 The Result of Demonstration Experiment

Fig. 3 shows the T-N removal situation throughout the experimantal period. In the period of warm water temperature, the T-N removal rate was as high as 80-90%, and the effluent from final sedimentation tank remained in the vicinify of 5 mg/l except in the period of Run-2. But in the winter time the nitrogen removal condition deteriorated because of the residue of nitrate and ammonium nitrogen. It is pointed out that the nitrification and denitrification rate would decrease below the temperature of 15°C.

In this experimental period, a more detailed investigation was carried out four times per year by taking the water samples along the aeration tank every two hours for 24 hours. The measurement result of this detailed investigation is summurized in Table 6,7,8,9, where the water quality of nitrogen, phosphorus and alkalinity in each cell of aeration tank was measured with the filtrate sample of mixed liquor. It is considered that Table 6 shows the representative result in Run 1. The nitrogen behavior corresponding to the data of Table 6 is illustrated in Fig. 4. In this case it is clearly observed that the anmonium nitrogen is oxidized to nitrate in oxic cell A,B and that the nitrate is removed in the anoxic cell C,D by denitrification. Also, the alkalinity is consumed in oxic cells and is produced in anoxic cells though the pH values are almost constant at 6.8 - 6.9. It is considered that typical nitrification and denitrification occured along the series of cells of the aeration tank.

Table 7 and Table 8 show the operation data in the higher temperature and middle temperature period in Run 3 respectively and the corresponding nitrogen data are shown in Fig. 5 and Fig. 6. Though the detention time of aeration tank calculated by inlow base was as short as 21 hours in Run 3, the total nitrogen was removed to 5 mg/l. This operation result was worth to that of Run 1 when the detention time had been deoubled to 43

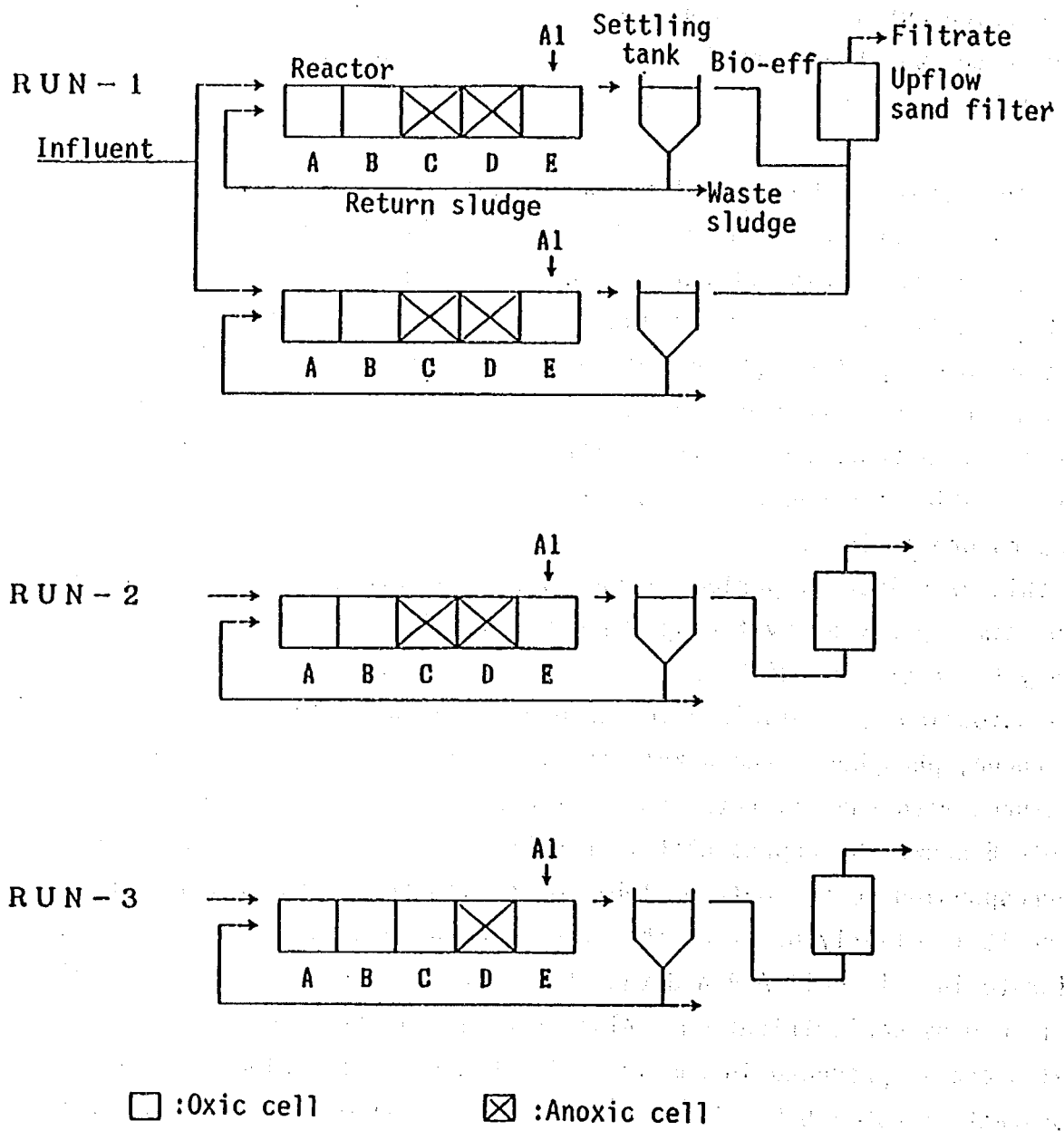


Fig.2 Flow diagrams of Kotoh WTP for each experiment.

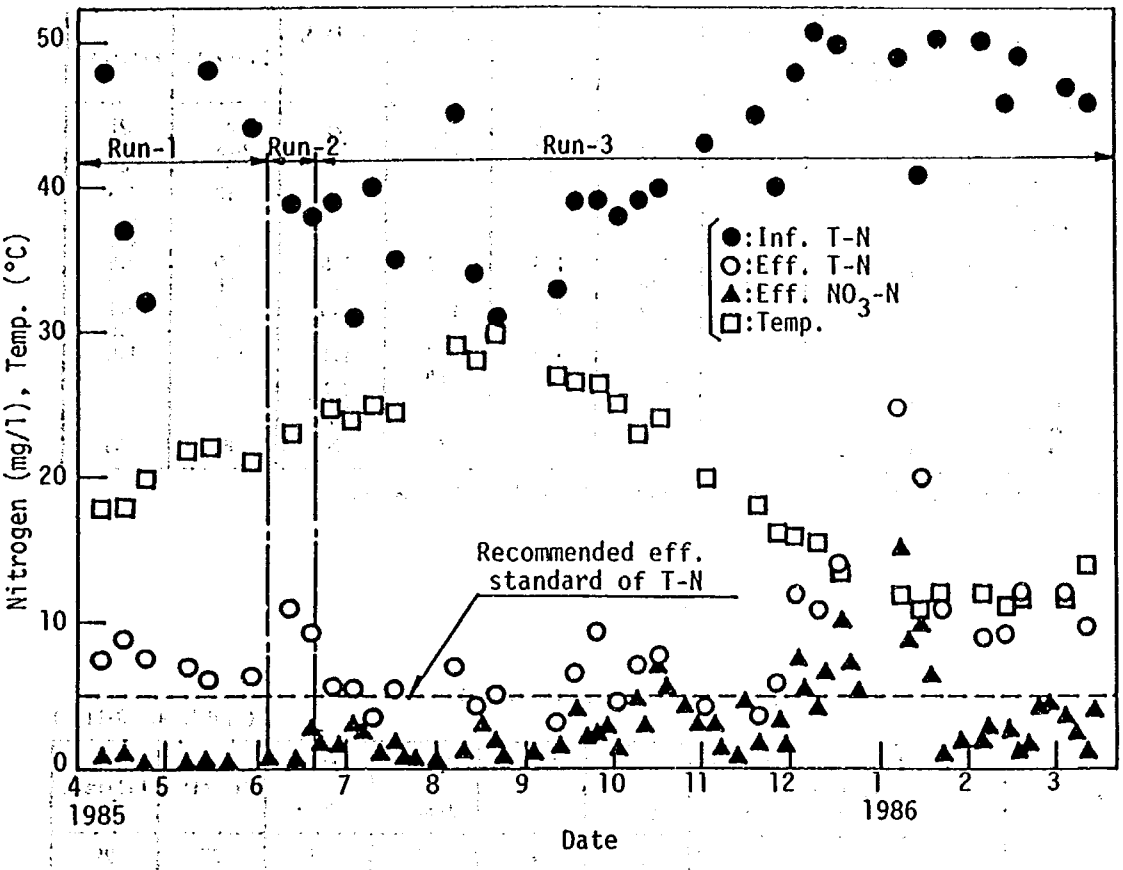


Fig.3 Annual variations of nitrogen and water temperature at Kotoh WTP.

Table 6 Data of water quality in Run-1

(1985. 5. 28-29)

Parameter		Inf.	Inf.+ Return Sludge	Cell A Oxic	Cell B Oxic	Cell C Anoxic	Cell D Anoxic	Cell E Oxic	Bio-eff.	Filtrate
Temp	℃	21	-	21	21	21	21	21	21	21
pH		7.1	-	6.9	6.8	6.8	6.8	6.5	6.8	6.9
SS	mg/ℓ	140	-	-	-	-	-	-	38	9
COD	mg/ℓ	55	32.3	-	-	-	-	-	13	7.5
BOD	mg/ℓ	250	125	-	-	-	-	-	19	4.6
T-N	mg/ℓ	38	20.0	-	-	-	-	-	4.7	4.2
NH_4^+ -N	mg/ℓ	19	9.8	5.0	1.0	1.5	1.8	1.5	2.0	2.5
NO_2^- -N	mg/ℓ	TR	TR	TR	TR	TR	TR	TR	TR	TR
NO_3^- -N	mg/ℓ	TR	TR	2.3	5.2	2.0	0.3	0.3	TR	0.1
T-P	mg/ℓ	3.8	2.6	-	-	-	-	-	1.6	1.5
PO_4 -P	mg/ℓ	2.2	1.7	0.9	0.8	1.0	1.7	1.1	1.2	1.0
DO	mg/ℓ	-	-	0.4	0.4	ND	ND	0.5	-	-
MLSS	mg/ℓ	-	-	-	-	-	-	4070	-	-
MLVSS	mg/ℓ	-	-	-	-	-	-	2910	-	-
Alkalinity	mg/ℓ	130	93.2	63	27	52	50	50	62	56

FLOW RATE 1017m^3 /day, RETURN RATIO 118%, RSSS 8070 mg/ℓ, WASTE SLUDGE 25m^3 /day

Table 7 Data of water quality in Run-3-1

(1985. 9. 10-11)

Parameter		Inf.	Inf.+ Return Sludge	Cell A Oxic	Cell B Oxic	Cell C Oxic	Cell D Anoxic	Cell E Oxic	Bio-eff.	Filtrate
Temp	℃	27	-	27	27	27	27	27	27	27
pH		7.0	-	6.8	6.7	6.6	6.6	6.6	6.8	6.9
SS	mg/ℓ	55	-	-	-	-	-	-	2	2
COD	mg/ℓ	54	33	-	-	-	-	-	7.8	6.4
BOD	mg/ℓ	190	107	-	-	-	-	-	12	3.9
T-N	mg/ℓ	30	18	-	-	-	-	-	4.3	3.8
NH_4^+ -N	mg/ℓ	11	6.5	2.6	2.0	1.3	2.0	1.8	1.3	1.2
NO_2^- -N	mg/ℓ	ND	0.1	0.1	0.2	0.2	0.3	0.2	0.2	TR
NO_3^- -N	mg/ℓ	TR	0.7	3.0	4.8	7.0	2.8	1.8	1.4	1.2
T-P	mg/ℓ	3.4	2.2	-	-	-	-	-	0.8	0.9
PO_4 -P	mg/ℓ	2.1	1.4	0.9	0.8	0.8	0.8	0.6	0.6	1.1
DO	mg/ℓ	-	-	0.6	0.6	0.5	0.3	0.4	-	-
MLSS	mg/ℓ	-	-	-	-	-	-	2940	-	-
MLVSS	mg/ℓ	-	-	-	-	-	-	2220	-	-
Alkalinity	mg/ℓ	130	93	69	47	30	43	46	50	49

FLOW RATE 1098m^3 /day, RETURN RATIO 87%, RSSS 6330mg/ℓ, WASTE SLUDGE 25 m^3 /day

Table 8 Data of water quality in Run-3-2

(1985. 11. 19-20)

Parameter		Inf.	Inf.+ Return Sludge	Cell A Oxic	Cell B Oxic	Cell C Oxic	Cell D Anoxic	Cell E Oxic	Bio-eff.	Filtrate
Temp	℃	17.5	-	18	18	18	18	18	18	17
pH		7.1	-	6.9	6.6	6.5	6.7	6.7	6.7	6.8
SS	mg/ℓ	140	-	-	-	-	-	-	6	1
COD	mg/ℓ	65	37	-	-	-	-	-	4.9	7.6
BOD	mg/ℓ	240	132	-	-	-	-	-	9.7	1.8
T-N	mg/ℓ	39	23	-	-	-	-	-	4.9	4.9
NH_4^+ -N	mg/ℓ	19	10.5	3.5	TR	0.8	1.7	1.5	0.8	0.6
NO_2^- -N	mg/ℓ	TR	ND	0.1	0.1	0.1	0.1	0.1	0.1	TR
NO_3^- -N	mg/ℓ	0.1	1.0	4.1	4.5	4.1	3.3	2.3	2.0	2.8
T-P	mg/ℓ	3.8	2.3	-	-	-	-	-	0.5	0.6
PO_4 -P	mg/ℓ	1.2	0.8	0.6	0.6	0.5	0.6	0.4	0.4	0.6
DO	mg/ℓ	-	-	0.2	0.1	0.1	ND	ND	-	-
MLSS	mg/ℓ	-	-	-	-	-	-	3110	-	-
MLVSS	mg/ℓ	-	-	-	-	-	-	2410	-	-
Alkalinity	mg/ℓ	140	94	55	31	28	43	43	41	38

FLOW RATE 959m³ /day, RETURN RATIO 88%, RSSS 6280 mg/ℓ, WASTE SLUDGE 25m³ /day

Table 9 Data of water quality in Run-3-3

(1986. 1. 19-20)

Parameter		Inf.	Inf.+ Return Sludge	Cell A Oxic	Cell B Oxic	Cell C Oxic	Cell D Anoxic	Cell E Oxic	Bio-eff.	Filtrate
Temp	℃	12.0	-	12	12	12	12	12	13	12
pH		7.2	-	6.9	6.6	6.6	6.7	6.8	6.8	6.8
SS	mg/ℓ	92	-	-	-	-	-	-	13	4
COD	mg/ℓ	62	40	-	-	-	-	-	12	10
BOD	mg/ℓ	220	143	-	-	-	-	-	42	7.8
T-N	mg/ℓ	47	31	-	-	-	-	-	11	10
NH_4^+ -N	mg/ℓ	21	15.3	11	9.8	9.1	8.5	8.7	7.9	7.0
NO_2^- -N	mg/ℓ	0.1	0.1	0.2	0.2	0.2	0.1	TR	0.1	TR
NO_3^- -N	mg/ℓ	0.1	0.4	4.1	5.0	4.2	2.0	1.1	0.9	2.4
T-P	mg/ℓ	2.1	1.6	-	-	-	-	-	1.0	0.8
PO_4 -P	mg/ℓ	-	-	0.8	0.4	0.3	TR	TR	-	-
DO	mg/ℓ	-	-	0.6	0.4	ND	ND	ND	-	-
MLSS	mg/ℓ	-	-	-	-	-	-	3460	-	-
MLVSS	mg/ℓ	-	-	-	-	-	-	2830	-	-
Alkalinity	mg/ℓ	150	120	95	70	67	83	82	80	77

FLOW RATE 1097m³ /day, RETURN RATIO 77%, RSSS 6840 mg/ℓ, WASTE SLUDGE 25m³ /day

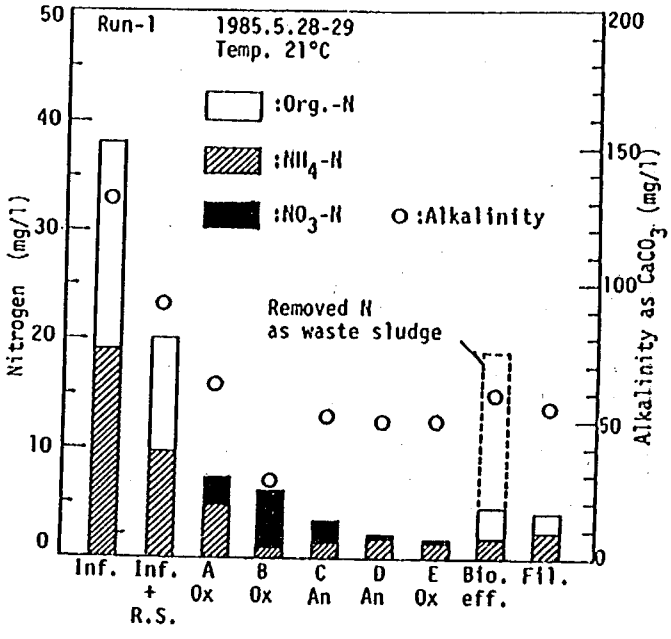


Fig.4 Changes of nitrogen and alkalinity through the process in Run-1.

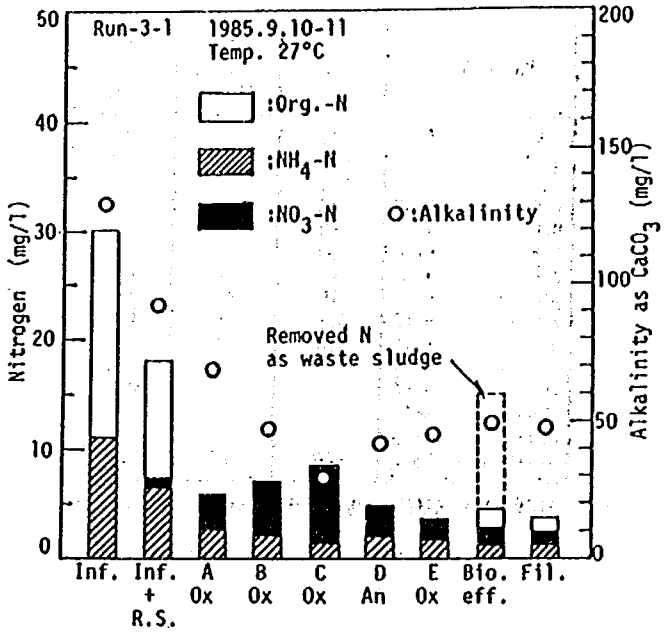


Fig.5 Changes of nitrogen and alkalinity through the process in Run-3-1.

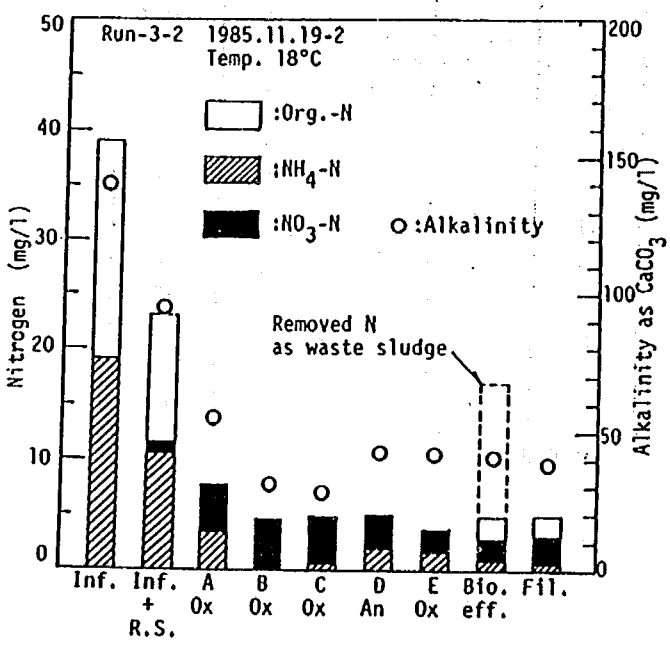


Fig.6 Changes of nitrogen and alkalinity through the process in Run-3-2.

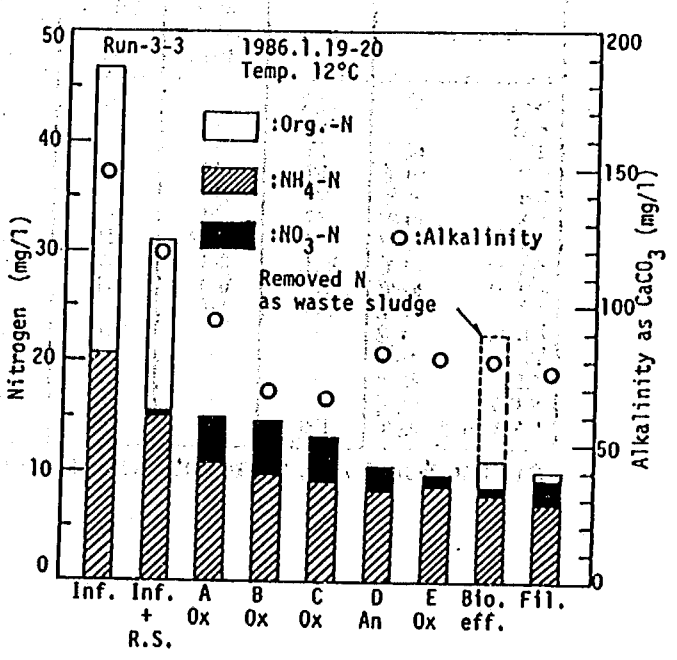


Fig.7 Changes of nitrogen and alkalinity through the process in Run-3-3.

hours.

On the other hand, in the low temperature period of Run 3 the nitrogen removal behavior was much deteriorated as shown in Table 9 and Fig. 7. The water temperature at this detailed examination was as low as 12°C which is the usual water temperature in the coldest winter for this plant. As Fig. 7 shows, the nitrification progress rate was considerably low and this low nitrification rate would be the limitting factor for the nitrogen removal operation in this period. This low nitrification rate may result from a low DO level in aeration tank as well as a low water temperature, because the DO level in oxic cell C was approximately zero.

In the Kotoh plant, the oxygen supply capacity of the aerator is considered to not be sufficient for the oxygen demand of carbon oxidation and nitrification. As the oxygen demand increases especially in winter because of the higher nitrogen concentration in the influent, the deficit of the aeration capacity would be revealed. Related to the relatively low DO operation in oxic zone, the phenomenon of simultaneous nitrification and denitrification in oxic zone is estimated, and can be confirmed by the nitrogen behavior in Fig. 4 and Fig. 6. As for the required aeration time for complete oxidation of nitrogen in the low temperature, we will discuss on it in the later paragraph.

The other miscellaneous phenomena which should be pointed out concerning the nitrogen behavior in the aeration tank are the nitrogen removal as excess activated sludge and the anmonium nitrogen production in anoxic zone. The removed nitrogen as waste sludge is as high as about 10 mg/l on the basis of the nitrogen concentration of the waste water. The resolved nitrogen in anoxic zone as anomonium nitrogen form is observed to the extent of 1 to 2 mg N/l.

3. Discussion

As mentioned above the total detention time of the aeration tank seems to not be sufficient to remove the nitrogen to the concentration of 5 mg/l in the winter time. To examine this problem, the estimation of the denitrification rate under endogenous respiration as well as of the nitrification rate. Recently, we arranged the design manual of the

biological nitrification-denitrification process with the nitrified liquor recycle.[2] In this manual the nitrification and denitrification rate were examined and the following Fig.8 and Fig.9 are presented.

Fig.8 shows the relation between the nitrification rate and the temperature which was obtained by the demonstration and pilot plant investigations. The experimental result at Kotoh WTP obtained in the former investigation is also added in a dotted line.[3] In this figure it can be estimated that the nitrification rate at 12°C of the coldest water temperature in Kotoh WTP is about 0.5mgN/gMLSS·hr

In the same manner, Fig.9 shows the denitrification rate regarding the temperature index. The solid lines describe the denitrification rate of the denitrification using raw sewage as carbon source. The dot line shows the experimental result of the denitrification using endogenous respiration which was obtained in the former investigation carried out in the Kotoh WTP.[3] The denitrification rate using endogenous respiration is generally considered to be lower than that using raw sewage as the carbon source, but this figure shows that there may not be so great a difference between them. The denitrification at cold temperatures for Kotoh WTP is estimated as 0.3 mgN/gMLSS·hr.

We can calculate the theoretically needed detention time of the aeration tank by the above mentioned nitrification and denitrification rates. Assuming that influent T-N 40mg/l, effluent T-N 5mg/l and the removed T-N as waste sludge 10mg/l, and MLSS 3g/l, the calculated detention time for nitrification is 20 hours as well as 28 hours for denitrification. This calculation shows that 48 hours of detention time is needed for the nitrification-denitrification process to meet the effluent standard of 5mg/l T-N for cold water.

In the nitrification-denitrification process using endogenous respiration, the nitrification precedes the denitrification. Therefore at the first stage, about 30mg/l anmonium nitrogen should be oxidized to nitrate, and this would consume more than 200mg/l alkalinity. However, as the alkalinity of the influent is less than 200mg/l, this process does not work at all. To solve this problem, some modification should be made on this process. We think that the application of simultaneous nitrification

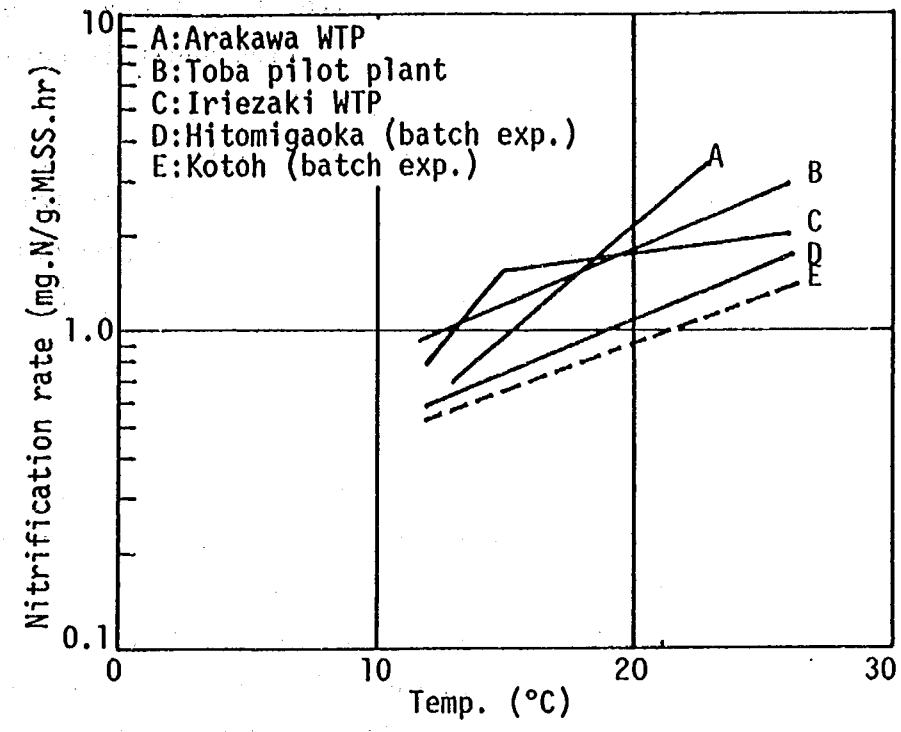


Fig.8 Relationship between nitrification rate and temperature.

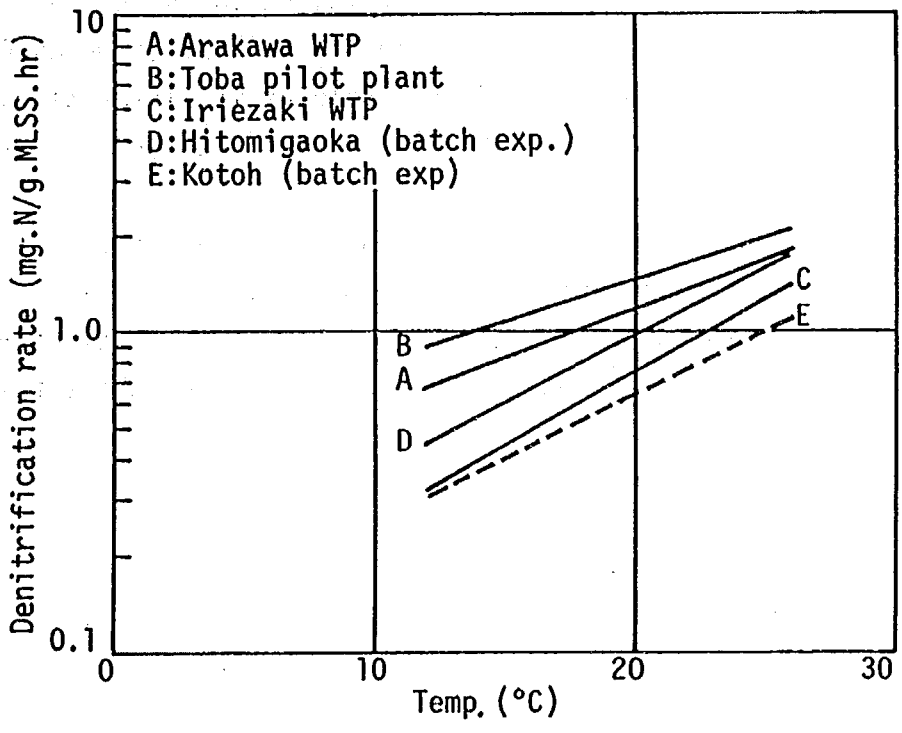


Fig.9 Relationship between denitrification rate and temperature.

denitrification would be one possible way to solve it. It may not be so difficult to accomplish the simultaneous nitrification-denitrification condition, because the phenomenon was commonly observed in the demonstration experiment and is easy to obtain only by controlling the DO level. It is expected that the application of simultaneous nitrification-denitrification would solve not only the alkalinity deficit problem but also shorten the design detention time of the aeration tank for high grade BOD and N removal.

4. Conclusion

The following results were obtained by demonstration experiments carried out in the aeration tank of Kotoh WTP on the nitrification and denitrification process using endogenous respiration.

1) A nitrogen removal efficiency of more than 80% was obtained except in the winter time under the total detention time of 21 hours of the aeration tank, which exceeded the nitrogen removal efficiency of the nitrification-denitrification process with the nitrified liquor recycle.

2) In the winter time, when the water temperature was below 15°C, anmonium and nitrate nitrogen concentration remained in the effluent at as high as about 10mg/l. Theoretical calculation shows that it needs 48 hours detention time to meet 5mg/l N of the effluent in winter time, assuming the nitrification rate 0.5 mgN/gMLSS·hr and the denitrification rate 0.3 mgN/gMLSS·hr.

3) The alkalinity deficit may occur in the process where the nitrification precedes the denitrification. The application of simultaneous nitrification-denitrification can solve this alkalinity deficit problem and also has the possibility of shortening the total detention time required.

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2) Ministry of Construction and Advanced Wastewater Treatment Committee , Design Manual of Phosphorus and Nitrogen Control (1986) (In Japanese)

3) Y.Harada et al, Simultaneous Removal of Nitrogen and Phosphorus by the Modified Activated Sludge Process -A Full-Scale Evaluation Proc. of Environ.& Sani. Eng. Research Vol.20 (1984) (In Japanese)

DIFFERENCES IN THE NITRIFICATION AND DENITRIFICATION BEHAVIOUR OF ONE- AND TWO-STAGE ACTIVATED SLUDGE PLANTS

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1. Introduction

Traditionally, the most important task of wastewater treatment is the elimination of oxygen-consuming organic chemical compounds. The most important measuring parameters for the above are the BOD_5 or the COD. The sludge loading rate SLR has been accepted as the most significant design parameter. Researches of BÖHNKE and DAMIECKI /4/ have shown that there exists a clear relationship between the sludge-loading rate SLR and the purification efficiency of the activated sludge process.

In the design guidelines of the German Sewage Works Association (ATV) for activated sludge plants with more than 10,000 inhabitants connected, it is assumed that a "good" nitrification can be achieved in case of a sludge loading rate SLR = 0.15 kg BOD_5 / kg SS·d. Up to now, more detailed recommendations of design do not exist, but a serie of design suggestions /2,5,8,14,16,18,20,23,25,27/.
In biological sewage treatment plants nitrification and denitrification are necessary for several reasons:

1. The nitrification, i.e. the oxydation of ammonium (NH_4^+) to nitrate (NO_3^-), has a high requirement of oxygen. This requirement of oxygen should be kept away from the receiving water bodies and should be shifted into the biological sewage treatment plant.

2. Ammonium (NH_4^+) and ammonia (NH_3) have to be seen in a proportion to each other, which is mainly influenced by the pH-value and the temperature. In case of high pH-values and high temperatures the concentration of ammonia increases up to an extent becoming toxic for fishes.

3. Nitrogen, together with phosphorus, is an essential nutrient for all kinds of organisms supporting an eutrophication of standing or slowly flowing waters.

4. A high concentration of ammonium in the raw water requires an increase of the chlorine dosage necessary for the desinfection to receive drinking water. The high chlorine dosage supports the formation of chloramines.

5. A high concentration of nitrate in drinking water causes injurious of health - especially for babies.

6. By means of denitrification in biological sewage treatment plants energy can be saved and the alkanity consumed by the nitrification can be partly gained back.

In recent years a couple of two-stage activated sludge plants have been built in Germany, among which there is a plenty of Adsorption-Biooxydation plants. In contrary to one-stage plants there are up to now few experiences and no design suggestions concerning the elimination of nitrogen by nitrification and denitrification in two-stage plants.

In the following we introduce a research concerning the differences between nitrification and denitrification in one- and two-stage activated sludge plants based on operational results of pilot plants and full-scale plants, whereby especially the AB-process is considered.

In this connection a designing method is suggested, which is based not only on pilot results of our own, but also on literature data and on a critical investigation of present design suggestions.

2. Experiences on Researches of AB - Plants

In the past many researches of the AB-treatment have been done with the goal to optimize the reduction of BOD_5 and COD. Measurements of the eliminatation of nitrogen have been sometimes executed, but not very intensively.

Table 1 and figure 1 show the measured NH_4^+-N influent and effluent values as well as the NO_3^--N effluent values. But all these investigations have not been done to optimize the denitrificaiton.

The plants' efficiency of nitrification is very good. Up to a sludge loading rate of approx. 0.2 kg BOD_5 / kg MLSS · d in the second stage an extensive nitrification is achieved. Concerning the effluent value of the pilot plant Rotterdam-Dokhaven of 23 mg NH_4^+-N/l you have to consider that the influent to stage B has shown an ammonium concentration of 218 mg N/l. Only beyond a sludge loading rate of 0.3 kg BOD_5 / kg MLSS · d nitrification is no longer recorded.

In this connection the pilot results of INGERLE /12/ are very interesting: with a B-stage's loading rate of 0.18 kg BOD_5 / kg MLSS · d the following NH_4^+-N effluent values have been achieved, depending on the temperature:

$T = 20^o$ C: $NH_4^+-N_e = 2{,}3$ mg/l
$T = 10^o$ C: $NH_4^+-N_e = 5{,}1$ mg/l
$T = 5^o$ C: $NH_4^+-N_e = 11{,}5$ mg/l

Even at 5° C more than 60 % of the nitrogen could be nitrified.

The values of NO_3^--N in the effluent of the 2nd stage have to be seen under the limitation that in no case a denitrification by means of anoxic zones has been the goal of the research. Nevertheless, the performance of denitrification shows approx. 40 % - 50 % for example in Krefeld and Haan Gruiten.

Encouraged by these results of investigation a design procedure has been developed, which considers the differences between the nitrification and the denitrification in one- and two-stage activated sludge plants.

3. General Design Procedure

After having fixed the influent values and the frame conditions as well as the nitrification performance of the 1st stage (primary clarification or 1st biological stage) the following 4 most important design steps have to be followed (the design procedure is shown in attachment 1):

1. Computation of the sludge age, which is necessary for the assumed growth of the nitrifiers.

2. Computation of the necessary retention time in an aerobic tank section depending on the nitrification rate; Choice of the largest aerobic volume, coming out of point 1 or 2.

3. Computation of the retention time necessary for denitrification in the anoxic tank section depending on the denitrification rate.

4. Control, if a sufficient BOD_5-elimination can be achieved with the existing loading rate according to the calculation of the entire volume.

The control required under point 4 will be always positive in case of normal wastewater composition and extensive nitrification and denitrification, i.e. the extensive elimination of nitrogen requires a larger tank volume than the extensive BOD_5-elimination.

4. Elimination of Nitrogen by Incorporation into the Biomass

Before the nitrogen to be nitrified and denitrified can be calculated, it has to be computed how much nitrogen is incorporated into the cell during the growth of the biomass and how much together with the excess sludge will be eliminated out of the system.

According to KAPP /15/ the nitrogen content of the organic share in the excess sludge amounts to 9 %, according to EKAMA et al. /5/ 9 % to 12 % including an approximate value of 10 %. KAYSER /14/ gives a value of 12 %, based, however, on the active bacteria mass, which is only partly included due to measuring of the volatile matter. In the following we have taken 9 % as a base.

The elimination by incorporation into the excess sludge computes with the equation:

$$C_{N,inc} = \frac{oES_I \times f_{N,inc} \times 1000}{Q}$$

where: $C_{N,inc}$ = Nitrogen eliminated by incorporation into the excess sludge (converted in an N-concentration) (mg/l)

oES_I = organic Excess Sludge production (g VSS/Inh·d or: kg VSS/d)

$f_{N,inc}$ = Nitrogen content of the organic excess sludge (g N/g VSS)

Q = daily wastewater flow (l/Inh·d or: m^3/d)

5. Design for the Nitrification

5.1. Design to Ensure a Sufficient Sludge Age

Only two specialized groups of bacteria, Nitroso- and the Nitro-group, are able to nitrify and in addition they grow only very slowly. For this reason, the sludge age has to be sufficient high so that the nitrifiers reproduce faster than they are washed out with the excess sludge from the system.

The equation most often used to compute the growth rate μ is the one of DOWNING/quoted in 27/. This equation is also suggested by other authors, for example KAYSER /14/ and GUYER /6/:

$$\mu_T = 0.47 \times 1.103^{(T-15)}$$

where μ_T = Growth rate at a temperature T (d^{-1})

T = Temperature (^oC)

At 15° C the term to correct the temperature becomes 1, the growth rate amounts to 0.47 d^{-1}. That means that a "change of generation" of the nitrifiers needs a time of 1/0.47 = 2.13 d. The reciprocal value of the maximum growth rate is the minimum sludge age, at which a washout of bacteria can be just avoided. The indicated growth rate is only valid in connection to conditions, under which the nitrifiers can actually grow. But this is only possible under aerobic conditions with a sufficient high oxygen concentration. This results in the conclusion that this sludge age must refer only to the aerobic tank section.

The minimum of the aerobic sludge age theoretically necessary amounts to:

$$t_{SS,theor.} = 1/\mu_T$$

where: $t_{SS,theor.}$ = aerobic sludge age theoretically necessary (d)

For T = 5° C, a necessary aerobic sludge age of 5.67 days is needed, at 15° C the such age amounts to 2.13 days and at 25° C only 0.8 days are necessary.

The sludge age calculated in that way cannot practically lead to a nitrification, because every short-term fall below this age would wash out the nitrifiers. In addition, nitrifiers are sensitive bacteria, the growth of which can be slowed down or even stopped by a variety of compounds.

Due to the above reasons, an aerobic sludge age is needed for designing of an activated sludge plant, which is much more higher than the one required in theory:

$$t_{SS,pract.} = SF \times t_{SS,theor.}$$

where: $t_{SS,pract.}$ = aerobic sludge age practically necessary (d)
SF = Safety Factor (-)

According to the investigations of GUJER/6/ and KAYSER/14/, the safety factor for a nitrification of 90 % should be at least 2, according to SCHLEGEL /24/ the factor should be 3 as minimum. A high safety factor is required as well, if the concentration of ammonium in the effluent shall be very low (e.g. $\leq$1 mg/l). When there are low NH_4^+-N-concentrations in the effluent - and therefore also in the tank itself - the nitrifiers' growth is slowed down corresponding to the Monod-kinetic. This slowing-down can be compensated by a high safety factor. A high safety factor is the pre-condition to achieve very low NH_4^+-N-concentrations in the effluent. In the following a minimum safety factor of 3 has been assumed.

The approx. retention time of the wastewater in the aerobic tank section can be computed with the aerobic sludge age practically required:

$$t_{aerob} = \frac{t_{SS,pract.} \times ES}{q_{24} \times MLSS}$$

where: t_{aerob} = average retention time in the aerobic tank section (h)

ES = Excess Sludge production (g SS/Inh·d
or: kg SS/d)

q_{24} = Average day value of effluent (l/Inh·h
or: m^3/h)

MLSS = Mixed Liqueur Suspended Solids in the tank volume (kg/m^3)

Here a clear advantage of a two-stage-system can be seen compared to a one-stage one: through the extensively reduced production of excess sludge in the 2nd stage such a high sludge age can be established that the necessary retention time in the aerobic tank section, and, due to this, also the volume of this tank section is much less than that one in a one-stage plant.

5.2. Design to Assure a Sufficient Retention Time

In a second step it has now to be checked, which average retention time in the aerobic tank is necessary to nitrify the ammonium as much as possible, with a nitrification rate still to be calculated and an assumed MLSS concentration. To achieve that, first of all the nitrogen concentration to be nitrified is calculated:

$$C_{N,nitr} = C_{Ntot,1} - C_{N,inc} - C_{NH4\text{-}N,e} - C_{Norg,e}$$

where: $C_{N,nitr}$ = Nitrogen Concentration to be nitrified (mg/l)

$C_{Ntot,1}$ = total Nitrogen Concentration in the effluent of the primary clarification or the 1st activated stage (mg/l)

$C_{N,inc}$ = Nitrogen Concentration incorporated into the excess sludge (mg/l)

$C_{NH_4\text{-}N,e}$ = Ammonium Concentration in the effluent (mg/l)

$C_{Norg,e}$ = Concentration of organic Nitrogen in the effluent (mg/l)

The incorporated nitrogen concentration has already been calculated above, the ammonium concentration in the effluent has to be given as measure for the nitrification performance of the system and, if the final clarifier works well, the process application has hardly an influent on the concentration of the organic nitrogen in the effluent, which lies approx. in the range of 2 to 3 mg N/l.

For the next step a nitrification rate has to be assumed. In a detailed literature review for one-stage systems HENZE & HARREMOES have computed nitrification rates of 1.7 and 5.1 g N/kg VSS·h at 20° C /9/. SCHLEGEL has measured at 20° C rates of 1.0 and 4.5 g N/kg /23/, SUTTON & JANKS give a rate of 4.5 g N/kg VSS·h /quot. in 23/. PÖPEL /18/ assumes for 15° C 3 g N/kg VSS·h.

This rate shall be taken in the following as a so-called "base"-nitrification rate for the calculations. The word "base"-nitrificaiton rate has been chosen because it is only valid for one-stage systems. For the second stage of two-stage plants, however, which is fed with a wastewater including an altered BOD_5/N ratio, it is to be converted corresponding to the subsequent step of calculation.

Only the nitrifiers can nitrify. The nitrification rate referring to the entire organic dry solid matter has to change if the nitrifier fraction in the organic dry solid matter changes. The nitrifiers' concentration cannot be measured directly, but is estimated by the following theoretical considerations /2,11,27/:

The nitrifier fraction in the entire biomass f_N depends on the growth of the nitrifiers proportionally to the the growth of the entire biomass consisting of heterotrophic, carbon compounds degrading bacteria and nitrifiers:

$$f_N = \frac{\text{growth of the nitrifiers}}{\text{growth of the entire biomass}}$$

The growth of the biomass is computed with the mass of eliminated substrate (ΔBOD_5 or ΔN) multiplied with the yield coefficient Y:

$$f_N = \frac{\Delta N \times Y_N}{\Delta BOD_5 \times Y_B + \Delta N \times Y_N}$$

where: f_N = fraction of the nitrifiers in the entire organic dry solid matter (kg/kg)

ΔN = nitrified mass of Nitrogen (e.g.: mg)

ΔBOD_5 = degraded mass of BOD_5 (e.g.: mg)

Y_N = Yield coefficient of the nitrifiers (g VSS/g N_{elim})

Y_B = Yield coefficient of the heterotrophic bacteria (g VSS/g $BOD_{5,elim}$)

The above equation can be changed and receives the following form:

$$f_N = \frac{1}{\frac{\Delta BOD_5}{\Delta N} \times \frac{Y_B}{Y_N} + 1}$$

The correct estimation of the yield coefficients Y_N and Y_B is a problem when applicating the equation. In the EPA-Manual the following rates are mentioned /27/:

$Y_B = 0.55$ (g VSS/g $BOD_{5,elim}$)

$Y_N = 0.15$ (g VSS/g N_{elim})

When calculating with these figures and assuming parallely that the BOD_5 as well as the nitrogen are mostly eliminated, the following familar equation produces:

$$f_N = \frac{1}{\frac{BOD_5}{N} \times 3.67 + 1}$$

Figure 2 shows the resulting curve.

The equation shows clearly that the nitrifier fraction in the organic dry solid matter depends directly on the BOD_5/N-ratio of the influent wastewater. A reduction of this ratio leads to an increase of f_N. Such a reduction can be achieved by using a first stage.

In Germany the normal municipal wastewater has an approx. BOD_5/N-ratio of 225/50 = 4.5 after primary clarification, from which a nitrifier fraction of

5.7 % is computed. In case of changing the ratio to 135/45 = 3.0, for example by using a highest-loaded activated sludge stage, it will result in a fraction of 8.3 %. Then the sludge contains (8.3 / 5.7 = 1.46) 46 % more nitrifiers and the nitrification rate increases also by 46 %.

According to this, HENZE & HARREMOES give also nitrification rates of two-stage systems of 9.2 to 13.8 g N/kg VSS·h /9/, MULBARGER has measured 7.5 g N/kg VSS·h in the second stage of a large-scale pilot plant /17/. Investigations of STOVER & KINCANNON showed that the nitrification reactor of a two-stage system achieved a nitrification performance, which was by 40 % higher than in a one-stage system, both having the same sludge age /26/.

In table 2 the mentioned nitrification rates for one- and two-stage systems are shown.

The comparison of one- and two-stage systems shows a further advantage of the two-stage one: The higher fraction of nitrifiers in the biomass results in a higher nitrification rate.

The nitrification rate, which already includes a correction factor concerning the nitrifiers' share in the sludge, has still to be corrected concerning the present temperature of wastewater. The same constant as for the growth rate of nitrifiers is used:

$$v_{N,T} = v_N \times F \times 1.103^{(T-15)}$$

where:
- $v_{N,T}$ = Nitrification rate at the Temperature T (g N/kg VSS·h)
- v_N = Nitrification rate at 15° C (g N/kg VSS·h)
- F = Correction Factor to consider the share of the nitrifiers in the sludge (-)
- $F = f_{N,actual}/f_{N,one\text{-}stage}$

With the nitrification rate $v_{N,T}$ calculated like this, the necessary retention time in the aerobic tank section can be calculated again:

$$t_{aerob} = \frac{C_{N,nitr}}{v_{N,T} \cdot MLSS \cdot IL/100}$$

where: t_{aerob} = necessary retention time in the aerobic tank section (h)

$C_{N,nitr}$ = Nitrogen concentration to be nitrified (mg/l)

MLSS = Content of Mixed Liqueur Suspended Solids in the activated tank (g/l)

IL = Ignition Loss (= organic share) of the dry solids (%)

In the next step it has to be checked according to which computation method - according to the necessary aerobic sludge age or according to the nitrification rate - the higher average retention time $t_{aerobic}$ has been achieved. This increased retention time has to be taken as the base for measuring.

It has proved that in general for one-stage plants the necessary aerobic sludge age, and for two-stage plants the nitrification time available, is the competent way of measuring.

The retention time computed with the preceding equations must not be kept for the wastewater flow during the daytime (e.g. q_{14}). For the further computation it is sufficient to go on calculating with the average value of the whole day (q_{24}). Also the sludge loading rate, the competent measuring parameter for the degradability of BOD_5 and COD is not calculated with the BOD_5-load received during the daytime, but with the average load of the whole day.

At that stage the volume of the aerobic tank section can be computed with the competent retention time:

$$v_{aerob} = t_{aerob} \times q_{24}$$

where: v_{aerob} = specific aerobic tank volume (l/Inh)
or: aerobic tank volume (m^3)

t_{aerob} = necessary retention time in the aerobic tank volume (h)

q_{24} = hourly wastewater flow (l/Inh·h or m^3/h)

In those cases where not the aerobic sludge age, but the retention time in the aerobic tank section is the competent design parameter - normally in two-stage plants - the actual aerobic sludge age can now be calculated.

$$t_{SS,act} = \frac{v_{aerob} \cdot MLSS}{ES}$$

where:

$t_{SS,act}$	=	actually present aerobic sludge age	(d)
v_{aerob}	=	specific aerobic tank volume	(1/Inh)
		or: aerobic tank volume	(m^3)
MLSS	=	content of Mixed Liqueur Suspended Solids in the activated tank	(g/l)
ES	=	Excess Sludge production	(g SS/Inh·d
		or:	kg SS/d)

If the sludge age is higher than necessary, the actual safety factor is also higher:

$$SF_{act} = \frac{t_{ss,act}}{t_{ss,theor}}$$

where:

SF_{act}	=	Safety Factor actually present	(-)

When the present aerobic sludge age increases according to this calculation, the actual safety factor increases as well. So it can be said that a high safety factor and also a higher process stability of the nitrification are characteristics of two-stage plants.

6. Design for the Denitrification

In contrary to the nitrification, the denitrification is not done by a group of specialized bacteria, but a lot of aerobic heterotrophic bacteria can be used. Due to this fact, no minimum sludge age is necessary for the denitrification; the design is executed under the assumption of a denitrification rate.

The result of a detailed literature review about denitrification is described in the review of HENZE & HARREMOES /10/. According to this review the

denitrification rate depends on the availability of the carbon source or - in other words - on its degradability. Under this condition and at 20° C the following denitrification rates are the result:

Methanol	:	10 mg NO_3^--N/g VSS·h
Raw wastewater	:	3 mg NO_3^--N/g VSS·h
Endogenous carbon source	:	0.4 mg NO_3^--N/g VSS·h

In two-stage plants the wastewater is already preliminarily clarified in the 1st stage and so it changes its composition and degradability. A useful parameter for the degradability of wastewater is the BOD_5/COD ratio. By using the biological sewage purification in one-stage plants, this ratio is strongly reduced. The lower this ratio is, the lower is the degradability of the wastewater and the more difficult it is to degrade the remaining organic compounds. In conventional two-stage activated sludge plants with an average-loaded 1st stage the BOD_5/COD-ratio is also reduced by the 1st stage, i.e. the degradability and the denitrification rate as well are reduced by using these wastewaters as carbon source. High-loaded 1st stages or adsorption stages, however, are an exception. The evaluation of the results of 15 test series with 11 various municipal wastewaters showed that the BOD_5/COD-ratio was not changed on average by pre-connecting an adsorption stage, but that the BOD_5/COD-ratio could even be increased by running the adsorption stage facultatively /3/.

Figure 3 shows the results of this evaluation.

This leads to the conclusion that the denitrifictation rate in the subsequent, low-loaded stage is not reduced, but can be even increased by pre-connecting a high-loaded 1st stage.

The denitrification rates mentioned by HENZE & HARREMOES have already been quoted above; in large-scale and laboratory tests SCHLEGEL got the result of rates ranging from 1 to 4 with an average rate of 3 g N/kg VSS·h at water temperatures of about 15° C /22/. If the process proceeds well, i.e. avoiding especially oxygen transfer into the anoxic denitrification zone, even higher values can be achieved. In a review of various references PÖPEL /20/ has mentioned the following values:

relatively small denitrification zone : 4.5 - 6.6 g N/kg VSS·h
relatively large denitirification zone : 2.1 - 8.6 g N/kg VSS·h

In table 3 the denitrification rates found in literature reviews are listed.

In the example calculation shown in the attachment, a denitrification rate v_D of 2 g N/kg VSS·h has been assumed for one- and two-stage plants. This assumption lies more in the lower range of the values normally assumed.
Also the denitrification rate is depending on temperature. BARNES & BLISS indicate the dependence, which is included in the following equation /2/:

$$v_{D,T} = v_D \times 1.07^{(T-15)}$$

where: $v_{D,T}$ = denitrification rate at the temperature T (g N/kg VSS·h)

Before the volume of denitrification is calculated, the nitrogen concentration to be denitrified has to be computed:

$$C_{N,deni} = C_{Ntot,1} - C_{N,inc} - C_{NH_4^+-N,e} - C_{NO_3^--N,e} - C_{Norg,e}$$

where: $C_{N,deni}$ = Nitrogen Concentration to be denitrified (mg/l)

Now the average retention time in the denitrification section can be calculated:

$$t_{anox} = \frac{C_{N,deni}}{v_{D,T} \times MLSS \times IL/100}$$

where: t_{anox} = necessary retention time in aerobic tank volume

and with t_{anox} then the anoxic volume follows:

$$V_{anox} = t_{anox} \times q_{24}$$

where: V_{anox} = specific denitrification volume (ℓ/Inh)
or: denitrification volume (m^3)
q_{24} = hourly wastewater flow (ℓ/Inh·h
or: m^3/h)

To achieve a denitrification, either the recirculation flow out of the nitrification tank has to be moved back into the denitrification tank (pre-connected denitrification) or the activated tank has to be designed like an oxidation ditch with constant recycling (simultaneous denitrification). In this case the sum of the returned flows of wastewater and activated sludge referred to the influent, is called total recycle ratio (TRR).

$$TRR = RR + RSR$$

where: TRR = Total Recycle Ratio (-)
RR = Recirculation Ratio (-)
RSR = Return Sludge Ratio (-)

Figure 4 shows the arrangement of the flows.

To calculate the necessary recycle ratio, the required efficiency of denitrification has to be known:

$$\eta_{deni} = \frac{C_{N,deni}}{C_{N,nitr}}$$

where: η_{deni} = Efficiency of the denitrification (-)
$C_{N,deni}$ = Nitrogen Concentration to be denitrified (mg/l)
$C_{N,nitr}$ = Nitrogen Concentration to be nitrified (mg/l)

After having calculated the efficiency of denitrification the necessary recycle ratio computes according to HEGEMANN:

$$TRR = \frac{\eta_{deni}}{1 - \eta_{deni}}$$

7. Influence of the BOD_5/N-Ratio

It is often said that two-stage plants cannot denitrify in a satisfactory way, because the organic carbon necessary for denitrification is not sufficient any more in the influent of the 2nd stage. This is true for two-stage systems operating with a normal-to-medium-loaded 1st stage. But if the 1st stage is run as a high-loaded adsorption stage, a sufficient high BOD_5/N-ratio is remaining in normal-composed wastewater to ensure a far-reaching denitrification.

Table 4 shows a comparison of 10 large-scale sewage treatment plants of the two-stage type with a medium-loaded 1st stage and 9 large-scale plants of the AB-type with a highest-loaded 1st stage. The BOD_5 concentration in the effluent of the medium-loaded 1st stage with an average of 51.4 mg/l is surely not sufficient enough to denitrify extensively by taking normal-composed municipal wastewater. Compared to this, the effluent of the Adsorption stages contains still 174 mg BOD_5/l.

According to HENZE & BUNDGAARD /8/ the BOD_5/N-ratio theoretically necessary by using wastewater as carbon source amounts to 2.00 g BOD_5/g N, but due to the fact that the denitrification is connected to a growth of bacteria, at least 3 g BOD_5 / g N are necessary. In practice this value should be somewhat higher, because always some oxygen, included in the recycled wastewater and the return sludge, is transferred into the denitrification zone. Nevertheless, the value necessary in practice does not have to be so much higher than the value necessary in theory, as it is required sometimes.

JANSEN & KRISTENSEN /13/ for example speak of a danish sewage treatment plant, in which, over a measuring period of 4 years, the BOD_5/N-ratio in the influent into the activated section was 3, for the anorganic nitrogen concentration in the effluent, however, an average value of only 2.9 mg N/l could be measured.

In figure 5 the relationship is explained by means of a diagram. With this diagram it can be seen - by taking the concentration in the influent of the 1st stage and the elimination efficiency of the 1st stage - if it is possible to achieve a still extensive denitrification.

The average values of the large-scale plants, shown in table 4, are indicated in the diagram. Because there are no or only very few nitrogen measurements

of these plants, in all cases a nitrogen concentration in the influent into the 1st stage of 55 mg N_{tot}/l and a N_{tot}-elimination of the 1st activated stage of 20 % is assumed; for a primary clarification η N_{tot}: 15 % are taken.

The 10 large-scale plants with a 1st activated stage conventionally-loaded, show a BOD_5/N-ratio in the influent into the 2nd activated stage of approx. 1.2, which is significantly lower than the one required in theory. The 9 large-scale plants with highest-loaded 1st stage show a BOD_5/N-ratio in the influent to the 2nd stage of approx. 4.0, which is more than necessary in practice. To calculate more precisely, however, not the ratio of BOD_5/N to the influent into the denitrification tank is the value to be considered, but the ratio of the BOD_5/ to the nitrogen to be denitrified. Because a nitrification and denitrification of 100 % are only seldom the goal and are also rarely achieved, and because a part of the nitrogen is eliminated by incorporation into the biomass and not by denitrification, the ratio then looks even more favourable.

8. Control of the BOD_5-Elimination

In the last design step it has to be controlled, if the actual sludge loading rate is sufficient enough to degrade the organic compounds. The sludge loading rate computes with the equation:

$$SLR = \frac{Q \times C_{BOD_5,1}}{V_{tot} \times MLSS \times 1000}$$

where:

SLR	=	Sludge Loading Rate	(kg BOD_5/kg SS·d)
Q	=	specific wastewater flow	(l/Inh·d)
		or: wastewater flow	(m^3/d)
$C_{BOD_5,1}$	=	BOD_5-concentration in the effluent of the 1st stage	(mg/l)
V_{tot}	=	specific total Volume	(l/Inh)
		or: total Volume	(m^3)
MLSS	=	Content of Mixed Liqueur Suspended Solids in the activated tank	(g/l)

By taking normal-composed municipal wastewater, an extensive nitrification and denitrification requires larger volumes of the activated tanks than they are necessary for a sufficient BOD_5- and COD degradation.

9. Application of the Computation Method

The computation method introduced above has been applicated in attachment 1 in order to compare the volume necessary in an aeration basin in a one-stage activated sludge plant to the one of the AB-process. Please take the assumptions chosen, especially concerning the concentrations of the influent and effluent and the degradation efficiency of the 1st stages, out of the attachment. The very rigid effluent requirements of:

$$NH_4^+\text{-}N_e \leqslant 1 \text{ mg/l}$$
$$NO_3^-\text{-}N_e \leqslant 4 \text{ mg/l}$$
$$N_{org,e} \leqslant 2 \text{ mg/l}$$

result in an entire volume of the aeration tank of approx. 124 l/inhabitant in a one-stage plant, for the AB-process the necessary volume amount only to 88 l/inhabitant.

According to the same schedule the specific volume for the two purification processes with a temperature ranging from 5° to 25° C has been calculated and is indicated in figure 6. The second, dotted line is valid for case b with some higher N-values in the effluent. The entire sludge loading rate (SLR) respectively neccessary and the partition between the aerobic and anaerobic tank section is indicated at the curves.

The curves have been calculated by assuming a denitrification rate v_D at 15° C of 2 g N/kg VSS·h. The influence of the denitrification rate on the necessary treatment volume, however, is significant, which is shown in figure 7. For denitrification rates (15° C) ranging from 0.5 to 5 g N/kg VSS·h the inhabitant-related volume of the aeration tank necessary to achieve the rigid requirements of the effluent above mentioned, is shown. The upper hached area applies to one-stage plants, the lower to AB-plants. In both cases the temperature ranges from 10° to 20° C.

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	Pilote-plant Aachen-Horbach	Pilote-plant Aachen-Horbach	Pilote-plant Aachen	Sewage treatment plant Krefeld	Pilote-plant Rotterdam-Dockhaven Netherland	Study Prof. Ingerle Innsbruck Austria	Full-scale study Haan-Gruiten	Pilote-plant Aachen	Pilot-plant Pulheim	Pilot-plant Lenzburg/Switzerland (Hager + Kuster)
Sludge loading rate	0.07	0.09	0.12	0.17	0.15	0.18	0.20	0.22	0.39	0.32
Influent of A-stage NH_4^+-N (mg/l)	23.7	15.9	24.3	(23.5)	N-Kj 290	38.3	26.0	24.3	55.0	-
Influent of B-stage NH_4^+-N (mg/l)	21.0	11.5	23.7	18.4	218	-	40.0	23.7	-	12.0
Effluent of B-stage NH_4^+-N (mg/l)	0.3	0.3	0.34	1.0	23.0	2.3*	2.0	9.4	40.0	11.2
Effluent of B-stage NO_3^--N (mg/l)	24.5	14.3	20.2	7.2	-	12.0	21.5	9.5	10.0	-
Time of the year			Dec.	May-June		Summer	Sept.-Jan.	Dec.	Aug.	Dec.
* Innsbruck NH_4^+-N = 5.1 mg/l at 10° C				Not optimized for denitrification						

Table 1

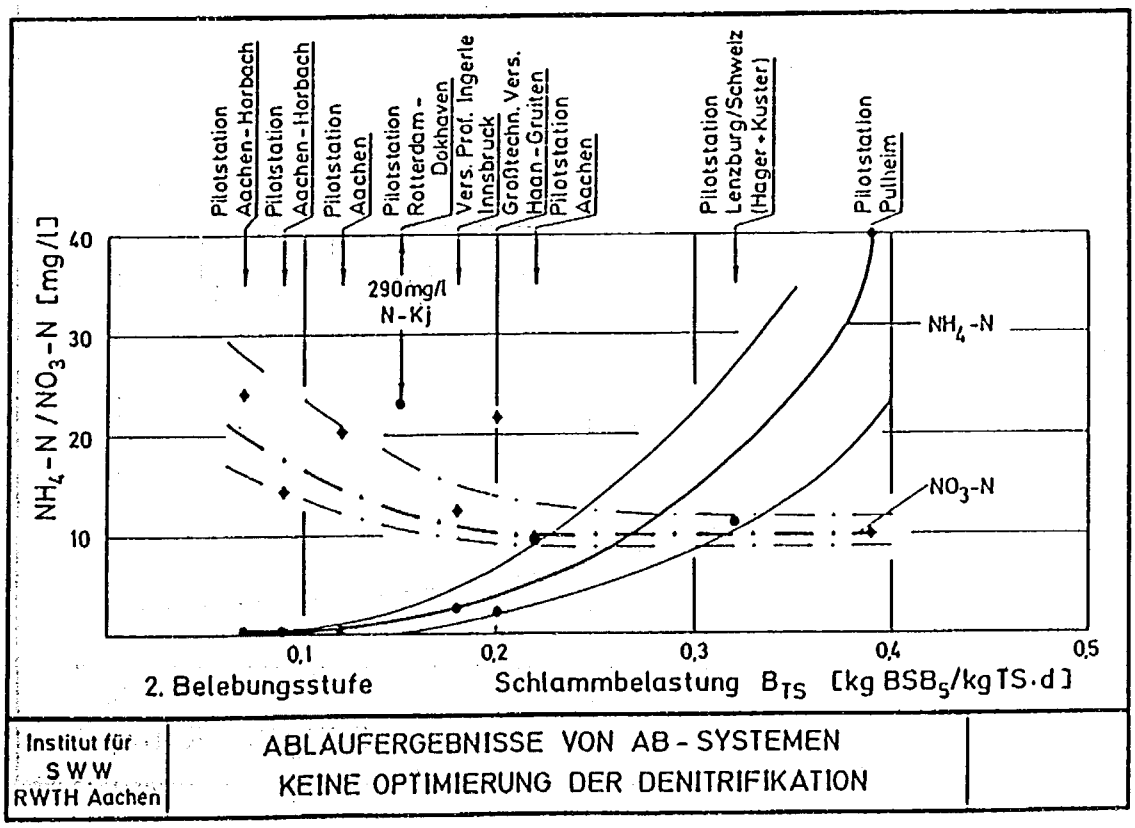


Figure 1

Author	Nitrification-rate g N/kg VSS.h	Temperature °C	Lit.
One-Stage Systems			
Barnes & Bliss	1 - 3	?	/2/
Pöpel	1,5	5	/19/
"	2,1	10	
"	3,0	15	
"	4,2	20	
Bishop et al.	2,3	20	/9/
Henze	1,7	20	"
Wuhrmann	3,2	20	"
Heide	5,1	20	"
Sutton	3,3	20	"
Barnard et al.	3,6	15	/1/
Sutton & Jank	1,8	10	/23/
"	3,0	15	"
"	4,6	20	"
Dijstra & Baenens	1 - 3,1	?	"
Schlegel	0,5 - 4	15	"
Two-Stage Systems			
Heidmann et al.	9,2	20	/9/
Mulbarger	7,5	15	/17/

Table 2

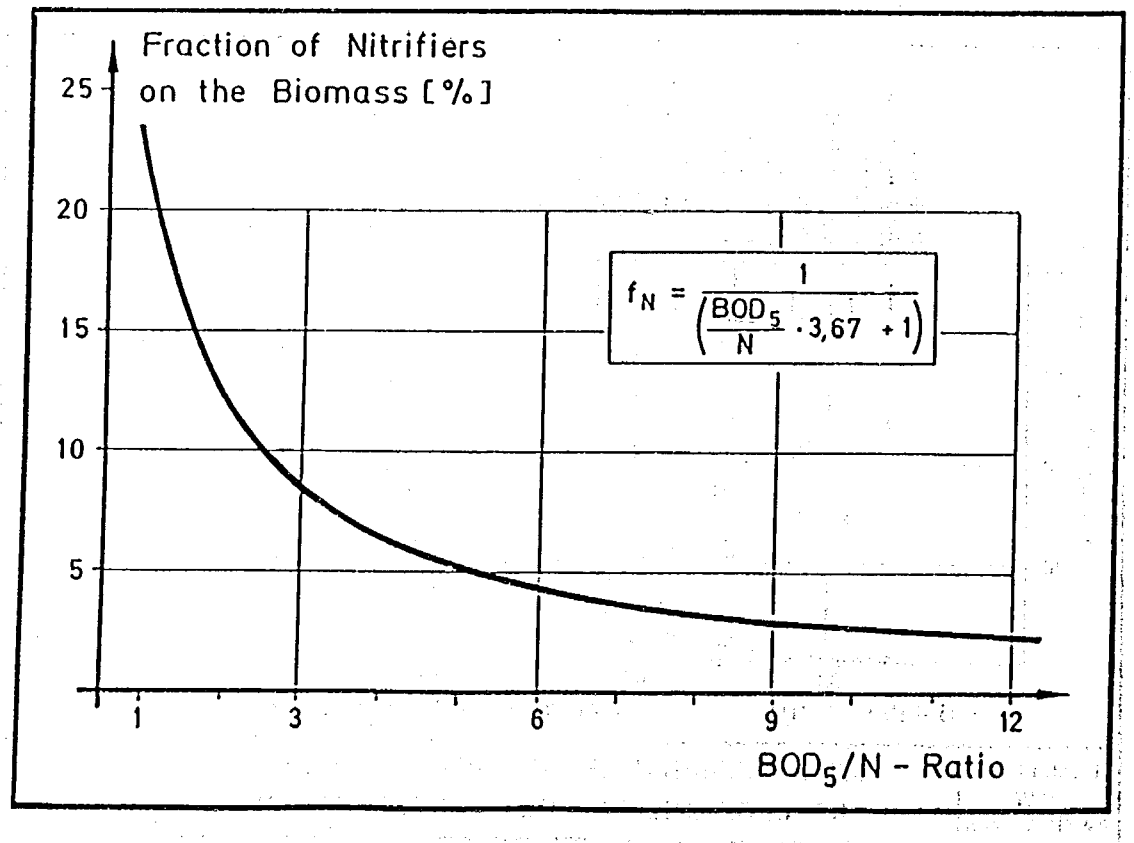
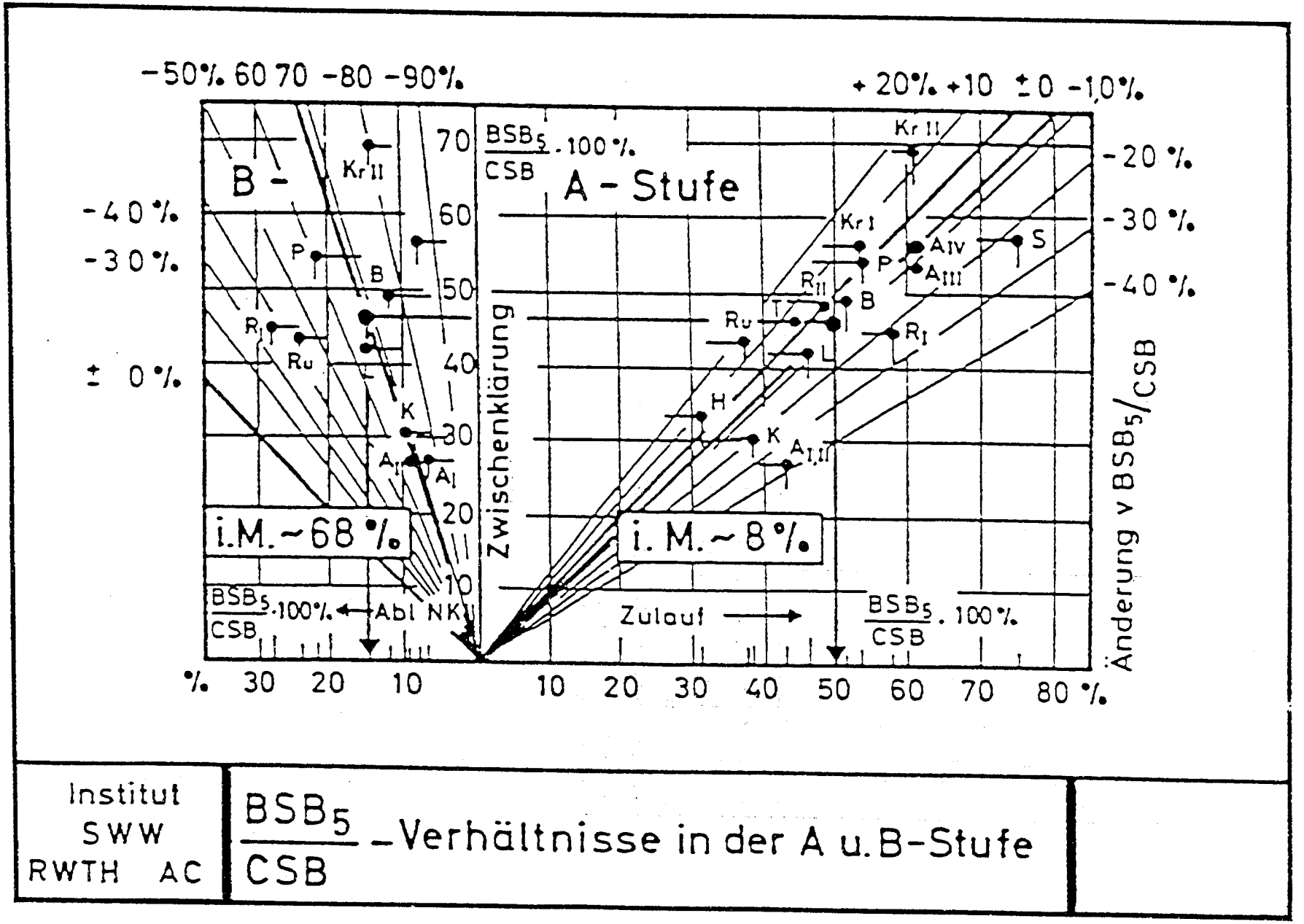


Figure 2

Author	Denitrification-rate g N/kg VSS.h	Temperature °C	Lit.
Kayser	3[1] - 4,46[2]	?	/9/
Krauth	2,1[1] - 6,6[2]	?	/9/
Pöpel	2,4[1] - 6,0[2]	?	/9/
Schlegel	4,8 - 8,6[1]	?	/9/
Henze & Harremoes	10[3]	20	/10/
	3[4]	20	
	0,45[5]	20	
Schlegel	1,8 - 4,8	10	/24/
	3,2 - 6,4	15	
	5,3 - 8,33	20	

1) fairly large denitrification zone: V_D/V 0,4
2) fairly small denitrification zone: V_D/V 0,2
3) Methanol as carbon source
4) Raw wastewater as carbon source
5) Sludge as carbon source

Table 3



Institut SWW RWTH AC	$\frac{BSB_5}{CSB}$ – Verhältnisse in der A u. B-Stufe

Figure 3

two-stage activated sludge plants										
No.	medium loaded 1st stage Effl. PC:					Adsorption stage Effl. IC:				
	I + PE design	I + PE actual	%	SLR	$BOD_{5,e}$ mg/l	I + PE design	I + PE actual	%	SLR	$BOD_{5,e}$ mg/l
1	9,500	6,020	98.1	0.13	22	55,000	132,600	65.0	2.90	109
2	33,000	14,700	92.9	0.30	20	3,200	41,580	67.6	3.80	290
3	12,500	13,866	70.0	0.80	41	5,000	5,100	62.2	1.10	68
4	64,000	70,875	96.9	0.65	25	10,000	4,932	38.7	4.20	168
5	120,000	58,100	90.0	0.57	13	800,000	800,000	51.1	2.40	180
6	40,000	43,000	48.0	1.10	150	104,000	53,333	49.0	6.70	204
7	46,100	27,763	91.6	0.40	11	40,000	31,200	41.0	2.20	230
8	26,400	14,235	80.4	0.45	43	350,000	793,000*	70.3	2.55	85*
9	90,000	87,197	77.3	1.06	75	92,000	128,428	52.8	4.53	145
10	215,000	134,065	77.9	0.43	50	* not contained in the average because the addition of chemicals				
	I + Pe	470,029	80.9	0.66	51.4	I + Pe	1,197,173	53.0	2.92	174

Table 4

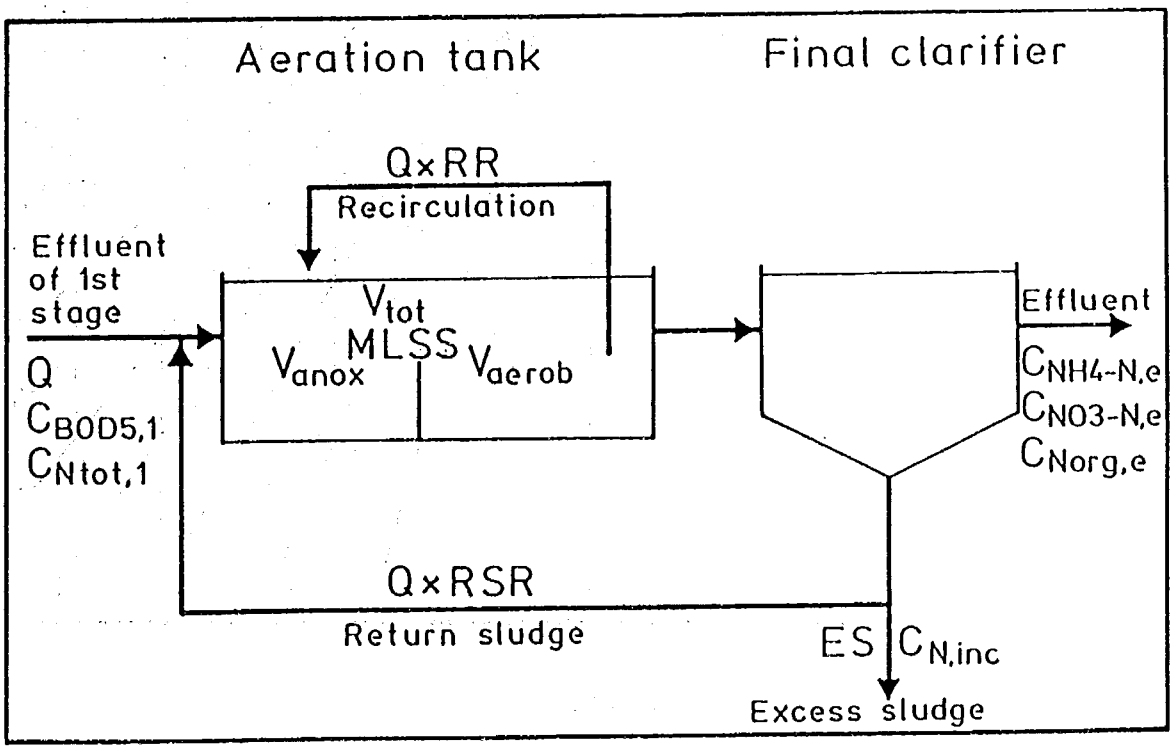


Figure 4

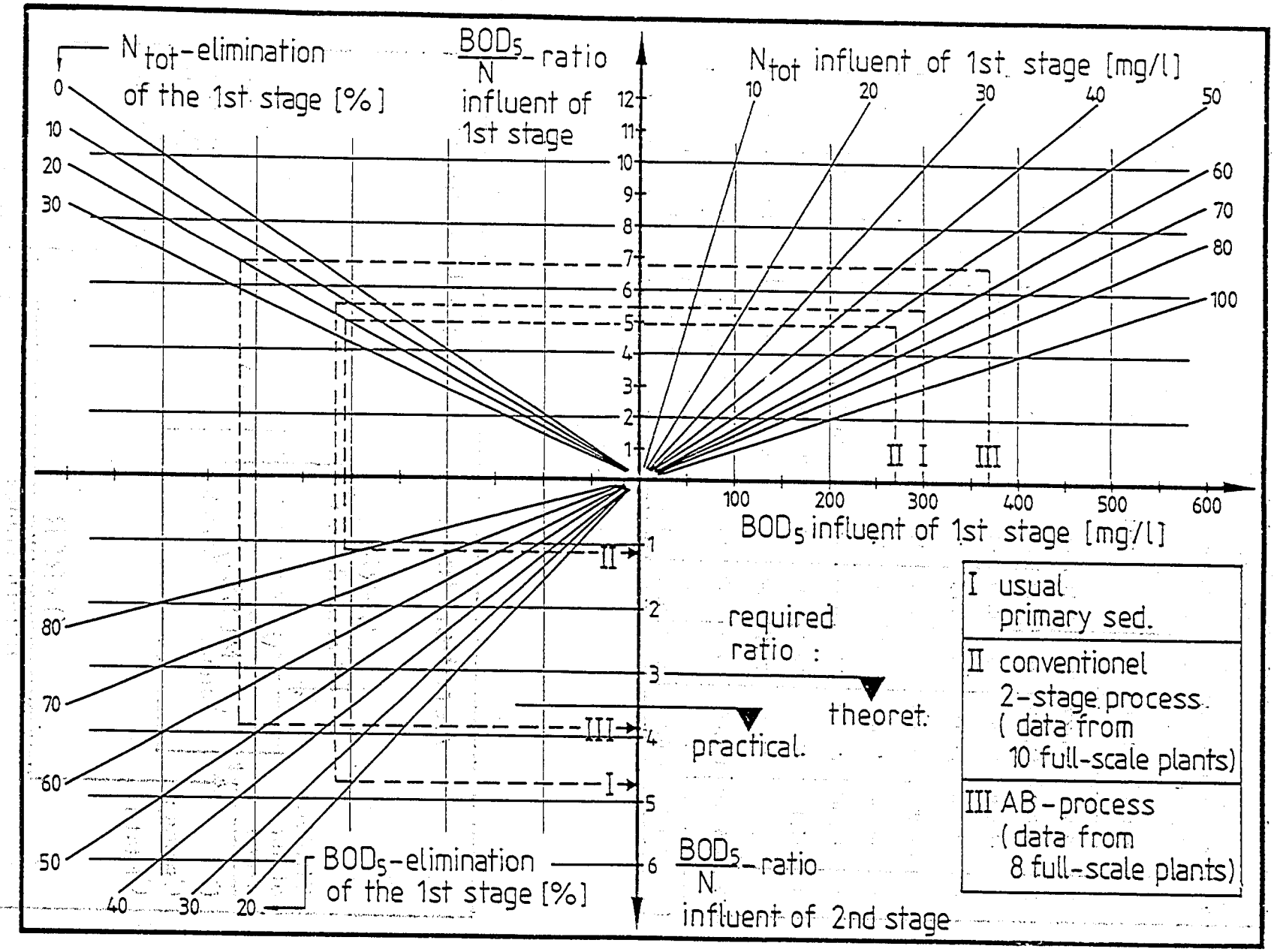

N tot-elimination of the 1st stage [%]
BOD5/N-ratio influent of 1st stage
N tot influent of 1st stage [mg/l]
BOD5 influent of 1st stage [mg/l]
BOD5-elimination of the 1st stage [%]
BOD5/N-ratio influent of 2nd stage
required ratio :
theoret.
practical.
I usual primary sed.
II conventionel 2-stage process (data from 10 full-scale plants)
III AB-process (data from 8 full-scale plants)

Figure 5

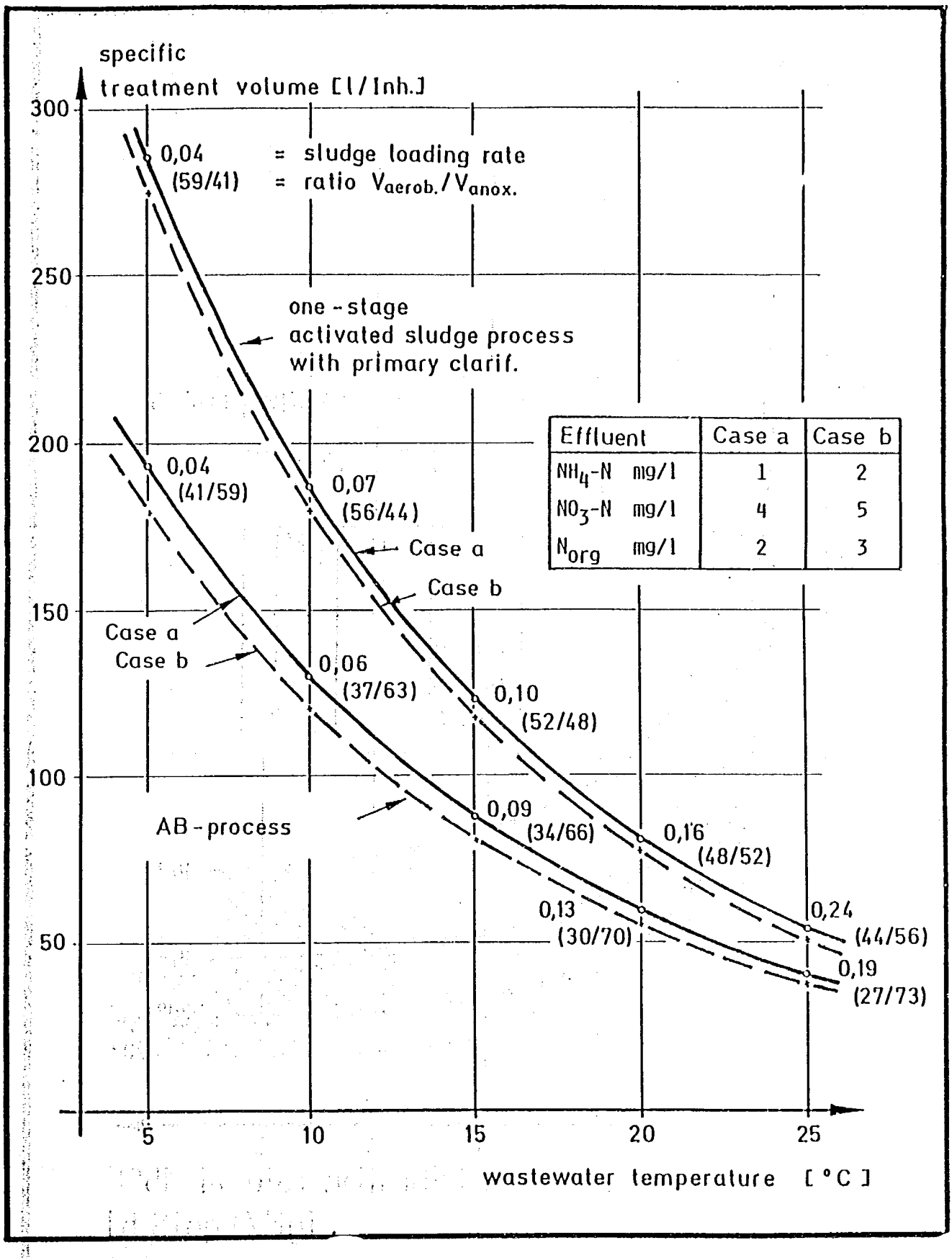

specific
treatment volume [l/Inh.]
300
250
200
150
100
50
0,04 = sludge loading rate
(59/41) = ratio $V_{aerob.}/V_{anox.}$
one-stage
activated sludge process
with primary clarif.
0,04
(41/59)
0,07
(56/44)
Case a
Case b
Case a
Case b
0,06
(37/63)
0,10
(52/48)
AB-process
0,09
(34/66)
0,16
(48/52)
0,13
(30/70)
0,24
(44/56)
0,19
(27/73)
Effluent | Case a | Case b
NH_4-N mg/l | 1 | 2
NO_3-N mg/l | 4 | 5
N_{org} mg/l | 2 | 3
5
10
15
20
25
wastewater temperature [°C]

Figure 6

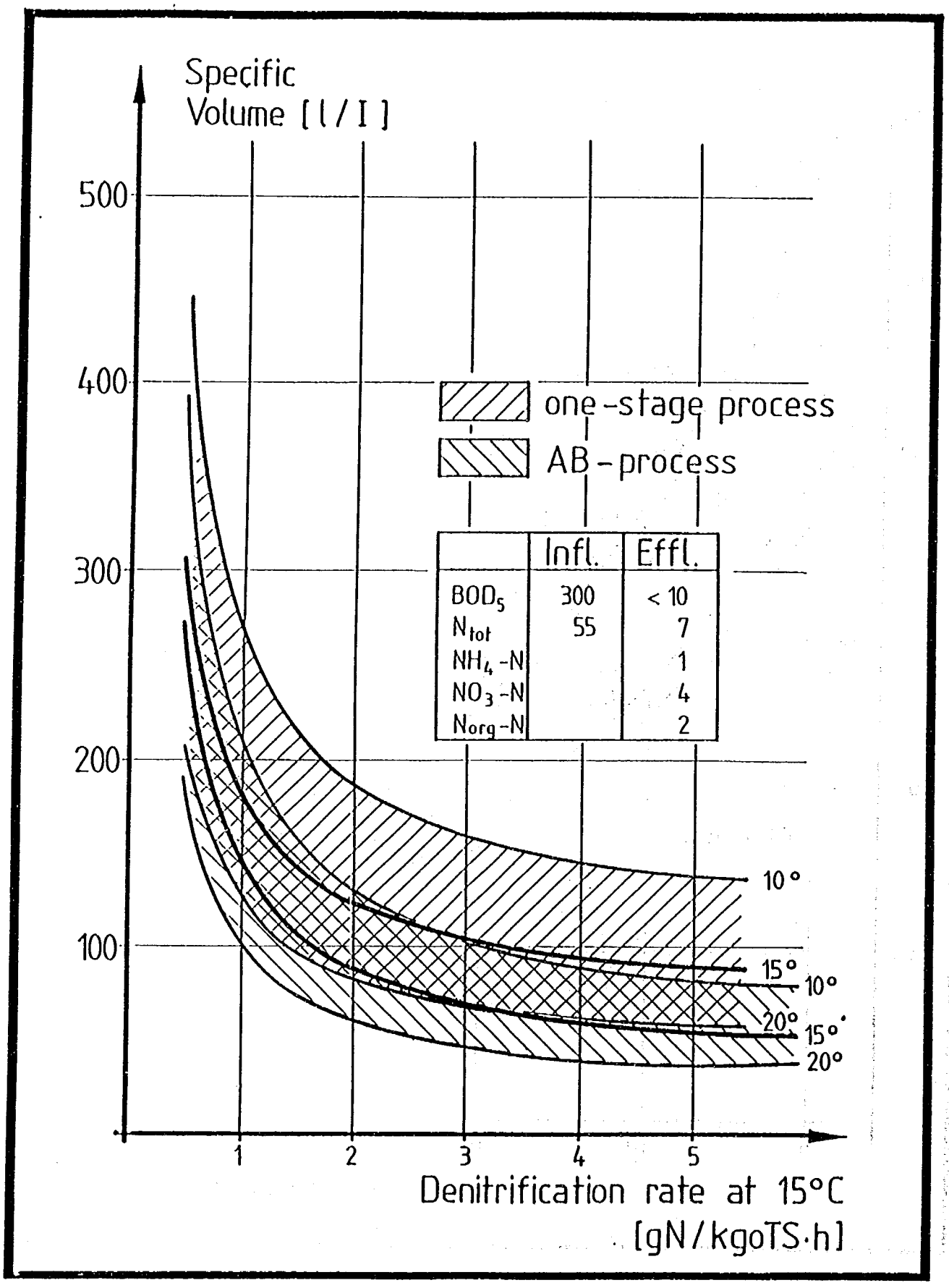

Specific
Volume [l / I]
500
400
300
200
100
one-stage process
AB-process
Infl.
Effl.
BOD5
300
< 10
Ntot
55
7
NH4-N
1
NO3-N
4
Norg-N
2
10°
15°
10°
20°
15°
20°
1
2
3
4
5
Denitrification rate at 15°C
[gN/kgoTS·h]

Figure 7

Attachment 1

			one-stage	two-stage
1	Assumption of the influent and effluent concentrations			
	$C_{BOD5,0}$	mg/l	300	300
	$C_{Ntot,0}$	mg/l	55	55
	$C_{NH4-N,e}$	mg/l	1	1
	$C_{NO3-N,e}$	mg/l	4	4
	$C_{Norg,e}$	mg/l	2	2
2	Assumption of the wastewater flow and other values			
	Q	l/Inh· d	200	200
	q_{24}	l/Inh· h	8,33	8,33
	q_{14}	l/Inh· h	14,29	14,29
	T	°C	15	15
	MLSS	g/l	3,5	3,5
3	Assumption of the treatment efficiency of the 1st stage (primary or A-stage)			
	$\eta_{BOD5,1}$	%	25	55
	$\eta_{Ntot,1}$	%	10	18
4	Calculation the influent concentrations of the 2nd stage (activated sludge or B-stage)			
	$C_{BOD5,1} = C_{BOD5,0} \cdot (1-\eta_{BOD5,1})$	mg/l	225	135
	$C_{Ntot,1} = C_{Ntot,0} \cdot (1-\eta_{Ntot,1})$	mg/l	50	45
5	Calculation of the N-elimination by incorporation into the excess sludge			ES_I after MÖLLER; DIERING $f_{N,inc}$ after KAPP
	ES_I	gSS/Inh· d	35	12
	IL	gVSS/gSS	0,65	0,70
	$oES_I = IL \cdot ES_I$	gVSS/Inh· d	35· 0,65 =22,75	12· 0,70 =8,40
	$f_{N,inc}$	gN/gVSS	0,09	0,09
	$C_{N,inc,2} = oES_I \cdot f_{N,inc} \cdot Q/1000$	mg/l	$\frac{22,75 \cdot 0,09}{200/1000}$ =10,24	$\frac{8,4 \cdot 0,09}{200/1000}$ =3,78

Attachment 1 (cont.)

			one-stage	two-stage
6	Assumption of the growth rate of nitrifiers and its dependency on temperature			after KAYSER and EPA-Manual
	μ T μ_T	d-1 °C d-1	0,47· 1,103(T-15) 15 0,47	0,47· 1,103(T-15) 15 0,47
7	Calculation of the theoretically and practically necessary aerobic sludge age			SF after GUJER and SCHLEGEL
	$t_{SS,theor.} = 1/\mu_T$	d	1/0,47 =2,13	1/0,47 =2,13
	Safety factor SF	-	3	3
	$t_{SS,pract.} = SF \cdot t_{SS,theor.}$	d	6,39	6,39
8	Resulting average retention time in the aerobic section			
	$t_{aerob} = tss \cdot ESI / (q_{24} \cdot MLSS)$	h	$\frac{6,39 \cdot 35}{8,33 \cdot 3,5}$ =7,67	$\frac{6,39 \cdot 12}{8,33 \cdot 3,5}$ =2,63
9	Calculation of the N-concentration to be nitrified			
	$C_{N,nitr} = C_{Ntot,1} - C_{N,inc} - C_{NH4-N,e} - C_{Norg,e}$	mg/l	50-10,24-1-2 =36,76	45-3,78-1-2 =38,22
10	Assumption of a basic nitrification rate			after PÖPEL and HENZE & HARREMOES
	v_N	gN/kgVSS· h	3	(3)
11	Calculation the nitrifier fraction of the biomass			after EPA-Manual and HENZE & HARREMOES
	$f_N = \frac{1}{\frac{C_{BOD5,1}}{C_{Ntot,1}} \cdot 3,67+1}$	%	$\frac{1}{(225/50) \cdot 3,67+1}$ =5,71	$\frac{1}{(135/45) \cdot 3,67+1}$ =8,33
	Correctionfactor f	-	1	8,33/5,71 =1,46
12	Calculation of the actual nitrification rate			
	v_N	gN/kgVSS· h	3	3· 1,46 =4,37

			one-stage	two-stage
13	Calculation of the temperature dependency of the nitrification rate			after EPA-Manual
	$v_{N,T} = v_N \cdot 1,103^{(T-15)}$	gN/kgVSS·h	$3 \cdot 1,103^{(15-15)} = 3$	$4,37 \cdot 1,103^{(15-15)} = 4,37$
14	Calculation of the average retention time in the aerobic section using the actual nitrification rate			
	$t_{aerob} = \frac{C_{N,nit}}{v_{N,T} \cdot MLSS \cdot IL}$	h	$\frac{36,76}{3 \cdot 3,5 \cdot 0,65} = 5,39$	$\frac{38,22}{4,37 \cdot 3,5 \cdot 0,70} = 3,57$
15	Selection of the crucial average retention time from 8 and 14			
	t_{aerob}	h	7,67	3,57
16	Calculation of the specific nitrification volume			
	$V_{spec,aerob} = t_{aerob} \cdot q_{24}$	l/Inh	$7,67 \cdot 8,33 = 63,89$	$3,57 \cdot 8,33 = 29,75$
17	Calculation of the actual aerobic sludge age			
	$t_{SS,act} = V_{spec,aerob} \cdot \frac{MLSS}{ESI}$	d	$63,89 \cdot 3,5/35 = 6,39$	$29,75 \cdot 3,5/12 = 8,68$
18	Calculation of the actual safety factor			
	$SF_{act} = t_{SS,act}/t_{SS,theor}$		$6,39/2,13 = 3$	$8,68/2,13 = 4,07$
19	Calculation of the N-concentration to be denitrified			
	$C_{N,deni} = C_{N,tot,1} - C_{N,inc} - C_{NH4-N,e} - C_{NO3-N,e} - C_{Norg,e}$	mg/l	50-10,24-1-4-2 = 32,76	45-3,78-1-4-2 = 34,22

Attachment 1 (cont.)

			one-stage	two-stage
20	Assumption of a denitrification rate			after HENZE & BUNDGAARD
	v_D	gN/kgVSS· h	2	2
21	Calculation of the temperature dependency of the denitrification rate			after BARNES & BLISS
	$v_{D,T} = v_D \cdot 1{,}07^{(T-15)}$	gN/kgVSS· h	$2 \cdot 1{,}07^{(15-15)} = 2$	$2 \cdot 1{,}07^{(15-15)} = 2$
22	Calculation of the necessary average retention time in the anoxic section			
	$t_{anox} = \frac{C_{N,deni}}{v_{D,T} \cdot MLSS \cdot IL}$	h	$\frac{32{,}76}{2 \cdot 3{,}5 \cdot 0{,}65} = 7{,}20$	$\frac{34{,}22}{2 \cdot 3{,}5 \cdot 0{,}70} = 6{,}98$
23	Calculation of the specific denitrification volume			
	$V_{spec,anox} = t_{anox} \cdot q_{24}$	l/Inh	$7{,}20 \cdot 8{,}33 = 60{,}00$	$6{,}89 \cdot 8{,}33 = 58{,}20$
24	Calculation of the necessary circulation ratio when using preapplied denitrification			
	$\eta_{deni} = C_{N,deni}/C_{N,nitr}$	-	32,76/36,76 =0,89	34,22/38,22 0,90
	CR $= \eta_{deni}/(1-\eta_{deni})$	-	0,89/(1-0,89) =8,09 (!)	0,90/(1-0,9) =9 (!)
25	Check of the ratio BOD_5/N in the influent			
	$= C_{BOD5,1}/C_{Ntot,1}$	-	225/50 =4,50	135/45 3,00 (!)
26	Check of the ratio BOD_5/N to be denitrified			
	$= C_{BOD5,1}/C_{N,deni}$		225/32,76 =6,87	135/34,22 =3,95

			one-stage	two-stage
27	Calculation of the retention times of the average wastewater flow during daytime			
	$t_{aerob,14} = V_{spec,aerob}/q_{14}$	h	63,89/14,29 =4,47	29,75/14,29 =2,08
	$t_{anox,14} = V_{spec,anox}/q_{14}$	h	60,00/14,29 =4,20	58,20/14,29 =4,07
	$t_{total,14}$	h	8,67	6,15
28	Calculation of the sludge loading rate			
	SLR	$\frac{kgBOD_5}{kgMLSS \cdot d}$	$\frac{0,200 \cdot 225}{123,89 \cdot 3,5}$ =0,104	$\frac{0,200 \cdot 135}{87,95 \cdot 3,5}$ =0,088
29	Calculation of the total specific volume			
	$V_{spec,tot}$	l/Inh	123,89	87,95
30	Calculation of the actual total sludge age			
	$t_{ss,tot} = \frac{V_{tot} \cdot MLSS}{ESI}$	d	$\frac{123,89 \cdot 3,5}{35}$ 12,4	$\frac{87,95 \cdot 3,5}{12}$ 25,7

Abbreviations and Symbols

C	: Concentration	(mg/l)
1. Index:		
	BOD_5 : Biochemical Oxygen Demand within 5 days	
	N_{tot} : total nitrogen	
	N_{org} : organic nitrogen	
	$N_{,inc}$: Nitrogen, incorporated into the excess sludge (related to the inflow)	
	NH_4-N : Ammonium-nitrogen	
	NO_3-N : Nitrat-nitrogen	
	$N_{,nitr}$: concentration of nitrogen to be nitrified	
	$N_{,deni}$: concentration of nitrogen to be denitrified	
2. Index:		
	o : influent	
	1 : effluent of 1st stage	
	e : effluent of 2nd stage	
η	: Efficiency	(%)
1. Index:		
	see above	
2. Index:		
	1 : Primary clarifier or A-Stage	
	2 : Activated sludge stage or B-stage	
V_{spec}	: specific Volume	(l/Inh)
$V_{spec,aerob}$	: specific aerobic Volume	(l/Inh)
$V_{spec,anox}$	: specific anoxic Volume	(l/Inh)
v_N	: nitrification rate	(mgN/gVSS· h)
v_D	: denitrification rate	(mgN/gVSS· h)
MLSS	: Mixed Liquor Suspended Solids concentration	(g/l)
MLVSS	: Mixed Liquor Volatile Suspended Solids concentration	(g/l)
t_{SS}	: sludge age	(d)
$t_{SS,aerob}$	: aerobic sludge age	(d)
SF	: Safety Factor	(-)
ESI	: Excess Sludge production	(gSS/Inh· d)
oESI	: organic Excess Sludge production	(gVSS/Inh· d)
IL	: Ignition Loss	(-)
μ	: growth rate (of the nitrifiers)	(d^{-1})
T	: Temperature	(°C)
f_N	: fraction of nitrifiers on the biomass	(%)
$f_{N,inc}$	: part of nitrogen on the organic mass of excess sludge	(%)
F	: Faktor for the consideration of an higher nitrifier fraction	(-)
t_{aerob}	: aerobic Volume retention time	(h)
t_{anox}	: anoxic Volume retention time	(h)
Q	: specific wastewater flow	(l/Inh· d)
q_{24}	: average wastewater flow during the whole day	(l/Inh· h)
CR	: Circulation Ratio	(-)
SLR	: Sludge Loading Rate	$\frac{(kgBOD_5}{kgMLSS.d)}$

PURE CULTURE OPERATION OF ANAEROBIC AEROBIC PROCESS FOR BIOLOGICAL PHOSPHORUS REMOVAL

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A B S T R A C T

It is required to determine and isolate the predominant bacterial species which are responsible for the phosphorus removal in the anaerobic aerobic process in order to clarify the biological mechanism of phosphorus accumulation in this process. A successful operation of anaerobic aerobic process with pure culture of Acinetobacter and Pseudomonas was attempted. Some problems involved in the pure culture operation were investigated using activated sludge mixed culture.

Acinetobacter did not exhibit the release of phosphorus and the uptake of organic matter under the anaerobic conditions, which are the metabolic characteristics of the anaerobic aerobic sludge. The precultivation on the strong medium with rapid growth rate and the elimination of sedimentation step which was forced by the poor settleability of microorganisms gave an adverse effect on phosphorus removal. The relationship among the phosphorus content of the sludge, anaerobic phosphorus release and anaerobic uptake of organic matter was presented.

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1. Introduction

In recent times, phosphorus removal from wastewater, which is considered essential for the prevention of rapid eutrophication, has become one of the "hot" topics of research in the field of environmental Engineering. Many kinds of phosphorus removal processes have been proposed and studied. Among them, much attention has been paid to biological phosphorus removal processes, because these are more energy-saving and economical than chemical processes. The anaerobic aerobic activated sludge process was developed as a biological phosphorus removal process in 1970's.[1) 2)] It is characterized by the presence of an absolute anaerobic stage before the aeration basin.

A lot of investigation has been made into the basic mechanism of the phosphorus removal in the anaerobic aerobic process. However, the biological mechanism of the phosphorus removal is not yet clear. Even the bacterial species that are responsible for the phosphorus removal are not known with surety.

The anaerobic aerobic process is already a practical technique. Good phosphorus removal efficiency has been reported when this technique has been applied to the municipal sewage treatment. However, when it is applied to the treatment of nightsoil or some kinds of industrial wastewater, instability resulted.[3)] In order to establish the anaerobic aerobic process as a stable method for phosphorus removal, it is required to describe what occurs in the process or in the cells of the sludge microorganisms and to clarify the biological mechanism of phosphorus accumulation.

For the purpose described above, the determination and the isolation of the predominant species which are responsible for the phosphorus removal in the anaerobic aerobic process would be very useful. In the present study, a successful operation of the anaerobic aerobic process using a

bacterial pure culture was attempted and some problems involved were investigated.

2. Literature review and Scope of the Study

2-1 Metabolic Characteristics of Anaerobic Aerobic Process

The anaerobic aerobic process for the biological phosphorus removal is defined as an activated sludge process which includes an absolute anaerobic zone prior to aeration where the influent wastewater contacts with the activated sludge. The phosphorus removal efficiency in the activated sludge process can be estimated according to a mass balance in a steady state as expressed in equation (1).[1) 4)]

$$\eta = \Delta P/\Delta TOC = Y \cdot Px \quad (1)$$

η : phosphorus removal efficiency
(mgP removed/mgTOC removed)
ΔP : Removed phosphorus (mgP/day)
ΔTOC : Removed TOC (mgTOC/day)
Y : Biomass yield
(mgMLSS produced/mgTOC removed)
Px : Phosphorus content in the sludge
(mgP/mgMLSS)

Equation (1) shows that the increase in Px is equivalent to the improvement of the phosphorus removal efficiency of the process. Several investigators have indicated[1) 5)] that the phosphorus content of the sludge tends to increase when the sludge undergoes repeated recirculation through the anaerobic and aerobic stage and that the absolute anaerobic stage is the key to the high phosphorus removal efficiency obtained.

This sludge that is produced in the anaerobic aerobic process having a high phosphorus content will henceforth be referred to as "anaerobic aerobic sludge". The metabolic characteristics of the anaerobic aerobic sludge are summa-

rized in the following points.[6]

(1) Accumulation of polyphosphate in the cell under aerobic conditions.

(2) Release of phosphorus under anaerobic conditions.

(3) Uptake of organic matter under anaerobic conditions.

This high phosphorus content and good phosphorus removal of the anaerobic aerobic sludge is due to polyphosphate accumulation. Under anaerobic conditions, the organic substrates are uptaken into the microorganisms and at the same time orthophosphate, which is generated from the degradation of polyphosphate, is released from the sludge. As a possible explanation to this phenomenon, the following theory is generally accepted.[2) 7)] The anaerobic aerobic sludge is composed of some particular microorganisms that have the ability to utilize the organic substrates under anaerobic conditions. The polyphosphate accumulated in such microorganisms serves as the energy source for the uptake of organic substrates under anaerobic conditions, which is possible because polyphosphate is a thermodynamically "high-energy" compound. The release of phosphorus under the anaerobic conditions is a result of the polyphosphate utilization. This hypothesis is not verified experimentally.

A survey of available literature indicates that the phosphorus content of the sludge is very high and good phosphorus removal is achieved, if both the release of phosphorus and the uptake of organic substrates are observed simultaneously in the anaerobic zone. However the relationship between the polyphosphate accumulation and the anaerobic metabolism in the anaerobic aerobic sludge is not clear. The essential factors which are required for the phosphorus accumulation remain to be clarified.

2-2 Phosphorus Accumulating Bacteria

It is recognized today that the high phosphorus accumulation in activated sludge is essentially a biological phenomenon. However, the bacterial population involved here are not yet clear.

Many researchers have tried to determine and/or isolate the bacterial species which are responsible for the phosphorus removal in the anaerobic aerobic process. Till now, Acinetobacter spp. have been recognized as being capable of accumulating phosphorus.[8)9)10)] Acinetobacter spp. are observed in almost all the anaerobic aerobic processes investigated and in relatively high numbers.[10)11)] They are able to accumulate 7-10% phosphorus in their cells and no other bacterial species have such a high phosphate accumulating capacity.[12)] However, large number of Acinetobacter do not always guarantee a good phosphorus removal[13)] and in some high phosphorus removing plants, large numbers of bacterial genera other than Acinetobacter, such as Pseudomonas[13)14)] or Aeromonas[14)15)], have been observed.

The successful and continuous operation of the anaerobic aerobic process by using only a pure culture of Acinetobacter or the mixture of some pure cultures has not been achieved yet. Some reports[16)] indicate that the release of phosphorus and the uptake of organic substrates are not observed in a type strain culture of Acinetobacter under the anaerobic conditions. We have to determine whether Acinetobacter is responsible for phosphorus removal or not and find the suitable conditions for the particular bacterial species to accumulate polyphosphate in the cells.

2-3 Scope of the Study

In the present study, two type cultures of bacteria which are appearing in the anaerobic aerobic process, Acinetobacter and Pseudomonas, were prepared and constant operation of the anaerobic aerobic process was performed using the pure culture of these two bacteria. The possibility of Acinetobacter being responsible for phosphorus removal is discussed.

Some anaerobic aerobic operations with activated sludge mixed culture were carried out and the differences between the pure culture operation and the activated sludge operation were investigated experimentally. The problems involved in the realization of the successful phosphorus removal by pure culture are discussed.

The characteristics of the anaerobic aerobic sludge are discussed from the results of the mixed culture operation. The deterioration process of phosphorus removal are also discussed.

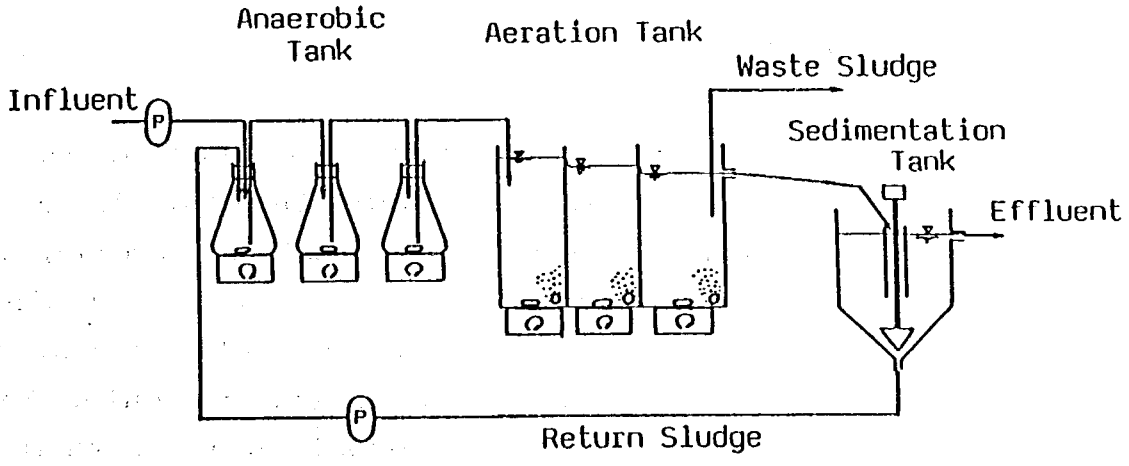


Fig. 1. Laboratory Unit of Anaerobic Aerobic Activated Sludge Process

3. Materials and Methods

3-1 Microorganisms

The type strains used in the present study were Acinetobacter calcoaceticus IAM12087 and Pseudomonas ovalis IAM1002 which are preserved in the Institute of Applied Microbiology, the University of Tokyo. Both of them were kept under freeze-dried conditions. They were plated on suitable agar mediums, then inoculated to liquid mediums for growth and incubated for a few days. When a sufficient amount of bacteria was obtained, it was transferred to the fermenter as described in the next section and anaerobic aerobic operation started. The organic components of the growth medium were almost the same as those of the substrates for the experimental unit.

Activated sludge was originally obtained from a sewage treatment plant in Tokyo. It was cultivated as a stock sludge in a laboratory scale unit of the anaerobic aerobic process as shown in Fig. 1.

3-2 Experimental Unit

A 1.6 liter fermenter was used as a reactor for the anaerobic aerobic operation as shown in Fig. 2. It was controlled by a sequence controller in semi-batch mode, viz., the addition of feed, anaerobic mixing, aeration, pumping of excess sludge, sedimentation and discharge of effluent were repeated periodically according to a preset schedule. Approximately, the initial 1 hour was anaerobic stage with mixing only and the following 2 hours were aerobic stage with mixing and aeration. During sedimentation, both mixing and aeration was stopped.

In some cases, especially in case of pure culture, the settleability of the microorganism was very poor. Therefore

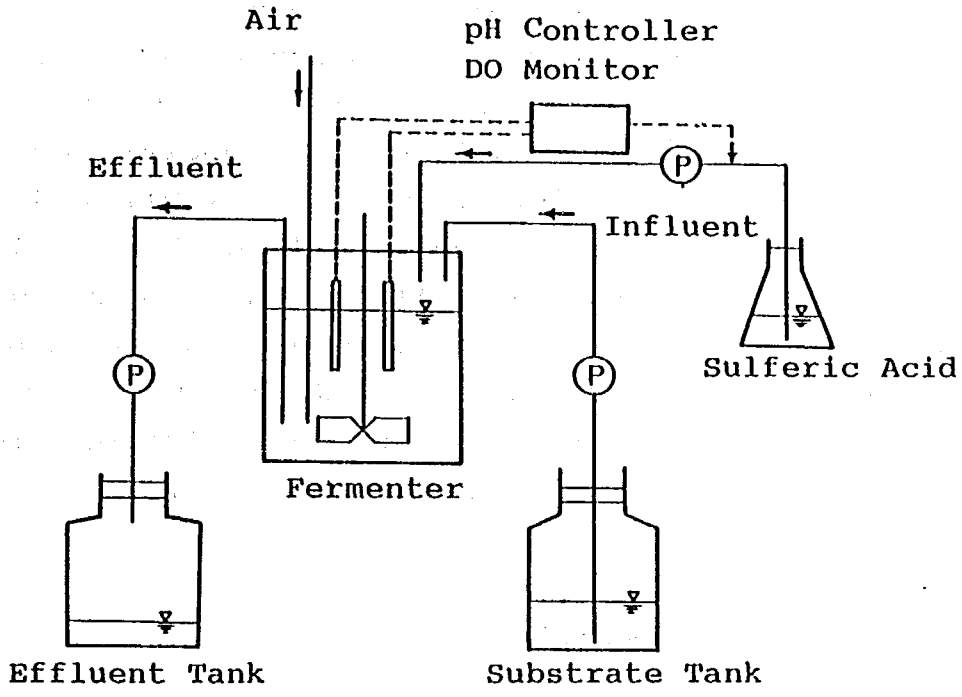
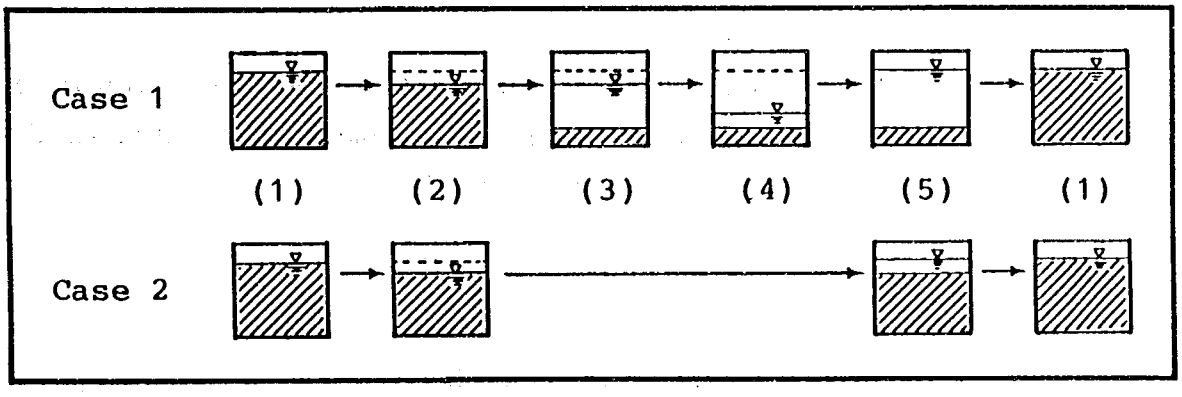


Fig. 2. Experimental Unit



(1) Anaerobic and Aerobic Stage
(2) Excess Sludge Drawing (3) Sedimentation
(4) Discharge of Effluent (5) Inflow of Substrate

Fig. 3. Difference between the Operation with and without Sedimentation Step

it was impossible to separate microorganisms from liquid by sedimentation. In such cases, the discharge of effluent water was abandoned and only the excess sludge was drawn by pumping. The difference between the operations with and without the sedimentation process is illustrated in Fig. 3. In the operation with sedimentation (case 1), dilute substrate solution was fed, but in the operation without sedimentation (case 2), a smaller volume of concentrated substrate solution was supplied and the hydraulic retention time was prolonged. As a result, the dilution rate of the process became much higher in Case 1 than in Case 2.

3-3 Experimental Conditions

Five experimental runs were carried out, two runs with pure culture and three with activated sludge. The experimental conditions are summarized in Table 1. The substrate compositions used in each run are shown in Table 2.

3-4 Analytical Methods

The analytical methods used in the present study are summarized in Table 3. The supernatant sample was obtained by filtration with Toyo Filter Paper No.5C.

Table 3. Analytical Methods

	Method	Reference, Equipment
PO4-P	Ascorbic Acid Method	Standard Methods
T-P	Potassium Persulfate Degradation	Standard Methods
COD	Potassium Dichromate Oxydation	Japan Industrial Standard 0102
TOC	TOC Analyzer	Shimadzu TOC 500
Acetate	Gaschromatography	Shimadzu GC-9A
MLSS	Weight Method	

Table 1. Experimental Conditions

Run	Micro-organism	Duration from to	Cycle (hrs) An	Ox	Sed	T	HRT (day)	SRT (day)	P/COD in feed (mg/mg)	COD Load (*1)	Note
1	Acineto-bacter	'84, 28 Nov-28 Dec	1.0	3.0	0.0	4.0	4.0	4.0	0.005 ∫ 0.015	190	Pure Culture
2	Pseudo-monas	'84, 09 Aug-19 Aug	1.0	2.0	0.0	3.0	6.25	6.25	0.025	130	Pure Culture
3 A	Labo. A.A.S. (*2)	'85, 01 May-11 May	1.0	2.0	1.0	4.0	8.0	8.0	0.009	170	No Precultivation
B											With Precultivation
4 A	A.S.Prl. (*3)	'85, 13 Aug-09 Sep	0.75	1.5	0.75	3.0	0.33 ∫ 0.67	6.0	0.038	180	Na 150mg/l
B											Na 1500mg/l
5 A	A.S.Prl. (*3)	'85, 18 Nov-28 Dec	0.92	1.42	0.67	3.0	0.33	6.0	0.038	180	With Sedimentation
B1		'85, 18 Nov-10 Dec					0.33	6.0	0.038		With Sedimentation
B2		'85, 11 Dec-19 Dec					6.0	6.0	0.031		No Sedimentation
B3		'85, 20 Dec-28 Dec					0.33	6.0	0.038		With Sedimentation

*1. mgCOD/l/cycle *2. Anaerobic Aerobic Sludge from Laboratory Unit

*3. Activated Sludge from Practical Plant

Table 2. Substrate Composition

Run	HAc (*1)	Glu	Pep	YEx	KH_2PO_4	Na_2HPO_4	$(NH_4)_2SO_4$	$MgSO_4 \cdot 7H_2O$	$CaCl_2 \cdot 2H_2O$	KCl	NaCl	COD	P
1	Na 10000	-	300	-	100 ~ 250	-	2000	500	100	-	-	4500	30 ~ 60
2	Na 4500	-	1000	1000	280	-	800	1700	300	1000	-	4000	100
3	Na 6000 K 2000	-	2000	2000	-	-	500	700	100	-	-	8000	72
4A	Na 550 (1100)	-	120 (240)	120 (240)	59 (118)	-	93 (186)	110 (220)	20 (40)	67 (134)	-	480 (960)	18 (36)
B	Na 550 (1100)	-	120 (240)	120 (240)	59 (118)	-	93 (186)	110 (220)	20 (40)	67 (134)	3800 (3800)	480 (960)	18 (36)
5A,B1,3	Na 160	70	240	100	60	-	90	110	20	60	200	480	18
B2	Free 1260	1260	4320	1800	250	1500	1000	55	20	-	-	7700	264

*1. Na:CH_3COONa K:CH_3COOK Free:CH_3COOH ( ) : After 29 Aug.

4. Experimental and Results

4-1 Pure Culture Study

(1) Acinetobacter calcoaceticus IAM12087 (Run 1)

Acinetobacter calcoaceticus IAM12087 was grown by the previously mentioned method and the anaerobic aerobic operation with Acinetobacter (Run 1) was started. The main carbon source was acetate. The sedimentation step was not provided, because Acinetobacter had very poor settleability. The operation was continued for about a month.

The filtered orthophosphate concentration in the fermenter and the phosphorus content of the cell throughout the run are shown in Fig. 4. During the initial period, the phosphate concentration in the fermenter was fairly high, although the COD:P ratio was kept as low as 100:0.5. This is because excess phosphate in the growth medium remained in the reactor. After several days' operation under the same conditions, the phosphate in the supernatant was completely taken up into the cells. Then the COD:P ratio was raised to 100:1.5. The phosphate began to leak to the supernatant. The phosphorus content of the cell increased up to 4.1% just after the rise of COD:P ratio, but it went down to the initial level soon.

A typical change of the concentration of orthophosphate and acetate during an anaerobic aerobic cycle is shown in Fig. 5. It is clear from Fig. 4 and 5 that both the release of phosphorus and the uptake of acetate under anaerobic conditions, the characteristic metabolism of the anaerobic aerobic sludge, are not observed even after Acinetobacter is exposed to the long-term anaerobic aerobic operation.

(2) Pseudomonas ovalis IAM1002 (Run 2)

The anaerobic aerobic operation with Pseudomonas ovalis IAM1002 (Run 2) was started in the same manner as Run 1. As

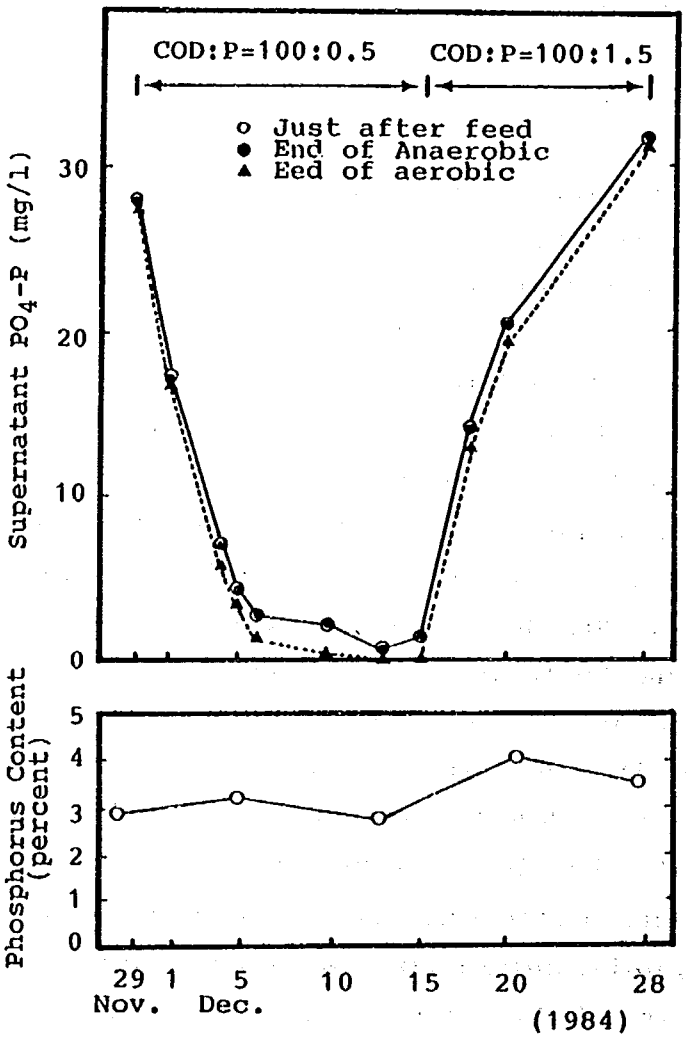


Fig. 4. Profile of PO_4-P and Phosphorus Content in Run 1

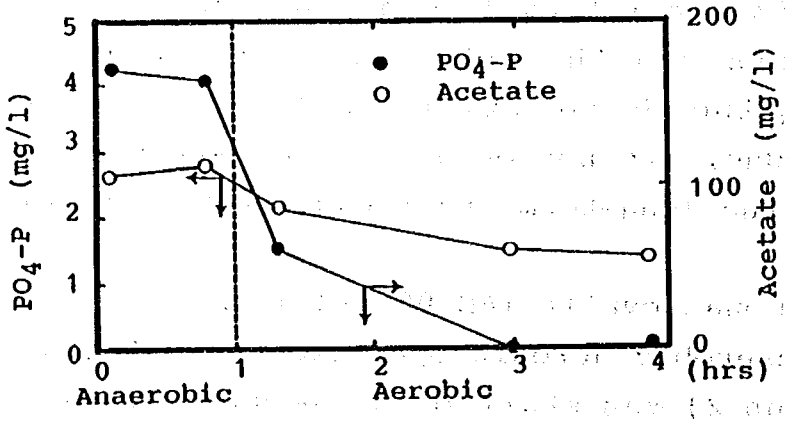


Fig. 5. Typical Change of PO_4-P and Acetate Concentration During One Cycle in Run 1

the settleability of Pseudomonas was not good, the sedimentation step was eliminated. After about one week of operation, a bacterial contamination occurred, which was noticeable from the fact that the apparent characteristics of the cells began to change and that at least two different kinds of bacteria were observed in the fermenter. Therefore the experiment was stopped after 10 days.

The phosphorus content of the cell was around 3.1% and it did not vary much throughout the experiment. A slight decrease in acetate concentration in the anaerobic stage was observed, but the simultaneous release of phosphorus from the cells did not take place.

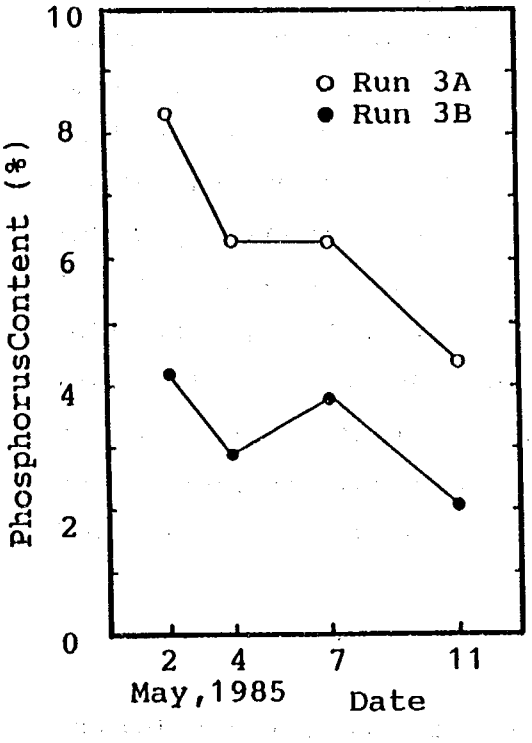


Fig. 6. Change of Phosphorus Content in Run 3

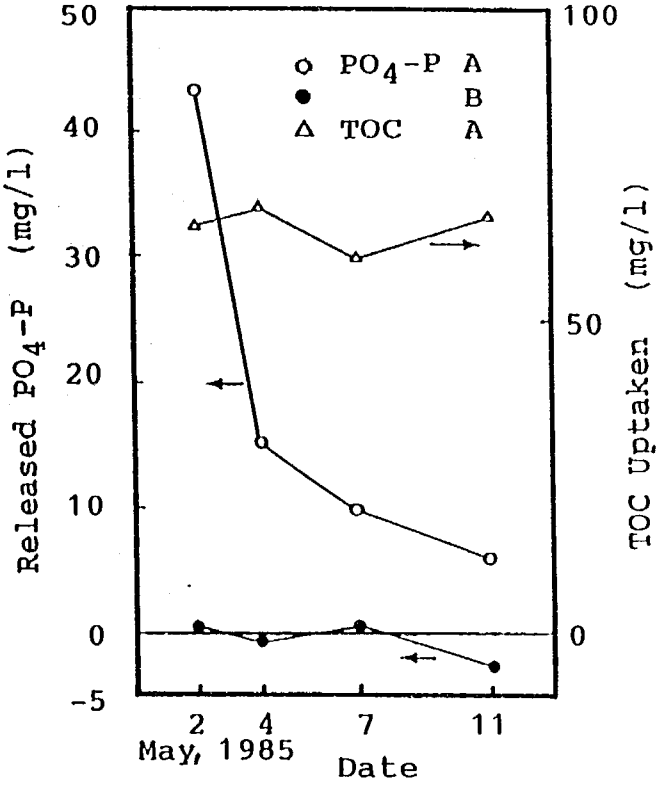


Fig. 7. Phosphorus Released and TOC Uptaken in Anaerobic Stage in Run 3

4-2 Influence of Cultivation Prior to Anaerobic Aerobic Operation (Run 3)

(1) Experimental

Anaerobic aerobic operation with pure culture requires considerable amount of cells for the start-up. It is, therefore, necessary to increase the bacteria on the growth medium with high substrate concentration prior to operation. During this kind of precultivation, the substrate concentration of the medium is kept much higher and the growth rate of cells is much larger than wastewater treatment systems. The ability of bacteria to accumulate large amount of phosphorus in their cells may vanish because of the exposure to high substrate concentration and the rapid growth during precultivation. Then, the influence of precultivation on phosphorus accumulation was examined using activated sludge (Run 3).

Two fermenter systems, A and B, were prepared for Run 3. System A was the control system, where the anaerobic aerobic sludge was transferred to the fermenter directly. In System B, the experimental system, a very small amount of the anaerobic aerobic sludge obtained from the laboratory unit was innoculated into the liquid medium containing high substrate concentration (3390mg/L in TOC). After a few days' precultivation, the whole medium was transferred to the fermenter. The organic composition of the precultivation medium was similar to that of the subsequent operation as shown in Table 2. System A and B were operated in anaerobic aerobic mode for 10 days. Sedimentation step was not provided for both System A and B.

(2) Results

The change of phosphorus content of the sludge during the anaerobic aerobic operation is shown in Fig. 6. The initial phosphorus content was 8.3% and 4.2% respectively for Run 3A and 3B. This means that there was marked reduction in

phosphorus content during precultivation. In the control system, Run 3A, a drastic reduction in phosphorus content from 8.3% to 4.4% was observed in 10 days. This indicates that the operational or environmental conditions in the control system was not suitable for phosphorus accumulation by microorganisms. In the experimental system Run 3B, the phosphorus content gradually decreased from 4.2 to 2.1%. High phosphorus accumulation in sludge was not observed in both Run 3A and 3B.

Phosphorus released and TOC uptaken during the anaerobic stage in Run 3 was plotted in Fig. 7. The uptake of organic matters was observed throughout the experiment in Run 3A. The observed TOC level in Run 3B was very high compared with its changes. Therefore, we cannot determine clearly whether TOC uptake occurred or not in the anaerobic stage of Run 3B. In the control system, Run 3B, phosphorus release was high in the anaerobic stage just after the start-up, but the amount of released phosphorus gradually decreased with time. Phosphorus release was not observed in Run 3B, in which the phosphorus accumulation to the sludge did not occur. The results from Run 3A and B shows that the sludge which has poor phosphorus accumulation capability uptakes organic matter without much release of phosphorus under anaerobic conditions.

4-3 Influence of High Sodium Concentration (Run 4)

(1) Experimental

The main carbon source in the pure culture operation was acetate, which was usually added as sodium acetate. Therefore much sodium ion was added to the culture together with acetate. Sodium ion is little utilized by microorganisms. It should be accumulated to a high concentration in the fermenter, if the dilution rate of the system is very small. It was

observed that sodium ion and some other mineral concentrations were very high in the operation without the sedimentation step. The influence of high sodium concentration on phosphorus accumulation was investigated (Run 4).

Two fermenter systems, A and B, were provided as control and experimental. Sodium ion concentration of system A was controlled at 150 ± 50 mg/l, and that of system B at 1500 ± 500 mg/l. Sodium level in System B was similar to that observed in the pure culture operation. Sedimentation step was provided for both systems, so the accumulation of inactive materials in the fermenter should have been small. The only difference between system A and B lay in the sodium ion concentration. The anaerobic aerobic operation in both systems was started by putting the laboratory anaerobic aerobic sludge into the fermenters, and operated for 25 days (Run 4).

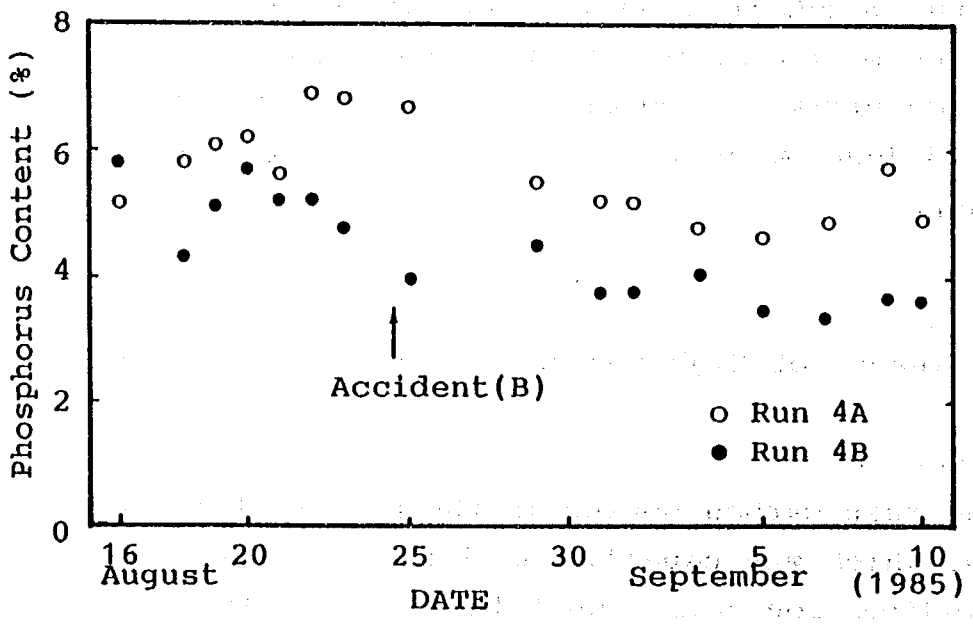


Fig. 8. Phosphorus Content of Sludge in Run 4

(2) Results

The phosphorus content of the sludge in Run 4 is shown in fig. 8. The profile of orthophosphate and acetate during one anaerobic aerobic cycle on the 24th day after start is shown in Fig. 9. In the low Na system (System A), good phosphorus accumulation in the sludge is observed throughout the run, which can be seen from the high phosphorus content of 6%. The release of phosphorus and the uptake of organic substrate in the anaerobic stage was prominent. It was observed that the sludge in System A continued to possess characteristics typical to the anaerobic aerobic sludge during the run.

In the high Na system (System B), the phosphorus content of the sludge was maintained as high as in System A initially. On the 10th day from the start, there was an accident in which a part of the sludge flew away from the fermenter and sludge concentration decreased. Subsequently, the phosphorus content began to decrease. However, even the sludge obtained at the end of Run 4B with a phosphorus content as low as 3.5% exhibited phosphorus release and uptake of organic substrate as shown in fig. 9. These results indicate that the high sodium concentration does not drastically affect the phosphorus accumulating capacity of the sludge, even if it is as high as 1500 mg/l. There remains a possibility that the phosphorus accumulation capacity of the sludge is damaged by a much longer exposure to high sodium concentration.

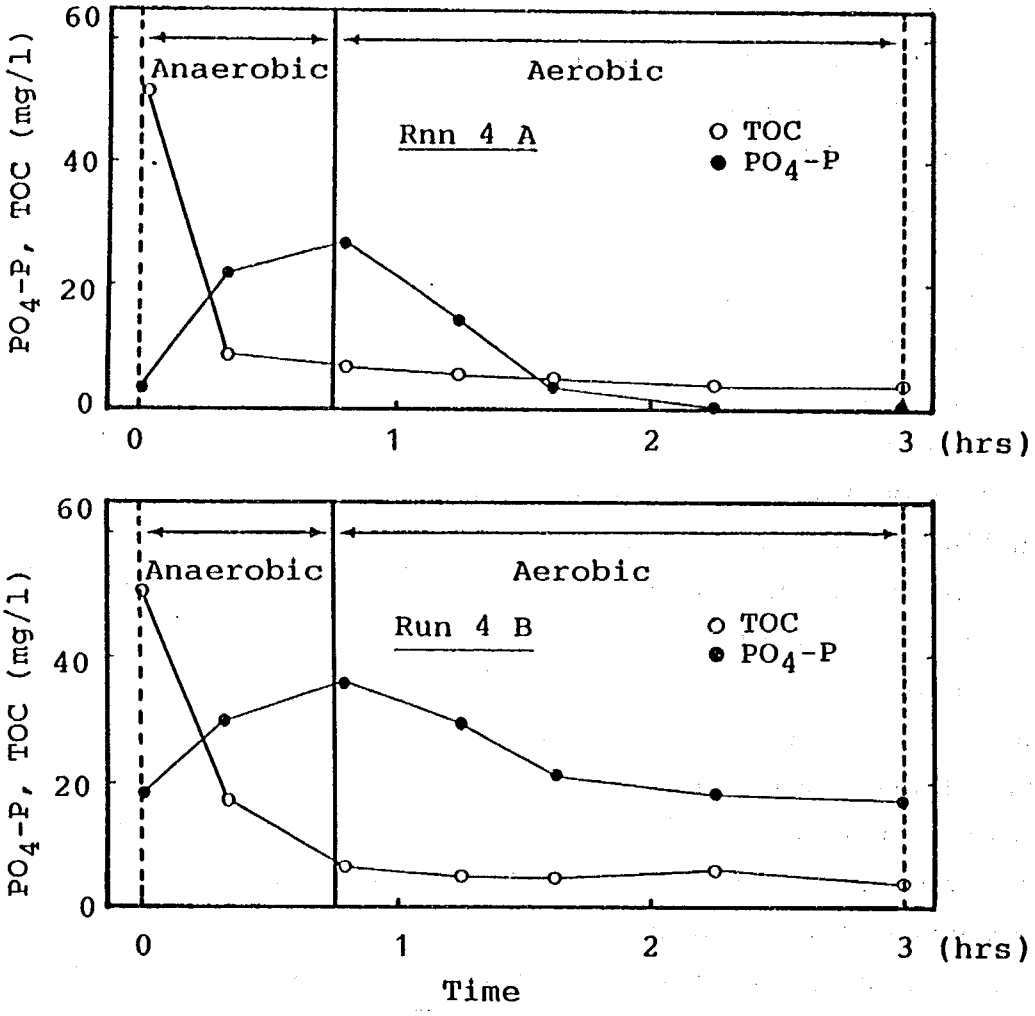


Fig. 9. Profile of PO_4-P and TOC during One Cycle on the 24th Day from Start-up in Run 4

4-4 Influence of Operation without Sedimentation Step (Run 5)

(1) Experimental

Sedimentation step can not be provided in most pure culture operations because of the poor settleability of microorganisms. In this section the influence of operation without sedimentation step on phosphorus removal was investigated experimentally.

Two fermenter systems were started with activated sludge obtained from a practical sewage treatment plant and operated in anaerobic aerobic mode for 43 days (Run 5). System A in Run 5 was the control system in which the sedimentation step was provided throughout the run. System B was operated under the same operational conditions as system A during the initial 23 days, where the sedimentation step was provided. After 23 days' operation, the sedimentation step was eliminated and the operational mode was changed from case 1 to case 2 as shown in Fig. 3. The organic loading per unit volume of fermenter was held constant, although the feed concentration was increased. Along with the change of operational mode the substrate composition was changed simultaneously as described below.

1) Sodium acetate was replaced by acetic acid so that much sodium ion might not be accumulated in the fermenter.

2) Loading of inorganic nutrients per unit volume of fermenter was reduced in order to prevent high accumulation of inorganic matter in the fermenter.

The operation without sedimentation step was continued for 9 days. On the 32nd day after start-up, the operational mode and the substrate composition of System B was returned to the initial conditions.

(2) Results

The experimental results of System A and B are summarized in Fig. 10. In system A, excellent phosphorus removal

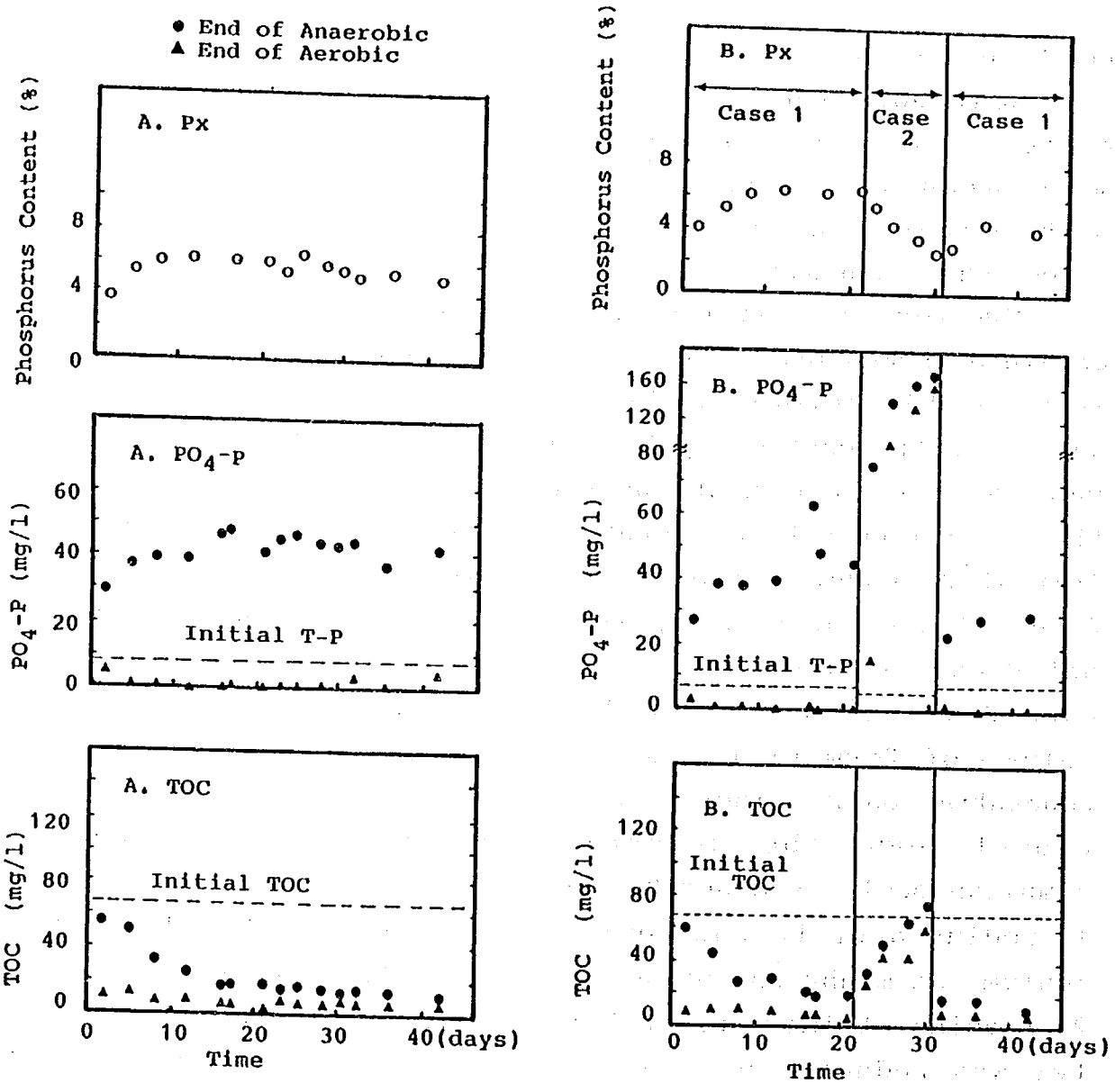


Fig. 10. Change of Px, PO_4-P and TOC in Run 5

was observed throughout the run. Phosphorus concentration at the end of aerobic stage was below 0.5 mgP/l in most cases. Phosphorus content of the sludge began to increase just after the start of operation and reached 6.1% in 8 days. After that it was maintained between 5.2 and 6.5%. TOC was largely removed in the anaerobic stage accompanied by release of phosphorus. System A in Run 5 was a typical example of the successful operation of anaerobic aerobic process.

System B also exhibited excellent phosphorus removal during the initial 23 days. The experimental results obtained during this period were almost the same as those in System A. When the sedimentation was stopped on the 23rd day, the decrease in phosphorus content of the sludge and the increase in PO_4-P and TOC in the supernatant was observed very clearly as shown in Fig. 10. The decrease in phosphorus content indicates that the ability to remove phosphorus was reduced by the elimination of sedimentation step. Release of phosphorus and uptake of organic substrate was observed even in the period in which the phosphorus removal was getting worse. When the sedimentation was started again on the 32nd day, phosphorus removal was enhanced immediately. The phosphorus content began to increase and PO_4-P and TOC in the supernatant was reduced to the previous level.

It was indicated that some changes in environmental conditions caused by the elimination of sedimentation step and/or the change of substrate composition gave a great negative effect on phosphorus accumulation in sludge microorganisms.

5. Discussion

5-1 Problems Involved in Pure Culture Operation

(1) Acinetobacter and Pseudomonas

Two type strains, Acinetobacter calcoaceticus IAM12087 and Pseudomonas ovalis IAM1002, did not release phosphate at all and did not uptake much organic substrate under anaerobic conditions, even when they were exposed to the anaerobic aerobic operation for days. Acinetobacter has been recognized as the predominant species of the anaerobic aerobic sludge.[8)10)] However, the observed metabolism of Acinetobacter examined here is completely different from that of the anaerobic aerobic sludge. The maximum phosphorus content of the cell was 4.1% and 3.1% respectively for Acinetobacter and Pseudomonas. These are lower than the phosphorus content of anaerobic aerobic sludge produced in laboratory units fed with synthetic wastewater, where the phosphorus content can get to 6-10%.[17)] Judging from the experimental results obtained, it seems that Acinetobacter calcoaceticus is not the bacterial species which is responsible for the removal of phosphorus.

As to the causes why Acinetobacter did not exhibit metabolism similar to the anaerobic aerobic sludge, the followings points can be made.

1) Acinetobacter is not the species which is responsible for phosphorus removal.

2) The type strain of Acinetobacter examined here had lost its ability to accumulate phosphorus because of the long-term preservation.

3) The conditions involved in the pure culture operation were not suitable for phosphorus accumulation.

We do not deal with the first two points in the present study. As to the third point, two factors are involved. First, the precultivation step should be provided for pure

culture operation. Secondly, the sedimentation step cannot be provided because of the poor settleability of pure culture.

(2) Precultivation

It was indicated from the results of the precultivation in Run 3 that rapid growth on strong medium changed the settleability of the sludge for worse. Only the microorganisms which had larger growth rate in the strong medium were selected by the precultivation procedure. Such selected microorganisms did not exhibit the characteristic metabolism of the anaerobic aerobic sludge. In order to isolate the bacterial species responsible for phosphorus removal from activated sludge and to operate the anaerobic aerobic process with it, it is necessary to increase the bacterial mass. Therefore we need to devise some kind of precultivation procedure in which the bacteria do not lose the ability to accumulate phosphorus.

(3) Elimination of Sedimentation Step

The influence of the elimination of sedimentation step was examined in Run 5. The amount of phosphorus removed during one cycle of the anaerobic aerobic operation is listed in Table 4. It is noticed that the amount of PO_4-P removed becomes smaller during the period in which sedimentation step was eliminated, although the amount of TOC removed was almost constant throughout the run. The ability to remove phosphorus

Table 4. Amount of Phosphorus Removed during One Cycle in Run 5

	Days from Start-up from to	Removed TOC in OneCycle (mg)	Removed Po4-P inOneCycle (mg)	Sedimentation
Run 5 A	13 - 43	118	10.2	Provided
B1	10 - 22	128	10.5	Provided
B2	24 - 31	118	5.3	Not Provided
B3	33 - 43	118	8.5	Provided

was undoubtedly reduced by the elimination of sedimentation step and accompanying changes. Similar results have been reported in a pilot plant test of anaerobic aerobic process, in which good phosphorus removal was not achieved in low dilution rate process.[18] The possible reasons for this are listed as follows.

1) Rise of inorganic salt concentration.
2) Accumulation of inactive organic matters.
3) Influence of acetic acid used instead of sodium acetate.

Although the inorganic nutrient concentration in the feed was a little decreased during the operation without sedimentation step, most of inorganic matter must have been accumulated in the fermenter because of the small dilution rate of the system. Concentration of Na and K were 230 mg/l and 100 mg/l respectively, which are not very high concentrations. Results from Run 4 shows that sodium concentration as high as 230 mg/l does not affect phosphorus accumulation. Therefore, the increase of sodium concentration can not be the cause of the deterioration of phosphorus removal capacity, although other inorganic materials might be the cause. In small dilution rate systems inactive organic matters could be also accumulated, which was observed in Run 5B and is shown in Fig. 10. Inactive organic matters are present in the substrate and they may be produced by the microorganisms in the fermenter. There is a possibility that high concentration of inactive organic matters has a inhibitory effect on phosphorus accumulation by microorganisms.

Acetic acid was used instead of sodium acetate during the operation without sedimentation in order to reduce the accumulation of sodium ion in the fermenter. Acetate exists as acetate ion in the pH range near neutral. Therefore there was no difference between acetic acid and sodium acetate as a substrate, if pH is kept around neutral. Reduction of alkalinity caused by the use of acetic acid might affect the phosphorus accumulation.

5-2 Characteristics of Anaerobic Aerobic Sludge

The anaerobic aerobic sludge is characterized by (1)high phosphorus content, and (2)ability to uptake organic matters accompanied by phosphorus release under anaerobic conditions. In this section, we will discuss whether high phosphorus content, anaerobic phosphorus release and anaerobic uptake of organic matter can be recognized as proper indicators of good phosphorus removal in anaerobic aerobic process. The ability of a sludge to remove phosphorus can be estimated by the ratio of removed phosphorus to removed TOC, which will be henceforth described as $\Delta P/\Delta C$ (mgP/mgC). Phosphorus content of the sludge, amount of phosphorus released in the anaerobic stage and TOC absorbed into the sludge in the anaerobic stage will be described as Px (Percent in Dry Matter), Rel-P (mgP/gMLSS) and Abs-C (mgC/gMLSS), respectively. The Relationship between $\Delta P/\Delta C$ and the three parameters, Px, Rel-P and Abs-C, are shown in Fig. 11, 12 and 13, where the data obtained from Run 5 and some additional from similar experiments with the same substrate as run 5 are plotted. In these figures, white circles represent the results obtained during the period in which phosphorus content of the sludge was decreasing drastically and dots represent the results in relatively stable period.

Under steady state conditions, the ratio $\Delta P/\Delta C$ should be proportional to phosphorus content Px as expressed in equation (1). In fact, a proportional relationship between $\Delta P/\Delta C$ and Px is observed in Fig. 11 as expressed by the following regression line, Equation (2), when some odd points are excluded.

$$Px = 46.8\ (\Delta P/\Delta C) + 1.20 \qquad (2)$$

It is reasonable that some points for sludges with decreasing phosphorus content, which are represented by white circles, appear apart from the regression line. High phosphorus content does not always indicate a good phosphorus removal

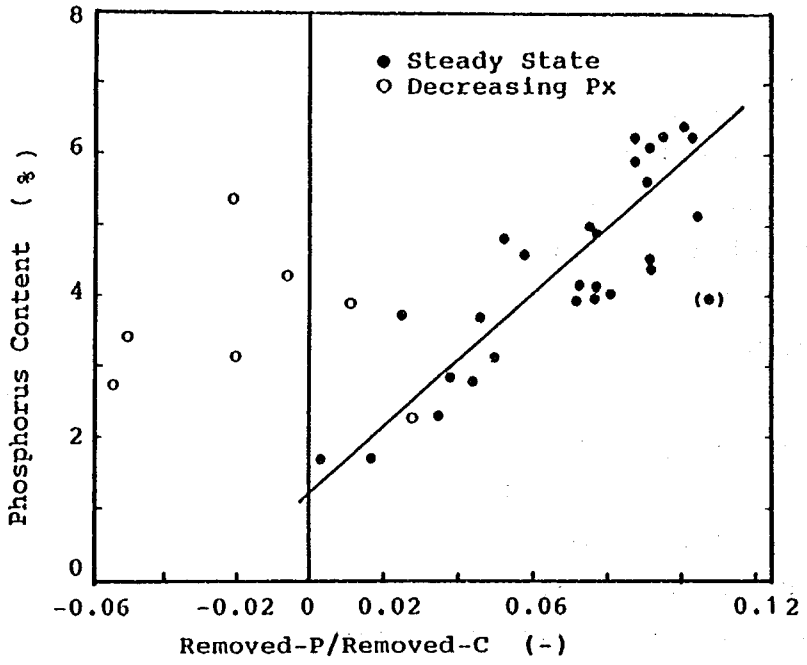


Fig. 11. Relationship between ΔP/ΔC and Px

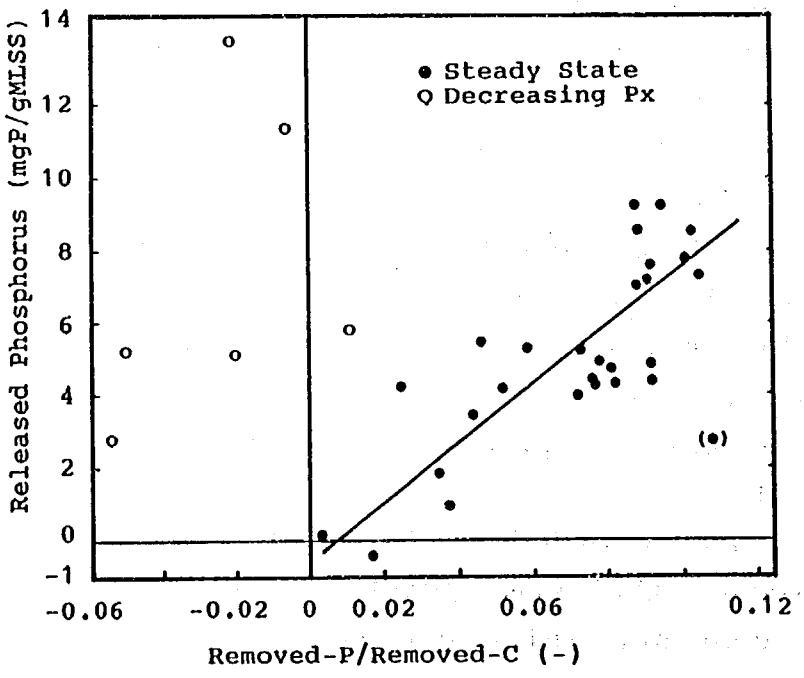


Fig. 12. Relationship between ΔP/ΔC and Rel-P

under unsteady state conditions. Similar relationship is shown in Fig. 12 between ΔP/ΔC and Rel-P. Generally, phosphorus content increased with the increase in anaerobically released phosphorus. The regression line expressing the steady state relationship between ΔP/ΔC and Rel-P is as follows.

$$\text{Rel-P} = 80.5\ (\Delta P/\Delta C) - 0.56 \qquad (3)$$

However there are some exceptions and all of them are the sludges with decreasing Px, which are represented by white circles in Fig. 12.

The relationship between ΔP/ΔC and Abs-C is quite different from the above as shown in Fig.13. The ratio of ΔP/ΔC does not affect Abs-C. In other words, the uptake of TOC in the anaerobic stage occurred regardless of the ability to remove phosphorus. These results are contradictory to the hypothesis,[2) 7)] which is described in "Literature Review" section, that only the bacteria which have the ability to accumulate polyphosphate and to supply energy for uptaking

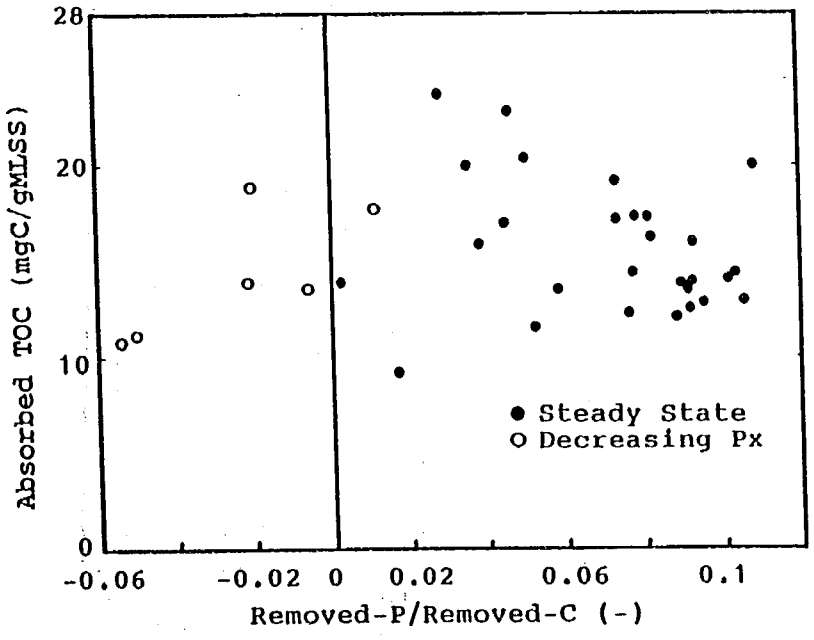


Fig. 13. Relationship between ΔP/ΔC and Abs-C

organic substrate under anaerobic conditions can become predominant in the anaerobic aerobic process. The results obtained here indicate that the anaerobic stage may function as a selector which selects the microorganisms with an ability to uptake organic substrate under anaerobic conditions, but that they do not always accumulate much phosphorus in their cells.

It has been generally accepted that the anaerobic release of phosphorus is governed by the anaerobic uptake of organic matter and that there is a proportional relationship between Rel-P and Abs-C.[7] However quite different results were obtained in the present study as indicated in Fig. 14, where the relationship between Rel-P and Abs-C in Run 5 is shown. Most sludges examined here had an ability to uptake most part of added TOC in the anaerobic stage. Therefore, Abs-C should be governed by organic loading which was kept almost constant throughout Run 5. As a result, most Abs-C was

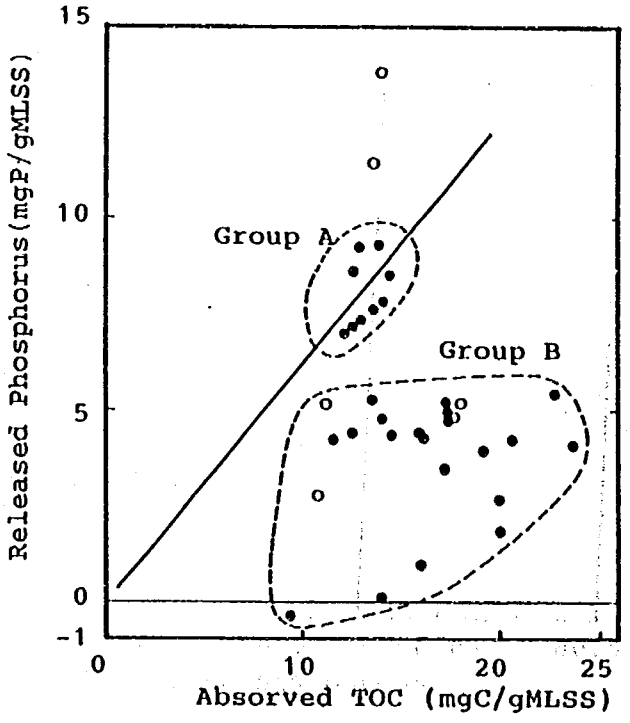


Fig. 14. Relationship between Rel-P and Abs-C

between 10 and 20 mgC/gMLSS irrespective of Rel-P. Group A, which is surrounded by a broken line, includes sludges with high phosphorus accumulating capacity. If we examine the relationship between Rel-P and Abs-C by changing the organic loading for the sludges of Group A, a linear relationship as expressed in the straight line (A) in Fig. 14 should be obtained. Group B includes sludges with poor phosphorus accumulating capacity. The sludges of Group B also have the same ability to uptake organic matter in the anaerobic stage but they do not release much phosphorus. Therefore, the latter was below the line (A).

Our results do not necessarily disagree with the previous researches. However a unique finding is that almost all the sludge examined here could uptake organic matter anaerobically irrespective of the capacity of phosphorus release and accumulation. How the sludge obtains energy for uptake of organic matters remains to be clarified. It can be accepted that the sludge produced in anaerobic aerobic process with a high phosphorus removal performance has a high phosphorus content and uptake organic substrate under anaerobic conditions accompanied by phosphorus release. However the converse is not necessarily true.

5-3 Mechanism of Phosphorus-Removal Deterioration

In the present study, prominent drop of phosphorus removal was observed in Run 3A and Run 5B. The change of Rel-P during these runs are shown in Fig. 15. It is obvious from Fig. 15 that the amount of anaerobically released phosphorus suddenly went up at the beginning of the deterioration of phosphorus removal and that it decreased with time. The level of anaerobically released phosphorus just after the deterioration of phosphorus removal began was much higher than that in the period of good phosphorus removal. It is suggested

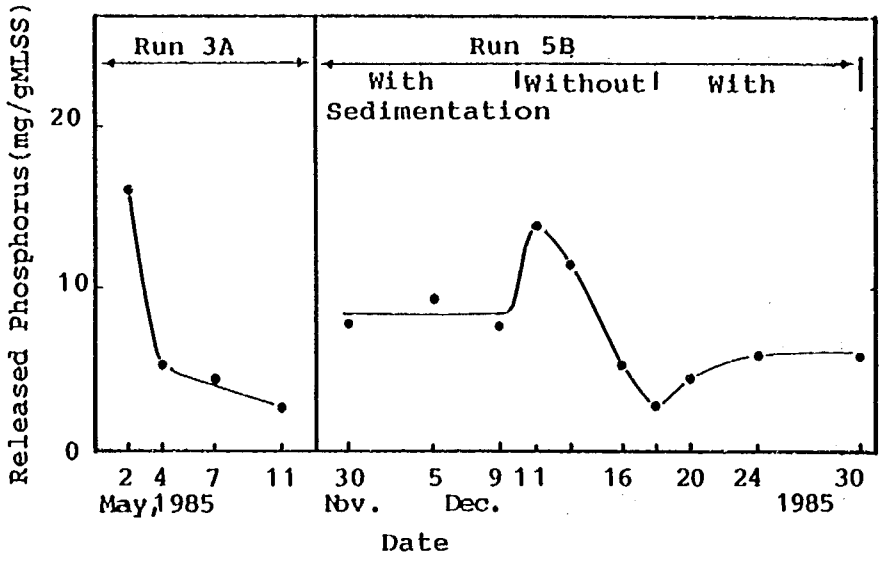


Fig. 15. Change of Anaerobically Released Phosphorus in Run 3A and 5B

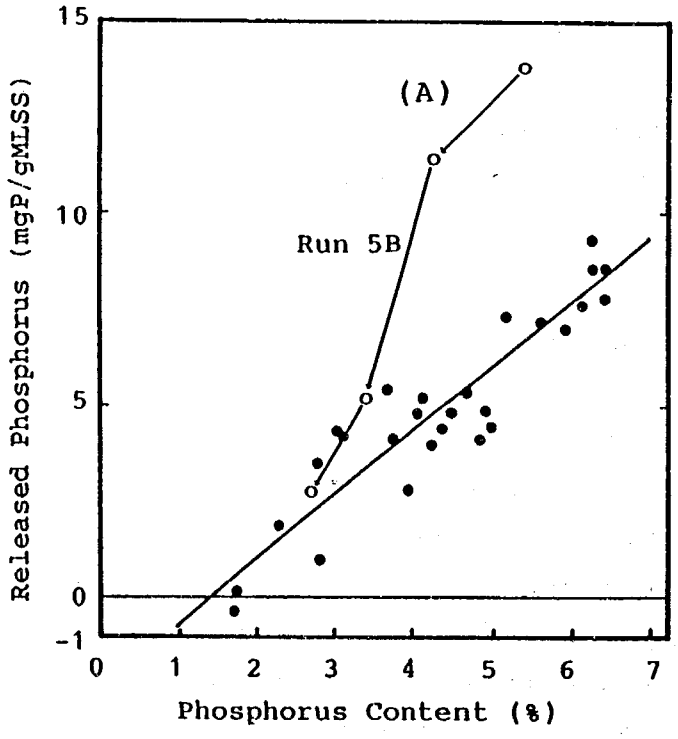


Fig. 16. Relationship between Px and Rel-P

that microorganisms can not afford to uptake all of the phosphorus, if it is excessively released in the anaerobic stage.

This suggestion is supported by Fig. 16 which shows the relationship between Px and Rel-P. Under steady state conditions with constant organic loading, the amount of anaerobically released phosphorus was proportional to phosphorus content of the sludge. The relationship between them is expressed in equation (4) and drawn in Fig. 16.

$$\text{Rel-P} = 1.68\ \text{Px} - 2.36 \qquad (4)$$

White circles represent the data obtained in Run 5B during the period in which the phosphorus content was decreasing. The arrows in the figure represent the chronological order of the data. The initial two data, which were obtained just after Px began to decrease, lie above the line. Then the points gradually come down to the steady state line. These results suggest that excessive release of phosphorus in the anaerobic stage should be a sign of deterioration of phosphorus removal ability.

It has been reported[4] that there is a critical BOD/MLSS ratio in the anaerobic zone which should not be exceeded for successful phosphorus removal in anaerobic aerobic process, and that the critical ratio is 0.1 kgBOD/kgMLSS. In other words, high BOD/MLSS ratio exceeding this value gives a negative effect on phosphorus removal. This fact may be explained by the results obtained in this study as follows. When as much BOD or organic substrate as the critical value is added to the anaerobic stage, excessive phosphorus may be released. If the release of phosphorus is too much, the phosphorus content of sludge could decrease, as the sludge can not uptake the released phosphorus completely.

6. Conclusions

The following conclusions were obtained from the experimental study on successful phosphorus removal with pure culture in anaerobic aerobic process.

(1) Pure culture of Acinetobacter calcoaceticus does not exhibit the metabolic characteristics of the anaerobic aerobic sludge.

(2) By undergoing the precultivation on strong medium with rapid growth rate, the sludge lose its ability to accumulate phosphorus.

(3) The accumulation of inactive organic matter or inorganic nutrients caused by elimination of sedimentation step may give an adverse effect on phosphorus removal. However sodium concentration as high as 1500 mg/l does not affect phosphorus accumulation greatly.

(4) Most sludges examined here could uptake organic matter under anaerobic condition, even though they had low phosphorus content. The mechanism of the anaerobic uptake of organic matter by the sludge with low phosphorus content remains to be clarified.

(5) The anaerobic aerobic sludge with high phosphorus removal capacity always has a high phosphorus content and uptakes organic substrate under anaerobic conditions accompanied by phosphorus release, but the converse is not necessarily true.

(6) Excessive release of phosphorus in the anaerobic stage is sometimes a sign of the deterioration of phosphorus removal capacity.

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BIOLOGICAL PHOSPHORUS REMOVAL BASIC PRINCIPLES AND APPLICATION

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Introduction

Nitrogen and phosphorus are the most important nutrients for all autotrophic beings like phanerogams, ferns, mosses and algae. In almost all cases in Germany phosphorus is the minimum factor for the growth of these plants and algae. Therefore a reduction of the phosphorus load is important for all sewage treatment plants situated at receiving waters in danger of eutrophication.

The northern part of Germany is low-lying country, such that many streams flow only slowly. Consequently the danger of eutrophication exists not only for lakes and reservoirs but also for these slow flowing creeks and rivers. In addition most of the German streams finally enter the North Sea which has to bear the sewage waters from great parts of Great Britain, Norway and the Netherlands. For a few years now the coastal parts of the North Sea - in fact just as the Baltic - has been considered to be in danger of eutrophication.

Phosphorus- and Nitrogen sources

The following sources are important for the introduction of phosphorus and nitrogen into the waters:

- point sources:
  * household sewage water
  * industrial sewage water
  * agricultural sewage water
  * rain water (via canalization)

- non-point-sources:
  * ground and drainage water
  * erosion
  * plant residues

FIRK&GEGENMANTEL /9/ have carried out intensive examinations to grant a better evaluation of the P- and N-loads from these sources. The phosphorus load of one inhabitant is calculated as follows:

P from food:	1.9 g P/I.d
P from detergents:	1.3 g P/I.d
P from household cleaners and dishwashing agents:	0.3 g P/I.d
Total:	3.5 g P/I.d

The nitrogen occurrence per inhabitant is in general considered to be:

11 -12 g N/I.d

For an average sewage water flow of 200 l/I.d these numbers lead to the following concentrations in the raw wastewater:

parameter	load g/I.d	concentration mg/l
BSB_5	60	300
P	3,5	17,5
N	11 - 12	55 - 60

However, these values show a great variation. The calculations carried out by FIRK and GEGENMANTEL had the results shown in figures 1 and 2. Considering the sewage treatment it is of importance to bear the fact in mind that the household and industrial sewage brings along 74% of the phosphorus load of the waters but only 51% of the nitrogen total. This points out that a phosphorus and a nitrogen removal in the sewage treatment plants are both possible.

Legal Requirements

Neither phosphorus nor nitrogen are parameters in the Wastewater Charges Act, nor in the minimum requirements for the clarification of communal sewage water which is effective nationwide. Consequently no sewage water has to be paid for phosphorus and nitrogen at present and there are no limiting values of general validity. However, limitation for the concentrations can be deduced from other legal regulations.

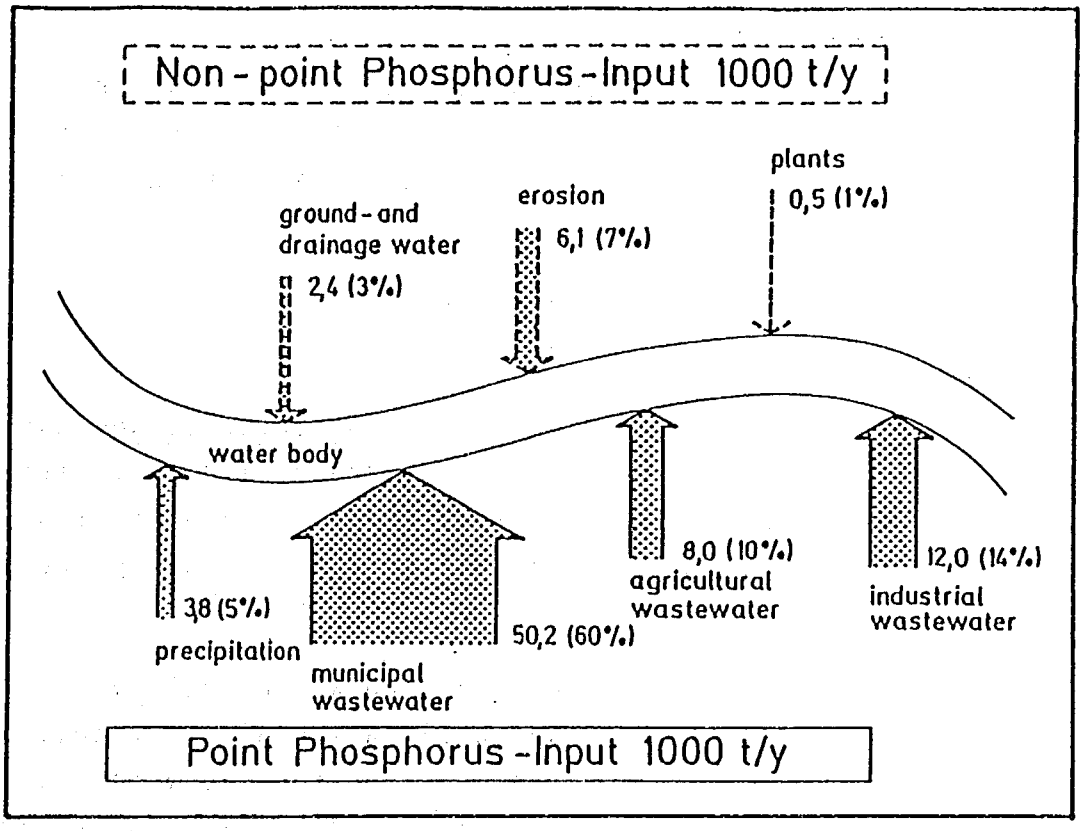


Figure 1: Proportion of phosphorus introduction into the waters of the German Federal Republic (FIRK & GEGENMANTEL)

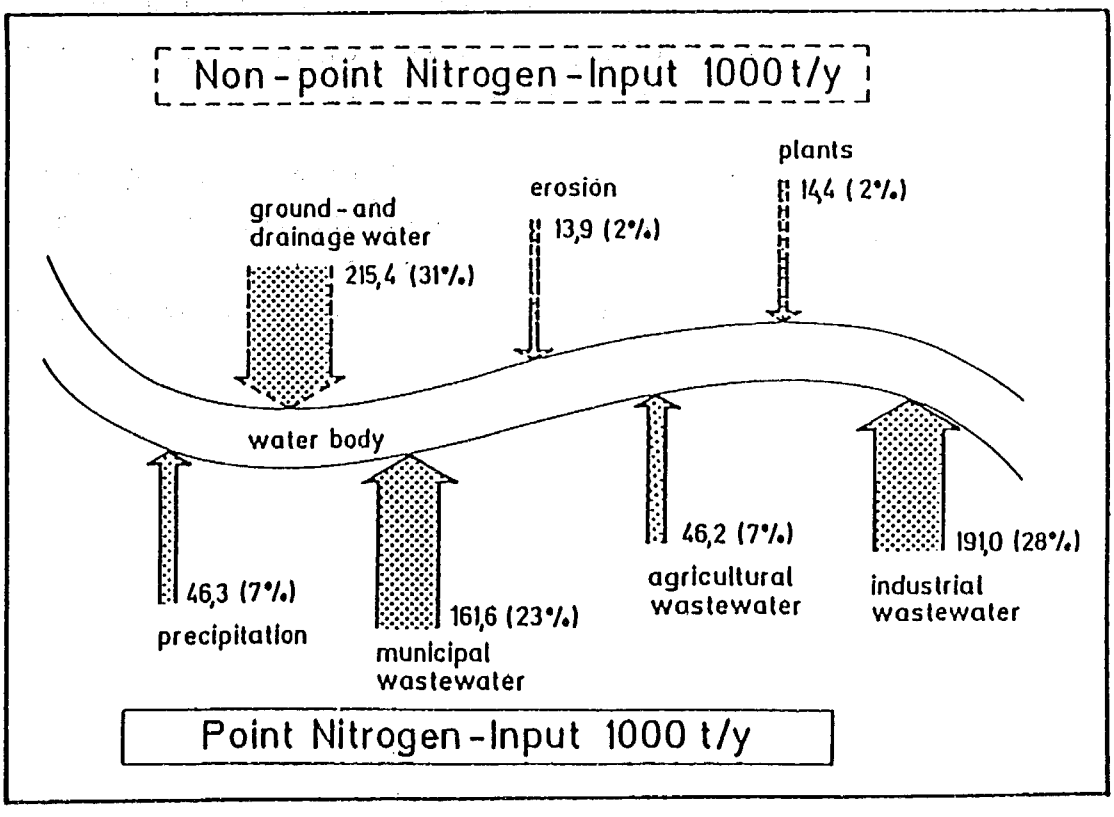


Figure 2: Proportion of the nitrogen introduction into the waters of the German Federal Republic (FIRK & GEGENMANTEL)

1. Basic Requirements

Basic requirements for every introduction of sewage water into the canalization or sewage treatment plant (STP) can be found in the ATV-worksheet A115. Phosphorus is not mentioned but there are regulations for nitrogen:

$$NH_3\text{-}N + NH_4^+\text{-}N \leq 200 \text{ mg/l}$$
$$NO_2^-\text{-}N \leq 20 \text{ mg/l}$$

2. Requirements For the Introduction Of Industrial Sewage Water

There are no federal limiting values for the P- and N-clarification of communal sewage water but they exist (with differing values) for certain industrial wastewaters, e.g. metal-processing and textile industry.

3. Requirements Depending On The Receiving Water Quality

Depending on the size, quality and utilization of the receiving waters additional limit values can be fixed for every STP. There are for example minimum quality requirements for all waters in the most heavily populated state Northrhine-Westphalia. Should a sewage water introduction cause the danger to fall short of the minimum quality , the effluent limit values can be reduced accordingly. The minimum quality requirements and a less strict 1. rehabilitation requirement are shown in table 1.

For waters which are specially utilized there are in addition limit values for certain parameters, which are required from the European Community. They are summarized in table 2 for the parameters nitrogen and phosphorus, which are of interest here.

	Parameter	Minimum water quality requirement	1st sanitation requirement
1	Temperature (T_{max}, °C)		
	waters, cool in summer	< 25	< 25
	waters, warm in summer	< 28	< 28
2	Oxygen (mg/l)	≥ 4	≥ 4
3	pH-value	6 - 9	6 - 9
4	Ammonium, NH_4^+-N (mg/l)	≤ 1	≤ 2
5	BOD_5 (without ATU) (mg/l)	≤ 7	≤ 10
6	COD (mg/l)	≤ 20	≤ 30
7	Phosphorus, total (mg/l)	≤ 0,4	≤ 1
8	Iron total (mg/l)	≤ 2	≤ 3
9	Zinc total (mg/l)	≤ 1	≤ 1,5
10	Copper total (mg/l)	≤ 0,05	≤ 0,06
11	Chromium total (mg/l)	≤ 0,07	≤ 0,1
12	Nickel total (mg/l)	≤ 0,05	≤ 0,07

Table 1: Minimal quality requirements for Northrhine-Westphalia

	Water use											
	leisure fishing				industrial fishing				direct drinking water supply		indirect drinking water supply	
	salmon water		carp water		salmon water		carp water					
	GA	SAF	GA	SAF	G	I	G	I	G	I	G	I
NH_4^+-N	≤1	≤2	≤1	≤2	≤0,03	≤0,78	≤0,16	≤0,78	≤0,78	≤1.17	≤0,04	
NH_3-N					≤0,004	≤0,02	≤0,004	≤0,02				
NO_3^--N										≤11,5	≤5,75	≤11,5
NO_2^--N	≤0,015	≤0,15	≤0,015	≤0,15	≤0,003		≤0,009					
TKN									≤2		≤1	
P_{tot}									≤0,3		≤0,17	

GA : Minimum quality requirement
SAF: Sanitation requirement
G : Guide-value
I : Imperativ-value

(mg/l)

Table 2: Limit values for nitrogen and phosphorus from the European Community

4. Other Requirements

In single cases further requirements for the STP effluents might be necessary depending on the state, the utilization or the size of the receiving waters. Maximum concentrations for different parameters are fixed for every introduction of wastewater into a water per decree by the controlling authority. Table 3 lists some exemplary legally permitted values for STPs of different sizes and receiving waters of different sensitivity.

Due to this legal situation the Federal Republic has

* a great amount of STPs without a legal limitation of the P-concentration in the effluent and
* a few hundred sewage treatment plants with very strict regulations concerning the phosphorus and the nitrogen.

Parameter: \ Town:		Würselen	Herzogenrath			Simmerath	Stolberg	Krefeld
			from 85	88	89			
Settleable Solids	mg/l	0,2	0,3	0,3	0,3	0,2	0,3	0,5(0,15)*
COD_{Chr}	mg/l	60	120	60	50	60	60	140(60)*
BOD_5	mg/l	15	25	15	7	15	15	30
P_{tot}	mg/l	-	-	-	-	0,6	-	-
NH_4-N	mg/l	5,0	-	5,0	1,0		1,0	-
Detergents	mg/l	← 2,0 →						
Extractable Subst.	mg/l	← 5,0 →						
pH	-	← 6,0 - 8,5 →						
Temperature	°C	← 301 →						
other		-	-	-	-	-	metals	metals
Connected P + PE		40.000	50.000			14.000	120.000	800.000
Receiving Water		creek				reserv.	river	Rhine

* Regulation according to the Wastewater Charges Act

Table 3: Some exemplary control values for STPs in Northrhine-Westphalia

This situation shows that up to now the biological phosphorus removal has not been applied on a large scale technology in Germany. There are however discussions whether phosphorus should become a parameter of the Waste Water Charges Act. This would probably start increased research and developmental work on the subject of biological P-removal.

Presently intensive research on the application of biological P-removal is being done in several places. We know about the following research projects:

- Technical University Hamburg, Prof. Sekoulov and Prof. Wilderer:
  - analysis of the P-removal potential of sludges.
  - biological removal at the SBR
- Technical University Braunschweig, Prof. Kayser:
  - comparison of PHO-STRIP and the PHOREDOX-process with small scale experiments
- Technical University München, Prof. Bischofsberger:
  - research on nitrification, denitrification and biological P- removal in sewage treatment plants with pure oxygen aeration (PHO-STRIP-process)
- University of Stuttgart, Prof. Krauth:
  - Nitrification, denitrification and P-removal in the A^2O-process
- Aachen University of Technology, Prof. Böhnke:
  - Biological P-removal in a two-stage activated sludge plant.

Furthermore there are experiments being carried out on some sewage treatment plants, as e,g. on the STP Ruleben in the city of Berlin or the STP Lippstadt (AB-process).

Mode of Operation Of The Biological Phosphorus Removal

The mechanisms in effect during the biological phosphorus removal have not been finally explained such that they might partly still be considered to be a hypothesis. Initially only the activated sludge process in all its modifications is taken into consideration, because only then the suspended bacteria of the activated sludge can be subjected to a change in environmental conditions. The main aim of all processes is first of all to increase the phosphorus content in all bacterial cells, with the

effect that a greater amount of phosphorus is eliminated with the excess sludge. The mechanisms will be explained with the help of figure 3 (according to ARVIN /1/).

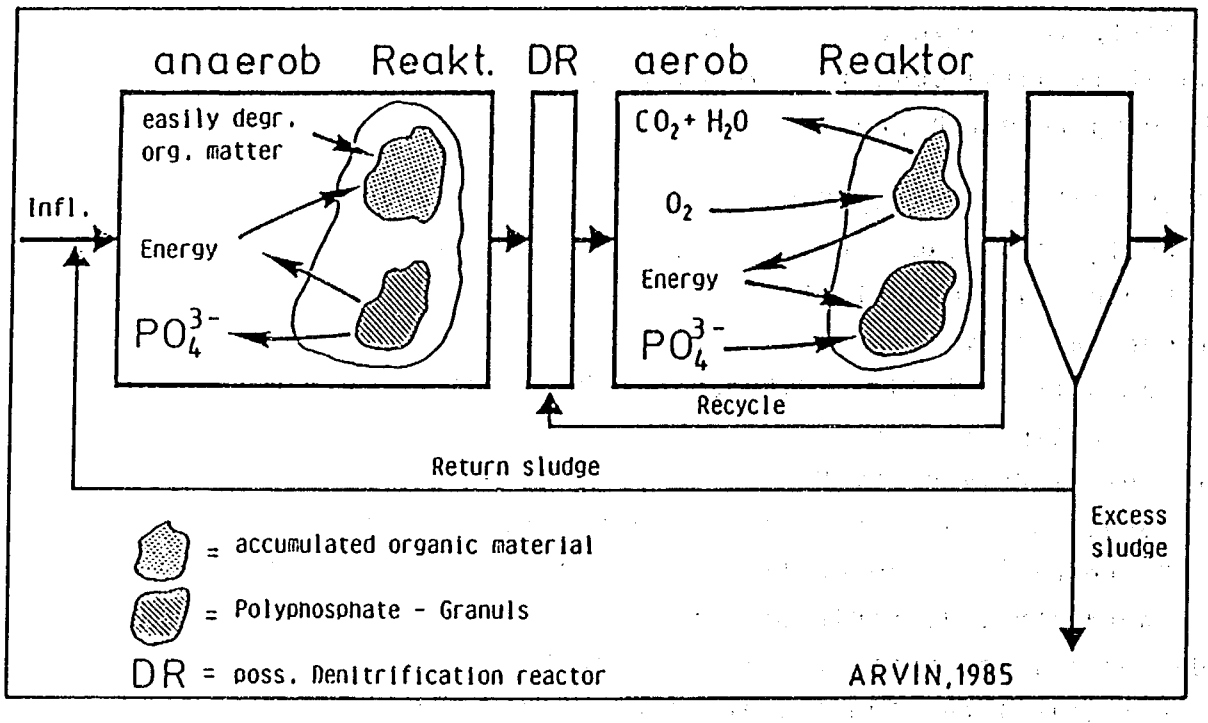


Figure 3: Basic mechanisms of the phosphorus accumulation

With regards to simplification the illustration does not take a possibly existing anoxic denitrification reactor into account. Also the proportions between the aerobic and the anaerobic basins cannot be inferred from the illustration. Due to the lack of dissolved oxygen the aerobic or facultative anaerobic bacteria of the activated sludge cannot use up the easily degradable organic material by respiration.

Therefore they deposit it initially in the cell material. The energy necessary for this process is extracted from the reduction of ATP to ADP:

$$ATP \rightleftharpoons ADP + \text{phosphorus}$$

This reaction sets PO_4-P free, the orthophosphate concentration in the liquid phase of the anaerobic basin increases to values that lie way beyond the influent concentrations.

In the aerobic basin the respiration of the organic compounds, accumulated before, takes place with the help of the dissolved oxygen; the energy set free is used to build up an ATP-storage.

A second possible explanation is that the polyphosphate-granules serve as a phosphorus storage. This explanation is not probable because the concentration of phosphorus in the aeration tank is always higher than necessary for the microbial growth /MATSCHÉ/.

Bacteria which are unable to build up this ATP-storage have a disadvantage in this system, the amount of bacteria with a high phosphorus content at the end of the aerobic stage increases.

Efficiency Calculation Of The "Normal" And "Increased" Biological Phosphorus Removal

The following considerations might help to evaluate the efficiency of a STP and to differentiate between "normal" and "increased" biological phosphorus removal. In addition they permit a better recognition of the relevant parameters.

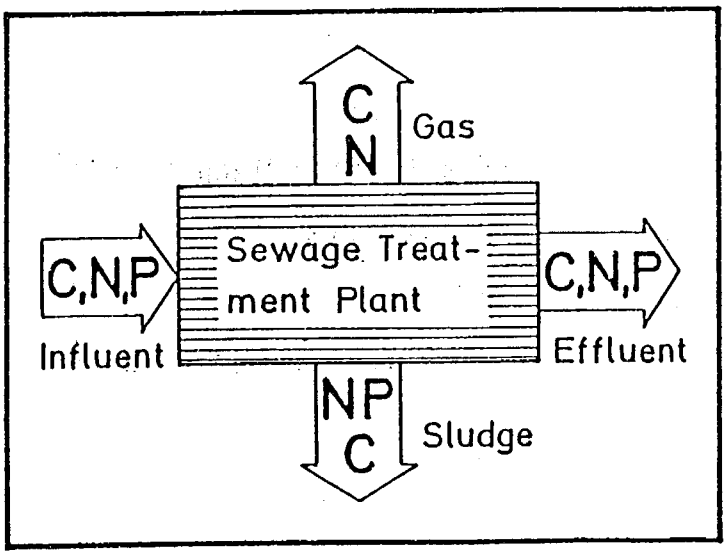


Figure 4: Flow scheme of the major polluters in a sewage treatment plant

The entire STP can, according to figure 4, be considered to be a "black box". All the polluting elements enter the plant via the influent but there are three possible ways of leaving the plant: gaseous, as solid or liquid via the sludge, or with the effluent

of the plant. Of the three elements mentioned carbon, nitrogen and phosphorus, the latter cannot take the exit into the atmosphere since gaseous phosphorus compounds do not occur under natural conditions. In consequence it is necessary to increase the load in the sludge in order to minimize the load in the effluent.

Since the phosphorus can only leave the plant via the sludge and since the amount of sludge produced depends mainly on the eliminated BOD_5 load, the relation between the BOD_5 and the phosphorus loads or concentrations in the effluent is a factor that greatly influences the phosphorus removal. This relation shall be termed m:

$$m = \frac{BOD_{5,infl}}{P_{total,infl}}$$

m divided by the phosphorus removal η_P results in:

$$\frac{m}{\eta_P} = \frac{BOD_{5,infl}}{P_{total,elim}}$$

or transposed:

$$\eta_P \cdot \frac{1}{m} = \frac{P_{total,elim}}{BOD_{5,infl}}$$

With the introduction of the BOD_5 removal η_{BOD} follows:

$$\frac{\eta_P}{\eta_{BOD}} \cdot \frac{1}{m} = \frac{P_{total,elim}}{BOD_{5,elim}}$$

If this equation is now divided by the specific excess sludge production

$$Y = \frac{SS}{BOD_{5,elim}}$$

the equation is:

$$\frac{\eta_P}{\eta_{BOD}} \cdot \frac{1}{m} \cdot \frac{1}{Y} = \frac{P_{total,elim}}{SS}$$

The term to the right of the sign of equation however corresponds to the phosphorus content in the sludge, hereafter called P_X, such

that the equation now looks as follows:

$$\frac{\eta_P}{\eta_{BOD}} \cdot \frac{1}{m} \cdot \frac{1}{Y} = P_X$$

or transposed:

$$\eta_P = \eta_{BOD} \cdot m \cdot Y \cdot P_X$$

This equation is of general validity, principally for plants with precipitation or for mechanical plants too.

Looking at each of the four elements determining the phosphorus removal we get an idea about a sewage treatment system with optimum p-removal.

The BOD_5 elimination is not a parameter that would be changed. Due to legal regulations a removal of at least 90%, mostly about 95%, is necessary in most cases.

There is only one way to markedly change the BOD_5/P-relation in the influent during the sewage treatment process: by operating a primary sedimentation. The BOD_5-efficiency of a primary sedimentation is about 30%, the P_{total}-efficiency - depending on the relation between dissolved and particular phosphorus compounds - can be estimated to be about 15%. This causes the relation m to be decreased by about (1 - 0.70/0.85 =) 18% to the unfavorable side!

In the forfield of the STP several possibilities exist to shift the BOD_5/P-relation: e.g. introduction of industrial wastewater with a high concentration of easily degradable organic substances and low phosphorus concentration; with a further reduction of the p-content in detergents or with a fertilization that is more appropriate to agricultural demands.

The specific excess sludge production (Y) depends on many factors, mostly however on the sludge load, or rather, on the nutrition situation of the biomass. In case of increasing the sludge loading rate (SLR) the excess sludge production increases too.

Figure 5 shows the relationship between sludge loading rate and specific sludge production Y according to specifications found in the "Sewerage Handbook" /8/ and other publications.

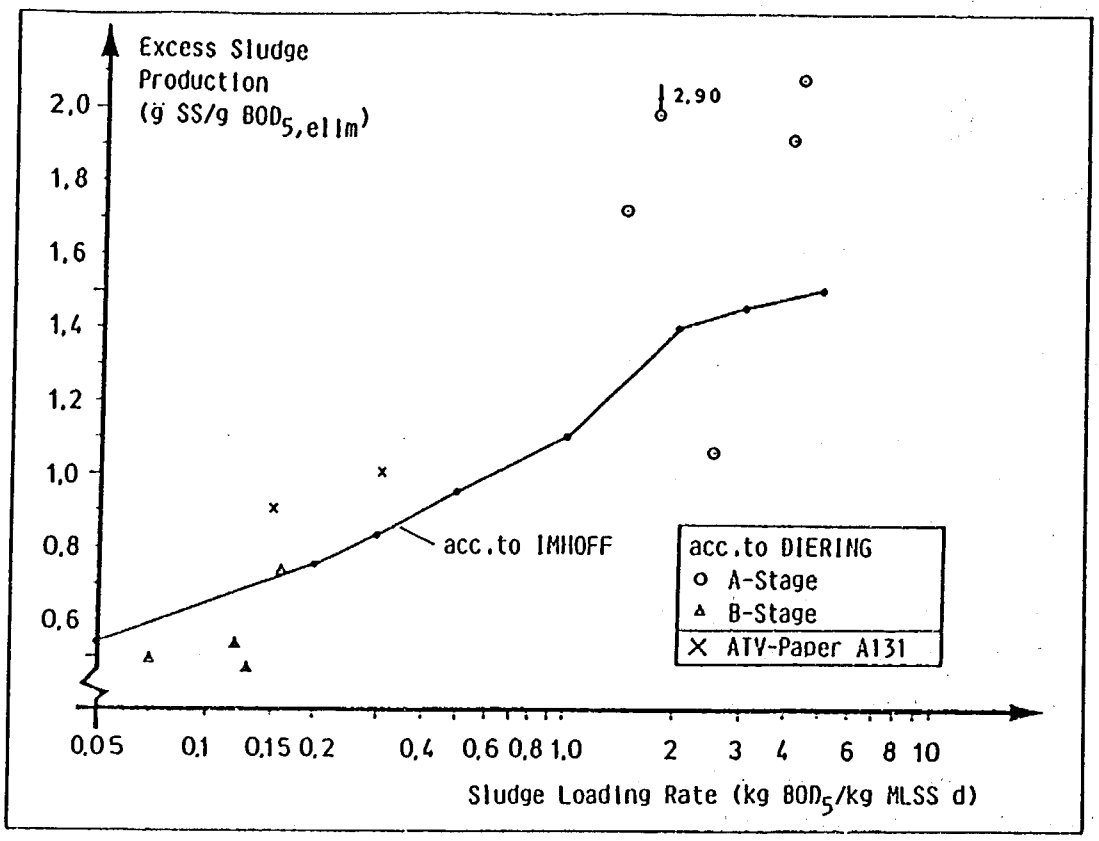


Figure 5: Relationship between sludge loading rate and specific sludge production

Especially in case of highly loaded plants without primary clarification the sludge exists not only of excess sludge but is constituted by a mixture of primary and excess sludge.

The phosphorus content values in the sludge , P_X, have so far not been thoroughly examined. The University of Stuttgart carried out the most extensive measurement program, under a different perspective /10/.

The analysis of 134 activated sludges from 90 activated sludge plants with primary clarification had the results shown in the following illustration. It can be inferred that none of these sewage treatment plants was specially directed towards an increased biological phosphorus removal. It becomes apparent that the phosphorus content in the sludge increases when the volumetric loading increases, that is seen in relation to the organic dry matter.

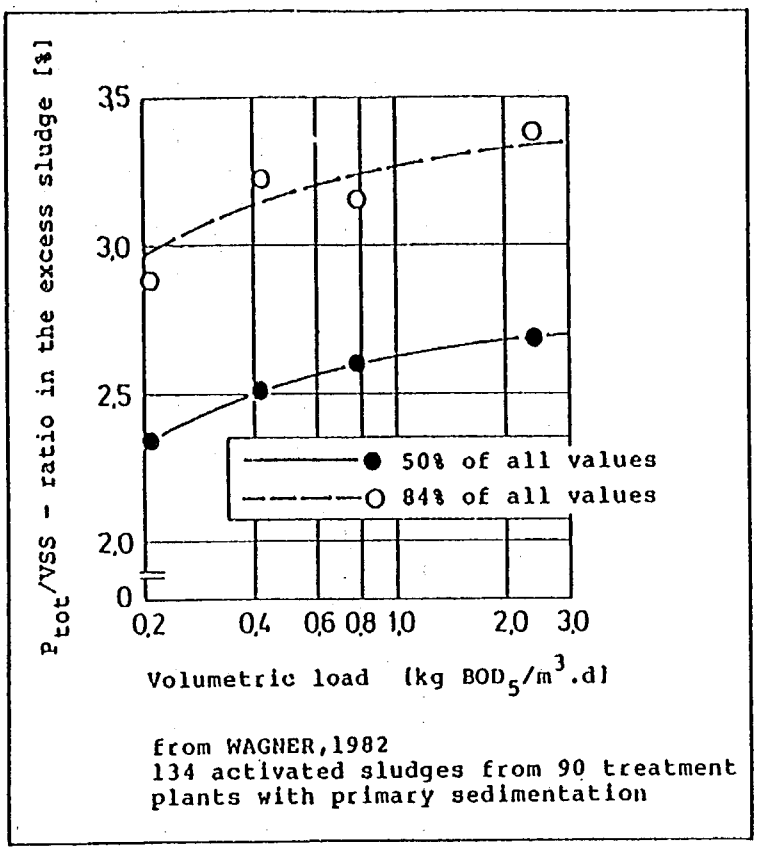


Figure 6: Phosphorus content in the organic dry matter of excess sludges from 90 large scale activated sludge plants

Figure 7 shows a graphic representation of the equation mentioned above. The diagram features the graphs for 3 supposed clarification procedures. A difference is made between two BOD_5/P_{total} relations in the influent: 300/15 = 20 for normal wastewater and 300/10 = 30 for a wastewater with relatively low phosphorus content. The considered clarification processes are:

Case I: one stage activated sludge plants with a sludge loading rate = 0.15 kg BOD_5/kg MLSS.d.. Due to a primary clarification the BOD_5/P relations change as listed in the table of figure 7. Depending on the sewage a phosphorus removal efficiency of 30% - 45% is calculated. These values are typical for normal treatment plants.

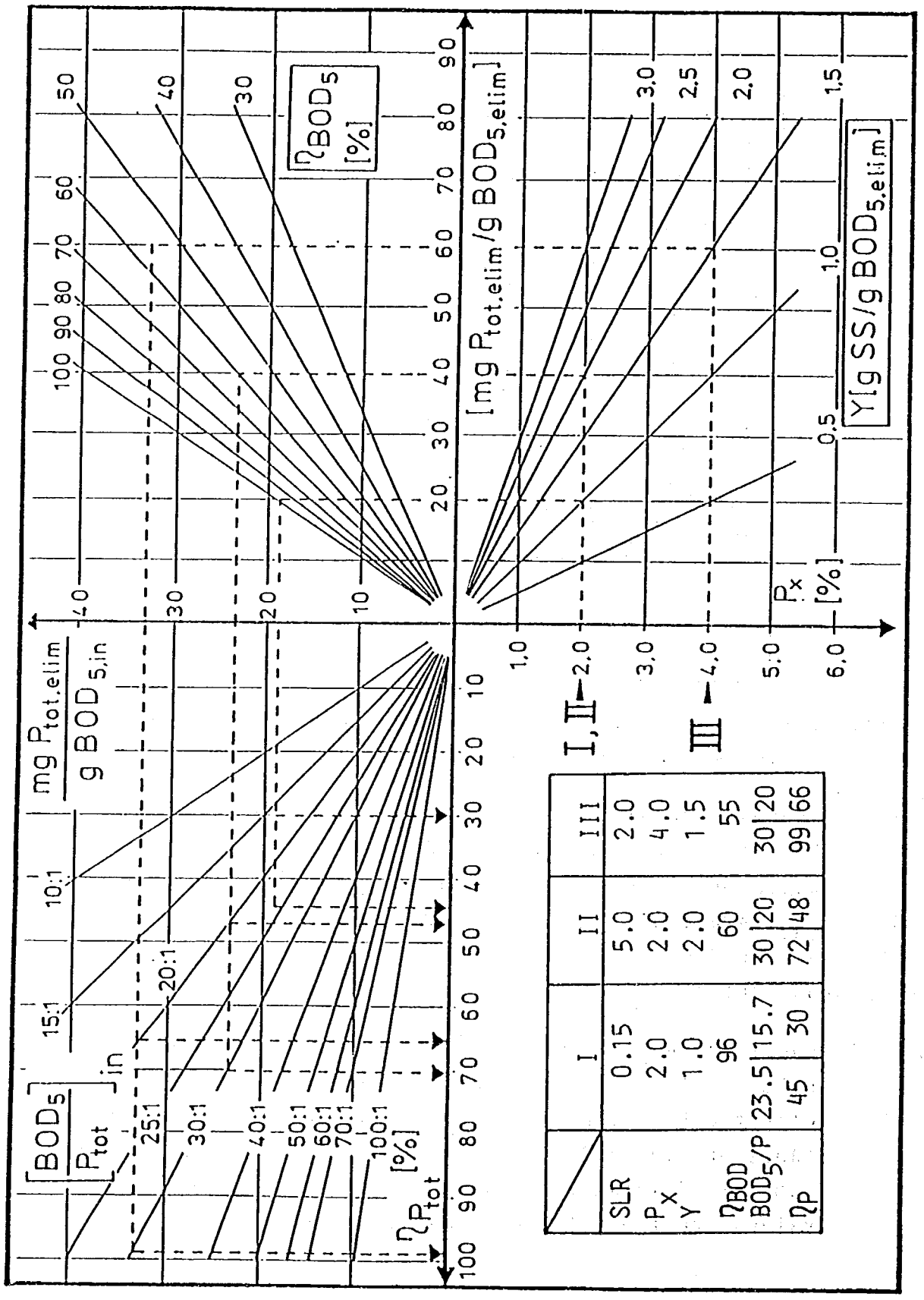


Figure 7: Diagram for the graphic calculation of the phosphorus removal efficiency of sewage treatment plants.

Case II: This is the Adsorption stage of an AB-plant. The sludge loading rate is high (SLR = 5.0), in consequence the specific excess sludge production is high too (Y = 2.0). The phosphorus content of the sludge lies in the normal range (P_X = 2.0), the BOD_5 removal efficiency is BOD = 60%. The calculated phosphorus removal efficiency lies between 48% and 72%. Such high values have in fact been measured in a large plant /5/.

Case III: This is a hypothetical case. In the Adsorption stage the phosphorus content of the sludge, P_X, is raised to 4% by a change of aerobic and anaerobic conditions. This is brought about by a somewhat lower sludge loading rate, a lower excess sludge production and a slightly decreased BOD_5 efficiency. The diagram shows that in this case the phosphorus removal efficiency could increase to 66% or up to 99%. This case is, as was said, still hypothetical, but it points out the direction for future work.

Necessary for the functioning of the phosphorus removal in the A-stage however is that microorganisms capable of increased phosphorus uptake are at hand. Although several microorganism species are able to do so showing apparently symbiotic effects with other species, the literature always names the Acinetobacter group /2,3/.

A detailed examination of biocenosis in the A- and the B-stage of the STP Krefeld, carried out by the Hygiene Institute of the University of Bonn, showed that Acinetobacter are present in both stages, in the A-stage with 5.3% of all bacteria even higher than in the weakly loaded B-stage (see fig. 8). This leads to the conclusion that in general both the A- and the B-stage are suitable for increased biological phosphorus removal.

SEWAGE TREATMENT PLANT KREFELD (1982)		
	A-Stage % of all	B-Stage organisms
Gram-negative Cocci	24.2	34.7
Enterobacter	15.5	14.7
Pseudomonas	8.6	8.7
Flavobacterium	7.1	9,3
Pasteurella	6.5	5.1
Acinetobacter	5.3	4.0
Aeromonas	5.3	3.3
Moraxella	3.0	1.2
Filamentous Microorg	2.5	1.5
remaining 25 species	22.0	17.5
Total	100.0	100.0
A-Stage: 769 tests B-Stage: 750 tests		

Figure 8: Bacteria population of the A- and B-stage of the STP Krefeld/11/

Large Scale Results

Measurements carried out at the STP Krefeld which has bee operating as adsorption activated sludge plant for several years now, have in fact yielded the values listed in table 3 /6/.

	A-STAGE INFLUENT	A-STAGE EFFLUENT	η_A	B-STAGE EFFLUENT	η_B	η_{TOTAL}
COD	652 mg/l	299 mg/l	54%	52 mg/l	83%	92%
BOD_5	234 mg/l	124 mg/l	47%	3 mg/l	97%	99%
N_{total}	51.9 mg/l	34.4 mg/l	34%	18,3 mg/l	47%	65%
P_{total}	14.7 mg/l	7.5 mg/l	49%	3.1 mg/l	57%	79%

Table 3: Results of a research project on the STP Krefeld /6/

Conflict Between Nitrification/Denitrifikation And Biological Phosphorus Removal

In figure 3 the basic mechanisms of phosphorus accumulation in the cell have been explained. A denitrification reactor was named but not mentioned any further. It is in fact not allowed to introduce nitrate into the anaerobic reactor. The presence of nitrate leads to a competition between the phosphorus accumulating bacteria and the denitrifiers for the easily degradable organic material.

The return-sludge however flows through the anaerobic basin and contains at least as much nitrate as the effluent of the STP. For one-stage sewage treatment plants this means that with the denitrification the nitrate content of the effluent - and in the return sludge at the same time - has to be pushed below the critical value of appr. 5,0 mg NO_3-N/l. This in turn leads to additional basin volumes, to a lower sludge loading rate and therefore to a lower specific excess sludge occurrence. The BARDENPHO-plants developed in South-Africa are an example for this. In these plants large basin volumes are available for an almost complete nitrification and denitrification. The plants therefore work in the loading range of activated sludge plants for aerobic sludge stabilization and compensate the high phosphorus content in the sludge with a low sludge production. Two-stage plants with the following distribution of tasks can lead the way out of this conflict:

1. Highly loaded stage:
   Partial BOD_5 and COD removal (50% - 60%),
   high excess sludge production,
   therefore high phosphorus removal,
   no nitrogen recycle from the second stage,
   nitrogen removal only by incorporation into the cells and by withdrawal of the particular nitrogen.

2. Weakly loaded stage:
   Removal of the remaining BOD_5 and COD,
   additional biological phosphorus removal,
   nitrogen removal by nitrification and denitrification.

Table 5 summarizes the advantages of a high sludge loading rate for the biological phosphorus removal:

ADVANTAGES OF A HIGH SLUDGE LOADING RATE FOR THE BIOLOGICAL PHOSPHORUS REMOVAL
- High excess sludge production - Higher phosphorus content in the sludge - No nitrification, no impediment of the phosphorus release by the nitrate content of the return sludge

Table 5: Advantages of a high sludge loading rate for the biological phosphorus removal

Effects Of The Sludge Treatment On The Biological Phosphorus Removal

Activated sludge which has been adopted to the biological phosphorus removal, gives of the stored polyphosphate under anaerobic conditions. This can lead to a phosphorus release during the sludge treatment since there anaerobic conditions prevail in general. Table 6 however shows that for each of the common sludge treatment processes there is also an aerobic one:

Sludge treatment stage	anaerobic	aerobic
Thickening	Gravity thickener	Floatation, centrifugation of excess sludge
Stabilization	Digestion	aerobic-thermophilic sludge stabilization, chemical stabilization
Dewatering	Sludge drying beds	Machine-run dewatering

Table 6: Confrontation of anaerobic and aerobic sludge treatment processes

Thickening in gravity thickeners and stabilization in anaerobic digesters are common and approved process stages. In case these procedures are involved a subsequent lime precipitation of the

process-waters is recommendable. For the lime precipitation - in contrast to the precipitation with metal-salts - the dosage is calculated according to the amount of water and the alkalinity of the wastewater. But the process-waters constitute only 1% to 2% of the influent such that the amount of lime to be applied is extremely low. The precipitation products that occur are insoluble such that a sedimentation basin is not needed.

A STP operating according to this process scheme is shown in figure 9.

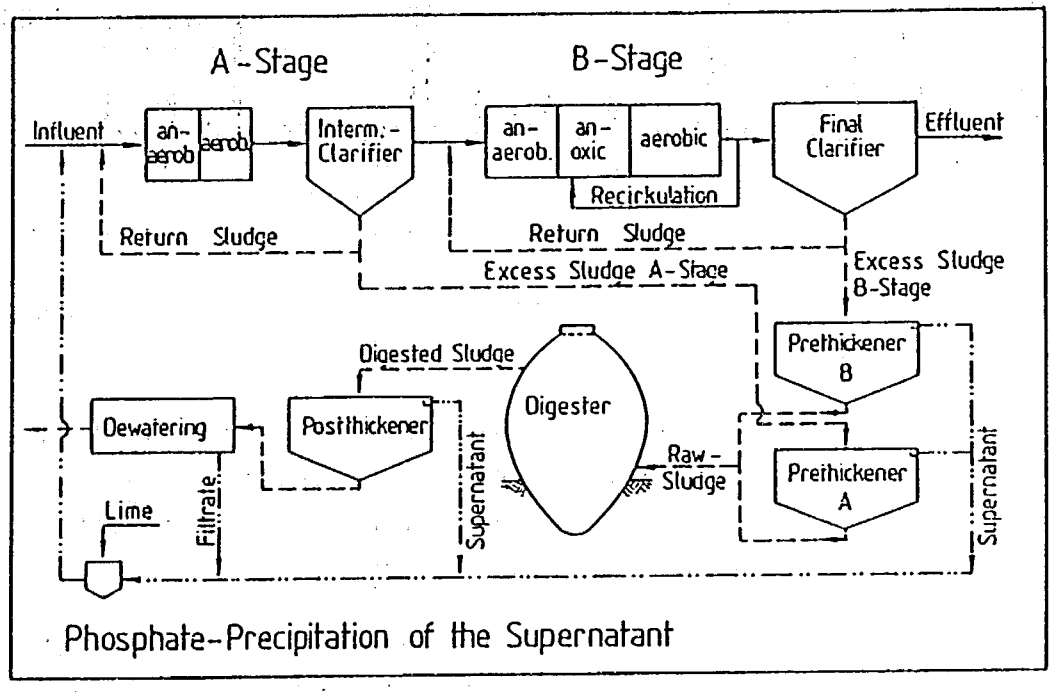


Figure 9: Flow-scheme of a two-stage STP with the aim of extensive phosphorus removal

Pilot Plant Results

A treatment system like the one shown in figure 9 is under investigation now on the research ground of the Aachen University of Technology. Figure 10 shows the orthophosphate-concentrations in the effluent of the 2nd stage. In July, most of the values have been rather high, no biological P-removal can be seen. At the end of July and in August, most of the values have been lower than 0.5 mg P/l with the exception of some mavericks.

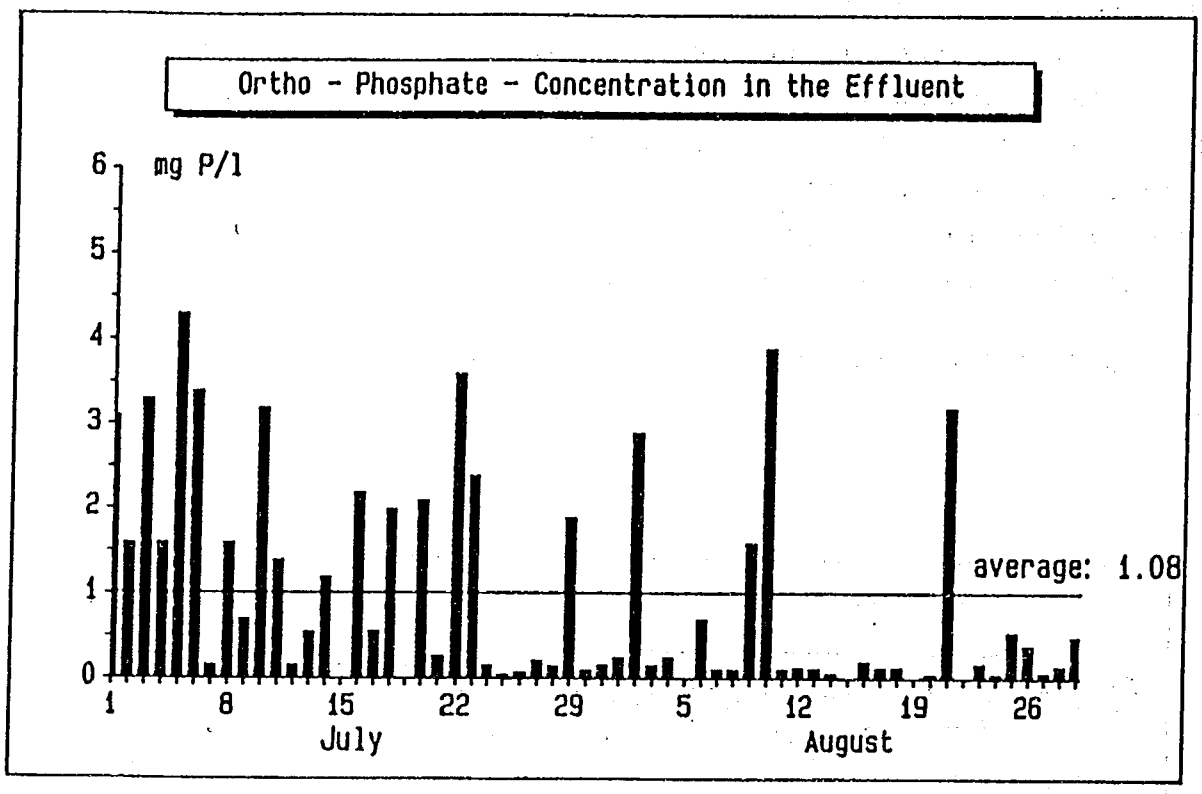


Figure 10: Orthophosphate-concentrations in the effluent of the 2nd stage

The high values are caused by an oxygen deficiency in the last, aerobic section of the B-stage. The oxygen deficiency caused a phosphorus release. The investigation will be continued with a better oxygen control in the aerobic section.

The next figure 11 shows the cumulated frequencies of the orthophosphate-concentrations in the influent, effluent A-stage and effluent B-stage. The removal efficiency of the A-stage seems to be low because the reduction of the orthophosphate-concentration by incorporation into the excess sludge is compensated by the hydrolysis of organic phosphorus to orthophosphate.

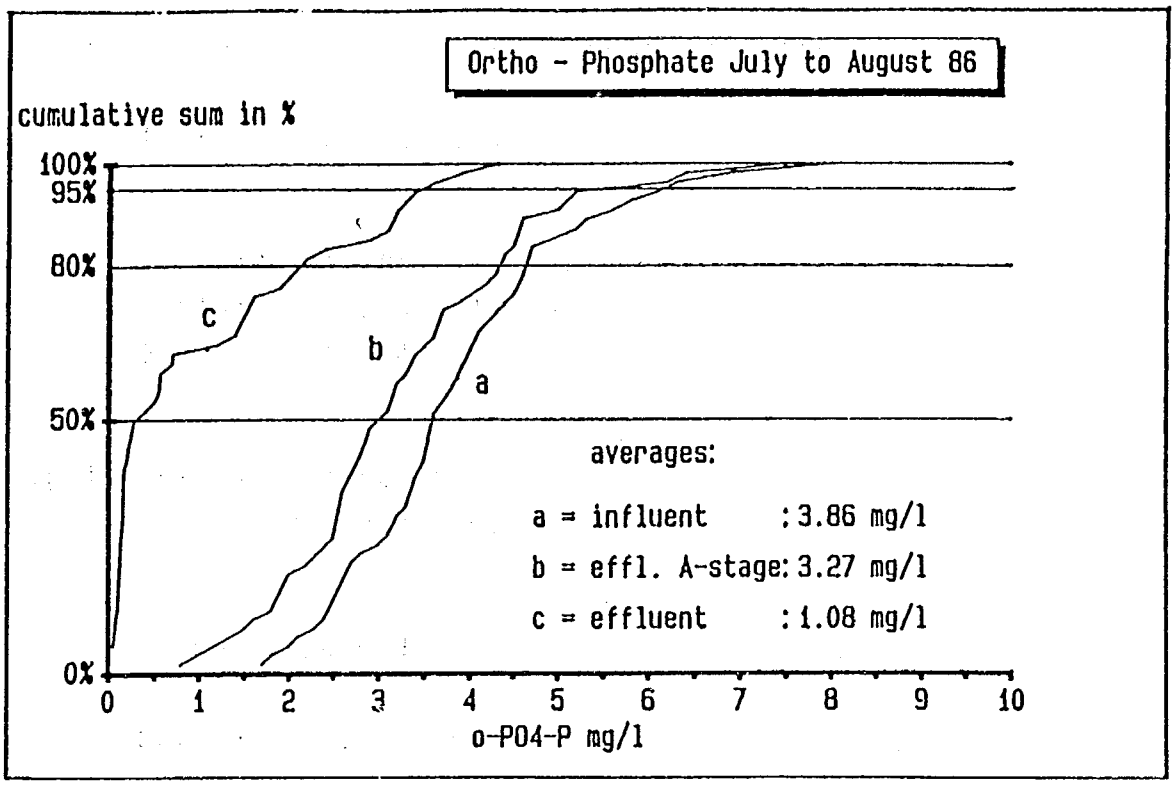


Figure 11: Cumulative Frequencies of the orthophosphate-concentrations in the influent, effluent 1st stage and effluent 2nd stage

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SESSION 2

ADMINISTRATION AND FINANCE OF SEWERAGE

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SESSION 2

ADMINISTRATION AND PREVENTION OF SEWERAGE

SIXTH FIVE-YEAR PROGRAM FOR SEWERAGE CONSTRUCTION

Itaru Nakamoto
Director
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ABSTRACT

In a last few decades, the Japanese economy achieved a remarkable growth. On the contrary, however, preparation of social capital is lagging way behind. The estimated coverage rate of publicly owned sewerage system to population at the end of FY 1985 is as low as 36 percent.

To promote systematic sewerage construction the Japanese Government has been carrying on the five-year programs since 1963. The program decides the national framework of sewerage construction for the coming five years by coordinate the national grant program with the local government's program.

This paper describes outline of Sixth Five-Year Program which started FY 1986, including background, priority target, target propagation, scale of investment and new measures.

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1. STATUS OF SEWERAGE AND SEWAGE PURIFICATION IN JAPAN

1.1 Brief History

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 km^2, Japan is nearly as big as whole Germany. The greater part of the national land is mountainous. Less than 25% of the total area in inhabitable but 120 million people, equivalent to the double of the total population of the West Germany 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 sixth five-year program started in fiscal 1986, there have been five five-year programs. Under the past five programs, tremendous amount of expendetures has been invested as stated later. 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 1985, sewerage systems had been developed for 43,300 thousand (with the coverage rate at 36%).

1.2 Present Situation

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 maintenanced 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 system are higher than those public sewerage systems. In addition, the improvement of urban storm drainage 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.

If fiscal 1986, about 1,770 billion yen is to be invested to the construction of sewerage facilities. Fig. 1 illustrates total amount of the investment to the sewerage facilities in last nine years.

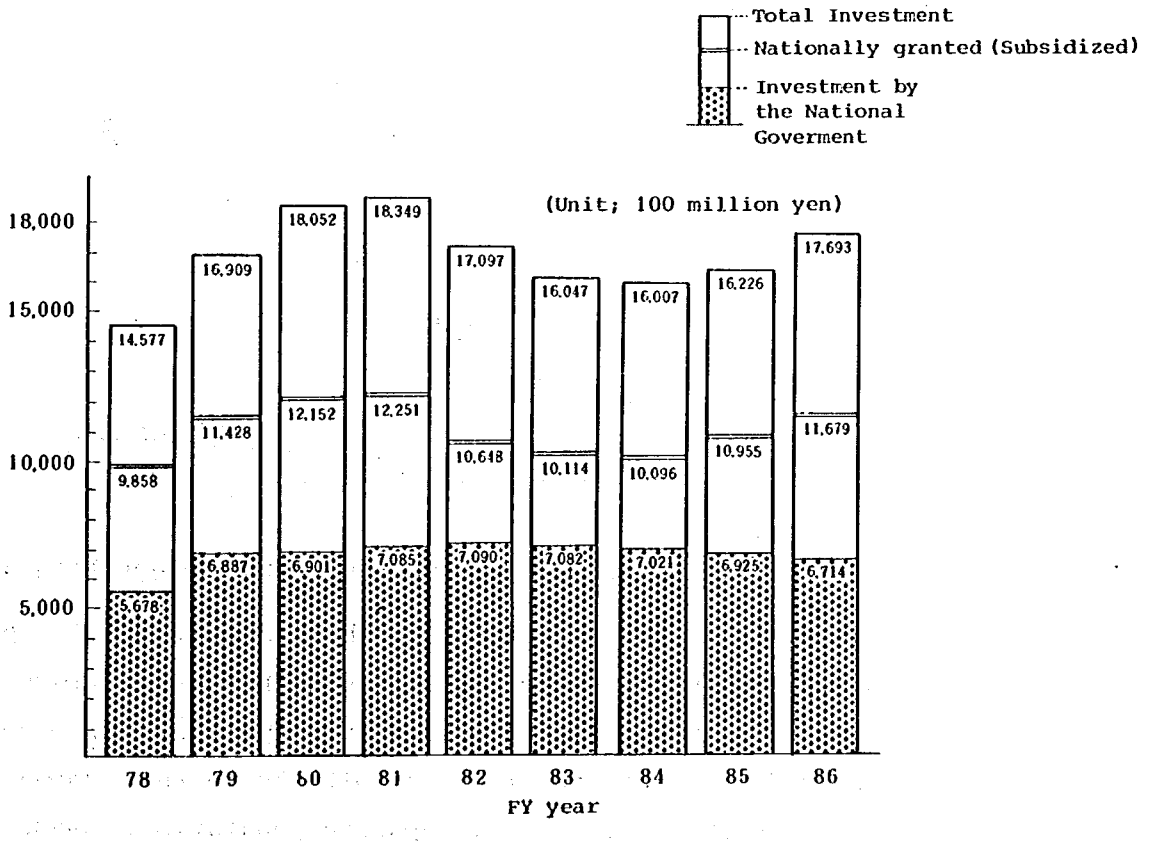


Fig. 1 Annual Changes in Investment to Sewerage Construction

During such period, the scale of investment to the sewerage programs went through rapid progress. It had once grawn equivalent to the defense budget. However, it has been at a standstill there years reflecting the severe financial situations.

The number of municipalities having started Municipal Sewerage Program is 857 at the end of FY 1985. This number holds a quarter of the total 3,256 Japanese municipal bodies. Four-hundred and eighty-three (531) municipalities out of such 857 are currently carrying out treatment of their municipal wastewater. Regional sewerage program has been carried out at 85 places in the 40 prefectural divisions, and 46 wastewater treatment plants are now in operation. Moreover, the Special Environment Protection Municipal Sewerage Program for agricultural/mountain/fishing villages and national parks have been carried out at 104 places, and the Special Public Sewerage Program for the areas containing industrial wastewater over two third (2/3) of the total quantity have been practiced at 8 places (Fig. 2).

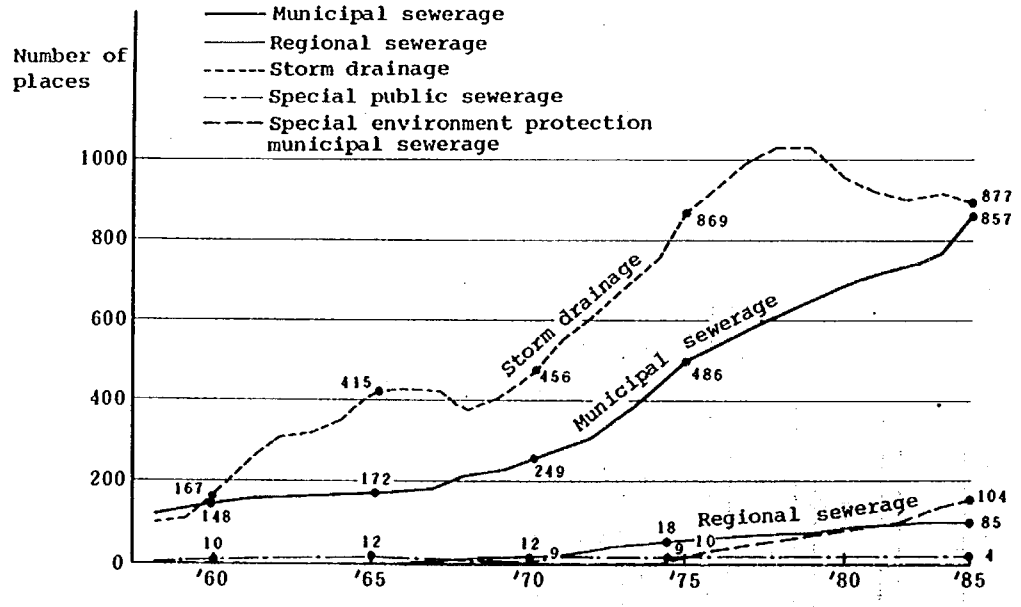


Fig. 2 Trend of the Number of Cities Carrying Out Sewerage Programs

In spite of increase in the investment to the Sewerage and Sewage Purification Programs and in the number of places carrying out sewage works, improvement has been slow in the spread of sewerage systems. The fiscal year of 1985 has proved that the number of inhibitants in the sewered areas holds no more than 36% of the total Japanese population. Such percentage held by designated cities with population over one million is

as high as 80%, while general cities 34% and small towns and villages of only 4%

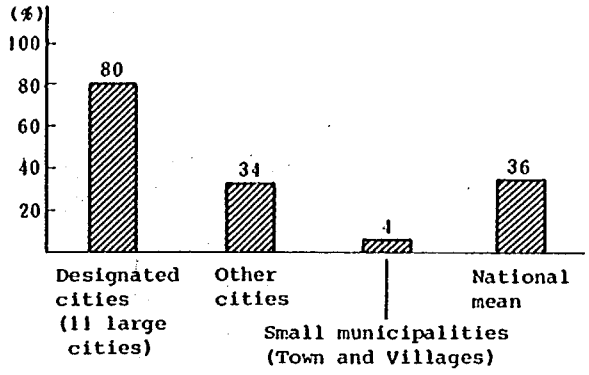


Fig. 3 Coverage Rate of Sewerage Systems to Population (as of the end of FY 1985)

This means that there is a big disparity in the service by sewerage systems among areas. It is urgently demanded to spread sewerage systems to comparatively less-inhibitied local cities thereby decreasing the disparity among areas.

Fig. 4 shows state of sewage works implementation by the size of the municipalities. As is seen, among the total 2,569 municipalities of less than 30 thousand population, only 398 (15%) have started sewage works including urban drainage system.

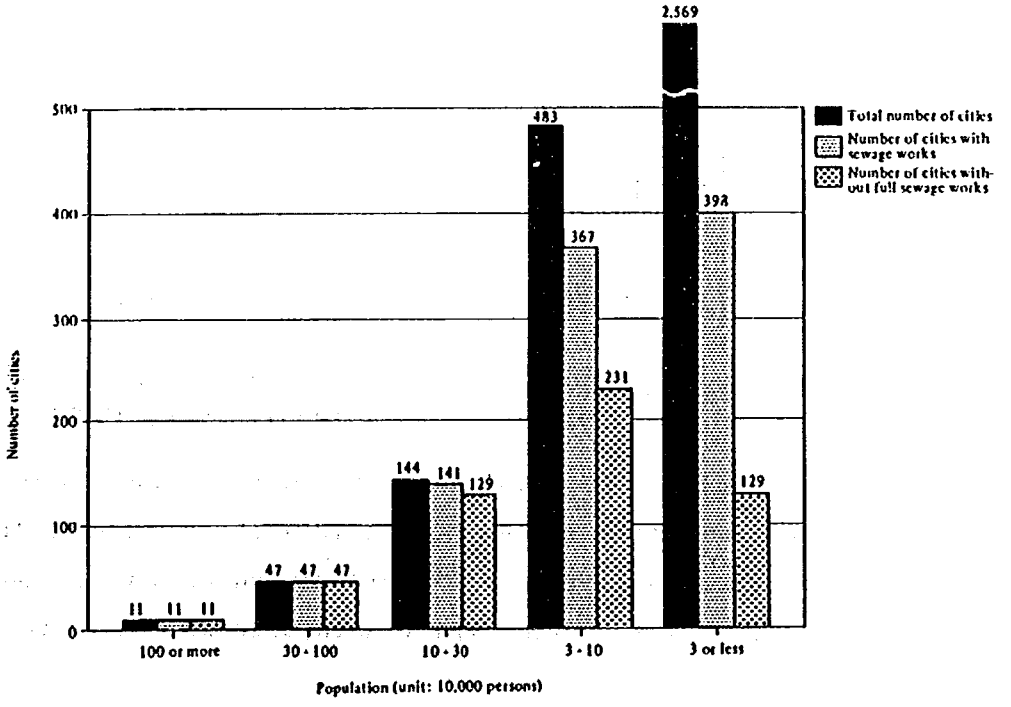


Fig. 4 State of Sewage Works Implementation

Recently, the Prime Minister's Office made a survey concerning peoples' requirements against social infrastructures in their communities. Brief results of the census is illustrated in the Fig. 5.

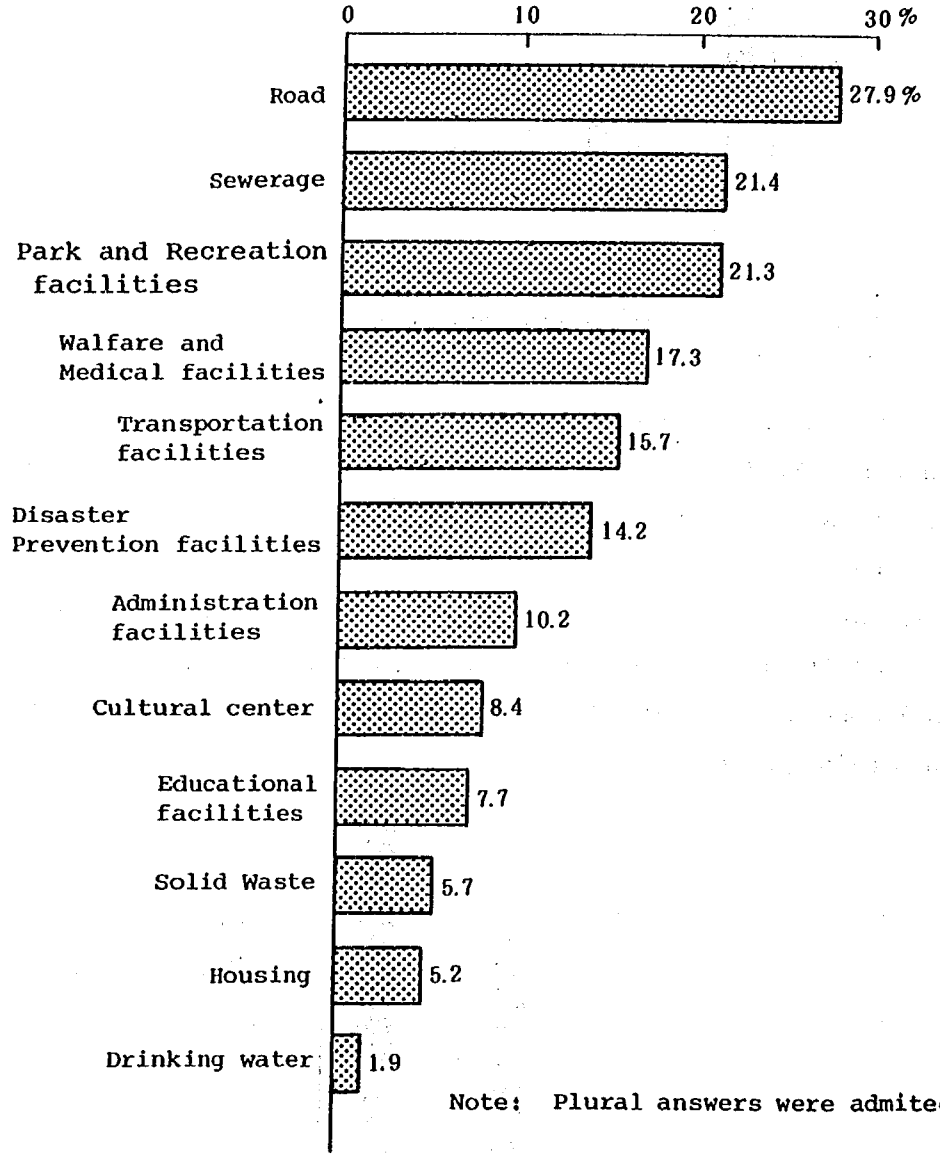


Fig. 5 People's Requirements Against Social Infrastructures in their Community (Survey by Prime Minister's Office, Oct, 1985)

As is shown in the figure, peoples' needs against sewerage facilities are ranked the second among other eleven public facilities. About 21 percent of the people want sewerage facilities for their communities while only 19% of them want drinking water systems which have already completed as a National lebel.

As stated earlier, investment to sewerage facilities has been at a standstill these years reflecting the severe financial situation of the

nation. In other words, how to acquire neccessary budget for the investment has been the concern of the responsible organization of the each public works.

The Ministry of Construction has announced several measures in these years, to let as many people as possible understand the benefits of sewerage systems and to smoothly promote the sewerage programs. Table 1 presents the list of cities to which the model project of sewage works called Aquatopia (from Aqua Utopia) are applied and the purposes of the project. In these works, it is tried to retrieve "rest by the water". In the Acuatopic cities, the sewerage programs are to be promoted with priority. Besides such works, the other model projects such as Appeal Sewerage and Idea sewerage have been practiced, thereby trying to make citizens recognize the benefit of sewage works.

2. FIVE-YEAR SEWERAGE CONSTRUCTION PROGRAM

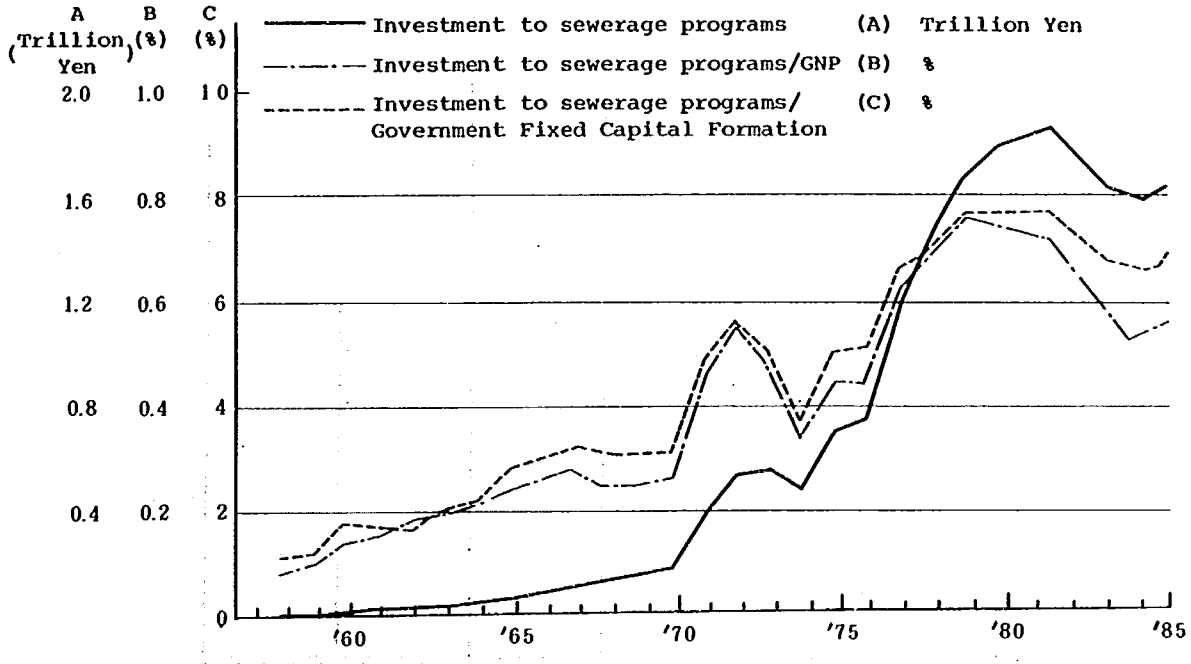
The systematic development of sewerage systems from the 1960s have evolved under five-year programs, the first one of which started in 1963. As has been introduced earlier, the year 1986 was the first year for the Sixth Five-Year Sewerage Program. In Japan, most public works are carried out under a five-year 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. 6 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

Table 1 List of Aquatopia projects

Prefecture	City/Town	Major water body	Major purpose
Hokkaido	Ikeda	Kiyomi Nisen River	Clean water ways in the town and improve water quality of the Koyomi Nisen River to create resort places for citizens.
Aomori	Hirosaki	Dobuchi River	Improve water quality of the streams in the city and stock the water ways with carp so as to give citizens the opportunities to familiarize with water.
Yamagata	Tendo	Kuratsu River	Clean all water ways in the city so as to offer the citizens the resort places where carp are stocked.
Ibaraki	Itako	Itako Cannals	Clean all cannals in the town so as to improve the environment of the town of "Iris and Cannals".
Gunma	Haruna	Lake Haruna	Clean all water ways in the town so as to maintain water quality of Lake Haruna which is the precious property of the citizens.
Gifu	Takayama	Miya River	Improve water quality of all water ways in the city so as to maintain the precious environment of the town which is called "small Kyoto".
Shizuoka	Hamamatsu	Hamana Lake	Improve water quality of all water ways in the city so as to offer the citizens the opportunities to familiarize with water.
Osaka	Toyonaka	Intown Rivers	Restore the "home of fireflies" at the artificial water way in the Tokura district as well as to improve water quality of all water ways in the city so as to offer the citizens the resort places.
Shimane	Matsue	Ohashi River	Improve water quality of all water ways in the city so as to restore the Matsue-city as "city of water".
Okayama	Kurashiki	Kurashiki River	Improve water quality of water ways in the city so as to create the environment matching the white walls (symbol of the city) and to offer the citizens the opportunities to familiarized with water.
Yamaguchi	Hagi	Niihori River	Improve water quality of all water ways in the city so as to restore the precious environment of Hagi, a historical castle town.
Fukuoka	Yanagawa	Yanagawa Cannals	Improve all water ways in the city so as to restore the environment of the cannal district which is remembered with connection to Hakushu (historical Japanese post), the property of the citizens.
Kumamoto	Hitoyoshi	Yamada River	Maintain water quality of Kuma River (famous for rifting) as well as to improve water quality of all water ways in the city so as to offer the citizens the opportunities to familiarized with water.

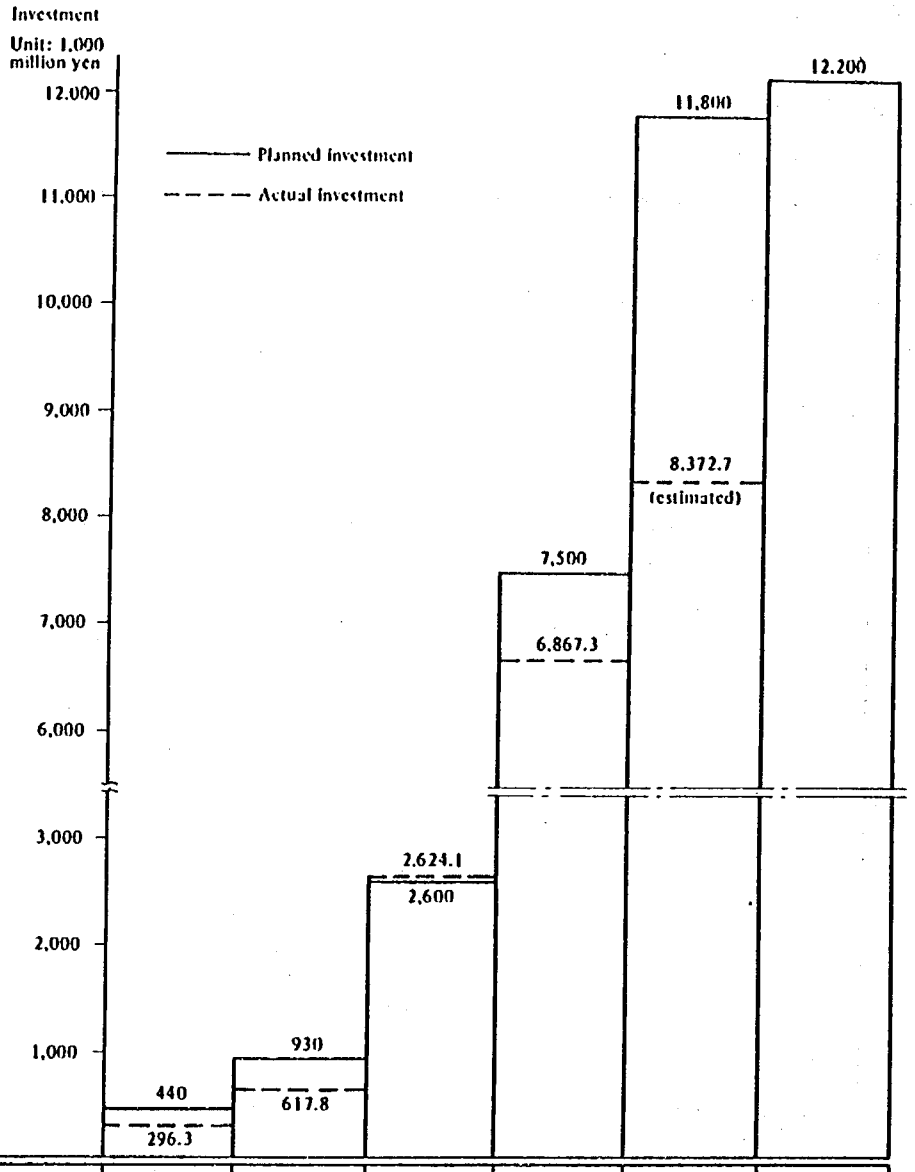
since 1958. As is discernible from these figures, there has been a sharp rise in the outlay for sewerage since the latter half of the 1960s. Particularly in the late 70's, the outlay has increased about tenfold. When it is taken into account that GNP has increased about 3.9 times and the formation by 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, and after early 80's unfortunately, rate of investment has been decreasing due to the recent economical recessions.



Note: GNP and government fixed capital formation of each year before 1964 are based on the old SNA.

Fig. 6 Trends of the Investment to Sewerage Programs

Fig. 7 indicates the investment amounts (planned and actual) under the past five five-year programs and current Sixth Plan. 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 until 5th program.



Planned period	1st 5-year Plan (Fiscal 1963 – 67)	2nd 5-year Plan (Fiscal 1967 – 71)	3rd 5-year Plan (Fiscal 1971 – 75)	4th 5-year Plan (Fiscal 1976 – 80)	5th 5-year Plan (Fiscal 1981 – 85)	6th 5-year Plan (Fiscal 1986 – 90)
Implementation period	(Fiscal 1963 – 66)	(Fiscal 1967 – 70)	(Fiscal 1971 – 75)	(Fiscal 1976 – 80)	(Fiscal 1981 – 85)	(Fiscal 1986 –
Rate of accomplishment	67.3	66.4	100.9	91.6	74.7 (estimated)	
Economic plan (Planned period)	Middle Years Economic Plan (Fiscal 1964 – 68)	Economic Society Development Plan (Fiscal 1967 – 71)	New Economic Society Development Plan (Fiscal 1970 – 75)	Economic Plan for 2nd Half of 1970s (Fiscal 1976 – 80)	7-year Plan for New Economic Society (Fiscal 1979 – 85)	Prospect and Guiding Principle for Economic Society of 80s

Fig. 7 Transition of Five-Year Plans for Sewerage Construction

3. SIXTH FIVE-YEAR PROGRAM FOR SEWERAGE CONSTRUCTION

Ministry of construction started sixth five-year program from FY 1986. Outline of the program is described below.

3.1 Objective

The 6th Five-year Program for Sewege Works Construction aims at promoting the systematic construction of the sewage works in succession to the preceding plan.

The Program targets "to raise sewered rate at the level of the advanced countries in Europe and America. For present, the Program calls for raising the level of coverage in major cities up to those of the advanced countries that is more than 90%. It also calls for further promotion of the sewerage construction in local cities. The Program seeks to improve water quality of stagnant water body of lakes, and of public waters such as rivers. The Program will promote the sewage works for storm-Water drainage of the frequently flooded areas in the cities."

3.2 Priority Targets of the Program

In the 6th Five-year Program, the following ten items are made priority targets:

(a) Sewerage construction shall be promoted to provide the people with better living environment and to respond to the regional environmental pollution control program, earlier accomplishment of environmental water quality standard, Areawide Water Pollutant Reduction Programme, and the lake water quality protection program.

(b) Level of the sewered rate in major cities shall be raised to those of the countries in Europe and America, while the construction of sewerage in regional cities shall be positively promoted.

(c) Sewage works shall be promoted to improve the stormwater drainage of the frequently flooded urban areas, and to enhance general capabilities of stormwater drainage in major cities.

(d) Sewage works shall be positively promoted to conserve invaluable natural environment, and to improve on the living environment in the rural,

mountainous and fishing districts. Furthermore, small public sewerage works shall be constructed in the areas with a population of less than 1,000 if it is needed for specific reasons. (Object of public sewage work has been limited to the district with a population of more than 1,000.)

(e) As a part of the housing and housing lot development programme, the sewage works of newly urbanized areas shall be positively promoted.

(f) Advanced wastwater treatment shall be implemented at the plants in the areas indispensable for the prevention of eutrophication in lakes and for water quality conservation of the rivers of water supply source.

(g) "Model amenity sewage work projects" shall be promoted for creating more pleasant living environment through sewage works.

h) Regional sludge disposal project shall be promoted for promoting a stable sludge treatment and disposal in major city areas. For this, treasury investment and loan shall be introduced to the Japan Sewage Works Agency for executing regional sludge disposal program.

(i) To make the best of resources value of sewage works, effective use of effluent from sewage treatment plant, use of sludge as new resource, and multipurpose use of sewerage facilities shall be promoted.

(j) Resources and energy conservation program shall be promoted along with the development of new technology such as application of biotechnology to wastewater treatment.

3.3 Target Propagation

The new plan sets target for raising the rate of coverage to the population from estimated 36% at the end of fiscal 1985 to 44% at the end of fiscal 1990. Another traget set by the new plan is to raise the rate of storm sewer couvered area from estimated 35% at the end of fiscal 1985 to 43% at the end of fiscal 1990. (Table 2)

Table 2 Target of 6th Program

	FY1985	FY1990
Total Population (10 thousand)	12,030	12,283
Covered Population (10 thousand)	4,333	5,439
Covered Rate (%)	36	44
Storm Sewer Covered Rate (%)	35	13

3.4 Scale of Investment

Relecting the lack of governmental economic plan concerning the public facilities development and the current stringent financial situation, the new five-year plan failed to expand the scale of investment. Least necessary budget for the projects, however, are secured contrary to the actual invest amount for fiscal 1985 which fell short of the required amount. Economic society of Japan now faces a big turning point and her future is full of mobile elements. Therefore, the new plan must be carried out with flexibility, corresponding with economic and financial developments and the progress of the public works. This should explain the larger percentage given the adjustment cost. And the plan is subject to evaluation and revision after three years. Table 3 shows the scale of investment under the new five-year plan.

Table 3. 6th 5-Year Plan for Sewerage Construction

(Unit: 100 million yen)

Classification	5th 5-Year Plan (Fiscal 1981–1985)		6th 5-Year Plan (Fiscal 1986–1990)
	Plan	(Estimated) Actual results	
Subsided works	77,900	54,064	66,800
Works with out subsidy	34,200	29,663	33,000
Total	112,100	83,727	99,800
Adjustment cost	5,900	–	22,200
Grand total cost	118,000	83,727	122,000

3.5 New Measures

(i) Expanded scope of implementation of public sewage work for the protection of specific environment

(a) Objectives

Objectives are to improve on the living environments of the rural, mountainous, and fishing districts that lag far behind the others in modern sewage facilities, and to protect quality of public waters. To implement the objectives, public sewage work for the protection of specific environment (to be called the small public sewerage) shall be newly constructed in the districts with less than 1,000 minimum population requirement.

(b) Contents

Object district is the one with the population of less than 1,000 that requires urgent construction of sewage works specially for the protection of nearby water quality. To be concrete, the object district should fit to one of the following four items:

1) The district is located in the upper stream of the stagnat water area or public water supply sources.

2) The district has invaluable natural environment such as natural parks.

3) The district has heavily polluted peripheral water due to concentration of population such as sight-seeing resort or newly built housing projects.

4) The construction of sewage works in the district in question is more effectively done if integrated in the construction of public sewerage or regional sewerage system.

(ii) Implementation of regional sludge disposal project

(a) Background

In major city areas where three are sewerage system, increased generation of sludge presents problems of heavier cost of treatment and of securing places for final disposal.

To cope with the situation, the Japan Sewage Works Agency shall become the main body for the implementation of "Regional Sludge Disposal Program". Under this program, the Agency summarily collects and disposes sludge generated from more than two of the sewage treatment plants managed by municipalities in major city areas. Figure 8 shows the system of the sludge disposal program.

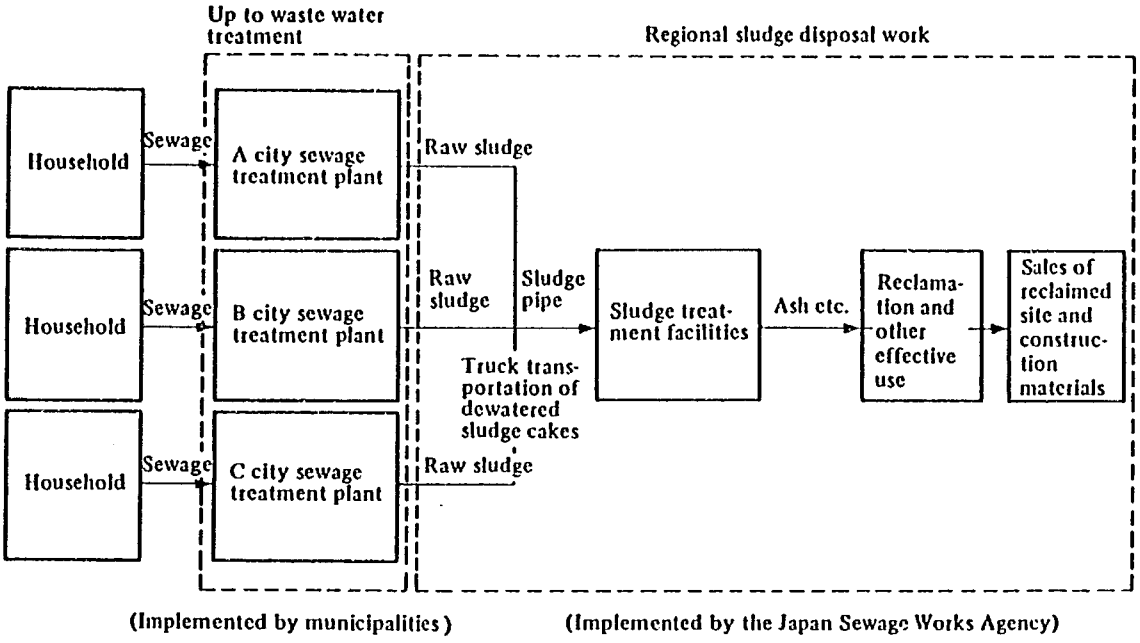


Fig. 8 Regional Sludge Disposal Program Diagram

(b) Outline of program

Japan Sewage Works Agency shall, upon receiving request from the municipalities, carry out construction, maintenance and management of the sludge disposal facilities.

The program shall be financed by treasury subsidy, investment and loan. The Agency shall refund the treasury investment and loan with the proceeds of reclaimed site and construction materials produced by sludge in addition to sludge disposal charges collected from the minicipalities.

(c) Contents of project scheduled for 1986

Regional sludge disposal program is scheduled for implementation in two districts in Hyogo Prefecture for 1986 at the cost of 5,000 million yen. Of the total construction cost, 4,700 million yen will be financed by the treasury investment and loan.

GERMAN ACTS FOR DISCHARGING OF WASTE WATER — OVERVIEW REGARDING TEH AMENDMENTS OF THE ACTS

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1. Introduction

Water resource management in the Federal Republic of Germany aims generally at achieving a reliable and lasting organization of water resources that will

- maintain or restore the ecological balance in bodies of water
- secure a trouble-free water supply for the population and industry
- guarantee also for the future the whole range of water uses.

Ways to achieve these aims are on one side the legislative constraints and on the other side a very intensive promotion of research and development of new technologies for reduction of environmental contamination. For example, the funds only for the water field (industrial and municipal waste water treatment, sludge treatment, drinking water supply, water reuse, etc.) given by the Federal Ministry for Research and Technology since 1975, are approximately 600 million D Marks for 700 Projects. (Funds for water research and also demonstration of new technologies in technical scale, are also given from other institutions, i.e., German Ministry for Interior, Ministries of the different countries of the Federal Republic and by public or private institutions).
The main aims of the BMFT-Programme are described in the "Environmental Research and Technology Programme" as follows:

a) Further developments and improvement of utilization of the existing disposal systems

b) Reduction of emissions of high-risk substances, as well as substitution of environmentally sound materials for questionable substances

c) Sludge treatment with the goal of non-hazardous overall disposal

d) Creation of closed-loop water use systems and reutilization of sewage.

2. General Remarks to the Water Quality

Since in the late 1960s, considerable efforts have been undertaken to clean-up our waste and keep them clean, given the fact that they are subject to pollution by large amounts of household, commercial and industrial effluents. This has been shown, for instance, in the development of local waste water treatment. In the public sector from 1957 to 1984

- the number of persons in households attached to public sewage systems increased from 32 to 56 million
- the percentage of the population whose sewage is treated, increased from 20% to about 80% of the total population
- the amount of treated waste water increased from 24% to about 88%.

The positive trend shown by domestic and industrial waste water treatment has led to considerable improvements in water quality in most of the bodies of water. For example, the following improvements in water quality in the Rhein were noted at the Bimmen/Lobith Monitoring Station on the German-Dutch border (measurements made by the International Rhein Protection Commission):

- The reduction of biodegradable substances brought about a marked rise in the oxygen concentration generally near to the saturation concentration of oxygen (Fig. 1).

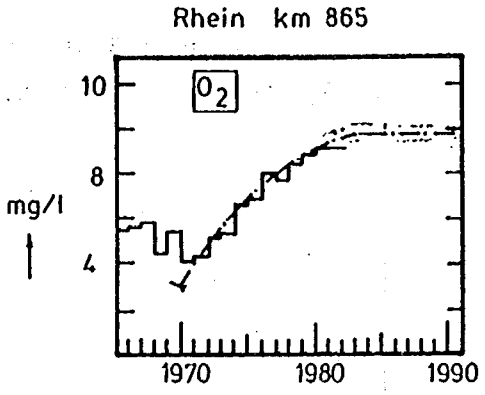


Fig. 1: Oxygen concentration in the Rhein River

- the concentration of biodegradable substances in the Rhein River was between 1973 and 1984 reduced by more than 90%.

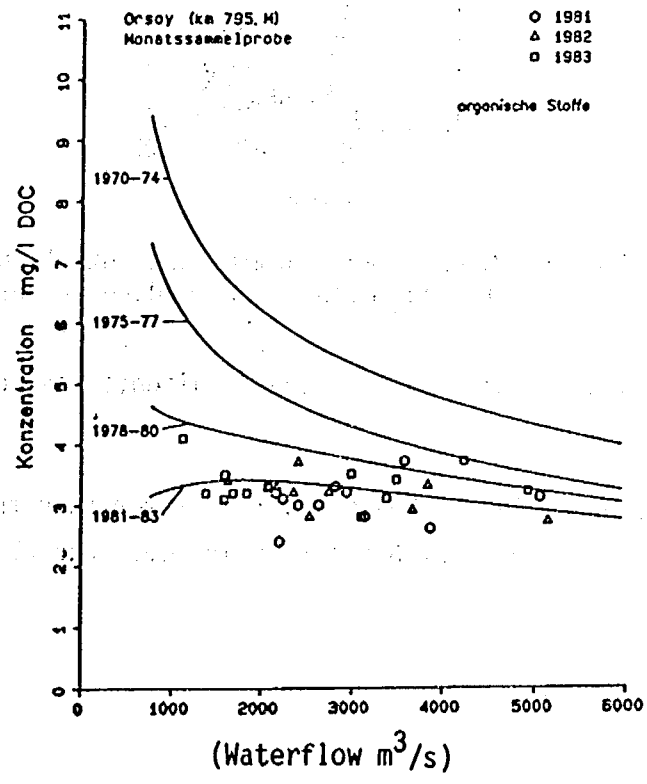


Fig. 2: Concentration of DOC (dissolved organic carbon) in the Rhein River.

- the total content of DOC is perceptibly reduced (Fig. 2).
- Mercury concentrations declied from an average of 3 microgrammes to values below 0.2 microgrammes (Fig. 3). Comparable reductions were observed in the case of other heavy metals

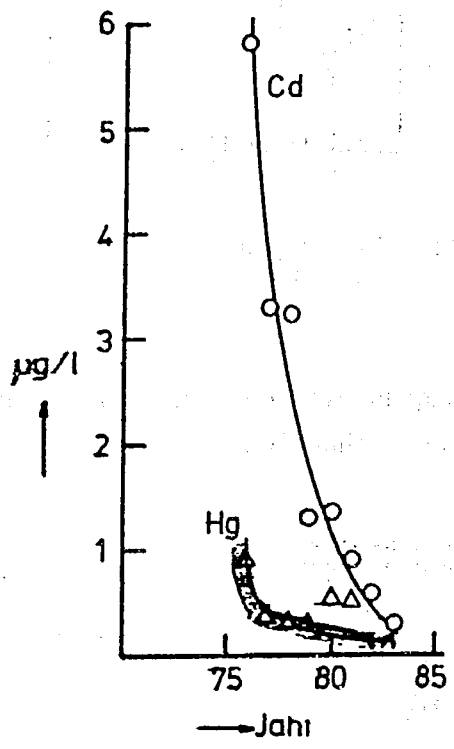


Fig. 3: Mercury and cadmium concentrations in the Rhein River (median values).

- also the content of heavy metals in the river sediments was reduced (Fig. 4).
- the ammonium concentration in the Rhein River is lower than the in the years before 1974, but in the last 3 years an increase was observed.

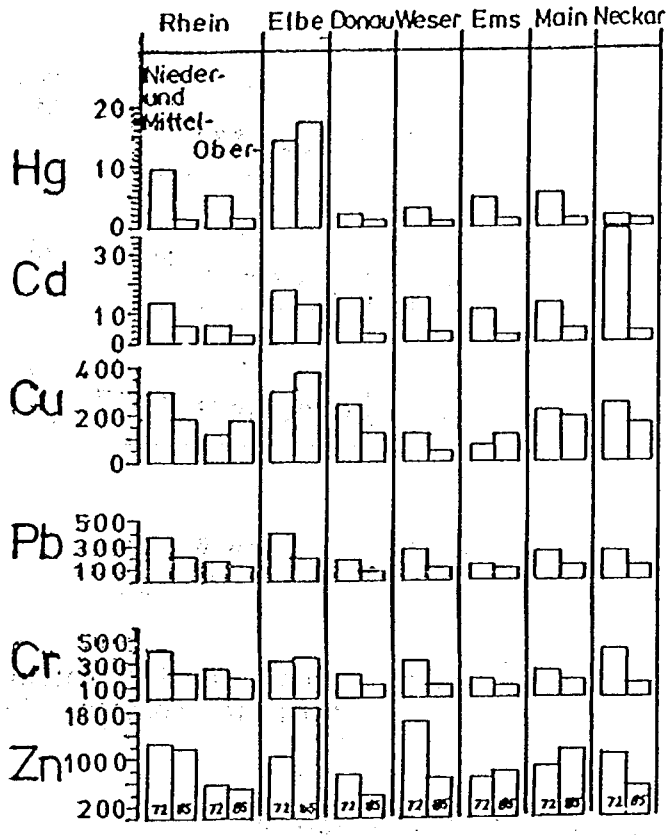


Fig. 4: Content of heavy metals in sediments of German Rivers 1972-1985

Not all questions could be solved, further research and development is necessary in the future. Some examples are:

- the reduction of concentration of hardly biodegradable substances in the Rhein River was only 40%. (No noticeable improvement was measured since 1979/1980!)
- the concentration of the chlororganic compounds are not lowered enough, the median values for the concentration for 1975-1978 were around 100 mg/l, 1979-1984 between 60 and 80 mg/l (measured by the Monitoring Station Wittlaer
- the ammonium concentration in the Rhein River is lower than in the years before 1974, but in the last 3 years an increase was observed
- in some cases, the concentration of inorganic and organic compounds in ground water is slowly rising, (Fig. 5). In particular, the nitrate content of the water, as a source for drinking water supply, is in some places a problem.

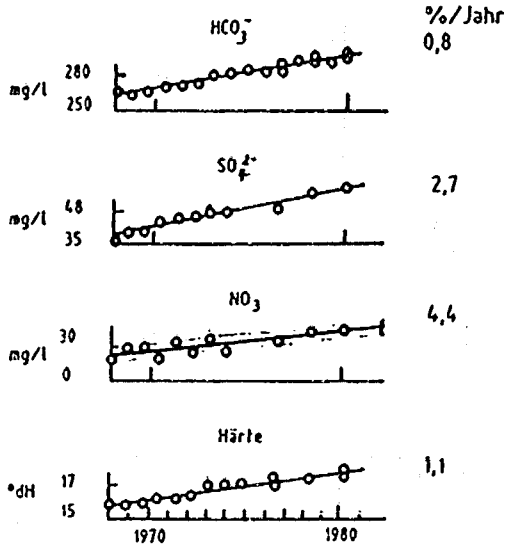


Fig. 5: Example for the content of inorganic substances in ground water

- the phosphorous content must be lowered
- the pollution of the waters from diffuse sources must be lowered

3) The Amendment of the Acts

Details on the main objectives of the most important acts ("old version"), the "Waste Water Management Act" and the "Waste Water Charges Act", were reported on the occasion of the 2nd Japanese-German Workshop, Karlsruhe 1984 by Dr. Kanowski, the 1st Japanese-German Workshop, Tokyo 1982 (short review in the report "Elimination and recovery of heavy metals..." by Dr. Robel) and by Dr. Hibbeln in Japanese papers. The "Waste Water Management Act" have been passed on to Parliament, the "Waste Water Charges Act" is now in discussion.

a) Waste Water Charges Act

Generally, the waste water charge must be paid to the States of the Federal Republic and the receipts must be used for the improvement of water quality, e.g., for the construction of water treatment plants.

The main objectives of this act were:

- a charge is to be paid for discharging waste water into water bodies ("polluter-pays-principle")(change in the new version!)
- the amount of the waste water charge depends on the pollution of the waste water and is measured in units of noxiousness
- the pollution is measured by the following parameters (change in the new version)
  - removable solids
  - chemical oxygen demand (COD)
  - mercury and its compounds
  - cadmium and its compounds
  - toxicity to fish.

When the treatment is better, the pollution in the effluents is lower than required by the minimum requirements", the charge is then lower. If the pollution in the effluents is higher, then the charge will increase.

The proposal for the Second Bill of Amendment to the Waste Water Charges Act from May 1986 is as follows:

The objective of this amendment to the Waste Water Charges Act is to increase the incentive function of waste water charges, thus encouraging further investment by polluters in pollution control with particular respect to the reduction of the water pollution load through dangerous substances, and at the same time to ease enforcement of the Act and to reduce administrative expenditure.

The introduction of the best available technology applying to the discharge of dangerous substances, as proposed in Article 7a Para. 1 of the Bill of a Fifth Amendment to the Federal Water Management Act is to be supported in the case of particular dangerous substances by means of appropriate levying of charges. Furthermore, the differentiated formulation of charge lists for both violation and respect of officially determined values is intended as a means not only of enforcing compliance with these values through liability of charges, but also of acknowledging, through the magnitude of charges, improved purification capacity beyond the requirements of generally recognized rules of technology.

The new Act contains the following fundamental adjustments:

- Levels of organic halogen compounds and of the heavy metals chromium, nickel, lead and copper will be taken into account in the assessment of charges to be paid
- officially determined values will replace maximum, standard and reference values as the basis for the levying of charges
- violation of officially determined values will have as its consequence significantly higher charges which will be oriented to the highest individual parameter
- the surpassing of generally recognized rules of technology and the achievement of the state of the art will be encouraged through substantial reductions in the magnitude of charges.
- for a specified period of time, 50% of investment made in pollution control measures which surpass the generally recognized rules of technology or which introduce state of the art technology can be offset against waste water charges.

Payment is also being planned for the following additional parameters in the future:

- Adsorbable organic halides (AOX),
- Chromium, lead, copper and nickel.

The "units of noxiousness" are also in discussion, the last proposal is as follows:

Pollutants	1 unit of noxiousness is:	Proposed values for concentration and amount per year
COD	50 kg	20 mg/l 250 - 500 kg/year
AOX	2 kg	100 µg/l 10 - 20 kg/year
Metals and its compounds:		
Hg	20 g	1 - 2 µg/l 100 - 200 kg/year
Cd	100 g	2 - 5 µg/l 0,5 - 1000 kg/year
Cr, Ni	500 g	30 - 50 µg/l 2,5 - 5 kg/year
Pb	500 g	50 - 100 µg/l 2,5 - 5 kg/year
Cu	1000 g	100 - 150 µg/l 5 - 10 kg/year

b) Waste Water Management Act

The fifth amendment of the Waste Water Management Act is intended to further improve protection of surface water and ground water. In particular, the intention is to further reduce the concentrations of dangerous pollutants such as heavy metals and organic halogen compounds in bodies of water.

The following changes, among others, are important:

a) in the future, waste water containing hazardous substances is to be treated using "the best available technologies" prior to being discharged into the sewer system or into water bodies

b) the possibility of establishing water protection zones is to be expanded

c) technical requirements for ground water protection are to be imposed on plants that produce, treat and use substances that pose a danger to water quality.

(Remark to a): The old version was -

(A licence for the discharge of waste water is only to be granted if the quantity and the pollution of the waste water are kept as low as possible when applying procedures that are based on the "generally acknowledged rules of technology").

One of the most important article is number 7:

a) A licence for the discharge of wate water may only be granted if the quantity of dangerous substances in the waste water is kept as low as possible through the observance of (adherence to) appropriate requirements in sentence 3, at least however, according to the best available technology.

The Federal Government, with the approval of Bundesrat (Federal States Chambers) enacts general administrative regulations about minimum requirements, which accord with "best available technology".
If there are substances of well-defined origin or groups of substances which are to be considered as dangerous due to suspicion of toxicity, longevity persistence, cumulative tendency, carcinogenic property or potential detrimental effects on crops or genetic codes, the requirements in the general administrative regulations have to accord to the standard of technology.....

The Federal Government, with the approval of the Bundesrat (Federal States Chambers) determines through statutory ordinance on areas of origin of waste water in the sense of Paragraph 3 which contains dangerous substances.

b)

"The countries of the Federal Republic shall also ensure that the necessary steps, in the sense of Environmental Protection, are enforced before the discharge of waste water with dangerous substances into the municipal sewerage systems".

The definition of "dangerous substances" is also in preparation. These are, for example, heavy metals and especially chlororganic coumpounds.

At this time, on proposal of the "Umweltbundesamt" and the "Bundesgesundheitsamt", a list of 393 substances is in discussion.

According to Article 7, the Federal Government enacts general administrative regulations concerning Minimum Requirements for the discharge of effluents. (Remarks: detailed discussions of these points, especially regarding the Minimum Requirmeents, see paper by Dr. Kanowski, 2nd Japanese-German Workshop 1984).

For waste water which contains "dangerous" substances, at this time are worked out new minimum requirements, which are based on the new request, that waste water is to be treated according to the "best available technology".

Some examples for the now discussed values are:

a) Leachates from dumping grounds: Tabs. 1 and 2

b) Pulp and Paper Industry:

AOX	1 kg/t pulp
COD	40 - 60 kg COD/t pulp
BOD_5	5 kg/ t pulp

	proposed effluent concentration (after treatment!)(mg/l)	purification efficiency (%)
BOD_5	15 - 30	η = 98 %
COD	50 - 300	η = 90 %
AOX	1 - 2 or 300	open
G_F (fish toxicity)		-
NH_4-N	5 - 30	

Tab. 1 : Proposed values for the Minimum Requirement according to the Federal Water Management Act § 7 for leachates from dumping grounds (now in discussion!)

Parameter		Concentration	
		min.	max.
pH		5.9	6.8
conductivity	μS/cm	$0.9\ 10^5$	$2.91\ 10^5$
BOB_5	mg/l	2600	5800
COB	mg/l	10800	20250
Phenols (total)	mg/l	104	260
Cyanide (total)	mg/l	0.23	1.1
Chloride	mg/l	57300	113100
NH_4-N	mg/l	3350	6800
Zn	mg/l	4.4	23.0
Cr. total	mg/l	<0.2	<0.5
Cd	mg/l	<0.1	<0.1
Hg	mg/l	0.002	0.05
Methylenchlorid	μg/l	11200	38600
Chloroform	μg/l	220	1060
Trichloräthylen	μg/l	370	8750
Tetrachloräthylen	μg/l	280	27100

Tab. 2 : Example for the concentration of some pollutants in leachates from dumping grounds

SESSION 3

SLUDGE TREATMENT

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NEWLY DEVELOPED GRAVITATIONAL THICKENING PROCESS FOR SLUDGE

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ABSTRACT

A new method to increase solid concentration of waste activated sludge thickened in the gravitational thickening process is proposed through experiments, and two basic designs of the thickening system based on the method are proposed to solve problems of low concentration of sludge thickened in Japan. The method utilizes the characteristics of gas production by waste activated sludge in the anoxic and anaerobic stages. Thickening of sludge by the method is conducted during a period in which the rate of gas production from sludge reduces, following a period of denitrification and being followed by the process of methane gas production. The period available for thickening ranges from 30 hours to 75 hours, depending on the temperature and initial nitrate concentration in sludge. Simulation results of the thickening process indicate that the solid concentration of sludge thickened by the method corresponds to that in the case of no gas production. As one design, the addition of a gas purging tank between the final settling tank and the thickener is proposed to purge gas produced by denitrification. Another is a thickener with double rakes, the upper one of which rotates faster than the lower.

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Newly Developed Gravitational Thickening Process for Sludge

1.Introduction

Solid concentration of waste activated sludge (WAS) thickened in gravitational thickeners is reported to range from 1.0 to 4.5% as total solids (TS) in Japan[1]. The solid concentration is low in comparison with that in European and American countries[2), 3)] and still has the tendency to decrease. This is due to the gradual change in living customs and diet of the Japanese. Because gravitational thickners in Japan have been designed for sludge to be withdrawn at 4% of TS, the decrease in the TS of the thickened sludge causes severe problems for sludge and waste water treatment such that the solid concentration of influent to digesters is lower than the design value and that returned liquid, that is, supernatant in thickners, to primary settling tanks is very high in solid

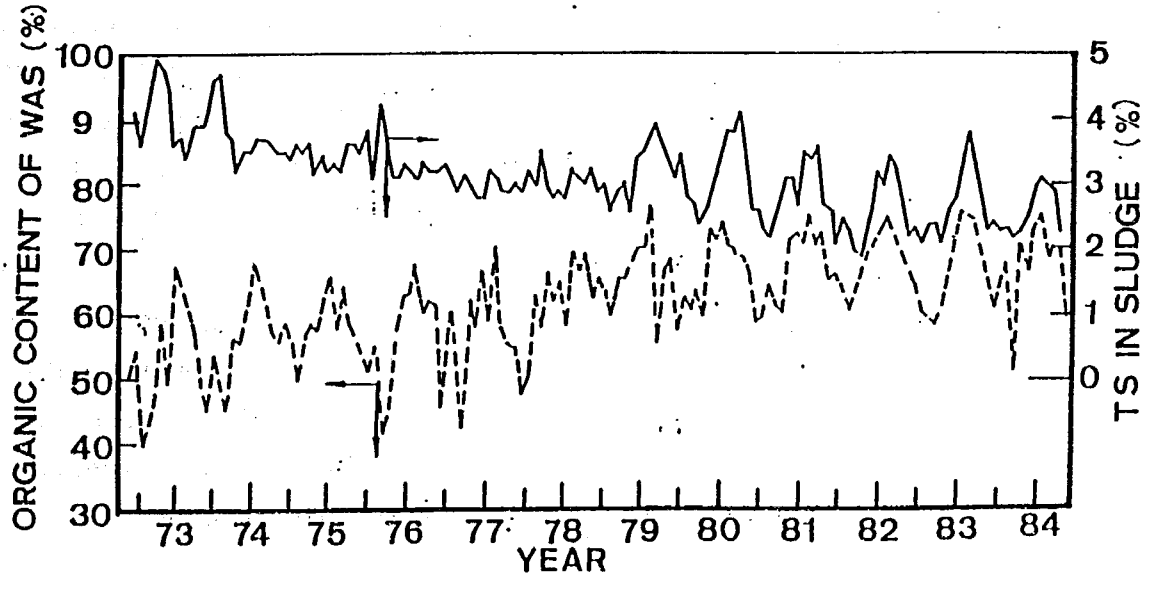


Fig. 1 Changes of TS in thickened sludge and organic content of WAS

concentration. Figure 1, for example, shows changes of TS and organic content of WAS in a municipal waste water treatment plant in Nagoya [4]. As a trend, the organic content of WAS in this case has been increasing while the TS, on the contrary, decreasing year after year. Until 1976, the increase in organic content of WAS had been related with the decrease in TS. This might be deeply related to the density changes in the sludge. Since 1976, the organic content of WAS periodically and concurrently increases with TS in winter, and decreases in summer. Since the recent seasonal changes of organic content of WAS and TS are related to water temperature and biological activity, the decrease in TS in summer is considered to be caused by gas production from sludge, not by the decrease in the density of sludge. Since the characteristics of sludge have been changing, research must have been conducted on processing and managing WAS. In the thickening process of WAS, the gravitational thickening process consumes less energy than mechanical ones. Although the former has been replaced to the latter to avoid low concentration problems for SS in Japan, the process should be employed on as large a scale as possible in order to save energy.

The foundamental method to increase or, at least, not to decrease the concentration of concentrated sludge in thickeners is to increase or not to decrease sludge particle density, the permeability of sludge, and solid fraction at which effective pressure begins to appear in sludge.

The most important one is the first one. Gas production in the thickening process decreases the final concentration of WAS. In previous works[5), 6)], the gas production from primary and mixed sludges and aged WAS in the stage of fermentation was studied for thickening. They expressed that the gas production rate in primary sludge during thickening is larger than that in WAS. Constituents of the gas are mainly CO_2 and CH_4. The CO_2 production rate depends on the concentrations of dissolved matters and dissolved total phosphate, and the CH_4 production rate depends on the concentrations of dissolved matters and COD. The gas production from fresh WAS in gravitational thickening, however, is less investigated. This study is directed at developing a new method to increase the withdrawal concentration of fresh WAS in the gravitational thickening process.

2. Gas Production from Sludge

Experiments were conducted in order to examine the mechanism of gas production in sludge.

2-1 Apparatus, Method, and Material

WAS from return sludge pits in the Eastern waste water treatment plant of Fukuoka in western Japan was employed as experimental material. The WAS was conditioned by either aeration for a prescribed period or nitrate addition. A gas collector to measure the volume of gas produced and to

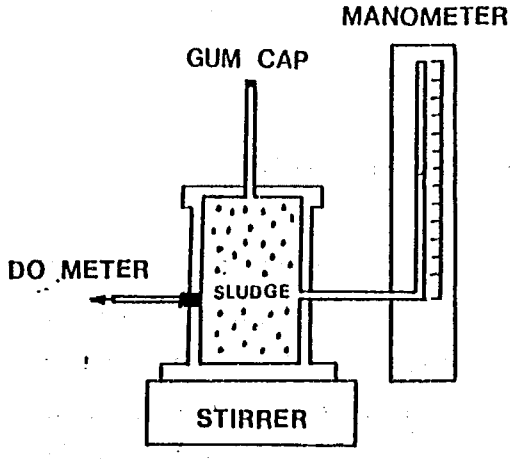


Fig. 2 Gas sampling device

sample the gas, as illustrated in Fig. 2, consists of two parts, the main tank (1.6 l in volume) and a manometer which is able to tilt to keep gas pressure equivalent to that of atmosphere. Another one, to sample sludge at prescribed times, as shown in Fig. 3, is 5l in volume with a balloon inside it to compensate for the loss volume of sludge withdrawn. Produced gas constituents were analyzed by a gas-chromatograph with TCD and Molecular Sieve 5A and Silicagel columns. The sampled sludge was centrifuged, and alkalinity, nitrate-nitrogen, nitrite-nitrogen, and ammoniacal-nitrogen in it were analyzed. All experiments were conducted at 20°C.

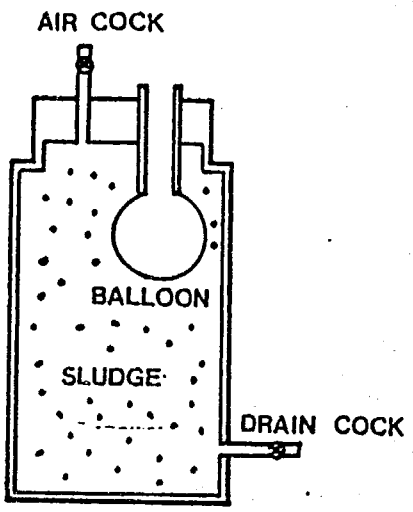


Fig. 3 Sludge tank

2-2 Experimental Results and Discussion

The properties of sludge employed in the experiments are listed in Table 1. The accumulated volume of gas produced by sludge is depicted in Fig. 4 for Series 1(nitrate addition) and in Fig. 5 for Series 2(preaeration). Gas production for each run shows a similar pattern. The constituents of gas produced are depicted in Fig. 6 and Fig. 7. In case of

Table 1 Properties of sludge

Series	RUN	SS (mg/l)	SVI	Ig-Loss (%)	Wet Sludge Density ρ_s (t/m³)	Fluid Density ρ_l (t/m³)	ε_s	Aeration Time(h)	Added NO_3-N (mg/l)	Alkalinity (mg/l)	NO_3-N (mg/l)	NH_4-N (mg/l)
1	1	5.800	178	70.7	.....	.....	.....	.....	0	90	12.0	0
	2								20	120	34.0	1.3
	3								40	114	54.0	1.3
	4	4.980	189	80.3	1.025	1.0006	0.092	.....	0	120	4.0	6.3
	5								10	135	11.0	6.7
	6								20	140	21.6	6.7
	7	5.180	180	76.9	1.004	1.0002	0.085	.....	0	83	14.6	2.0
	8								25	92	42.8	2.0
	9								35	90	48.5	2.0
2	1	5.280	186	82.8	1.03	1.000	0.087	0	.....	114	6.1	3.8
	2							3		81	10.3	0.54
	3							6		62	14.9	0.15

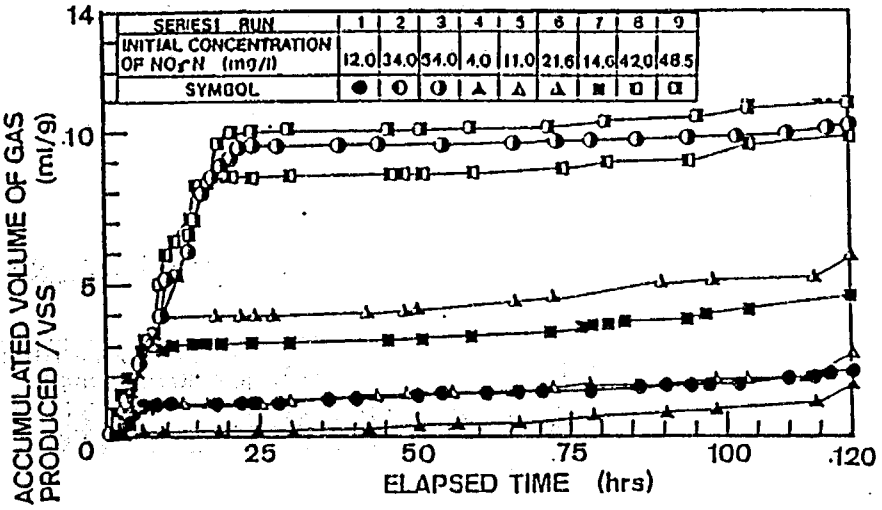


Fig. 4 Accumulated volume of gas produced (Series 1)

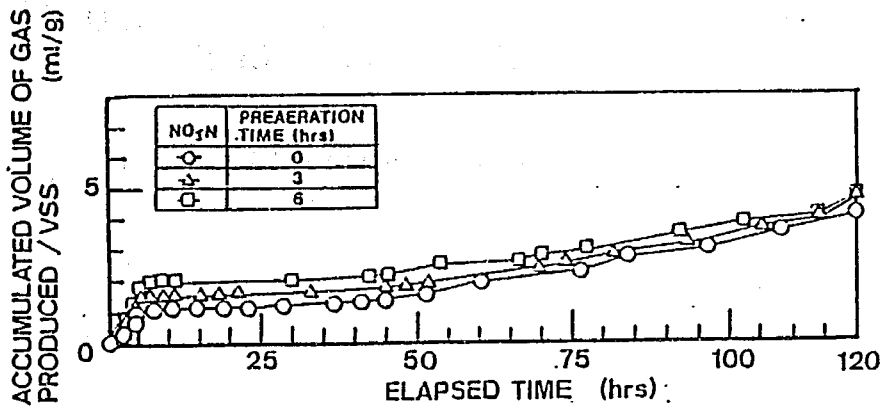


Fig. 5 Accumulated volume of gas produced (Series 2)

Run 3 in Series 1, gas is produced rapidly for the first 25 hours which includes nitrogen(95%) and carbon dioxide(5%). After that, the gas production rate drops drastically. This period is hereafter called the period of reduced rate of gas production, followed by the period of methane gas production which shows a large gas production rate again, after 110 hours from the test started. In case of Run 2 in Series 2 as shown in Fig. 5, the period of large gas production rate is 10 hours, and the period of reduced rate of gas production is 35 hours. The concentrations of NH_4-N, NO_2-N, and NO_3-N, and alkalinity concurrently measured with gas production volume in interstitial water are displayed in Fig. 8 to 12.

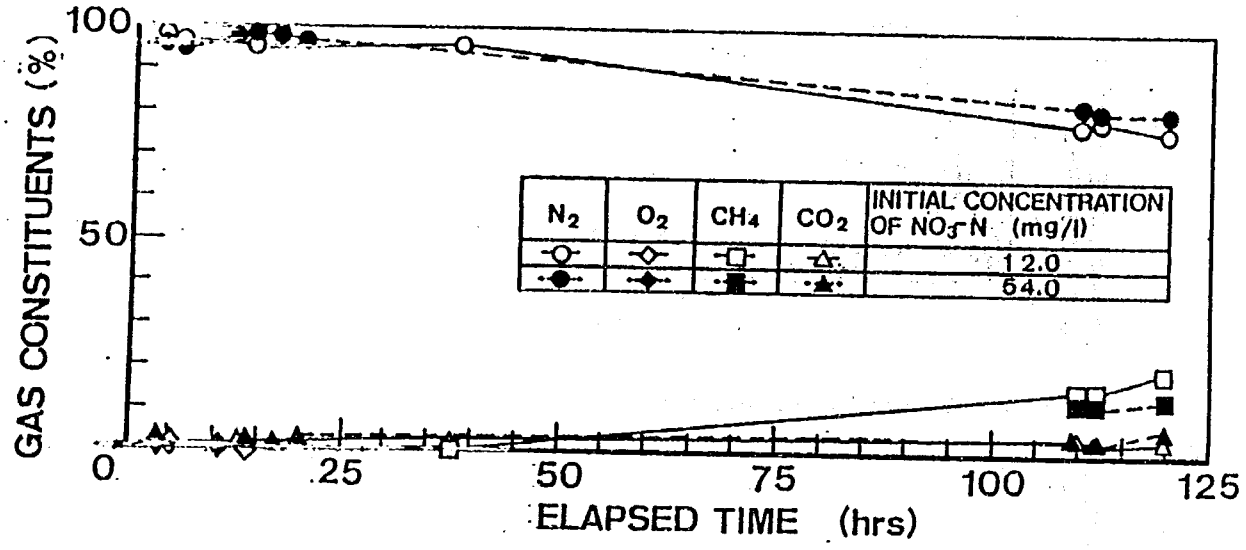


Fig. 6 Change of gas constituents (Series 1)

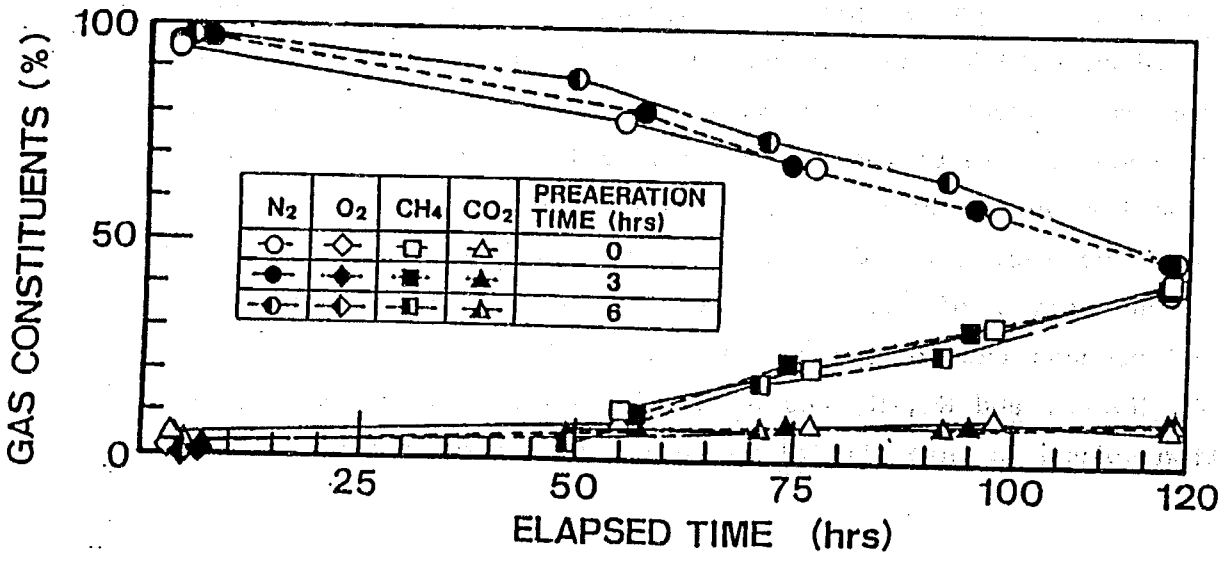


Fig. 7 Change of gas constituents (Series 2)

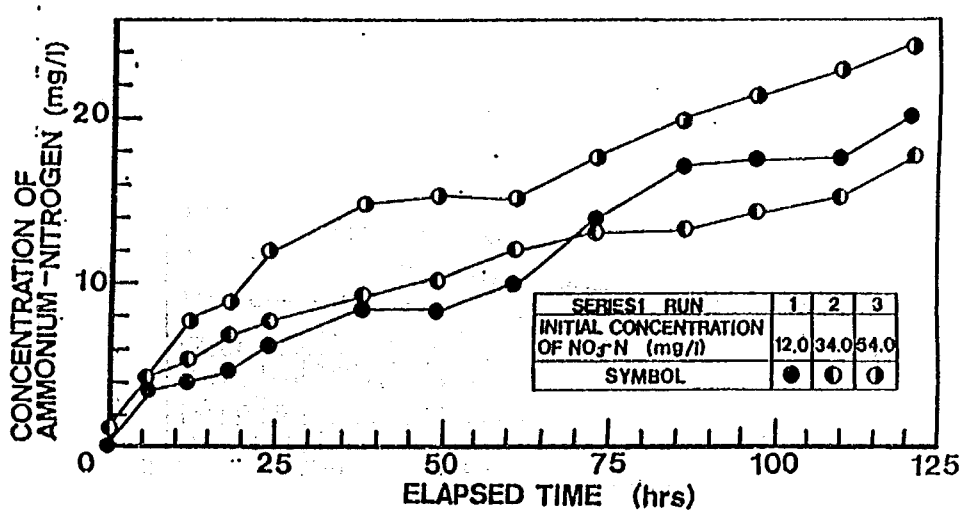


Fig. 8 Ammoniacal nitrogen concentration in interstitial water (Series 1)

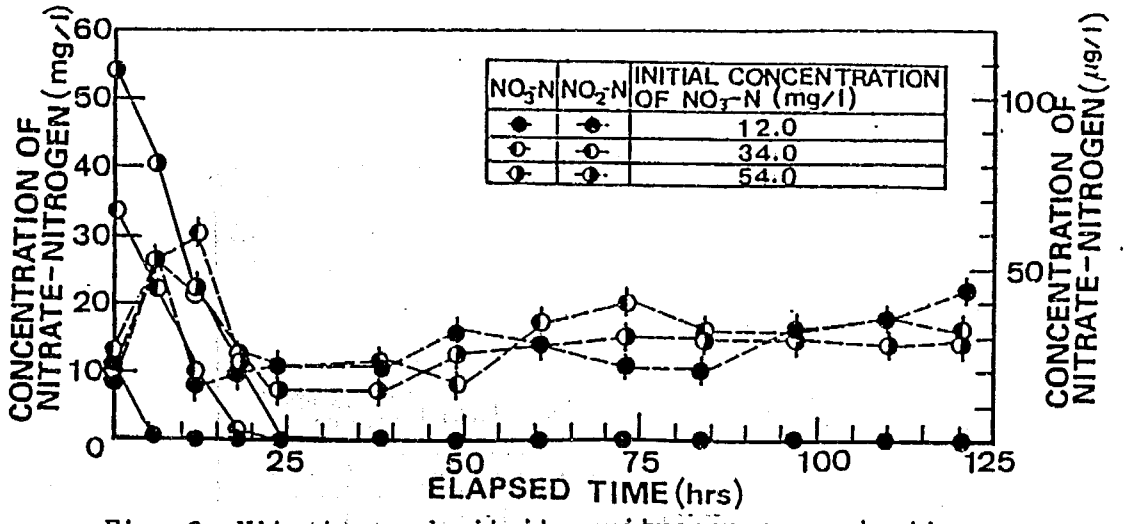


Fig. 9 Nitrate- and nitrite- nitrogen concentration in interstitial water (Series 1)

Initial concentrations of nitrate-nitrogen of Runs 1,2 and 3 in Fig. 9 were 12, 34, and 54 mg/l. The first one, 12 mg/l, is the concentration in raw sludge. In other runs, nitrate-nitrogen was added to raw sludge. After the depletion of dissolved oxygen within 30 min after tests started, the reduction of nitrate-nitrogen started and finished after 6, 18, and 25 hours depending on each initial concentration. Alkalinity in these runs in Fig. 10 increases as time elapses. The results in Fig. 4, 6, 8, and 9 express that gas in the period of large gas production rate is produced by denitrification and the increase in NH_4-N results from the decomposition of organic matter under anaerobic condition. Even in Series 2 in which pre-aeration was conducted, the concentrations of NH_4-N and NO_3-N and

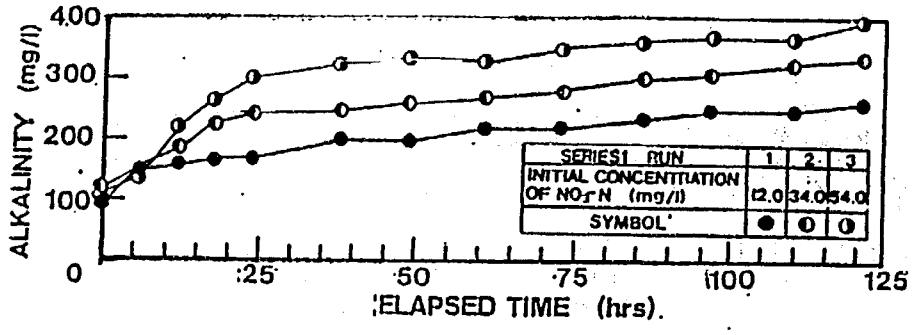


Fig.10 Alkalinity in interstitial water (Series 1)

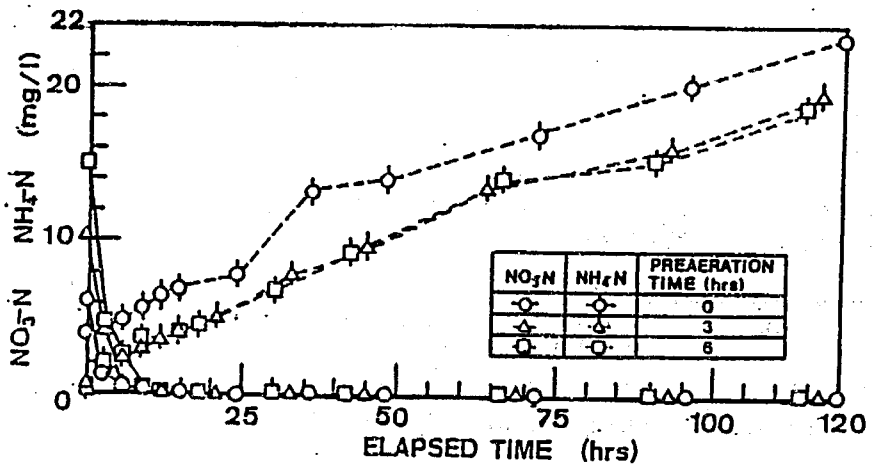


Fig.11 Ammoniacal and nitrate nitrogen in interstitial water (Series 2)

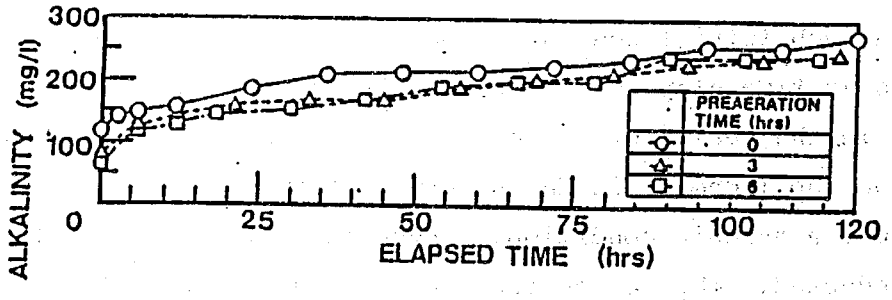


Fig.12 Alkalinity in interstitial water (Series 2)

alkalinity change in similar ways to those in Series 1. This explains that ammoniacal nitrogen is converted to nitrate nitrogen by aeration, and that aeration corresponds to nitrate addition. The nitrate-nitrogen reduction in these series is the 0th order reaction because of a small value of the saturation constant, about 0.1 mg/l, of the Monod equation [7]. A rate constant of the reaction is 0.00035 NO_3-N/g VSS hr. The reduction of 1 mg NO_3 makes 0.858 ml of N_2 gas at 20°C. Produced gas volume shown in Fig. 13 is in good agreement with the theoretical value. Up to 3 mg/l of

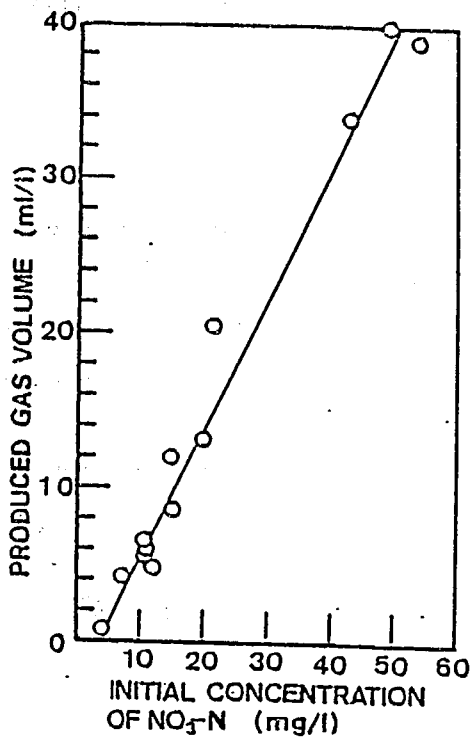


Fig.13 Produced gas volume

NO_3-N, no gas production is observed. This corresponds to the soluble amount of nitrogen gas. Since the rate of denitrification is linear with respect to NO_3-N concentration, the denitrification period is related to the NO_3-N concentration per unit mass of VSS as shown in Fig. 14. Alkalinity after denitrification increases in propotion to the initial NO_3-N concentration. The alkalinity increased to 3.57 mg for 1 mg of NO_3-N in

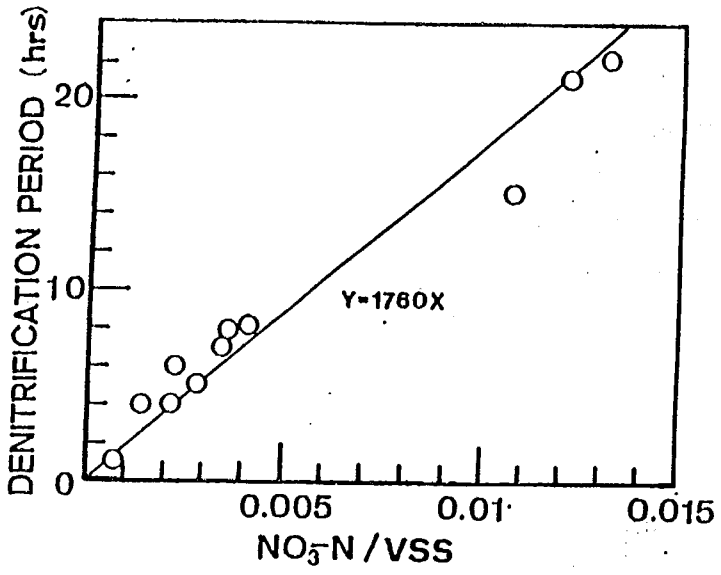


Fig.14 Denitrification period

these series, which coincides with the theoretical value. The period of floatation of sludge is defined as a period from the completion of denitrification to the initiation of floatation of sludge by the attachment of all bubble of gas produced. Figure 15 illustrates the relationship

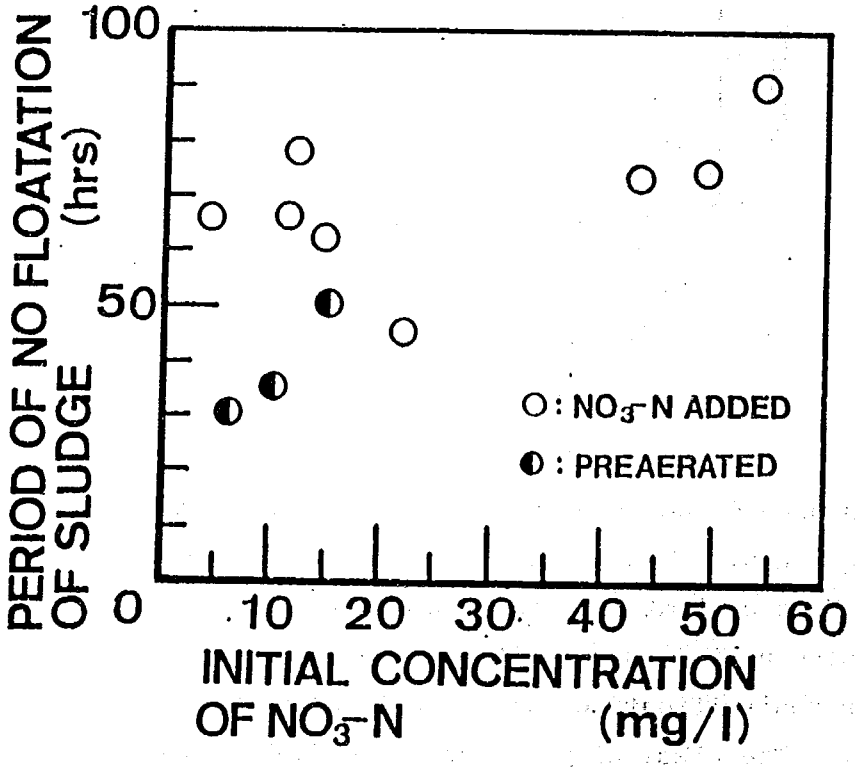


Fig.15 Sludge floatation time in the period of reduced rate of gas production

between the period of no floatation of sludge and the initial concentration of NO_3-N. The minimun period of no floatation is 30 hours. In Series 1, the relationship is not related to the initial concentration of NO_3 -N, in Series 2, however, it is. The inhibition of methane production by nitrate addition [8] and the reduction of the rate of methane production by ammonia [9] were not clearly detected in this study. As a result of these experiments, the gas production process is divided into three phases: the period of large gas production rate by denitrification; the period of reduced rate of gas production; the period of methane gas production. According to the results mentioned above, gravitational thickening should be conducted after purging gas bubbles by denitrification besides within the period of reduced rate of gas production. The completion of denitrification is detected by the change of NO_3-N concentration and ORP change. Tanaka et al [10] obtained -60 to -200 mV of ORP as values of the end of nitrification. In this study, -100 mV was obtained for it, its accuracy, however,is not high.

3. Sludge Thickening Test

Sludge thickening tests in the period of reduced rate of gas production were conducted and the test results were compared with those of simulation without gas production.

3-1 Apparatus, Method and Material

WAS from the same treatment plant mentioned before was employed as experimental material. After the completion of denitrification and gas purging, WAS was poured into a test column (21.5 cm inner diameter and 3 m in height) which was installed short pipes along its wall to detect excess water pressure as shown in Fig. 16. A pressure transducer is wire gauge type one (PC-10GC, Kyowadengyo Co.). Sludge particle movement was measured by the movement of markers which are 3 mm in diameter and 1 cm long and made of polystyrene with steel strips for the adjustment of submerged density. At the initial stage, these makers are fixed along the wall by magnet. When sludge goes into the thickening phase, the makers are released and move with sludge particles. After the end of test, thickened

sludge was taken out layer by layer and its water content and density were measured. Gas production tests were concurrently conducted with the thickening tests.

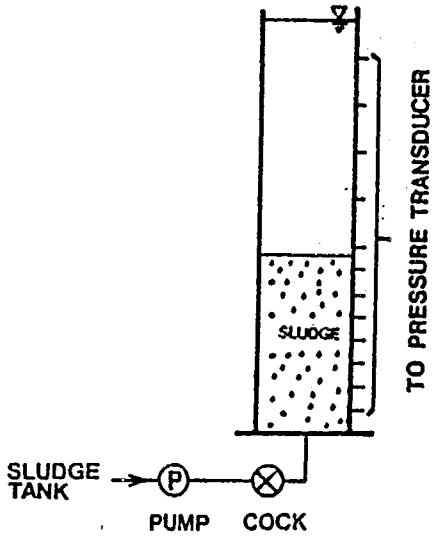


Fig.16 Settling column

3-2 Theoretical Expression of Gravitational Thickening Process

Sludge consists of three phases, solid, liquid, and gas phases. Kinetic and mass conservation equations for the thickening process are to be established for each phase. Assumptions to establish the equations are as follows:

1) The properties of sludge are homogeneous;
2) No compression of solid parts of sludge exists;
3) Interstitial water is drained one-dimensionally;
4) Sludge thickens one-dimensionally;
5) Reynolds numbers of the movements of liquid and solids are smaller than unity;
6) Produced gas distributes homogeneously;
7) Produced gas completely attaches to solid particles of sludge and moves with them;

8) The outer boundaries of gas bubbles function like solid boundary in movement due to very small bubbles;
9) The volume reduction of sludge by gas production is negligible;
10) Wall effects on movement is not considered;
11) The compression of gas produced by water pressure is negligible;
12) Sludge is saturated with gas produced during the period of reduced gas production.

Since the submerged weight of solid particles is supported by effective stress and excess pressure, the kinetic equations of total, solid and liquid phases are written as follows[11] :

TOTAL:

$$\frac{\partial P_e}{\partial x} + \frac{\partial P_s}{\partial x} + \{\varepsilon_s(\rho_s-\rho_e)+\varepsilon_g(\rho_g-\rho_e)\}g = 0 \tag{1}$$

SOLID:

$$\frac{\partial P_s}{\partial x} - \frac{\mu}{K}\left(V_e - \frac{\varepsilon_e}{\varepsilon_s}V_s\right)+\{\varepsilon_s(\rho_s-\rho_e)+\varepsilon_g(\rho_g-\rho_e)\}g = 0 \tag{2}$$

LIQUID:

$$\frac{\partial P_e}{\partial x} + \frac{\mu}{K}\left(V_e - \frac{\varepsilon_e}{\varepsilon_s}V_s\right) = 0 \tag{3}$$

Mass conservation equations of total, solid, liquid, and gas phases are derived as follows:

TOTAL:

$$V_s+V_e+V_g = q_b(t)+\int_0^x q_g\,dx \tag{4}$$

SOLID:

$$\frac{\partial \varepsilon_s}{\partial t} + \frac{\partial V_s}{\partial x} = 0 \tag{5}$$

LIQUID:

$$\frac{\partial \varepsilon_e}{\partial t} + \frac{\partial V_e}{\partial x} = 0 \tag{6}$$

GAS:

$$\frac{\partial \varepsilon_g}{\partial t} + \frac{\partial V_g}{\partial x} = q_g \tag{7}$$

$$\varepsilon_g + \varepsilon_s + \varepsilon_\ell = 1 \tag{8}$$

$$\varepsilon_g = \varepsilon_s \rho_s \alpha t \tag{9}$$

$$V_\ell = \varepsilon_\ell \cdot v_\ell \tag{10}$$

$$V_s = \varepsilon_s v_s \tag{11}$$

$$V_g = \varepsilon_g v_s \tag{12}$$

where V: volume flux per unit area, v: real velocity, ε: solid fraction, ρ: density, suffix g, s, and ℓ: gas, solid, and liquid phases, P_s :effective pressure, P_ℓ : excess pressure in liquid phase, μ: viscosity, K: coefficient of permeability, q_b : total volume flux per unit area at the bottom, q_g : gas production rate, α : gas production coefficient.

The coordinate (x, t) is, for the convenience of computation, to be converted to a mass coordinate(ω, t), where ω is mass of sludge, based on the following equation.

$$\omega = \int_0^x \varepsilon_s \rho_s \, dx \tag{13}$$

Differentiation of the kinetic and mass conservation equations is converted as follows;

$$\left(\frac{\partial}{\partial x}\right)_t = \varepsilon_s \rho_s \left(\frac{\partial}{\partial \omega}\right)_t \tag{14}$$

$$\left(\frac{\partial}{\partial t}\right)_x = -\rho_s V_s \left(\frac{\partial}{\partial \omega}\right)_t + \left(\frac{\partial}{\partial t}\right)_\omega \tag{15}$$

3-3 Experimental Results and Discussion

The properties of sludge employed in experiments are listed in Table

2. Since α is given, P_s and v_s are directly measured, K, coefficient of

Table 2 Properties of sludge

Series	RUN	SS (mg/l)	SVI	Ig-Loss (%)	Wet Sludge Density ρ_s (t/m³)	Fluid Density ρ_l (t/m³)	ε_s	Alkalinity (mg/l)	NO_3-N (mg/l)	NH_4-N (mg/l)
3	1	5,980	164	76.4	1.023	1.0021	0.1	95	9.5	1.0
	2	4,550	214	76.6	1.022	1.0016	0.08	85	7.1	1.0

permeability, is obtained from Eqs. 3,7,8,10, and 11, letting $q_b=0$. In a similar way, putting

$$\frac{\partial P_s}{\partial x} = \frac{dP_s}{d\varepsilon}\frac{\partial \varepsilon}{\partial x} \tag{16}$$

and using Eq.2, P_s is obtained.

Gas production in Run 1 of Series 3 is given in Fig.17. The period of reduced rate of gas production starts 10 hours after the test is started.

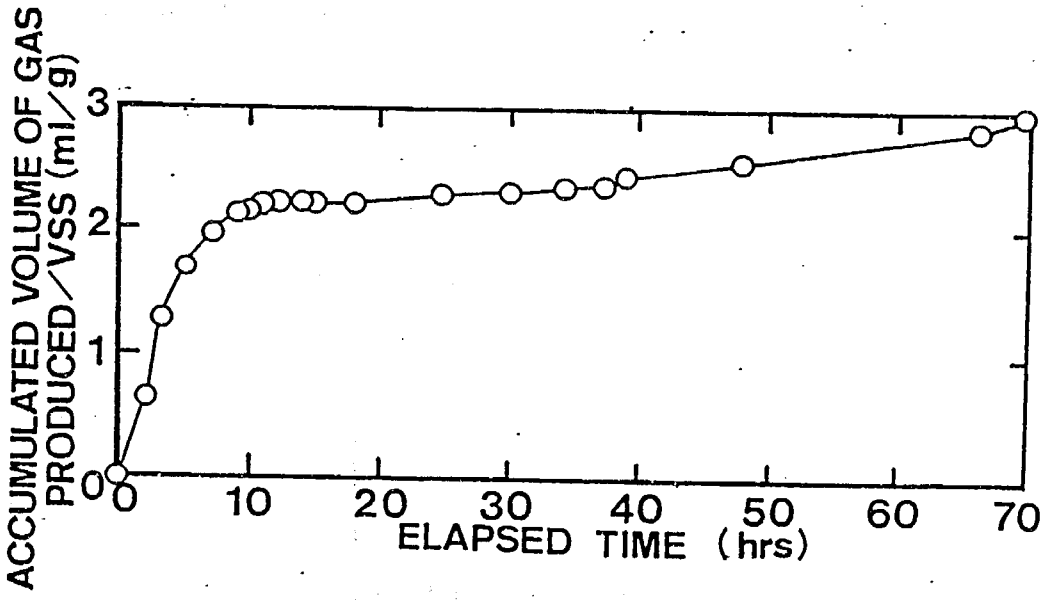


Fig.17 Accumulated volume of gas produced (Series 3, Run 1)

A coefficient of permeability and effective pressure were obtained as shown in Figs. 18-20. The results are, for Run 1 of Series 3,

$$\rho_l g K = 3.22 \times 10^{-6} \{ \varepsilon_v / (1 - \varepsilon_v) \}^{1.73} \tag{17}$$

for Run 2 of Series 3

$$\rho_l g K = 2.43 \times 10^{-5} \{ \varepsilon_v / (1 - \varepsilon_v) \}^{1.52} \tag{18}$$

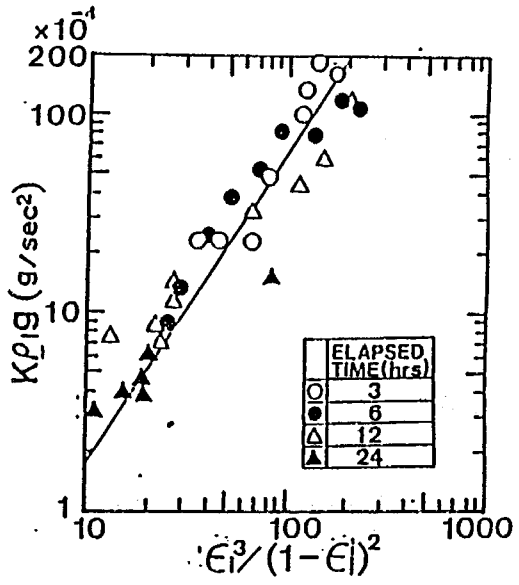


Fig.18 Coefficient of permeability (Series 3, Run 1)

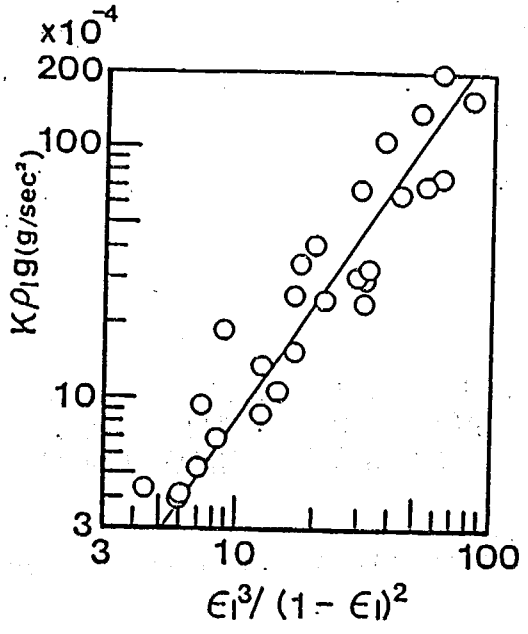


Fig.19 Coefficient of permeability (Series 3, Run 2)

for both

$$P_s/(\rho_1 g) = 2.1(\epsilon_s - \epsilon_{sc})^{1.6} \tag{19}$$

where ϵ_{sc} : critical solid fraction at which effective pressure is generated. Gas production coefficient, α, in the period of reduced rate of gas production is for Run 1 of Series 3,

$$\alpha = 7.718 \times 10^{-3} (\mathrm{ml/gs}) \tag{20}$$

for Run 2 of Series 3,

$$\alpha = 1.2025 \times 10^{-7} (\mathrm{ml/gs}) \tag{21}$$

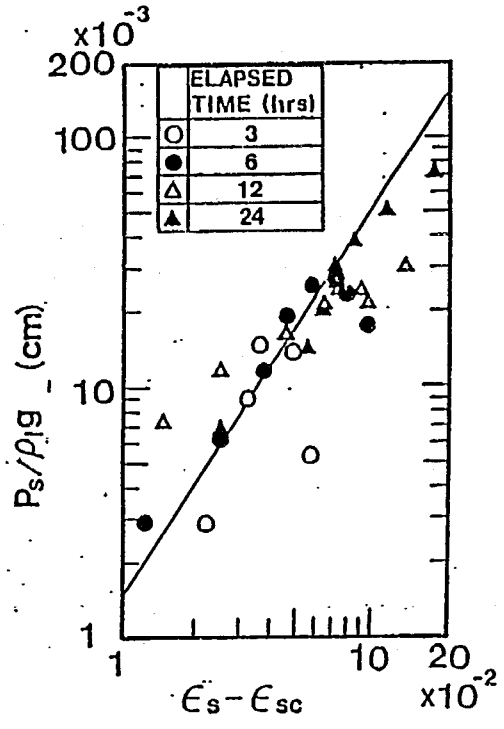


Fig.20 Effective pressure (Series 3)

4. Numerical Simulation of Thickening Process

Gravitational thickening in the period of reduced rate of gas production is simulated and discussed here, based on the relationships expressed in the previous section.

4-1 Simulation Scheme on Batch Test

When ε_s at time t is known and K and P_s are given as functions of ε_s, V_s at time t is expressed by Eq. 22 by use of the relationships in terms of the mass coordinate(ω, t).

$$V_{s,t,m+\frac{1}{2}} = \left[q_b(t) + \int_0^{w} q_g(w)dw - \frac{K(\varepsilon_{s,m})}{\mu} \cdot \varepsilon_{s,t,m} \left\{ (\rho_s - \rho_e)g + (\rho_g - \rho_e)g + \rho_s \frac{dP_s}{d\varepsilon_s} \cdot \frac{(\varepsilon_{s,t,m+1} - \varepsilon_{s,t,m})}{\Delta w} \right\} \right] \cdot \varepsilon_{s,t,m}$$

$$\varepsilon_{s,t,m} = \frac{1}{2}(\varepsilon_{s,t,m+1} + \varepsilon_{g,t,m+1} + \varepsilon_{s,t,m} + \varepsilon_{g,t,m})$$

$$\varepsilon_g = \varepsilon_s \rho_s dt \qquad (21)$$

where, suffix t, m means the m th division at time t, and $m+1/2$ indicates the border between m th and $m+1$ th divisions; Δw is an increment of sludge mass. $q_g(\omega)$ is the volume of gas produced per unit mass and unity time.

$$\int_0^{w} q_g dt = \alpha \Delta w N \qquad (23)$$

where N is total division number of sludge layers.

ε_s at time $t+1$ is expressed as Eq. 24 in terms of forward difference.

$$\varepsilon_{s,t+1,m} = \frac{\varepsilon_{s,t,m}}{1 + \varepsilon_{s,t,m} \cdot \rho_s \left(\frac{V_{s,t,m+1/2}}{\varepsilon_{s,t,m}} - \frac{V_{s,t,m-1/2}}{\varepsilon_{s,t,m-1}} \right) \frac{\Delta t}{\Delta w}} \qquad (24)$$

When the solid fraction of sludge at a time as the initial condition, and the solid fraction of sludge at the interface and the flux of the solid phase at the bottom as boundary conditions are given, computation on the simulation is to be conducted.

4-2 Simulation Results on Batch Test

Simulation results on Run 1 and Run 2 in Series 3 are illustrated in Figs. 21 and 22. The interface movement in computation coincides, as a

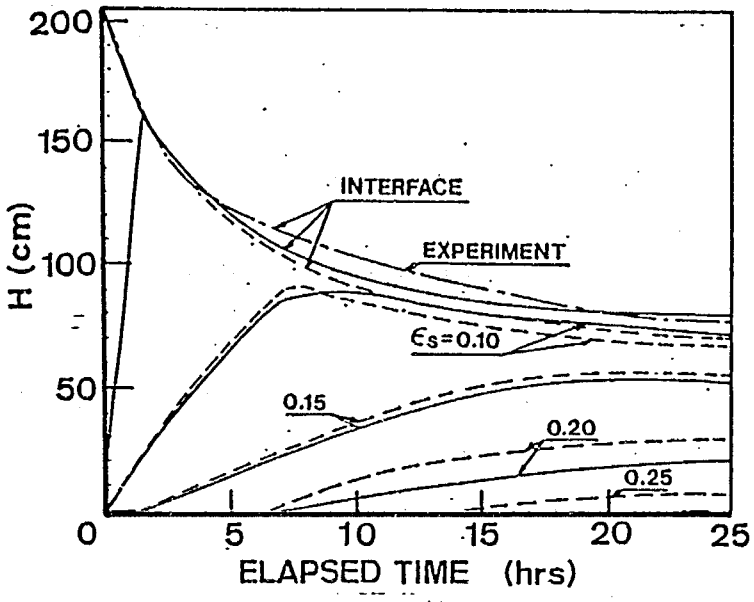


Fig.21 Numerical simulation results on batch test (Series 3, Run 1)
—·— observed ——— Simulation with gas production
– – – – Simulation without gas production

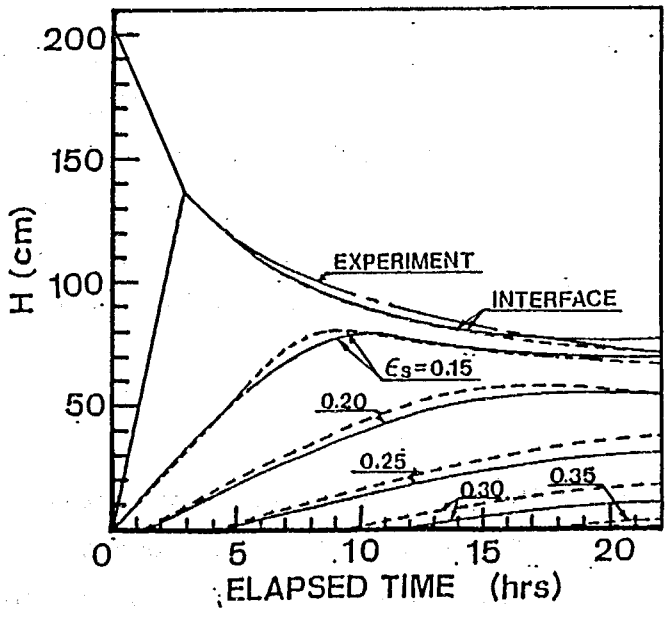


Fig.22 Numerical simulation results on batch test (Series 3, Run 2)
Lines: refer to Fig.21

whole with that in experiment, however, the experimental value becomes larger around the consolidation point and smaller than the computed one at the end of the test. Gas production in the period of reduced rate is shown not to influence thickening results. The difference of solid fraction at the bottom in Run 1, is 0.03. Because the difference becomes larger as the gas production rate becomes large, the control of gas production from sludge is required, especially in summer.

4-3 Simulation Scheme in a Continuous Flow

When q_b is not zero in a steady state, the computation scheme is written as follows:

$$\Delta \varepsilon_s = \frac{d\varepsilon_s}{dP_s}\left[\frac{\mu}{K}\left(q_b + \int_0^x q_g\,dx - \frac{V_s}{\varepsilon_s}\right) - \{\varepsilon_s(\rho_s - \rho_e) + \varepsilon_g(\rho_g - \rho_e)\}g\right]\Delta x \tag{25}$$

$$V_g = \int_0^x q_g\,dx \tag{26}$$

In a steady state, V_e and V_s take constant values in the entire region. On the other hand, V_g depends on position in the column.

4-4 Simulation Results in a Continuous Flow

An example of computation results is shown in Fig. 23, in which stands for wet solid fraction of inflow. Solid loading rates, 60-90 kg/m^2 day correspond to 0.00001-0.000015 $m^3/m^2/s$ of q_b. In computation, the hydraulic retention time is taken as 12 hours. Gas production in the period of reduced rate does not deteriorate thickenability. In this case, the major factor determines thickenability is the solid loading rate.

5. Application to Practical Use

In the previous section, the performance of gravitational thickening in the period of reduced rate of gas production is shown to be as almost same as that without gas production. Sludge with nitrate and nitrite can produce gas under anaerobic condition. Such sludge should not be

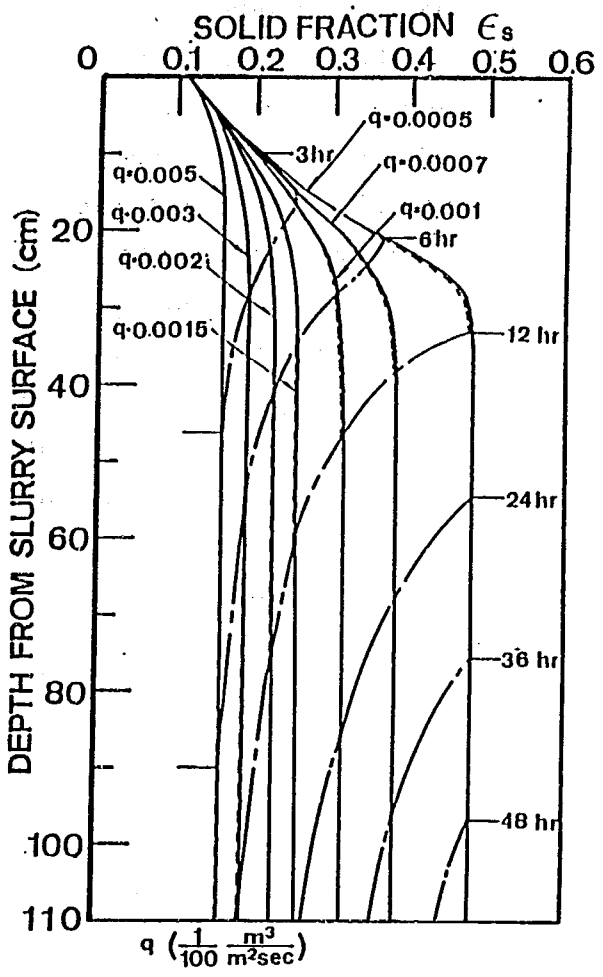


Fig.23 Numerical simulation results in a continuous flow
——— No gas produced ---- Gas produced

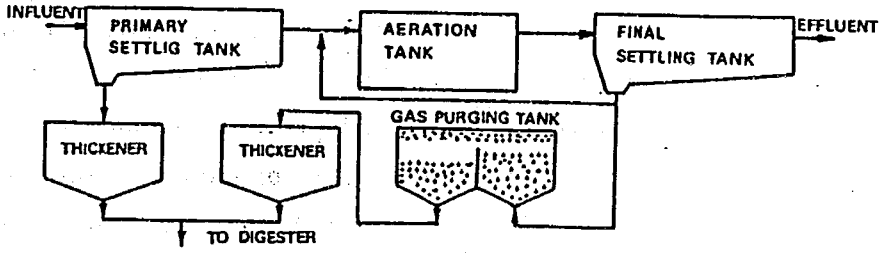


Fig.24 A proposed process with a gas purging tank

thickened before the completion of denitrification. When sludge is not in a fermentation stage, that is, sludge is fresh, the sludge should be thickened in the period of reduced rate of gas production. Once methane fermentation starts, such a period is not found.

As the application of the method developed here, a couple of practical designs are considered. The first one is an addition of a gas purging tank between the final settling tank and thickener as shown in Fig. 24. WAS in the denitrification period is stored in the gas purging tank and after denitrification and gas purging, the WAS is transported to the thickener and thickened. The sludge retention time in the gas purging tank is to be slightly longer than the denitrification period. The second one is double rake system in thickeners. Double rakes consist of the lower and the upper one which rotates much faster than the lower to make produced gas purged. The lower works for the disruption of floc structure and the establishment of water channels as conventional.

6. Conclusion

Gas production tests, thickening tests, and simulation for thickening were conducted. Results obtained are as follows:

1) The gas production process by fresh WAS consists of 3 periods, a period of large gas production rate by denitrification, a period of reduced rate of gas production, and a period of methane gas production;
2) The volume of gas production and the gas production period are functions of the initial concentration of nitrate;
3) The period of reduced rate of gas production continued more than 30 hours during which the thickening operation is able to be conduced;
4) The performance of thickening in the period of reduced rate of gas production is as almost same as that without gas production;
5) Newly developed thickening processes are proposed, the addition of a gas purging tank between the final settling tank and the thickener, and a thickener with two rakes, the upper one of which rotates faster than the lower to purge gas produced.

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STABILIZATION AND PASTEURIZATION OF SEWAGE SLUDGES BY AEROBIC-THERMOPHILIC TREATMENT

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Abstract

Sewage sludges have to be stabilized and prior to agricultural utilization be pasteurized according to the new pertinent regulations.

Aerobic-thermophilic stabilization takes place at temperatures high enough for appropiate pasteurization when the solids content is increased by (mechanical) dewatering corresponding to the ambient climatic conditions. The influence of the increased solids content upon the process of biochemical degradation is experimentally investigated and the first-order reaction constants for decay of volatile solids and COD are determined.

The oxygen uptake rate of the sludge is determined by monitoring the oxygen content of the fresh and exhaust air and measuring the applied air flow. This allows control of the process for minimizing heat loss and aeration cost and for ascertaining the degree of stabilization.

1. Introduction

Due to its low first but higher operation (aeration) cost, aerobic-thermophilic stabilization (ATS) is a sludge stabilization process especially suited for small to medium size (1) communities (50.000 pop. equivalents). The thermophilic temperatures, produced by biochemical oxidation of part of the volatile solids, increase the rate of degradation and decrease the required stabilization time to 4 to 10 days. A renewed interest for ATS is caused by the "sewage sludge regulation" (2), requiring pasteurization prior to agricultural application of the sludge in many cases. Goal of the investigations is to theoretically and experimentally datermine, at what total solids content the temperature requirements for pasteurization can be fullfilled, depending on the climatic conditions, especially on the ambient air temperature regime.

2. Pasteurization

For safe pasteurization of sewage sludge a temperature of 70 °C over 30 minutes of time ist required. This effect can be reached also at lower temperatures by increasing the retention time (3) as is shown in figure 1. From this figure it can be read, that a temperature of 50 °C at a minimum retention time of one day is sufficient for safe pasteurization. The retention time of one day is given by practical reason of daily operation of an ATS-plant with one charge per day. At (semi-) continuous charge of the ATS-reactor, a storage tank can be provided after ATS for appropriate pasteurization.

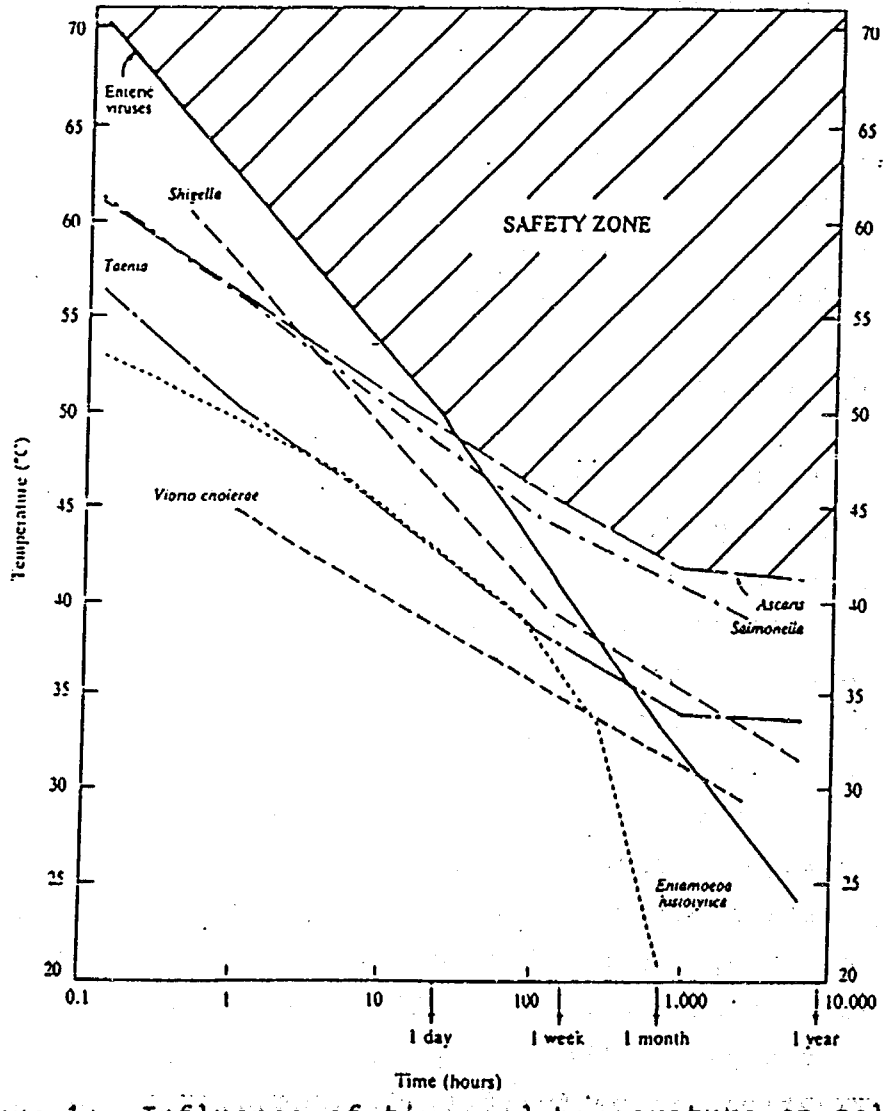


Figure 1: Influence of time and temperature on selected pathogens in night soil and sludge (3)

Under the above conditions, the requirements of pasteurization can, therefore, be translated into reaching a temperature by ATS not falling below 50 ^{o}C.

3. Conditions for simultaneous aerobic-thermophilic stabilization and pasteurization

The required temperature of at least 50 ^{o}C can safely be reached independent of the climatic conditions only when the sludge has a sufficiently high concentration of degradable substrate. In many cases this can be obtained solely by prethickening; sometimes mechanical dewatering may be required. The substrate content is generally measured by the concentration of total solids, volatile solids or COD.

The heat-energy input to the reactor is given by two factors: firstly by the liberated biochemical energy of some 15 kJ per g of COD removal (4); secondly by the mechanical energy input via aeration and mixing. The higher viscosity at elevated total solids (TS) contents requires an increased energy input. At 3,5 % TS, 100 W/m^3 power input can be regarded as sufficient. Our investigations have shown, that the required power input increases very roughly by the second power of TS-increase beyond this point, i.e. by $(TS/3,5)^2$.

Heat-energy is lost by in total four mechanisms: (a) warming up of sludge (solids and water), (b) evaporation (intensified by aeration), (c) warming up of the air used for oxygen supply, and (d) heat radiation from the reactor.

The interdependence of the above six factors is illustrated by two examples representing the required minimum and maximum sludge concentrations (COD and TS) for reaching pasteurizing temperatures:

- high ambient temperature (summer conditions) and substantial substrate degradation (30 %)
- low ambient temperature (winter conditions) and modest organics decay (17 %).

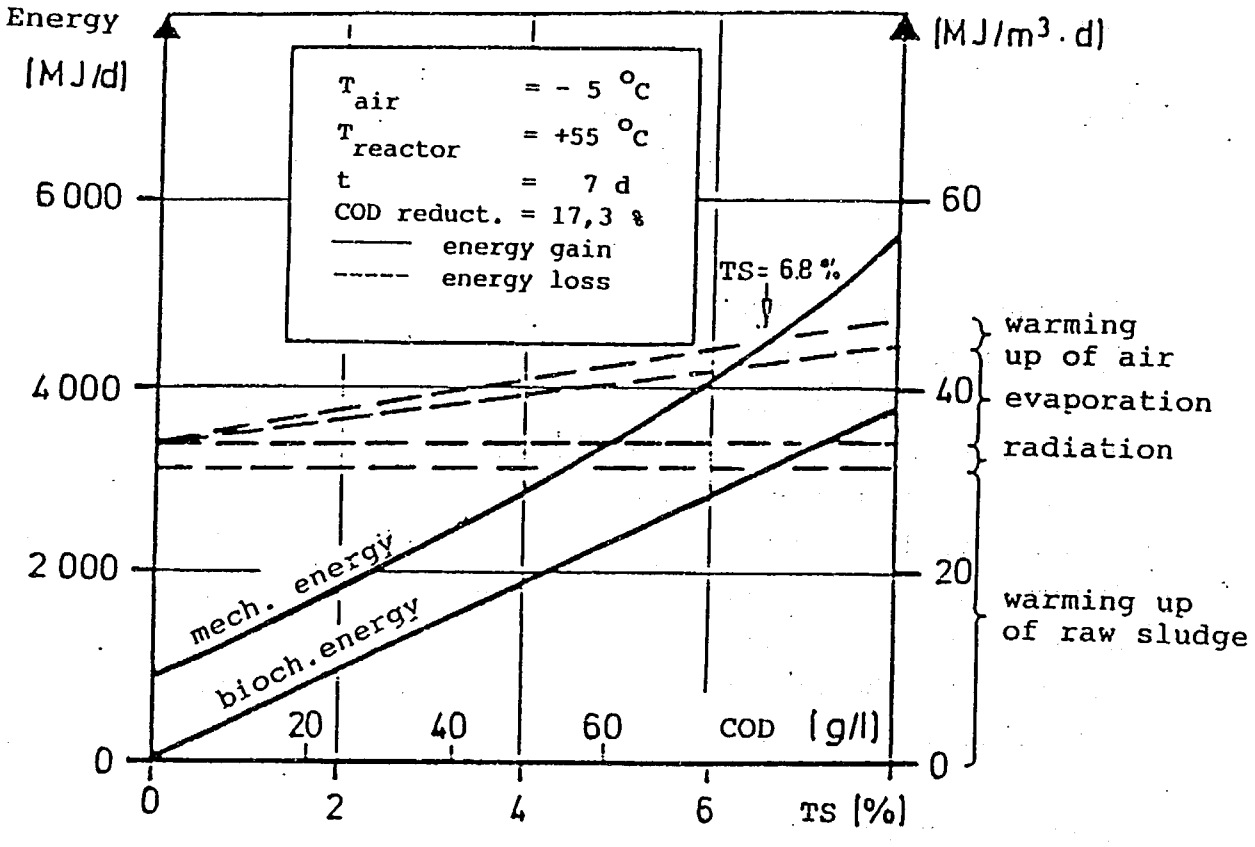


Figure 2: Heat energy balance "winter" 17,3 % COD reduction (10)

The conditions are based upon a reactor volume of 100 m^3 and a retention time of 7 days. Other sample parameters used for the heat energy balance can be found in the appended table "Sample Data". The results are grafically represented in figures 2 and 3 by plotting the sum of the two energy gains (solid lines) and the sum of the four energy losses (dashed lines) as a function of the total solids content. The intersection of both energy sum lines (energy balance) shows that substrate concentration between 31 g COD/l (2,5 % TS) and 84 g COD/l (6,75 % TS) are required for safe pasteurization, depending mainly on season temperatures and substrate degradability.

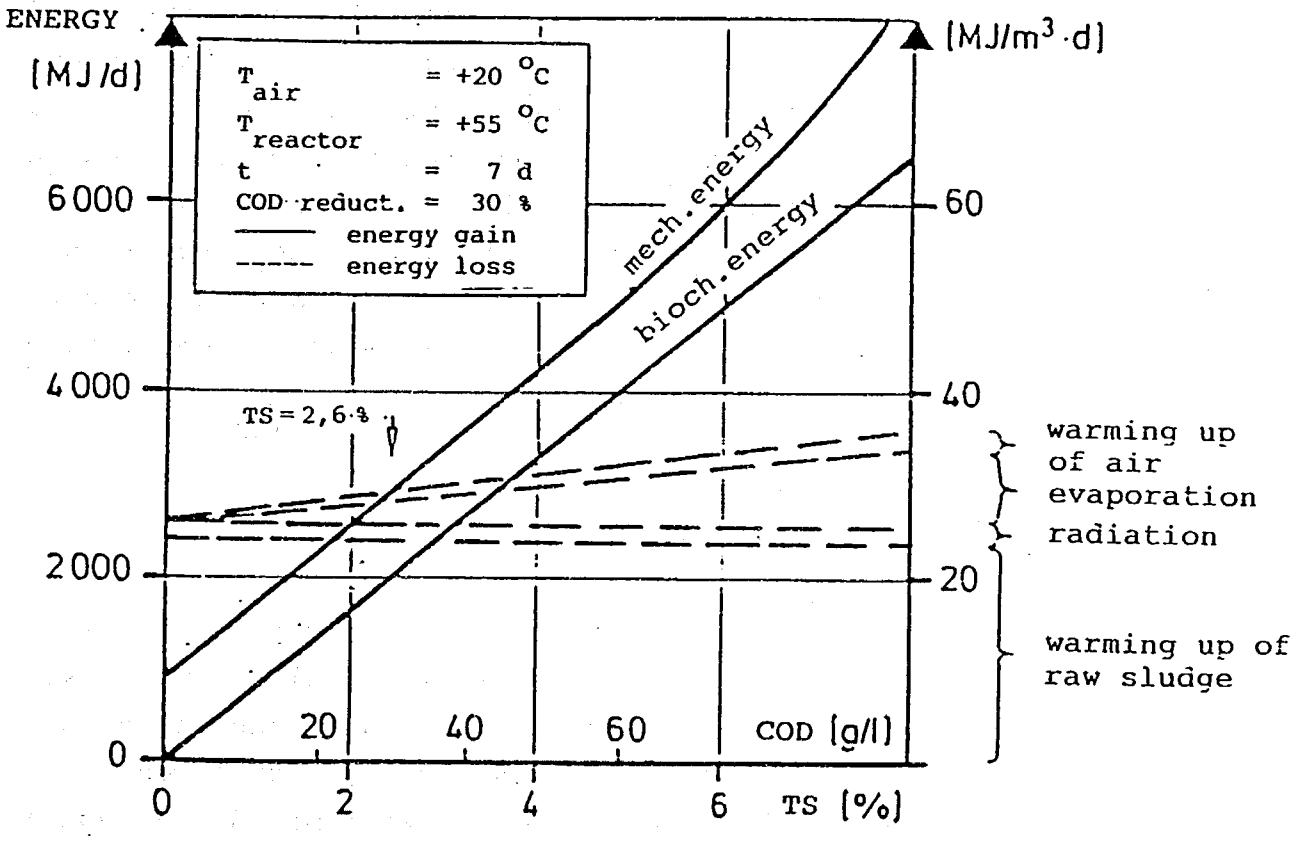


Figure 3: Heat energy balance "summer" 30 % COD reduction (10)

4. Experimental investigations

4.1 Research goals

Since substrate concentration as well as bacterial activity are measured by the same parameters (TS, VS, COD), the question arises whether high solids concentration influence the rate of degradation and at what minimum retention times a stable end product can be obtained additionally to the effect of pasteurization. Stability requirements and parameters are discussed in this context, also.

4.2 Pilot plant

Degradation experiments were performed in four insulated ATS-reactors of a volume of 70 l each. The pilot plant setup is illustrated by depicting one reactor plus appurtenances in figure 4. The four reactors were placed in a constant temperature room at 35 ^{o}C. The air, introduced into the lower part of the reactor, was measured with a rotameter and preheated and moistened by a water bath to minimize evaporation losses. By these means a temperature of 51 ± 4 ^{o}C could be maintained.

4.3 Material and methods

The raw sludge was obtained from a municipal activated sludge plant (40.000 pop. eq.). Primary sludge was taken from the sludge hoppers, homogenized and thickened to some 6,5 % TS. Return sludge was mechanically dewatered by a centrifuge to 7 % TS. Both sludges were mixed in equal volume portions and tap water was added to obtain four sludges of predetermined total solids content of 2,7 %; 4,0 %; 5,4 %; and 6,7 % TS.

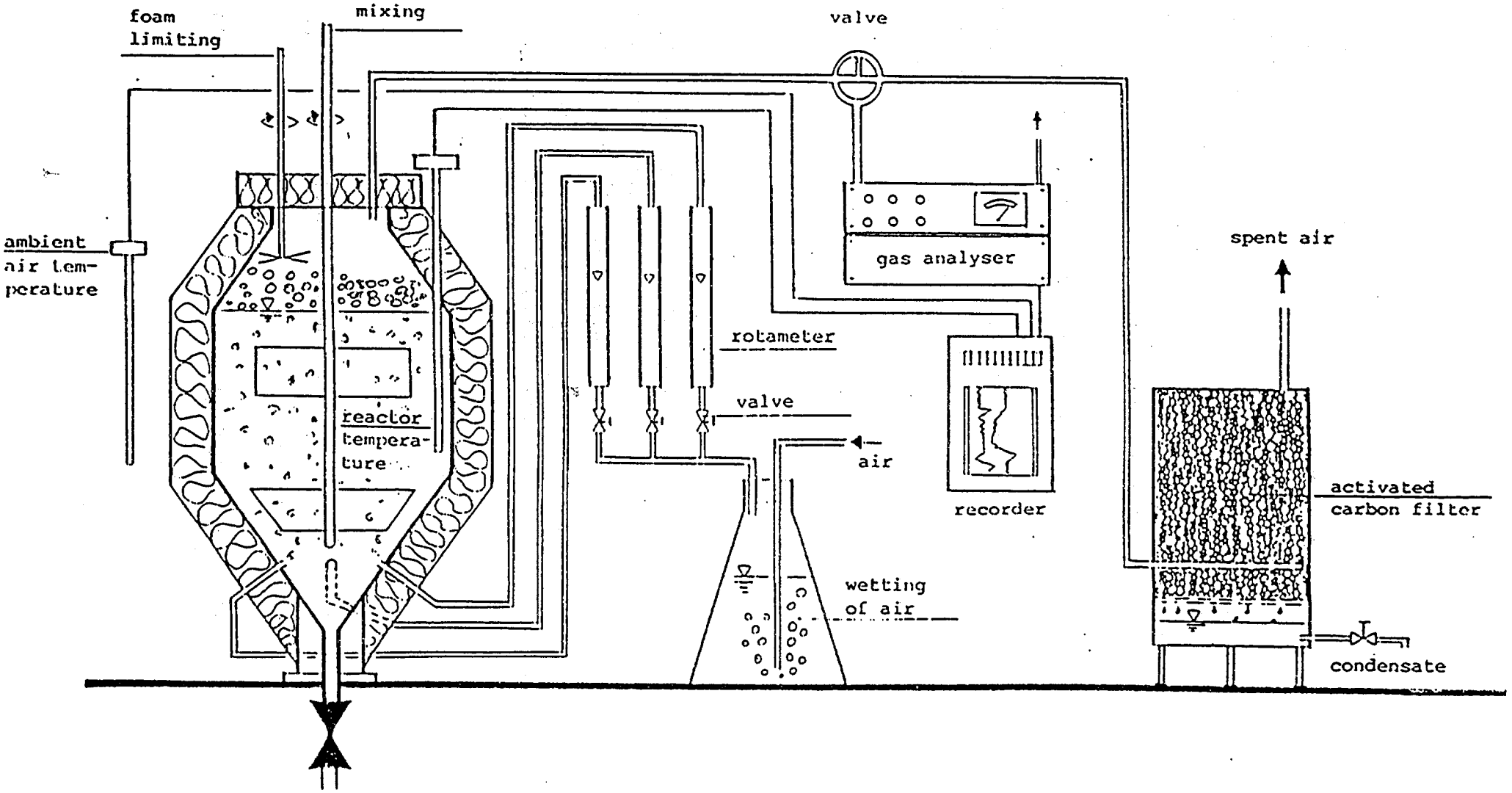


Figure 4: Pilot plant (scheme of one 70-liter-reactor)

All ATS-runs were performed as flow-through experiments with one charge per day. By varying the amount of the daily charge the retention time was fixed at five different values, viz. 2, 3, 5, 7, and 10 days. In total, therefore, four times five, i.e. twenty experimental runs were performed, each lasting for 2 to 3 times the retention time after reaching steady state conditions.

The course of degradation was followed by analyzing total solids (TR), volatile solids (VS), biochemical oxygen demand (BOD_5), and chemical oxygen demand (COD). The oxygen uptake rate of the sludges was determined by measuring the applied flow of air and the difference of the oxygen content of fresh and spent air.

5. Experimental results

5.1 Substrate degradation

Substrate degradation is explored by analyzing the change of TS, VS and COD in dependence of the retention time. The time dependent data (TS_t, VS_t, COD_t) were divided by the raw sludge data (TS_o, VS_o, COD_o) to give the relative substrate concentration S (-,%). Also, a kinetic model was fitted to the results.

5.1.1 Kinetic model

A first-order kinetic model for the degradable part $S-S_{oo}$ of the substrate was applied to the data:

$$- dS/dt = k \cdot (S-S_{oo}) \qquad (1)$$

S_{oo} is the not degradable fraction of the sludge. The reactors are regarded as completely mixed vessels fed with sludge of a substrate content of S_o at an amount to provide a retention time of t days. Then, equation 1 integrates into

$$S_t = S_{oo} + \frac{S_o - S_{oo}}{1 + k \cdot t} \qquad (2)$$

where S_t ist the reactor (effluent) substrate concentration.

It has been shown (5, 6, 7, 8, 9), that this approach can be successfully applied to unspecific substrate parameters like VS and COD, especially with respect to aerobic-thermophilic stabilization. This will also be illustrated by the data evaluation in the coming sections.

5.1.2 Total solids (TS)

The course of the relative substrate concentration (TS_t/TS_o) on the basis of total solids (TS in %) is depicted in figure 5 in dependence of the retention time. Although the data show a tendency as forecasted by equation 2, the essential model parameters k and S_{oo} could not be determined as significant statistics. This is caused by intensive data scattering and little degradation of some 20 % only, being indepent of the initial TS_o.

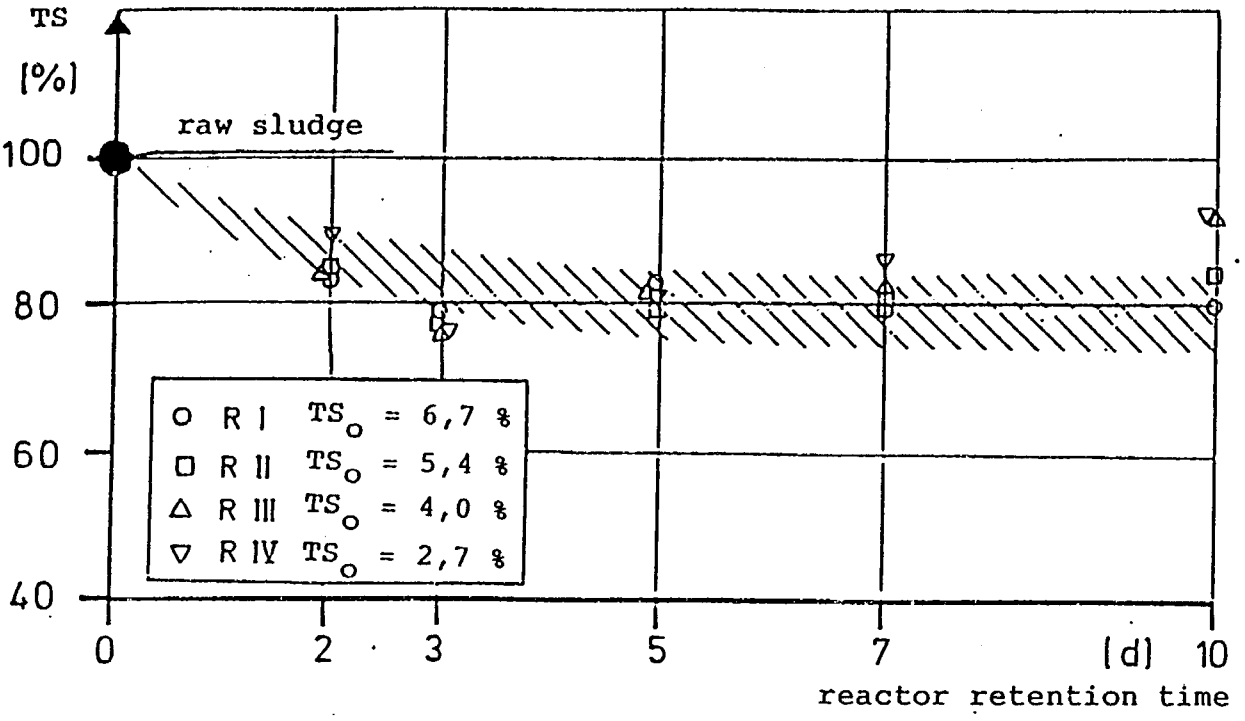


Figure 5: Reduction of total solids (TS) after (10)

5.1.3 Volatile solids

The degradation as indicated by the decay of volatile solids (VS_t/VS_o) is given in figure 6 as a function of the stabilization time. It is seen - with exception of four data points - that

- all values can be interpreted by the first order model with high accuracy (r = 0,98);
- the rate constant is not influenced by the (volatile) solids content;
- 30 % of the total volatile solids is degredable (S_{oo} = 0,70);

- the major part is already decomposed within 3 days of stabilization.

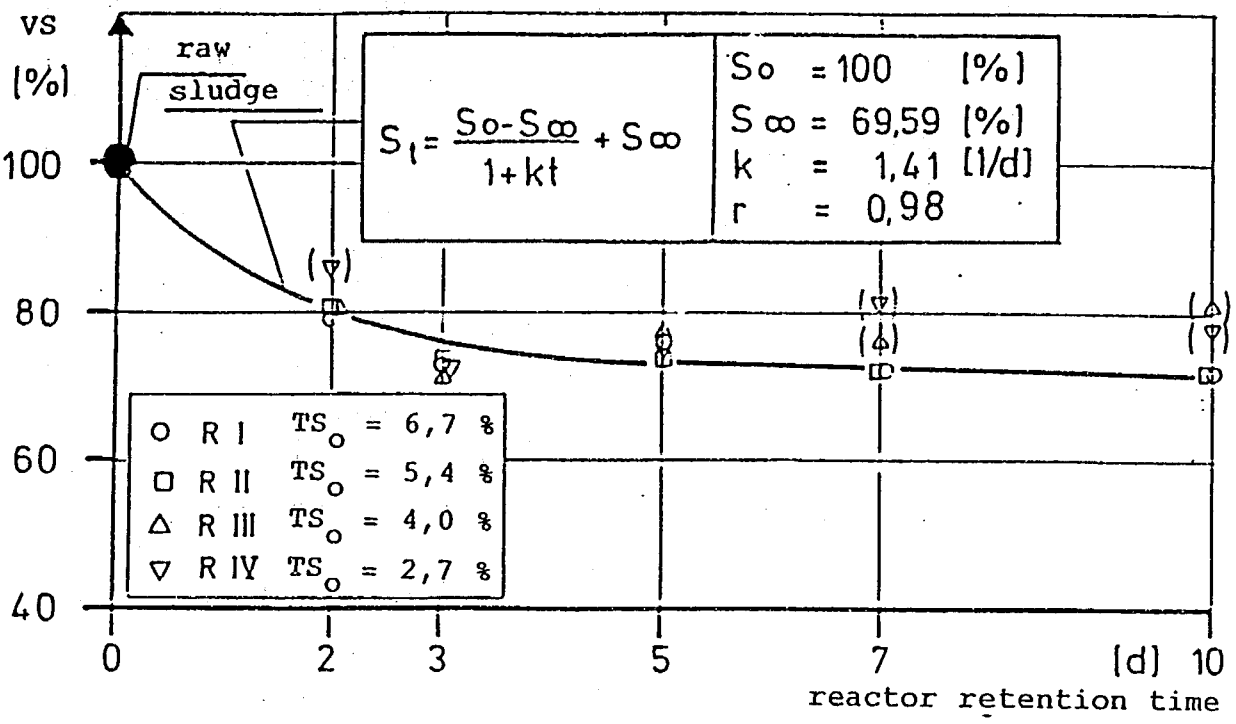


Figure 6: Reduction of volatile solids (VS) after (10)

5.1.4 Chemical oxygen demand

The degradation of COD is shown in figure 7 by plotting COD_t/COD_o versus stabilization time. Again, all values - except those for the least concentrated sludge - can be interpreted very accurately (r = 0.98) by only one equation. The substrate content again shows no influence on the rate constant. This constant (0,274 1/d), however, is much smaller than for volatile solids (1.41 1/d) as is the degradable fraction (26 % versus 30 %).

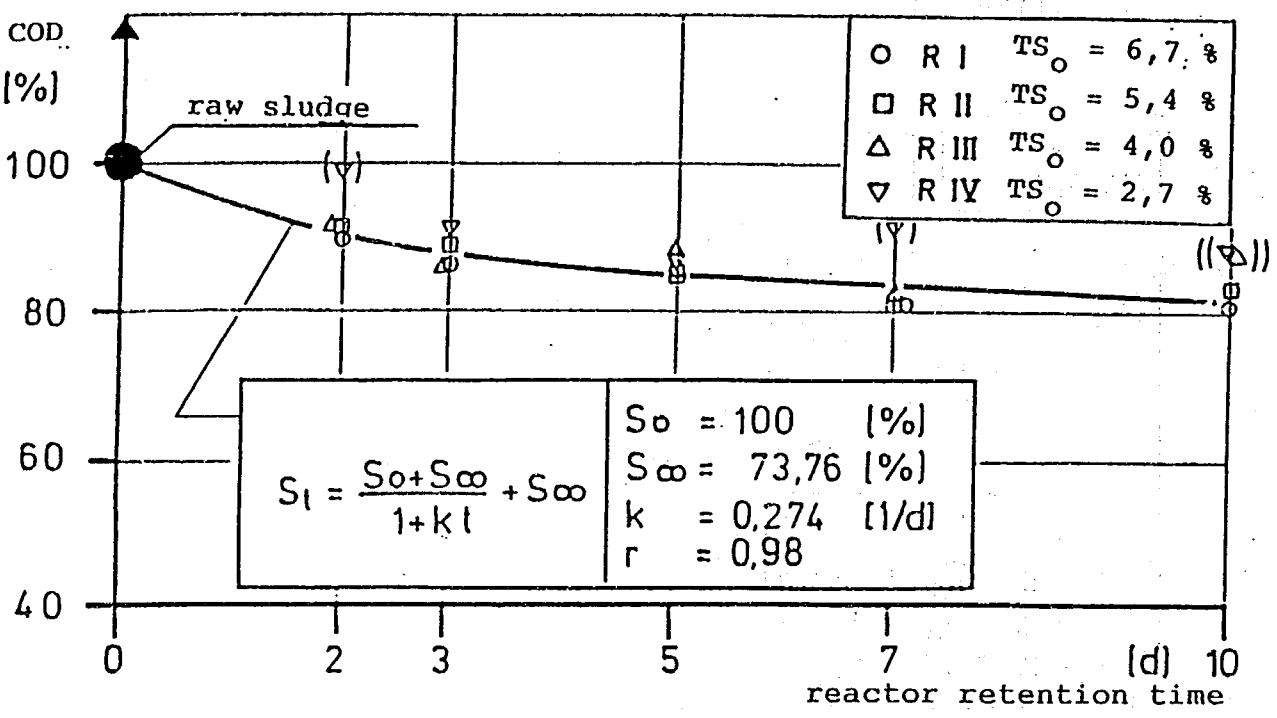


Figure 7: Reduction of COD (10)

5.1.5 Conclusion from substrate degradation

The course of degradation of volatile solids and of chemical oxygen demand can be described very accurately with a first-order model; the decomposition of total solids shows this tendency, also. In all cases, the rate constant is independent of the initial concentration; the degradable part varies from 20 % to 30 %.

As a consequence, the extent of stabilization is not influenced by the substrate concentration, but only by the time of stabilization.

This can additionally be shown by further analysing the basic parameter "volumetric substrate loading" (VL) of the reactor (kg substrate per m^3 of reactor volume per day). There are two ways to influence this parameter:

- the substrate concentration can be changed (at constant daily charges and hence retention time);
- the daily amount of sludge added and hence the retention time can be altered (at constant substrate concentration).

In figure 8 the effect of both methods is shown by plotting the obtained percentage degradation versus the volumetric COD loading. It is clear from this figure, that

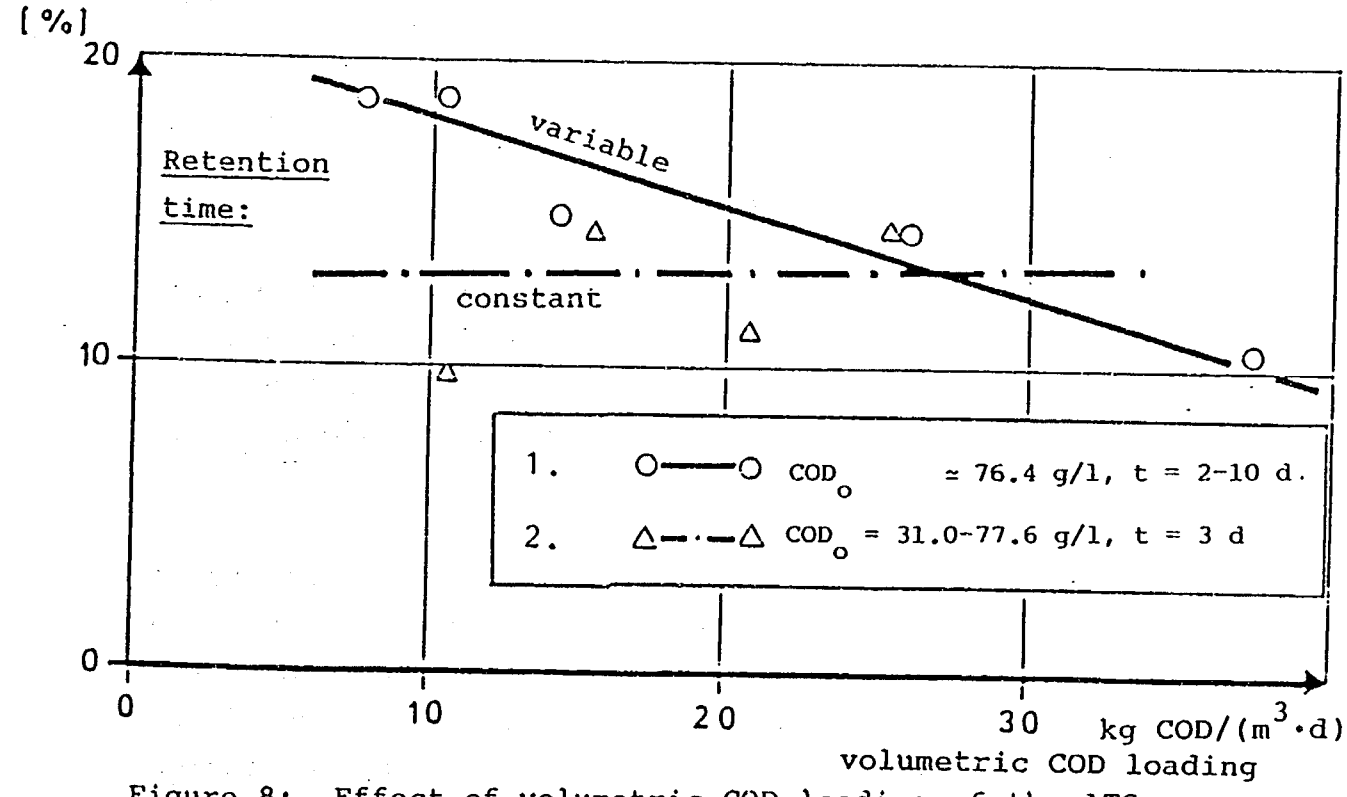


Figure 8: Effect of volumetric COD loading of the ATS-reactor on COD reduction at variable and constant retention time (10)

- at variable substrate concentration 13 % of the COD is degraded irrespective of the volumetric loading rate, ranging from 10 to 29 kg COD/(m^3·d) (retention time was 3 days in this case);
- at constant substrate concentration (76 g COD/l) the COD-degradation is strongly affected by the daily charge (and hence the VL). Decreasing the retention time from 10 to 2 days implies in this case increasing the VL from 7,6 to 38 kg COD/(m^3 d) and decreasing the percent COD-decay from 20 % to 10 %.

It is evident from these results, that stabilization time, not the volumetric loading rate determines the degree of stabilization and that retention time is the only appropriate parameter for designing ATS-reactors, therefore.

5.2 Oxyggen uptake and aeration

Sufficient supply of oxygen is a prerequisite for appropriate functioning of aerobic-thermophilic stabilization. On the other hand, excessive aeration adds to cost of operation and loss of heat energy and hence of safety of pasteurization.

Methods for control of aeration by monitoring the oxygen tension common in the activated sludge process cannot be applied to ATS. Oxygen probes for high temperature are hardly available, approach velocities to the probes of the viscous sludge cannot be generated and the oxygen tension will always be very close to zero at saturation concentrations of only some 5 mg/l.

Hence, the oxygen uptake rate of the sludge was determined by measuring the amount of air supplied and the difference of the oxygen content between fresh and exhaust air (see fig. 1). This method avoids sampling and interferences with the reactor content and allows, furthermore, continuous monitoring and control like with oxygen probes.

The uptake rates were specifically determined directly before and shortly after the daily charge of the reactors, with obviously low rates in the first case and high rates in the latter. The specific uptake rates (g O_2/(kg VS·d)) are plotted versus stabilization time in figure 9 for both situations.

The intensive fall of the uptake rate (prior to charging the reactor) from some 45 g/(kg VS·d) to values below 20 g/(kg·d) when increasing the retention time from 3 to 10 days signifies the decrease of biochemical activity and the increase of stability. This latter aspect will be covered in section 5.3 in more detail.

When decreasing the retention from 3 to 2 days, the uptake rate is curtailed also. This is caused by a low concentration of bacteria, attenuated by daily exchanging 33 to 50 % of the reactor volume with fresh sludge.

Finally, the continuous monitoring of the oxygen uptake rate by the described method allows optimum adjustment of aeration to the needs of the bacteria and thereby minimization of heat loss and aeration cost. From practical experience with full scale plants it can be concluded that for obtaining optimum results the air flow should be adjusted to yield 30 to 50 % air oxygen uptake.

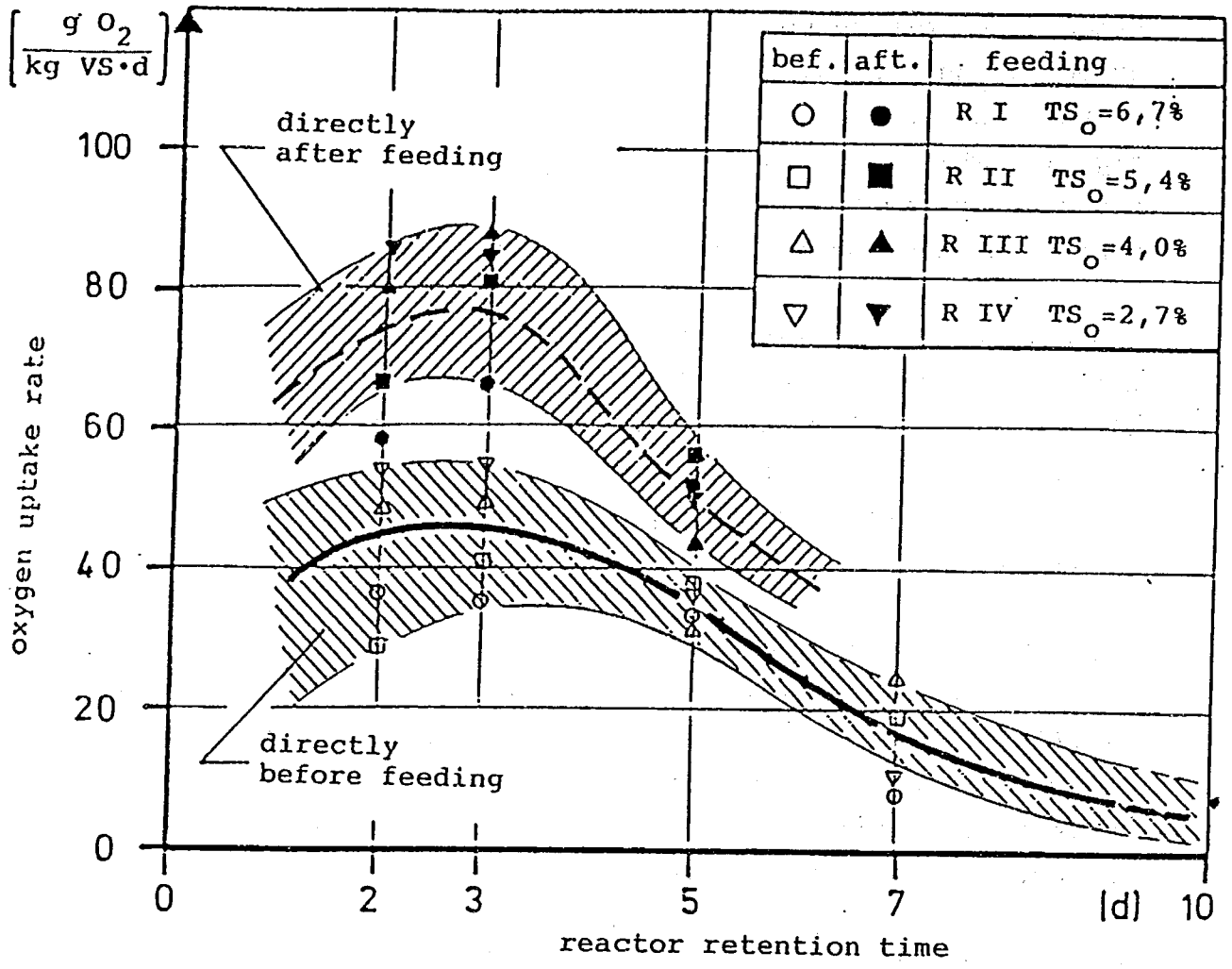


Figure 9: Oxygen uptake rate (10)

5.3 Stability

Whereas the operational requirements for pasteurization can be defined by 50 °C for at least 24 hours, stability has been characterized in a variety of ways. In fact, it seems impossible to uniquely define stability by only one (or two) parameters.

Depending upon further treatment steps between stabilization/pasteurization and final disposal by agricultural utilization or controlled tipping - a time span from one week up to some three months - the following criteria for stability can be put foreward:

- no foul odours during storage for some months followed by application in agriculture;
- well thickening sludge for minimizing storage volume;
- no reinfection even during storage over several months in covered tanks;
- easy to dewater mechanically up to solids contents required for landfilling.

Basically, stability can be defined by the (low) level of biochemical activity to which the sludge has been reduced. This level can be indicated by various parameters. Some of greater importance for storage of sludge have been investigated: (a) the change of the volatile solids content during 30 days of storage, (b) pH-changes during this time, and (c) the obtainable degree of thickening after stabilization and storage (figure 10).

It is evident from figure 10, that with increasing stability, i.e. with increasing retention time, less pronounced effects are to be noticed during storage:

- the further reduction of the volatile solids content of the sludge decreases, indicating less further biochemical activity;
- pH-drops due to anaerobic processes during storage become less striking;

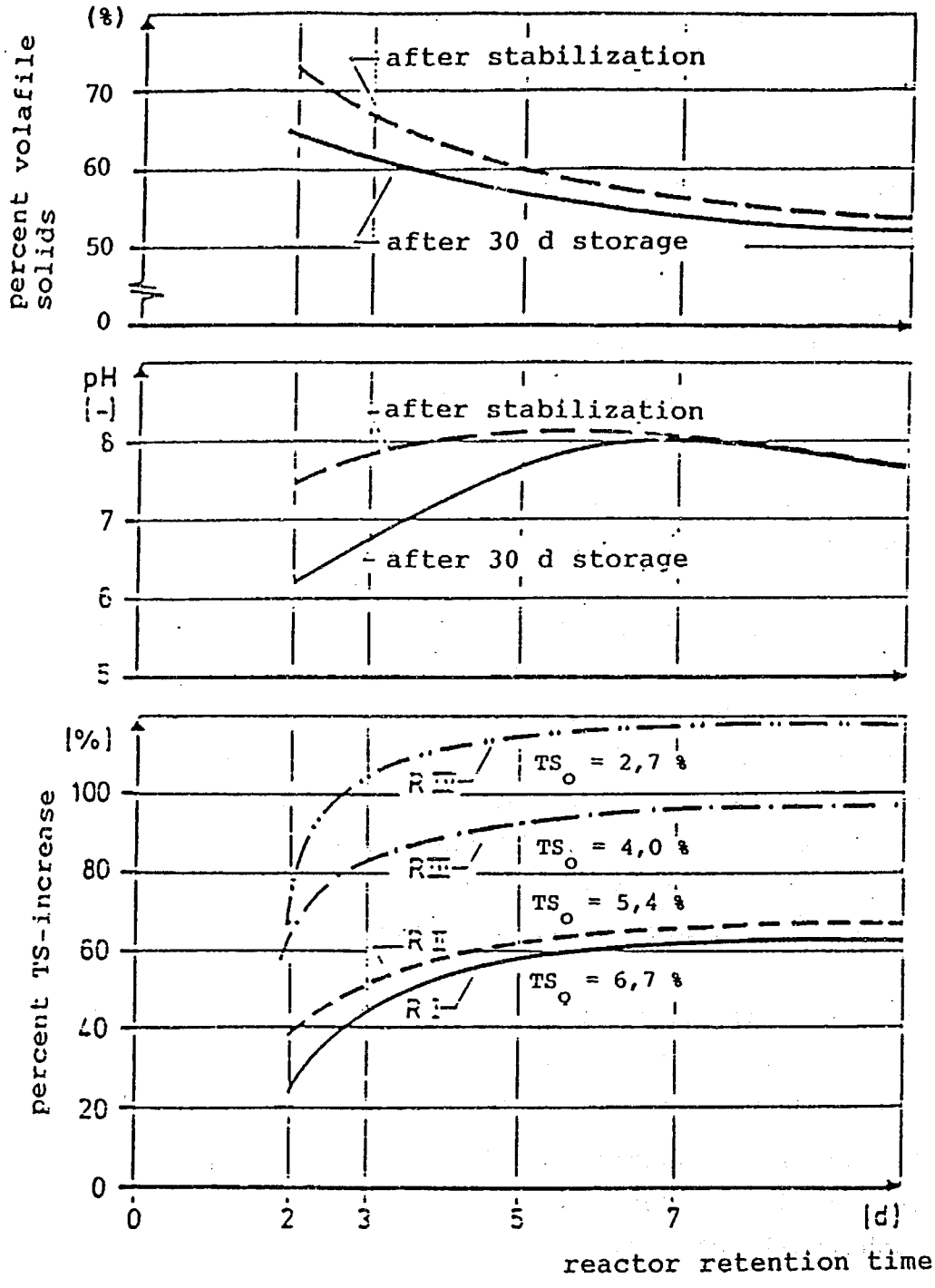


Figure 10: Change of sludge properties during storage (10)

- finally, it is evident from figure 10 that increasing the retention time, i.e. increasing the degree of stability, also means higher TS-contents obtained during storage. The sludge with TS_o = 2,7 %, for example, shows after 2 days stabilization time a solids increase by 70 %, i.e. from 2,7 % to 4,6 % TS; at 7 days, however, by 117 %, i.e. from 2,7 % to 5,9 % TS. At an initial TSo = 6,7 % the respective figures are:
  at 2 d: by 25 % from 6,7 % to 8,4 % TS and
  at 7 d: by 62 % from 6,7 % to 10,9 % TS.

A stability parameter, indicating most accurately the reduction in biochemical activity, however, is the rate of oxygen uptake. From the experiments it can be concluded (see figure 9), that 18 to 20 g O_2/(kg VS·d) can be taken as a limiting uptake rate providing sufficient stability. Further advantages of the oxygen uptake rate as stability parameter are:

- close correlation with the extent of biochemical activity of the sludge;
- convinient monitoring by the ΔO_2-method without sampling and interference with the reactor content;
- a reliable yardstick for stability is obtained by relating the rate to the volatile solids (g O_2/(kg VS·d)) .

6. Summary

Thickening and/or mechanical dewatering of sewage sludges up to some 7 % TS provides the means for aerobic-thermophilic stabilization at temperatures above 50 °C even under cold climatic condition. A safe pasteurization is

obtained simultaneously, therefore, which allows agricultural application of the sludge even under stringent hygienic regulations.

The increase in solids content does not change the rate of substrate degradation, nor does it impair the transfer of oxygen provided aeration and power input are increased accordingly.

The increased solids content, therefore, provides not only a means for safe pasteurization but at the same time the basis for a reduced reactor volume for the process. The time of stabilization, not the volumetric loading rate of the reactor with total or volatile solids or with COD, determines the final degree of stability.

By measuring the air flow to the reactor and the oxygen difference between fresh and spent air, the oxygen uptake rate of the sludge can be determined and the course of stabilization be accurately monitored. At the same time, the process of aeration can be controlled in order to minimize the cost of operation and losses of heat energy. Finally, the oxygen uptake rate serves as a yardstick for stability. The experimental results suggest 18 to 20 g O_2/(kg VS·d) as a limiting value.

At temperatures above 50 ^{o}C safe pasteurization is reached after one day of retention time already. Full stabilization, giving no reasons for complaints even during sludge storage of several months, require 6 to 7 days retention time, however.

7. Outlook to further application

Since a retention time of one day is sufficient for pasteurization, the remaining stabilization can be performed anaerobically (11, 12, 13, 14). The digestion gas from the anaerobic stage can be used for aerating and mixing the aerobic-thermophilic reactor. Energy balance calculations have shown, that this selfsupporting approach is technically feasible at (a) elevated total solids contents (5 to 6 %), (b) higher raw sludge temperatures (25 to 30 ^{o}C), and (c) heat energy economy (heat exchangers) within the process. This combination is being investigated as a means for treating collected faecal sludges and night-soil prior to safe application in agriculture.

Acknowledgement

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Appendix: Sample Data

Parameter	Notation	Value	Dimension
Reactor volume	V_R	= 100	m^3
	D	= 5,03	m
	H	= 5,03 + 1,00	m
Insulation	d	= 0,10	m
		= 3,5	kJ/(d m °C)
Retention time	t	= 7	d
Sludge flow	Q	= 14,3	m^3/d
Raw sludge: solids	TS	8	%
	COD[1]	100	g/l
Viscosity	at 8 % TS	= 55	mPas
	at 3,5 % TS	= 16	mPas
COD reduction (7 d)	(a)	= 17,3	%
	(b)	30	%
Raw sludge temperature			
- summer	T_1	= 20	°C
- winter	T_2	= 8	°C
Reactor temperature	T_R	= 50-60	°C
Air temperature			
- summer	T_S	= 20	°C
- winter	T_W	= -5	°C
Air moisture			
- fresh air	b_1	= 80	%
- spent air	b_2	= 100	%
Air pressure	p_a	= 103,0	kPa
Soil temperature	T_B	= 15	°C

1) Assumption: 1 % TS = 12,5 g COD/l

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PRESENT TREATMENT AND DISPOSAL OF SEWAGE SLUDGE IN KYOTO

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Abstract

PRESENT TREATMENT AND DISPOSAL OF SEWAGE SLUDGE IN KYOTO

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Sludge treatment and disposal systems have several important problems such as the improvement of sludge thickening performance, reduction in ashes and energy conservation measures.

This report therefore introduces the sludge thickening performance by using the pressurized flotation thickening method and the improvements made by modifying dewatering and incineration methods used in the City of Kyoto.

Also presented here is an outline of the present situation regarding the co-combustion of municipal waste with sewage sludge and the utilization of digested gas as well as the results of experiments on the recovery of waste heat from incinerators.

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1. INTRODUCTION

Treatment and disposal of the ever-increasing amount of sewage sludge resulting from the expansion of sewer service area is one of the major problems facing sewerage works.

About 450 municipal governments were operating public sewerage systems at the end of fiscal 1983 in Japan, and the quantity of sewage sludge generated from the total annual volume of treated wastewater of 7.1×10^9 m^3 will amount to 180×10^6 m^3. The situation of the treatment and disposal of sewage sludge is as indicated in Table 1.

Current sludge disposal is conducted mainly in the form of dewatered sludge. The method of sludge disposal varies, with 67% of the sludge being used for land fill, 9% for seashore reclamation and 14% for effective utilization such as returning to farmland and green districts. Most of the sludge is therefore disposed of by land fill and reclamation.

Table 1 Situation of sewage sludge treatment and disposal in Japan (FY 1983)

(Unit: 1,000 m^3/year)

Form of treatment \ Disposal method	Land fill	Seashore reclamation	Effective utilization	Others	Total
Dewatered sludge	1241	127	246	44	1658 (75)
Incinerated ashes	229	75	30	0	334 (15)
Compost, dried sludge	10	0	30	0	40 (2)
Digested, and thickened sludge	2	0	4	168	174 (8)
Total	1482 (67)	202 (9)	310 (14)	212 (10)	2206 (100)

Percentage is shown in parentheses

References: Sewerage in Japan (1985), compiled by the Ministry of Construction.

The City of Kyoto is located almost in the central part of Japan and is a cultural and sight-seeing city with a history of about 1200 years. It is also an inland city with a population of about 1.5 million mainly centering around the middle reaches of the Yodo River System. Its municipal wastewater pours into Osaka Bay through the Yodo River. Therefore, sewerage system in this city has to carry out the extremely important mission of not only improving the citizens' living environment but also of maintaining the water quality of the Yodo River as a water source for about 12 million people living in the lower reaches.

Therefore, the city is actively carrying out sewerage construction, and the situation of sewerage system as of March 1986 is as shown in Table 2.

The quantity of sewage sludge in the city has increased continuously as shown in Fig. 1 as a result of the expansion of sewer service area and a higher grade of the treatment method, and treatment and disposal measures for sewage sludge have become a great problem.

This report will explain the progress and present situation of treatment and disposal of the sewage sludge in the city.

Table 2 Situation of sewerage system in the City of Kyoto

(As of March 1986)

Total area of the city	61,061 ha
Total population of the city	1,471,663
Area of urbanization zones	15,009 ha
Sewer service area	10,204 ha
Population in sewer service area	1,153,580
Coverage of urbanization area	68.0 %
Coverage of total population	78.4 %
Total length of sewer pipelines	3,199 km
Treatment capacity	1,263,000 m^3/day
Toba Treatment Plant	909,000 "
Kisshoin Treatment Plant	125,000 "
Fushimi Treatment Plant	149,000 "
Ishida Treatment Plant	80,000 "

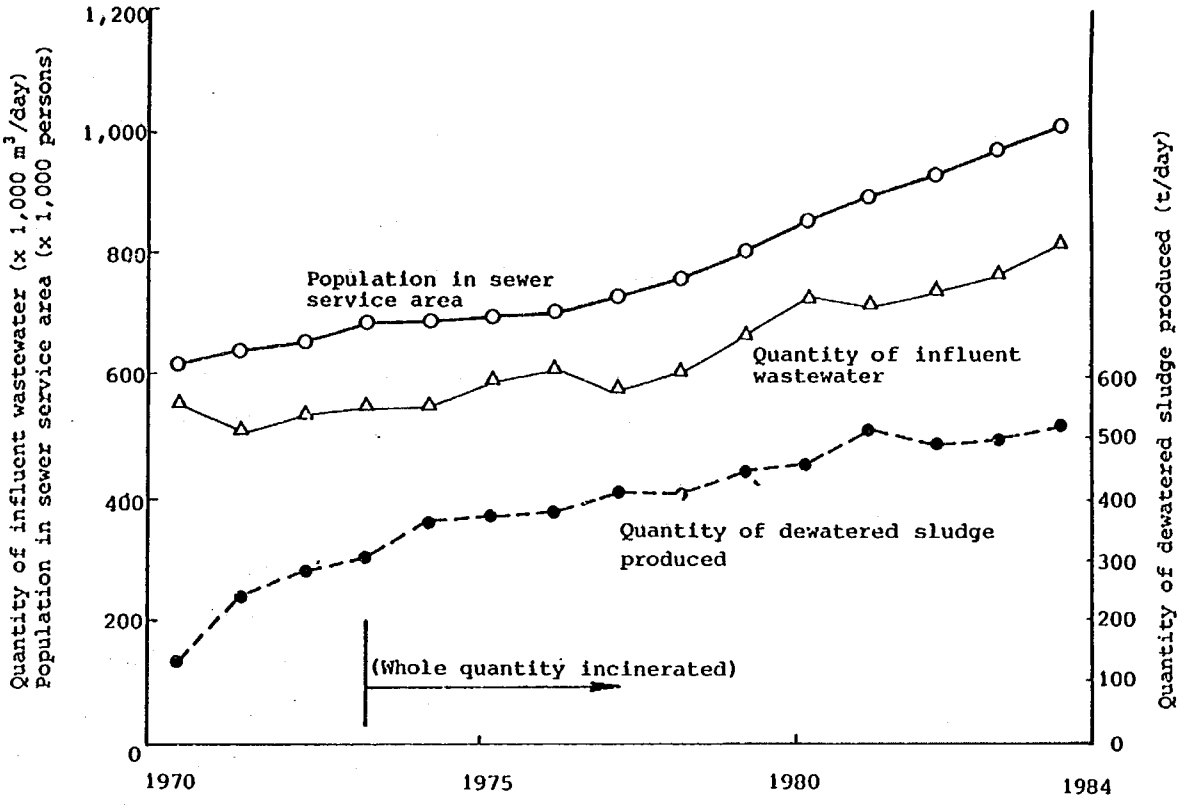


Fig. 1 Progress of sludge treatment in Kyoto

2. PROGRESS AND PRESENT SITUATION OF SEWAGE SLUDGE TREATMENT AND DISPOSAL

2-1 Changes in the Sludge Treatment Methods

Currently four wastewater treatment plants of Toba, Kisshoin, Fushimi and Ishida are in operation in the City of Kyoto. The change in the sludge treatment method in the city will be described below with respect to the Toba Treatment Plant as an example.

The Toba Treatment Plant was first opened in 1939 with a capacity of 78,000 m^3/day. Its present capacity is 909,000 m^3/day, and the capacity is scheduled to be expanded to 1,225,000 m^3/day in the future to make it the largest treatment plant in the city. Since the start-up of operation, sludge treatment and disposal in the plant had been performed by means of anaerobic digestion, solar drying and land fill. However, the quantity of sludge generated increased as the quantity of treated wastewater increased as a result of the facilities expansion works, and treatment by conventional methods had become difficult to perform. Therefore, a belt type vacuum filter was introduced in 1965 for improving the efficiency of dewatering

treatment and, in addition, the incineration of dewatered sludge by a vertical multiple-hearth furnace was commenced in 1968 in order to decrease the amount of sludge dewatered to be finally disposed of and to stabilize the final disposal. Therefore, raw sludge dewatering without anaerobic digestion has been performed in the facilities expanded after 1969, and all of the dewatered sludge generated in the city has been incinerated since 1973.

Moreover, in each unit treatment process, various improvements were made in order to perform even more efficient and economical sludge treatment and disposal.

With respect to thickening, mixed sludge comprising raw sludge and waste sludge had been conventionally thickened by gravity but, in 1982, the pressurized flotation thickening method was adopted for the waste sludge.

With respect to dewatering, a belt press dewatering equipment was adopted in 1984 in addition to the conventional belt type vacuum filters, and the construction of a stoker furnace for incineration is currently under way. In addition, the co-combustion of municipal refuse with dewatered sludge is being performed at a municipal refuse incineration plant adjacent to the Ishida Treatment Plant. These new sludge treatment methods will be respectively explained later.

2.2 Present Situation of Sludge Treatment and Disposal, and Energy Consumption

The sludge treatment processes and the situation of treatment in four plants of the city as of 1984 are as outlined in Fig. 2.

In the Toba Treatment Plant, not only the sludge generated within the plant but also the sludge from other plants and night-soil from unsewered area are treated and disposed of.

The sludge generated in the Kisshoin Treatment Plant is pumped to thickening tanks in the Toba Treatment Plant, and the sludge generated in the Fushimi Treatment Plant is transported by trucks in the form of dewatered sludge to the Toba Plant and incinerated there.

In the Ishida Treatment Plant, the sludge generated has been disposed of by co-combustion at the adjacent municipal refuse incineration plant since the autumn of 1985, but before that, the sludge was thickened, transported to the Toba Plant and treated there.

The quantity of wastewater treated in the four treatment plants in fiscal 1984, was about 750,000 m^3/d in average, as shown in Fig. 2, and the quantity of dewatered sludge generated from the wastewater reached 511 tons/d in average. All of the dewatered sludge was incinerated, and 76 tons of dry ashes generated (no water) was disposed of by land fill and so on.

A large amount of energy is consumed in wastewater treatment. Fig. 3 shows the energy consumption by treatment process in the four treatment plants in fiscal 1984. Energy consumption per cubic meter of influent wastewater amounts to 613 kcal, and the proportion of energy consumption by sludge treatment process is the highest at 62.5% of total consumption. The breakdown of this is 23.6% for thickening and digestion, 5.4% for dewatering and 71% for incineration. Thus, most fuel is consumed in the incinerators and the heating of digestion tanks. Heavy oil and digested gas are mostly used as fuel, and it is of course important to conserve fuel by improving each process of sludge treatment.

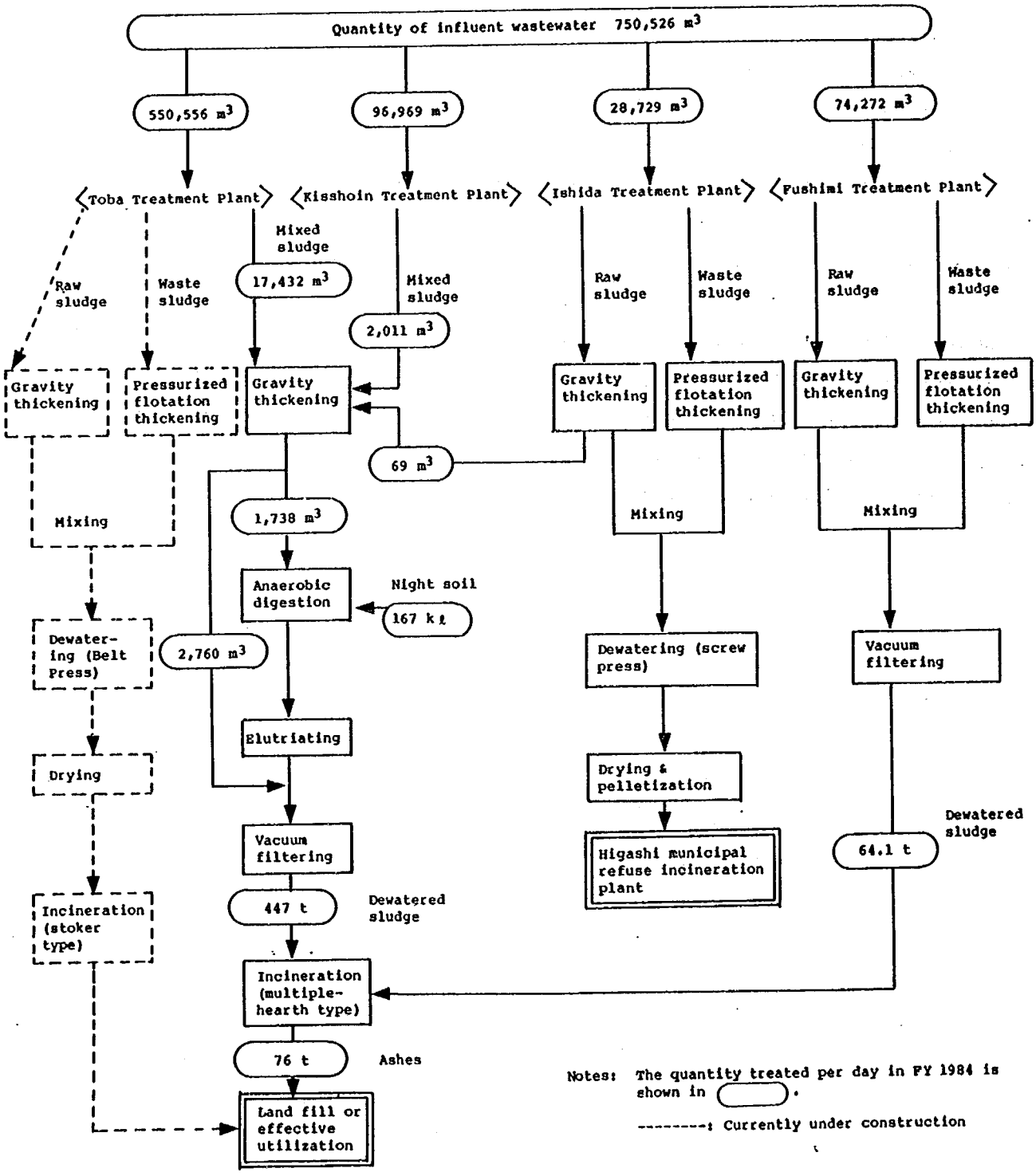
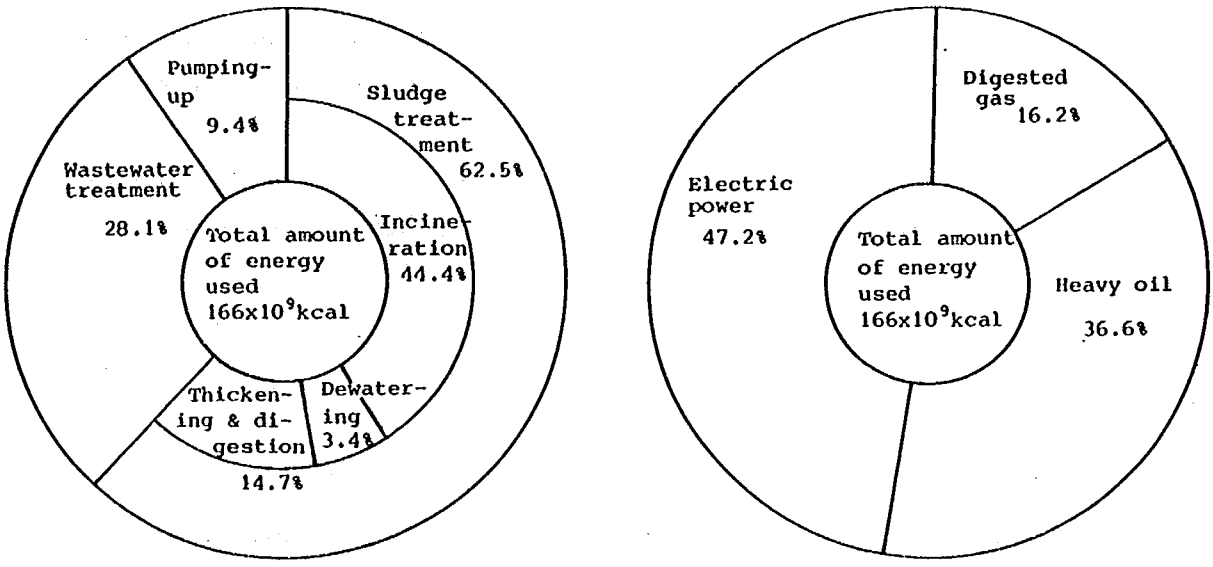


Fig. 2 Flow sheet of sludge treatment



Note*: Conversion factors
Electric Power: 860 kcal/kWh
Heavy oil : 9,000 kcal/l
Digested gas : 5,500 kcal/Nm^3

Fig. 3 Situation of energy consumption in treatment plants

3. PROBLEMS IN SLUDGE TREATMENT AND DISPOSAL AND THEIR COUNTERMEASURES

3-1 Employment of the Pressurized Flotation Thickening Method for Waste Sludge

3-1-1 Effect of the Deterioration of Thickening Performance on the Treatment Efficiency

Sludge thickening is the basics of sludge treatment, and its result greatly affects the over-all efficiency of sludge treatment.

In the City of Kyoto, gravity thickening of the mixed sludge comprising raw sludge and waste sludge was conventionally performed. However, in recent years, the solid content of thickened sludge has decreased as a result of qualitative change in sludge, and by this, the subsequent sludge treatment processes have been adversely affected as explained below.

1) In the digestion process, the hydraulic loading has increased and such problems have occurred as lowered digestion efficiency accompanied by the shortage of detention time, increase of energy consumption in heating process, and qualitative worsening of digester supernatant liquor.

2) In the dewatering process, the dosage amount of inorganic coagulant has increased, and the moisture content of dewatered sludge has increased, and filtration rate has decreased, by which the dewatering efficiency is adversely affected.

3) In the incineration process, the quantitative loading has increased, fuel consumption has increased, and the quantity of ashes generated has increased.

The treatment efficiency is greatly affected as stated above, and the treatment and disposal costs begin to increase.

3-1-2 Improvement by Pressurized Flotation Thickening Method

Causes of the deterioration of sludge thickening performance are as indicated below, so that volatile solid (VTS%) in sludge seems to have increased.

1) Expansion of separate-type sewered area has resulted in an increase of the concentration of organic matter in the influent wastewater.

2) The rate of generation of waste sludge tends to rise as a result of the employment of a higher grade of treatment.

The volatile solid ratio as a determining factor of sludge density has a certain correlation with the solid content of thickened sludge as shown in Fig. 4. The volatile solid ratio has recently risen to about 70% and effort are even being made for improving the degree of thickening but solid content of mixed sludge can be only 2.5% by gravity thickening.

To cope with the deterioration of thickening performance, a review was made for the separate thickening of waste sludge and raw sludge, and it was decided to use the pressurized flotation thickening method for waste sludge.

In the Fushimi Treatment Plant, one of the existing gravity thickening tanks was remodeled into a pressurized flotation thickening tank, and the separate thickening treatment was started in 1982.

According to treatment flow of this method, the waste sludge which was conventionally returned to the primary sedimentation tank and mixed and treated with raw sludge, is now thickened in the pressurized flotation thickening tank while the raw sludge is thickened in a gravity thickening tank. The thickened waste sludge and raw sludge are then mixed together.

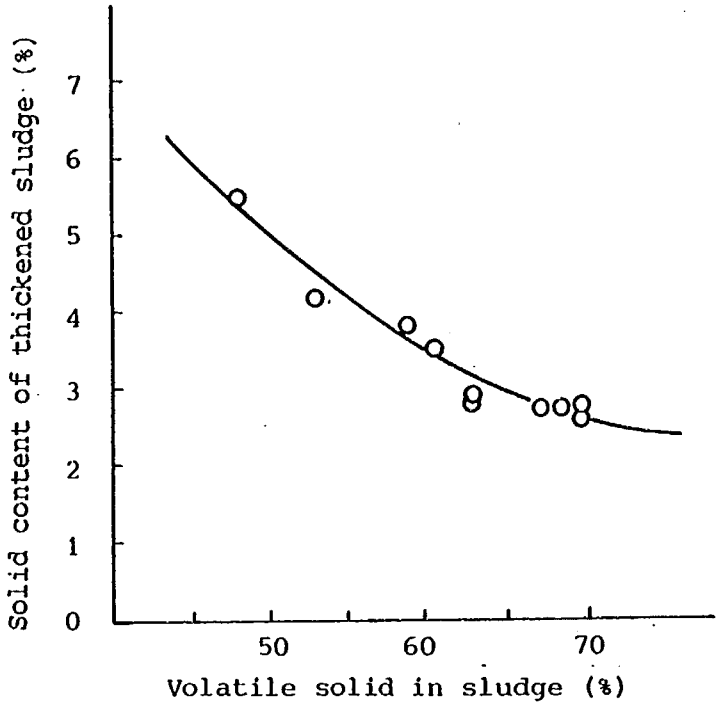


Fig. 4 Relationship between volatile solid in sludge and solid content of thickened sludge

For improving the thickening performance in the pressurized flotation thickening tank, both the solid loading and SVI are desired to be low. For obtaining 3% of solid content in sludge, the solid loading of 70 kg/m^2·day and the SVI of 200 are the upper limit in operation.

Shown in Fig. 5 is the effect of the solid content of mixed sludge on the chemical dosage ratio during dewatering and the moisture content of dewatered sludge. It will be known from this figure that the thickening performance and dewatering efficiency can be enhanced in the pressurized flotation thickening method to a greater extent than in the mixed gravity thickening method.

Table 3 shows the effect of the pressurized flotation thickening on the dewatering and incineration in the Fushimi Treatment Plant.

Though the solid content of mixed sludge was improved from 2.43% to only 2.86% as a result of the employment of the pressurized flotation and separate thickening method, the amount of chemical dosage during dewatering was saved and the moisture content of the dewatered sludge was lowered. Moreover, this method brought other advantages such as a decrease in the quantity of dewatered sludge to be transported and incinerated and the

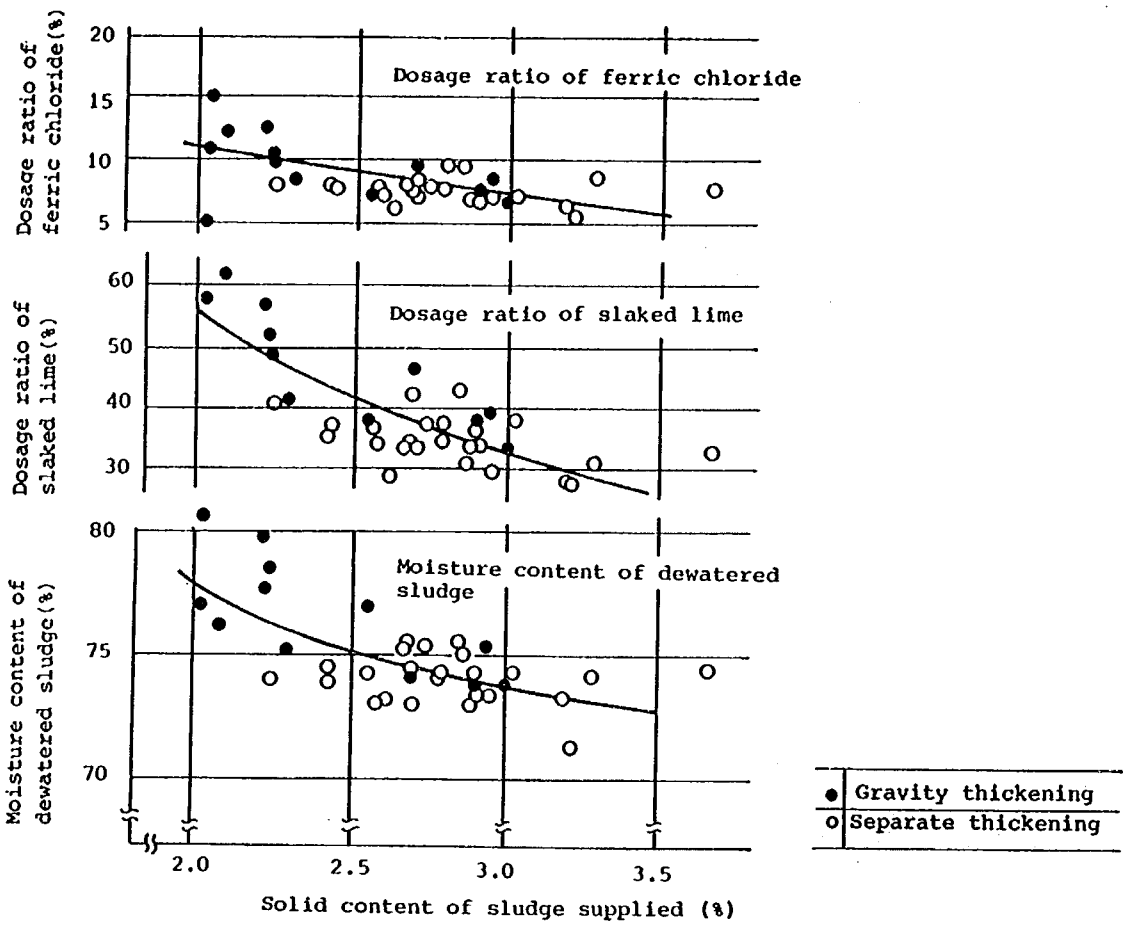


Fig. 5 Relationship between solid content of sludge supplied, chemical dosage ratio and the moisture content of dewatered sludge

Table 3 Effects of pressurized flotation thickening on dewatering and incineration

Item \ Method	Solid content of sludge supplied (VTS) (%)	Dosage ratio of ferric chloride (%)	Dosage ratio of slaked lime (%)	Moisture content of dewatered sludge (%)	Filtration rate ($kg/m^2 \cdot hr$)	Power consumption for dewatering (kWh/t · DS)	Consumption of auxiliary fuel[a] (l/t · DS)
Gravity thickening	2.43 (69.8)	9.3	46.8	76.8	8.3	195	251.5
Seperate thickening	2.66 (69.8)	6.8	33.5	73.4	9.9	153	99.3

DS = Dry solid
[a]: Trial calculations based on heat balance

saving of fuels for incineration. Also, the quality of return water from the sludge treatment facilities was improved so that the loading to waste-water treatment was reduced.

Based on the achievements at the Fushimi Treatment Plant, the expansion plan for the Toba Treatment Plant has incorporated the use of the pressurised flotation thickening method for waste sludge separately from raw

sludge. The expansion plan will be able to correspond to a wastewater treatment capacity of 475,000 m^3/day, and one-fourth of the facilities have been already completed and will soon be in operation. Table 4 shows the outline of pressurized flotation thickening equipments in the Toba Treatment Plant.

Table 4 Outline of pressurized flotation thickening equipments in the Toba Treatment Plant

Construction	Shape	Effective capacity	Solid loading	Sludge scratching-off method	Number of units installed
Reinforced concrete	Rectangular type 4.5 (W) x 19 (L) m	390 m^3	100 kg・DS/ m^2・day	Travelling type	5 units

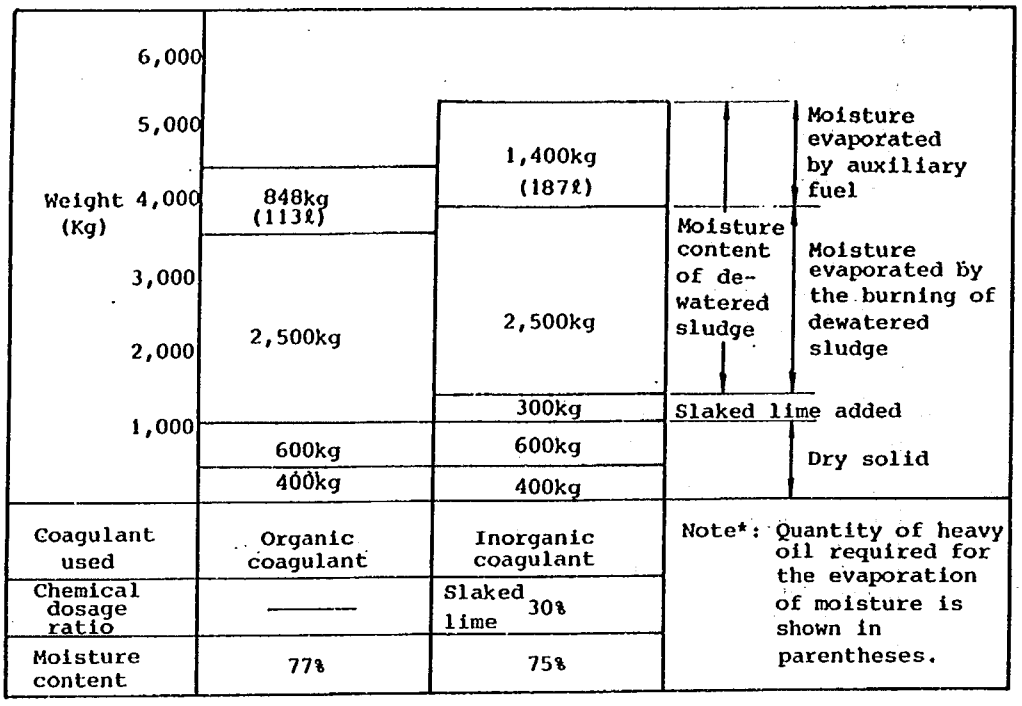
3.2 Improvement by Modifying Dewatering and Incineration Methods

Vacuum filters using inorganic coagulants were utilized for sludge dewatering in the city while vertical multiple-hearth furnaces were used for incinerating the dewatered sludge. These methods were both adopted as the best ones available at that time.

However in recent years, further reductions in the quantity of dewatered sludge and energy conservation have been greatly emphasized as the result of social change and technological innovation.

Because of this, the dewatering and incinerating facilities to be added are now selected by taking account of changes in the character of dewatered sludge as fuel, the method of disposal of ashes and so forth.

Belt press type dewatering equipments using organic coagulants have been adopted. Fig. 6 shows a comparison of dewatered sludges using organic and inorganic coagulants. From this figure, the following merits are apparent: (1) the fuel consumption per ton DS of dewatered sludge using an organic coagulant is 113 liters which is 60.4% of 187 liters of that using an inorganic coagulant (2) under the conditions of 60% of volatile solid ratio in sludge and 30% of dosage ratio of slaked lime, the quantity of ashes generated decreases to 57% of that of the case using an inorganic coagulant and (3) quantity of required auxiliary fuel can be greatly reduced because of the rise in the heat value of DS of dewatered sludge.



Trial calculations were made on the basis of 60% of volatile solid in the dry solid, heat value of volatile solid of 5,000 kcal/kg, and drying efficiency of 50% in incinerator.

Fig. 6 Comparison of dewatered sludge between the use of organic and inorganic coagulants

Shown in Table 5 is the outline of dewatering equipments which is installed in the Toba Treatment Plant and will be soon operated.

With respect to incinerators, the furnaces such as multiple-hearth furnaces, fluidized reactors and stoker furnaces widely being used at present were reviewed under the conditions of the incineration for 150 tons/day of dewatered sludge with a moisture content of 74%, 5,000 kcal/kg of heat value of volatile solid, and the use of A type heavy oil (heat value of 10,000 kcal/kg) as auxiliary fuel. They were reviewed and compared with respect to the construction, exhaust gas, operability, properties of ashes, maintenance costs and heat balance. As a result, the following evaluation was obtained:

1) In the case of stoker furnaces or fluidized reactors, dewatered sludge is pre-dried by exhaust gas and self-combustion becomes possible. However, in the vertical multiple-hearth furnace, the self-combustion is normally difficult to achieve and auxiliary fuel is required.

2) Ashes from a stoker furnace are in the half-melted state (clinker-like) so that there is no leaching of heavy metals and the ashes

are chemically stable. The ashes are therefore hardly scattered during transport and during land disposal.

3) Stoker furnace assure the least generation of NOx and also produce little dust and soot as in the case of multiple-hearth furnaces.

4) Thermal control of stoker furnaces is simple, and the furnace can easily follow fluctuations in the properties of dewatered sludge and in the loading. Co-combustion with screenings is also possible.

For the above reasons, a stoker furnace was finally selected and presently one stoker furnace with an incineration capacity of 150 tons/day is under construction.

Table 5 Outline of dewatering equipments in the Toba Treatment Plant

Type	Shape	Filtration rate	Number of units installed
Belt press	Effective width of filter cloth: 3.0 m	120 kg · DS/m · hr	6 units

3.3 Co-combustion of Municipal Refuse with Sewage Sludge

The Ishida Treatment Plant was mainly built for realizing economical operation by increasing efficiency, and for conserving natural resources and energy through the combined operation of the treatment systems in both the Ishida Treatment Plant and the Higashi municipal refuse incineration plant located adjacent to the Ishida Plant.

The combined systems can burn not only sludge but also screenings generated in the treatment plant together with municipal refuse in the incineration plant and the heat energy generated during the incineration is converted into steam and electric power. The power is utilized not only in the refuse incineration plant but also for every other purpose requiring energy in the wastewater treatment plant. Part of steam is also utilized for the dewatering and drying of sludge, for heating a swimming pool and for the air conditioning equipment. The wastewater from the refuse incineration plant is treated in the wastewater treatment plant.

Since furnaces with revolving grates exclusively for burning refuse were installed at the incineration plant, the properties of dewatered sludge must satisfy the conditions for co-combustion set forth in Table 6. Because

of this, a heating type screw press was employed for dewatering sludge and a drying and pelletization process was also added for producing dry pelletized sludge with a moisture content of about 40%.

The operation of the co-combustion system was started in October 1985 and is smoothly being operated with a heat value higher than the planned as shown in Table 6.

Records of long-term operation will be required for detailed review of the energy conservation and economy of this co-combustion system in the future.

Table 6 Conditions for the co-combustion of municipal refuse with dewatered sludge

		Municipal refuse (at Higashi refuse incineration plant)	Sewage sludge (Ishida Treatment Plant)	
			Planned	Actual 1985.10 ∿ 1986.3
Design quantity of treatment		400 t/day (200 t /day x 2 units)	Design quantity of solid: 30 t · DS/day	—
Solid content of sludge supplied		—	3.5%	3.6%
Moisture content of dewatered sludge		—	70%	70.1%
Municipal refuse & pelletized sludge	Moisture content	35%	40%	38.8%
	Low level heat value	2,280 kcal/kg	2,100 kcal/ kg · WS	2,301 kcal/ kg · WS

WS: Wet sludge

3.4 Fuel Conservation in Existing Facilities

As stated previously, about 60% of the total energy consumed is in the form of fuels mostly at the digestion tanks and incinerators. At the digestion tanks, digested gas was used as a fuel for heating tanks to conserve heavy oil. However after the oil crisis in 1973, it became necessary to fully utilize this digested gas and, for this purpose, a gas storage tank was added. Also, combustion devices suited to both heavy oil

and digested gas were employed in all the incinerators newly installed. Thereafter, the existing incinerators were gradually improved and at the end of fiscal 1984, five of seven incinerators could be operated with digested gas.

Table 7 shows the heavy oil savings through utilizing the digested gas. It shows that the proportion of digested gas to fuel is 11.7% for incinerators and 100% for digestion tanks. About 3,000 kl of heavy oil was saved per year at both the facilities. Also in the incinerators, part of the waste heat has been utilized as a heat source for preventing white smoke from the exhaust gas.

Moreover, in order to enhance the efficiency of utilizing waste heat, heat recovering from drain water (60 to 70°C) from exhaust gas cleaning scrubbers is being experimented with. The heat recovered from the drain water will be utilized as the heat source for heating the digestion tanks and thereby save the amount of heavy oil to be used. By the experimenting equipment a part of drain water from the scrubbers is pumped up to heat warm water sent from a warm water tank by means of a plate type heat exchanger.

The flow rate is 4 m^3/hr of drain water with a standard operating condition of 42 m^3/hr of drain water (68.8°C) for 6.2 tons/hr of incinerating sludge.

Fig. 7 shows the relationship between the flow ratio, the recovered quantity of heat recovered and the over-all coefficient of heat transmission of heat exchanger in the experiment. The over-all coefficient of heat transmission, which indicates the state of heat movement through the heat exchanger, will increase in proportion to the increase in the flow ratio. However, the recovered quantity of heat tends to level off when the flow ratio exceeds 0.5.

Now, assuming that the over-all heat loss is 40%, the area of heat transmission is 200 m^2 and the inlet water temperature is 37.5°C since the heat supplied for heating the digestion tank is 56 kcal/m^3 · hr (the actual), it is possible to recover heat of 1.6×10^6 kcal/hr, which is equivalent to the quantity of heat required for heating a digestion tank with the volume of 17,000 m^3, or to 40% of the total volume of primary digestion tank in the Toba Treatment Plant. With respect to the soil and corrosion on heat exchangers and the auxiliary facilities, it is required to make a further study.

Table 7 Situation of saving heavy oil in plants

Item			Fiscal year	1984
Incinerator		Quantity of incinerated sludge	10^3 t/year	187
	Quantity of fuel used	Heavy oil	kl/year	6,511
		Digested gas (converted to heavy oil)	"	859
		Total	"	7,370
		Percentage of dependency on digested gas	%	11.7
Digestion tank		Sludge supplied into digestion tanks	10^3 m^3/year	695
		Quantity of digested gas used for heating (converted to heavy oil)	Kl/year	2,124
		Percentage of dependency on digested gas	%	100

Conversion factors
Digested gas: 5,500 kcal/Nm^3
Heavy oil : 9,000 kcal/l

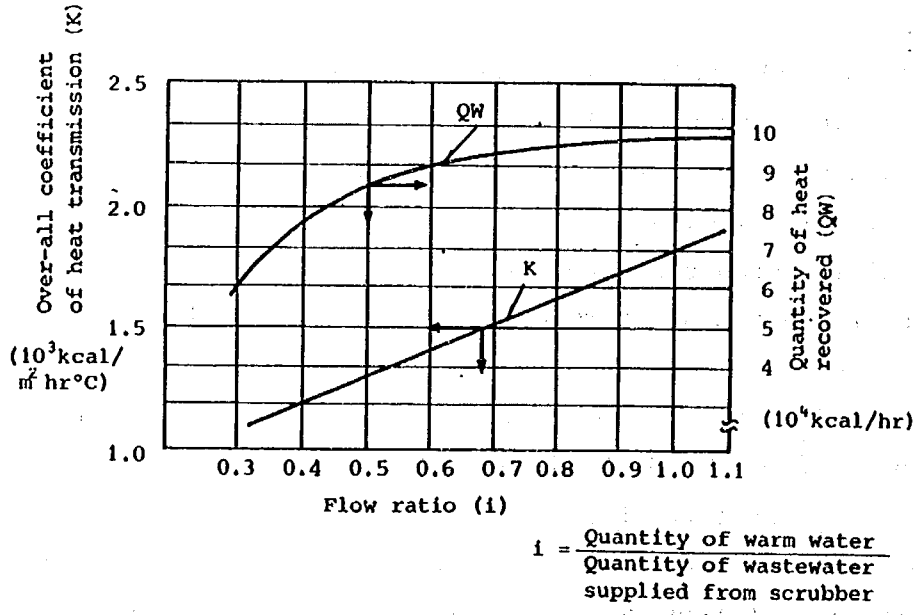


Fig. 7 Relationship between flow ratio and quantity of heat recovered

3.5 Final Disposal and Effective Utilization of Sludge

Urbanization has progressed since long ago in the City of Kyoto. The acquisition of land for filling and reclamation is very difficult to make

because the conservation of the cultural inheritance and the landscape is extremely important.

Therefore, for the treatment and disposal of sludge, the amount of dewatered sludge has been reduced to a minimum and all of dewatered sludge has been incinerated in order to reduce final disposal quantity and prevent the occurrence of pollution.

However, 511 tons/day of dewatered sludge was produced in fiscal 1984, and the amount of sludge produced will increase in the future in response to the annual growth of 4% of the sewered area. It is apparent that the capacity of the existing disposal site will become insufficient in near future because of this growth. Therefore, the City of Kyoto is promoting the effective utilization of sludge as natural resources and is also planning to dispose of sludge by land fill through participation in "Regional Treatment Plant Development Project in Osaka Bay Regions (so called Phoenix Plan)" formulated by the central government.

Utilization of sludge as resources must be promoted not only for reducing the amount of sludge to be finally disposed of but also for the purpose of resource conservation. The City of Kyoto is already using part of ashes as a soil conditioner for farmland and a soft ground. Moreover, ashes are being used as material for road construction and concrete products and dewatered sludge are being used as compost in order to expand the effective use of ashes and dewatered sludge. The effect of compost on green districts and farmland and the growth of plants are being studied at present. Marketing research for the compost will be surveyed in future.

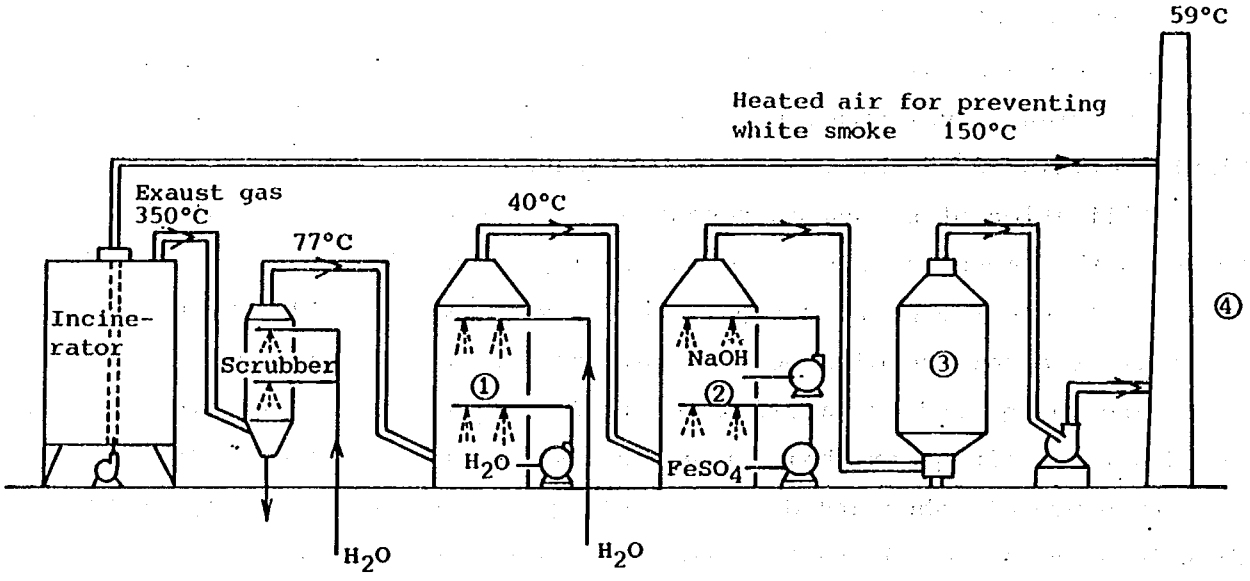
3.6 Measures for Pollution Control

In the treatment and disposal of sewage sludge, it is important to prevent the occurrence of secondary pollution. The Pollution Control Law restricts the emission of dust and soot, sulphur oxides, nitrogen oxides and hydrogen chloride contained in the exhaust gas from incinerators.

In the Toba Treatment Plant, the exhaust gas is treated by cooling tower, electric dast collectors and contaminant removing tower shown in Fig. 8. Table 8 shows the concentrations of various items after the treatment of exhaust gas, the emission of which is restricted by the law. The exhaust gas treatment facilities provide a sufficient removal effect for the restricted items, and the concentration of such items in exhaust gas after the

treatment is of course lower than the regulatory standards. Moreover, to prevent the exhaust of white smoke from smokestack, the heated air is introduced from the cooling device for rotating axis of the incinerator to the middle of the smokestack to raise the temperature of the exhaust gas.

Ash disposal site is legally restricted as the treatment facilities for industrial wastes. Because of this, the filling of ashes is being carried out in the disposal site with asphalt pavement at the bottom surrounded by reinforced concrete walls in accordance with the legal provisions. For disposal by filling, the leaching of hazardous substances is periodically tested in accordance with the laws, and the test results fully meet the standard values set forth in the laws.



(1) Cooling tower	(2) Contaminant-removing tower	(3) Electric dust collector	(4) Smokestack
Spray type, 161,300 m^3/hr, 1 unit	Spray type, washing with acid & alkali, 94,100 m^3/hr, 1 unit	Wet type, 80,000 m^3/hr, 2 units	Made of reinforced concrete, 2-m dia. and 30-m height, 120,300 m^3/hr, 1 unit

Fig. 8 Treatment processes of exhaust gas from incinerators

Table 8 Analytical results of exhaust gas from incinerator

(FY 1984)

Item	Measured values	Regulatory standards
Dust & soot (g/Nm^3)	Less than 0.013	0.7
Sulphur oxides (ppm)	1.33 (k = 0.08)	k = 2.34
Nitrogen oxides (ppm)	48	300
Hydrogen chloride (ppm)	28	700

4. AFTERWORD

In carrying out sewerage works in the City of Kyoto, a five-year plan is corresponded to the 6th five-year sewerage construction plan of the Japanese Government. For the purpose of complete coverage of the sewerage system in the whole urbanization area by the year 1994, the 1,200th anniversary of the foundation of Heiankyo (the ancient name of Kyoto capital), the works are now being carried out actively. The amount of generated sludge is increasing rapidly as the sewerage construction works progress, and the present problems in the sewage sludge treatment and disposal in Kyoto are that the sludge treatment consumes too much energy and that the final disposal is temporary in nature.

For sludge treatment in the future, it is necessary to efficiently operate the facilities, and to actively reduce the quantity of sludge to be finally disposed of. For final disposal, it is necessary to participate in the regional seashore reclamation projects by the Government and to also shift the disposal to the recycling type, that is, the effective utilization of sludge. For achieving these, it is necessary to make efforts while maintaining the vitality of the private sector from both the regional and long-term viewpoints.

MULTI-STAGE SLUDGE STABILIZATION

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1. Introduction

The government of the Federal Republic of Germany fixed with its first environmental program /1/ of September, 29th, 1971 the aim to purify the sewages from the municipal and industrial sectors until 1985 by methods of complete treatment. This purpose has almost been achieved today. As a consequence of these efforts an evident increase of the corresponding amounts of sludge had to be accepted besides the distinctly improved water quality situation.

The removal of the sewage sludges originating from the purification process gave the operators of sewage treatment plants more and more trouble. This development was caused by a changed knowledge in large parts of the population concerning emissions and immissions of odors, by new laws and novations and not at least by stricter terms of permission on the part of the authorities.

The problems of emission caused by sludge disposal can only be solved fundamentally for a longer period by stabilizing as well as dewatering the sludges to a greater extent than it is usual today. Therewith especially aspects of long-termed and raised safety in service as well as ecological and economical acceptability are emphasized.

The use of multi-stage systems offers itself as a possible solution of the sludge stabilization because the organisms involved

with the degradation processes show different physiological qualifications. By this the optimum environmental factors of the individual organisms can be adjusted and controlled more effevtively in order to achieve an intensification of the sludge stabilization processes.

2. Process Aims of Sludge Stabilization

The process aims of sludge stabilization are summed up in Fig. 1. The conversion of the organic sludge components are of superior importance in order to avoid negative influences to the environment. Secondary effects may be of extraordinary weight in a special case but they must not question or influence negatively the aforementioned primary aim of the stabilization measure. On the other hand a stabilization method always becomes doubtful if it is of an inadequate or worsening result with regard to the secondary effects as there are for example dewateribility or the necessary disinfection.

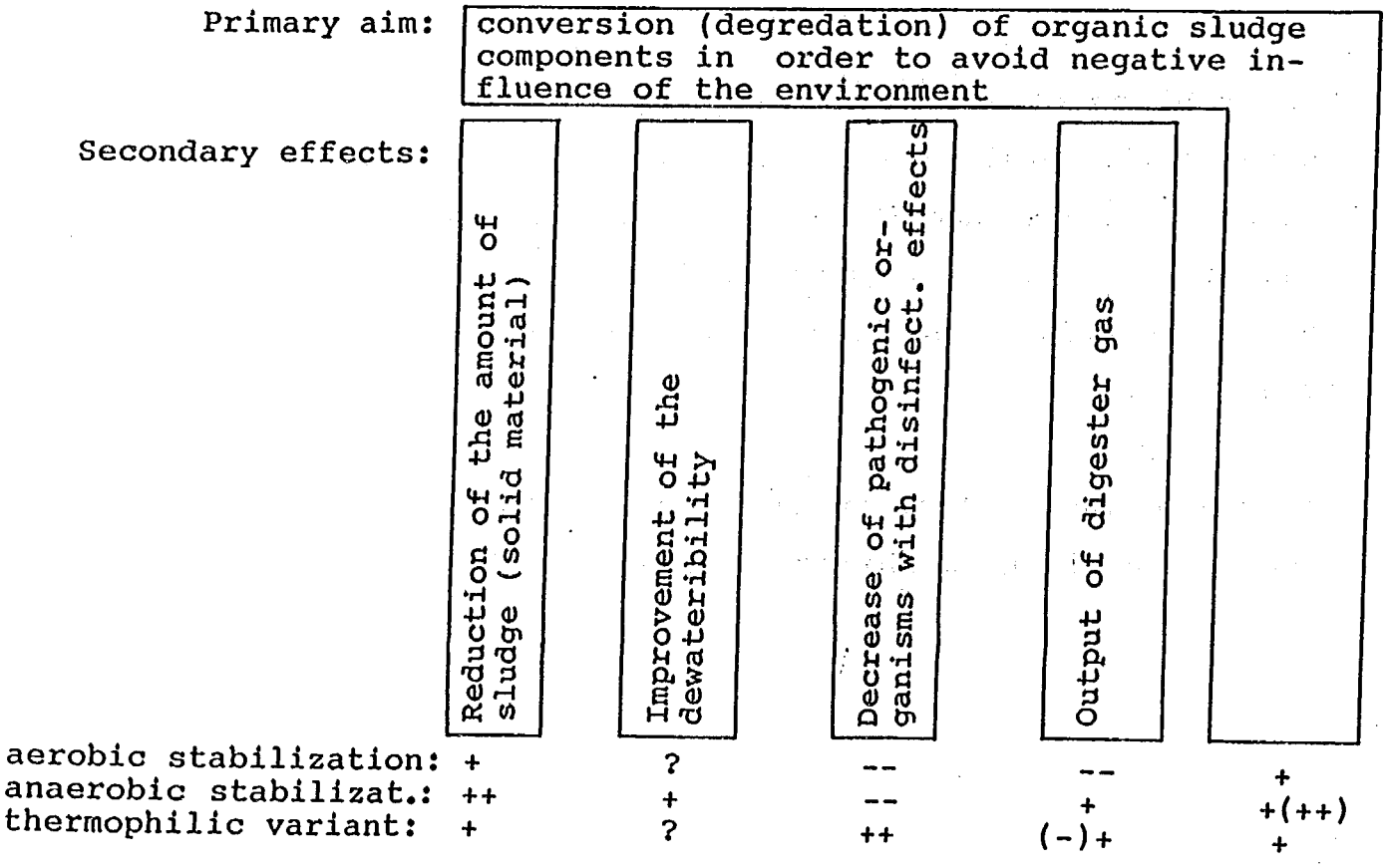


Fig. 1: Process aims of the sludge stabilization

3. Practicable Combinations of known Stabilization Methods

A schematic classification of stabilization methods is shown in Fig. 2. Besides the biological methods a stabilization of the raw sludges which become available can be achieved by a chemical and/or physical treatment. In practise these methods only have a bearing on sewage treatment plants which are very small or very large or which are working under special marginal conditions.

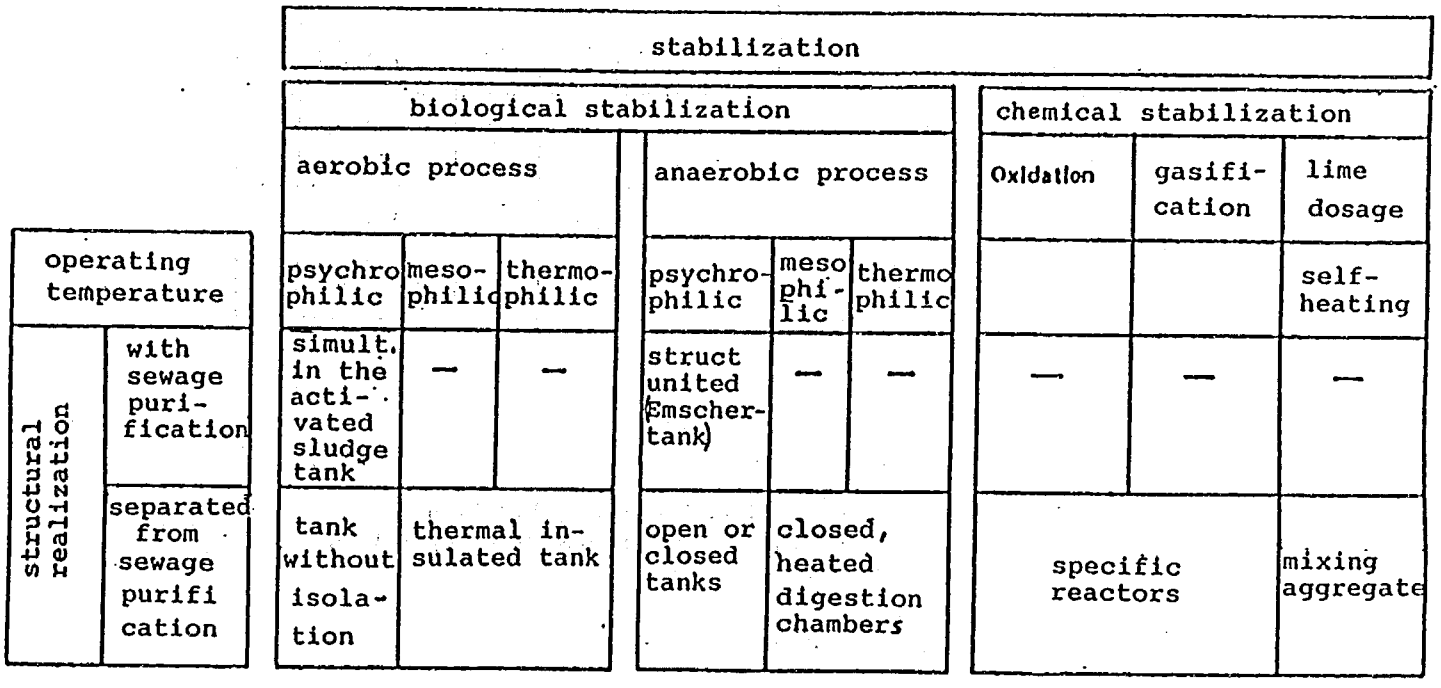
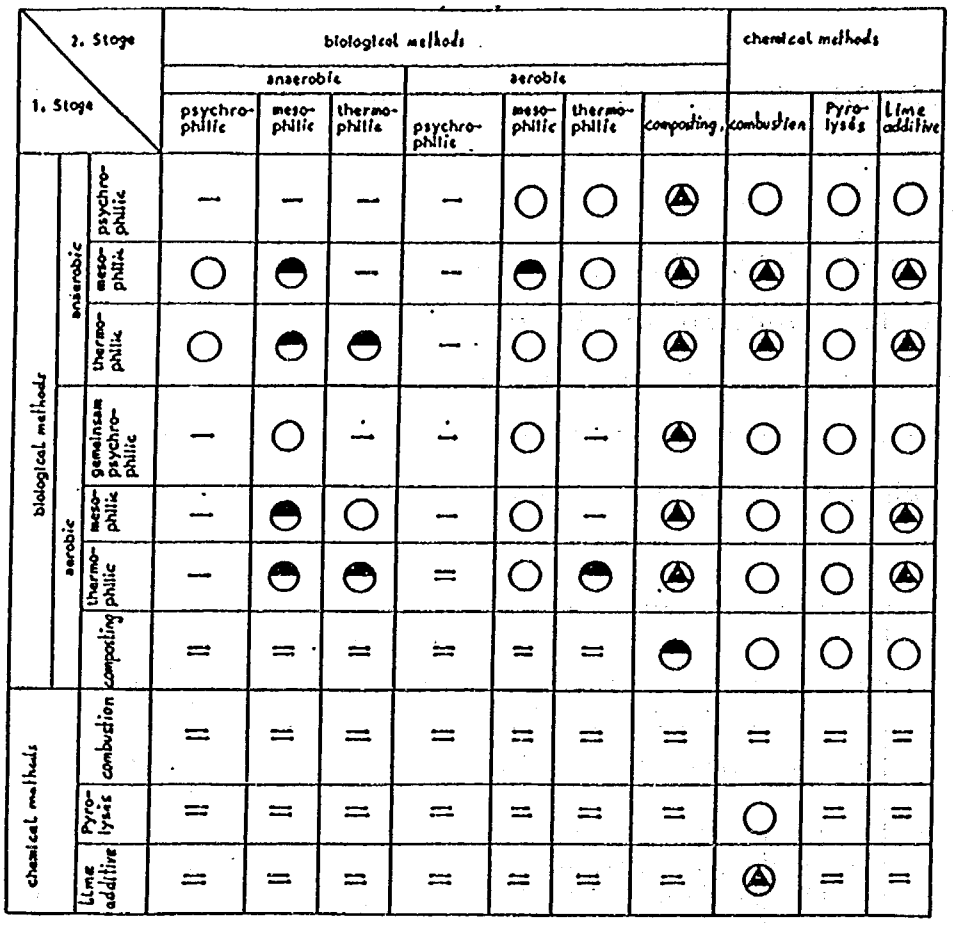


Fig. 2: Schematic classification of stabilization methods

An optimization of the basic operation "sludge stabilization" seems to be possible regarding the aforementioned increasing importance of the multi-stage mode of process as well as the combined technique. Fig. 3 shows the imaginable combinations of the two-stage treatment of sewage sludges with biological as well as with chemical-physical methods.



Legend:

= combination not possible

— { combination not sensible because of disadvantages compared with other combinations

○ irrelevant advantages as against one-stage operating

◭ { practicable use possibel with corresponding marginal conditions

◓ { better stabilization result to be expected with corresponding operating

Fig. 3: Two-stage combinations for the stabilization of sewage sludges.

The tabulation illustrates that some combinations from the sector of the biological stabilization (aerobically as well as anaerobically) have to be regarded as especially promising. Examinating the most essential possibilities of combination one can deduce 4 basic types:

3.1 Aerobic/aerobic mode of method

This mode of method is illustrated in Fig. 4. According to investigations of LOLL /2/ it shows, with appropriate construction, some advantages in contrast to a one-stage technique regarding the primary aim in view. Just as well a safe operating of this method is ensured within the meaning of a simultaneous disinfection at thermophilic process conditions.

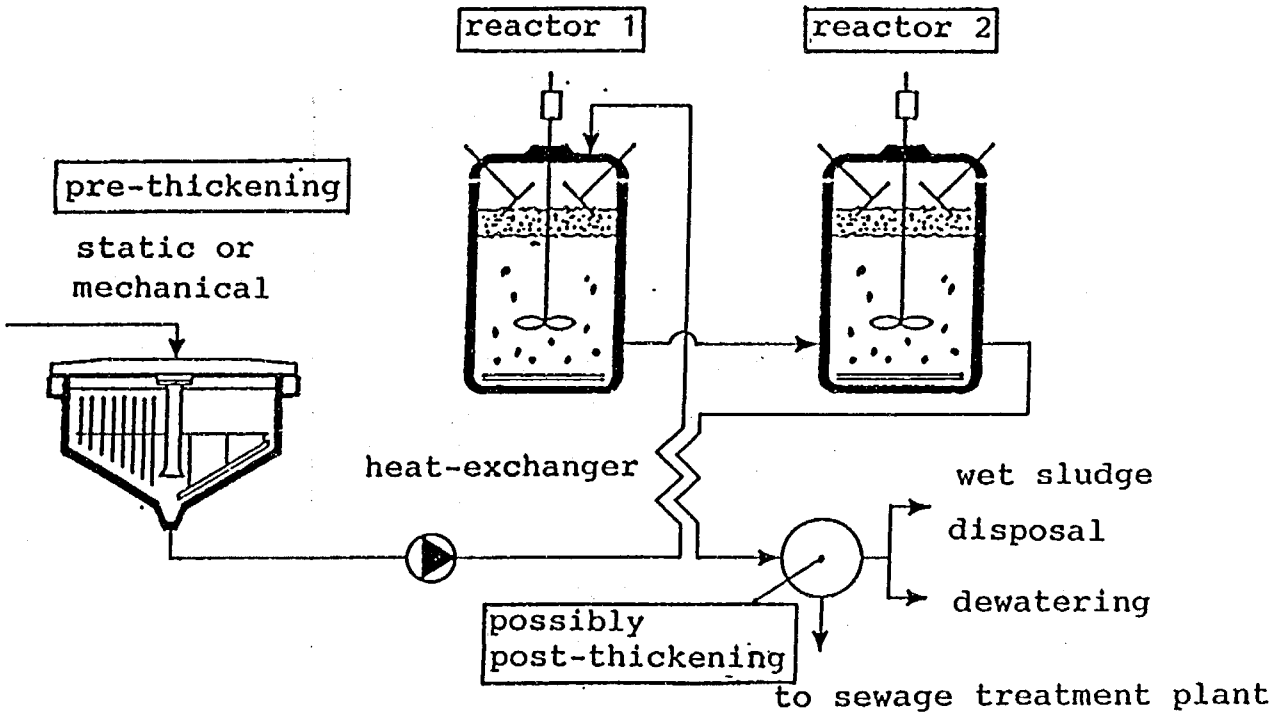


Fig. 4: Flow-diagram of an aerobic-thermophilic two-stage sludge stabilization

3.2 Anaerobic/anaerobic mode of method

The digestion of organic substances in an anaerobic environment, which is shown in Fig. 5, proceeds by 4 steps as is well known. The micro-organisms involved with the microbiological procedures need partly very different environmental factors for an optimum mode of operation. These can be realized by a partition of the technique into several stages.

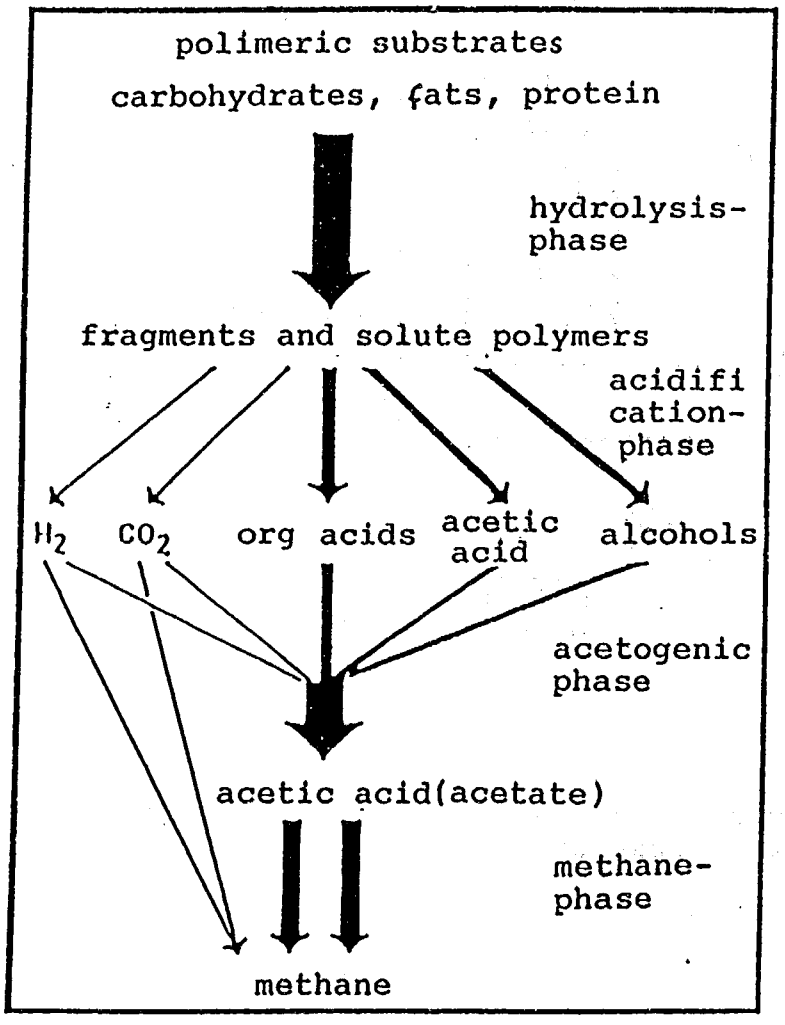


Fig. 5: Scheme of the multi-stage anaerobic degradation

The very different generation times of the micro-organisms as they are contrasted in Fig. 6 plead also for a multi-stage technique.

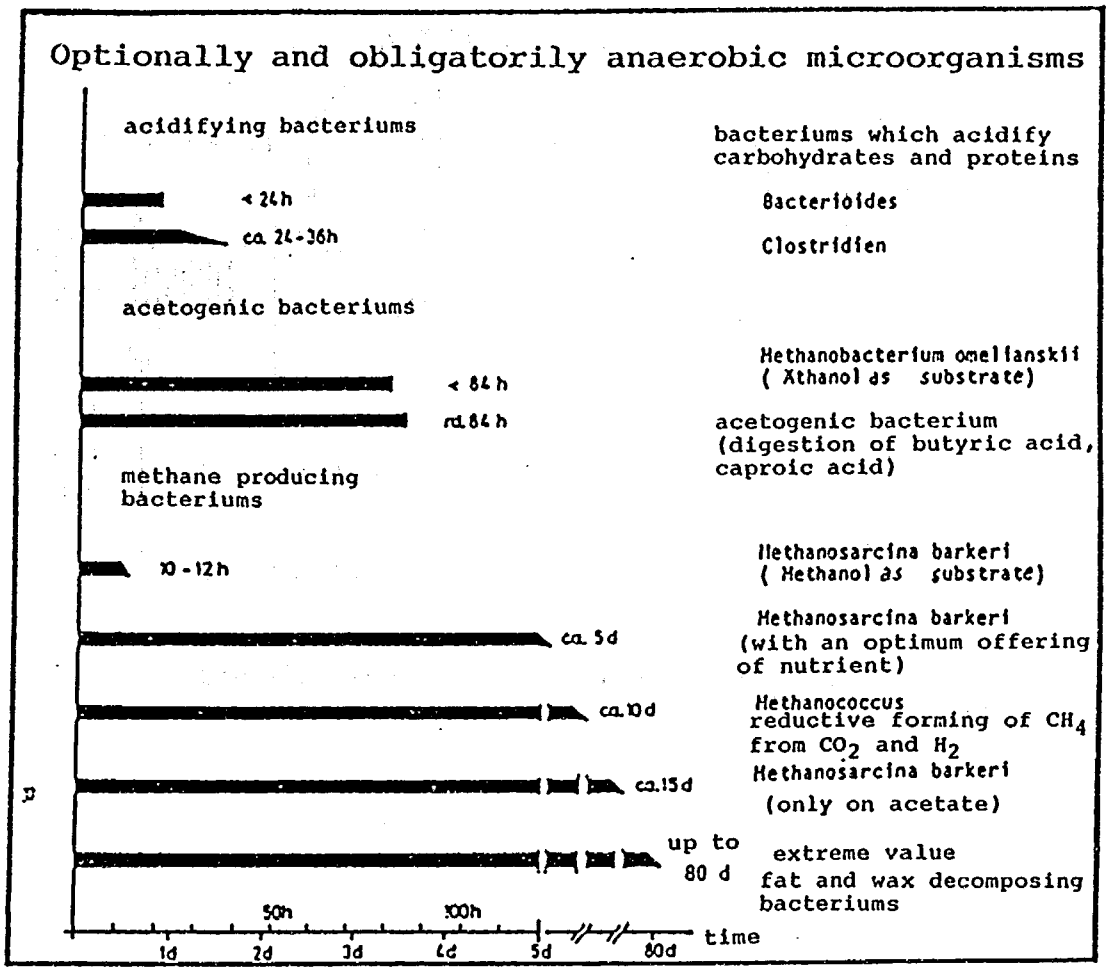


Fig. 6: List of the generation times of anaerobic micro-organisms (modified by /14/)

Fig. 7 shows a possibility for a technical realization of a two-stage process: the combination of a short-term digestion /6/ with a following conventional post-digestion seems to be a promising mode of process as will be discussed in detail later.

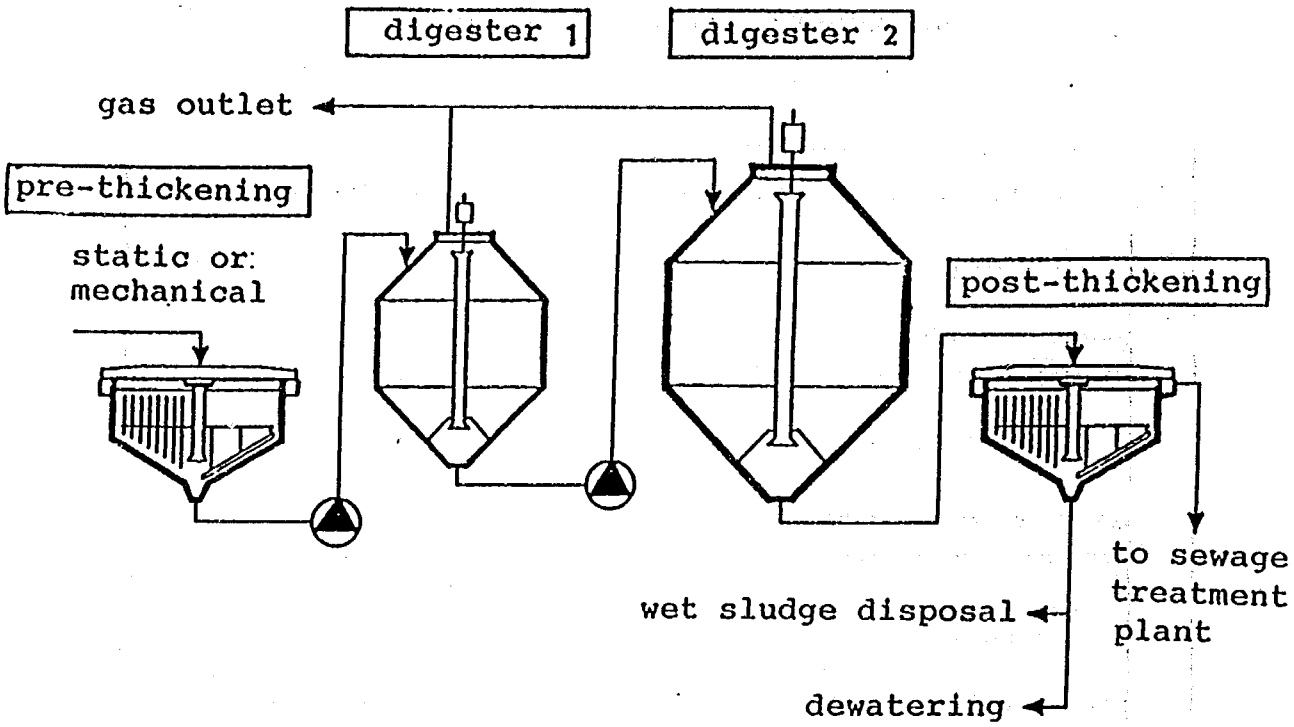


Fig. 7: Flow diagram of an anaerobic-anaerobic sludge digestion

A further two-stage process is being developed at the moment in a semi-scale pilot plant by the firm "Oswald Schulze" in Gladbeck, FRG, with the support of the "Oswald-Schulze" foundation: In the first stage a pre-acidification takes place whereby, after a retention time of 1-2 d, the biomass output is partly returned into the reactor. The methane production takes place in the second stage as well with a return of biomass and a retention time of circa 10 d.

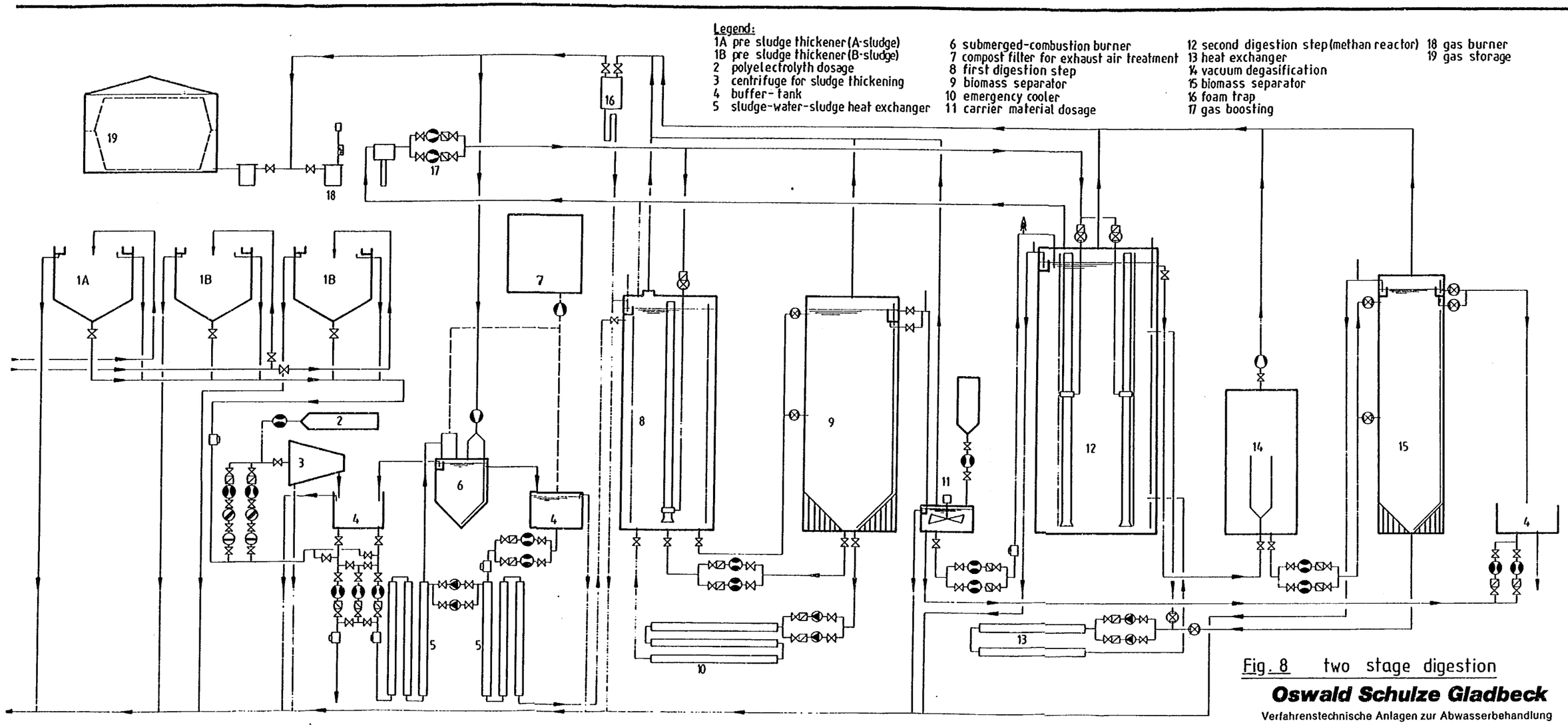
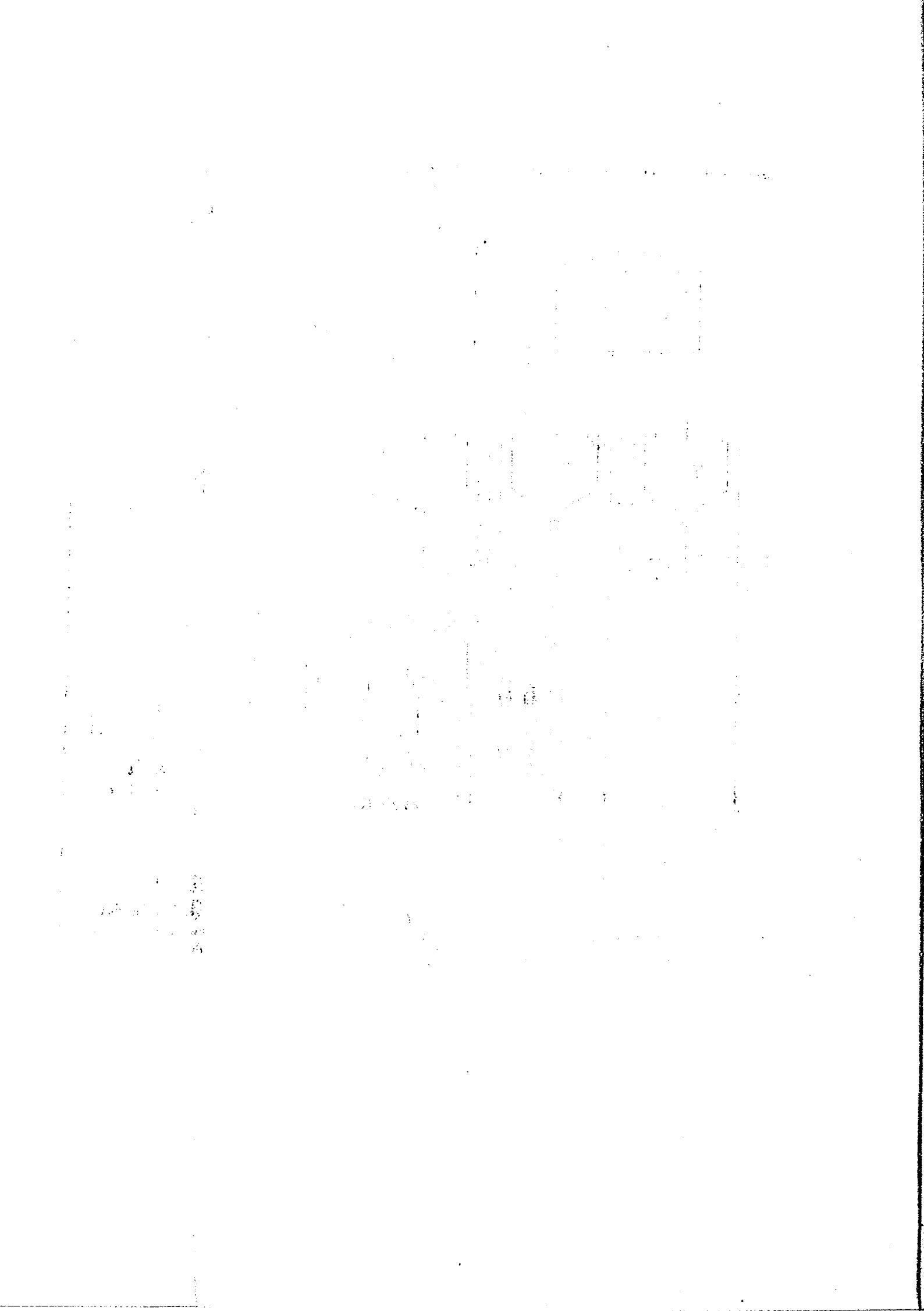


Fig. 8 two stage digestion

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Additionally a concentration of biomass is planned by adding carrier materials. At the moment the full-scale realization of this technique is being projected after having received excellent results from the preliminary tests for the treatment of problematic sludges as well.

3.3 Aerobic/anaerobic mode of method

Fig. 9 shows the flow diagram of a stabilization plant with an aerobic/anaerobic mode of process.

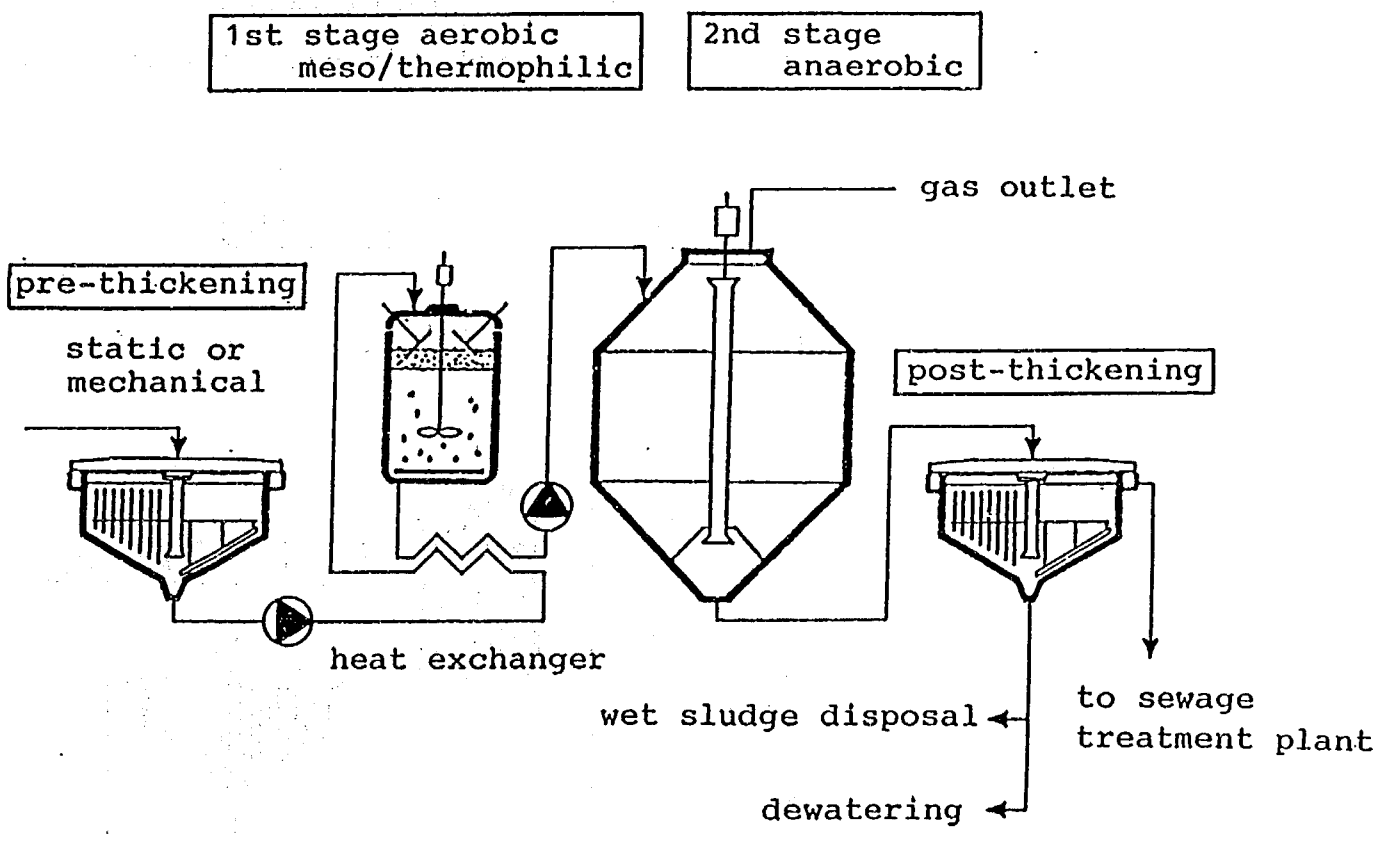


Fig. 9: Flow diagram of an aerobic-anaerobic sludge stabilization

This technique has been translated into large-scale plants for several times during the last years because a simultaneous disinfection is possible with a thermophilic mode of operation of the aerobic pre-stage. Beyond that the theoretical retention time in the second stage can be reduced. I will go into the results of some investigations more detailed later on.

3.4 Anaerobic/aerobic mode of method

This method is drawn as a flow-diagram in Fig. 10. It is known from the sector of the anaerobic sewage purification that organic ingredients can be eliminated respectively mineralized by an anaerobic pre-treatment with a following aerobic final purification.

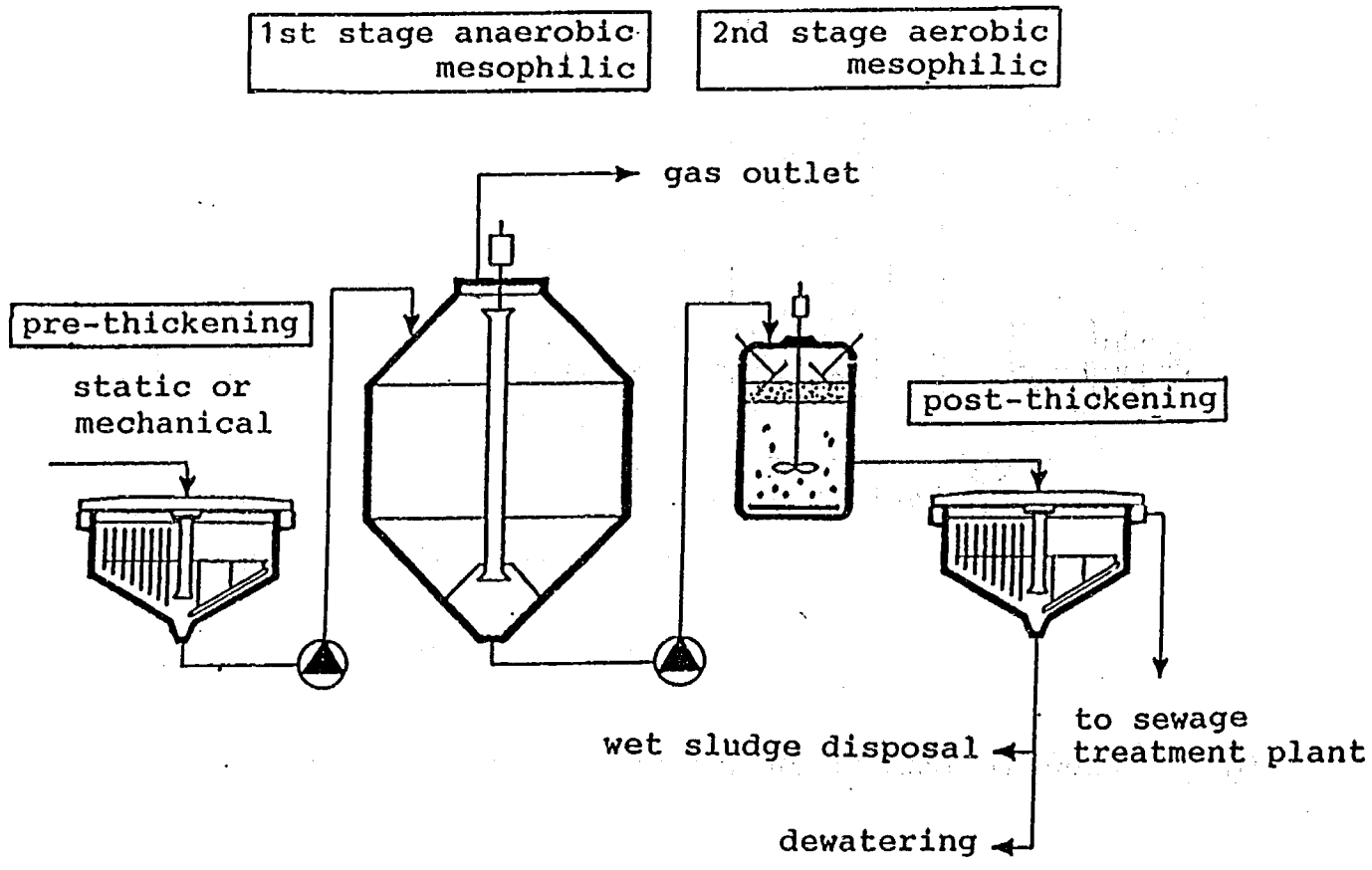


Fig. 10: Flow diagram of an anaerobic-aerobic sludge stabilization

Thus it seems to be sensible and promising to transfer this basic idea to the problems of the sludge stabilization. So far only probing laboratory tests have been carried out to examine this technique /7/; at the moment a research program of a semi-technical scale /8/ is run in this discipline. Results are surely to be expected in the course of the next two years.

4. Operating Results of the Executed Investigations

In the following the operating results of three research projects will be represented, which were executed between 1980 and 1985 at the universities of Munich, Aix-la Chapelle and Bochum /9/10/11/.

4.1 Thermophilic short term digestion with following mesophilic post-digestion

A mode of method within a thermophilic range offers itself for the first anaerobic stage considering a simultaneous disinfection of the received raw sludge. The second stage of digestion can be operated mesophilically as is generally known. Apart from a thermophilic, a mesophilic pre-digestion was examined as well in the mentioned investigations carried out by Wechs /9/,/12/.

The results of the production of digester gas, the content of methane and the total stabilization result are shown in the figures 11, 12 and 13, on the basis of the COD reduction for varied retention times with different temperatures.

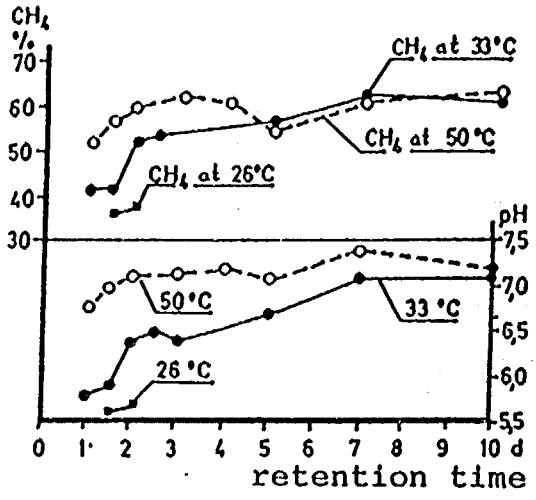


Fig. 11: Concentration of methane drawn above the retention time at different process temperatures /9/

Fig. 11 illustrates that the production of CH_4 attains a maximum (by thermophilic running) in the range between 2 and 3 days relating to the volume of the first anaerobic stage. In contrast to the theorem valid during the last few years Wechs proves that the anaerobic sludge digestion is possible and sensible in a mesophilic as well as in a thermophilic temperature range also at very short retention times.

Fig. 12 shows the CH_4 content of the received digester gas plotted as a function of the retention time. In this place it has to be marked that these are not experiments with a batch-reactor but investigations under the conditions of regular continuous processes each carried out for several weeks or months. From this it becomes clear that the running of a first anaerobic/ thermophilic digestion stage leads to a digester gas production of such a quality which enables, after being mixed with the digester gas of the second stage, its use in heating- and gas based power stations.

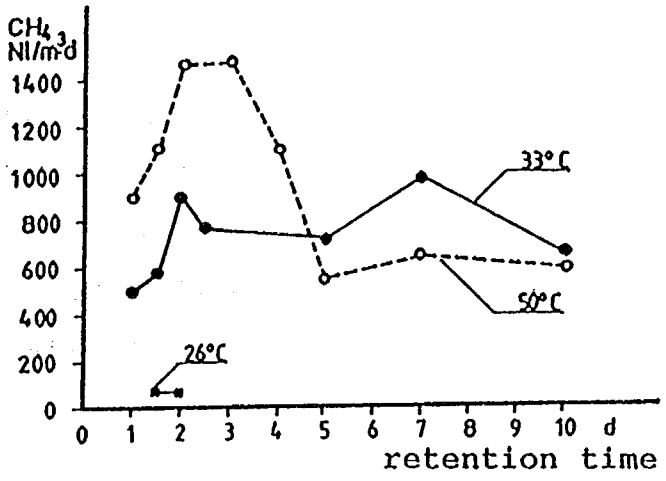


Fig. 12: Production of methane at different temperatures plotted as a function of the retention time

The first-mentioned relatively high space-related specific gas yield can be explained by the environmental factors prevailing in the first stage. With this technique organic acids which have to be regarded as a substrate for the methane producing biocenosis are to be found there in concentrations of about 2.000 up to 3.000 mg/l. The connection between metabolism and substrate concentration is known.

Furthermore one can derive from Fig. 13 that the permanently executed reference investigations (mesophilic operation with 20 days theoretical retention time) permit statements on an adequate dimensioning of such a two-stage combined technique regarding the COD reduction. A comparable COD reduction is the result already of 2 days retention time in the first thermophilic and 8 days in the second mesophilic stage. This means that a slightly more expensive mechanical and technological equipment of the stabilization plant

is opposed to a saving of a required digester volume of 50 %. As already mentioned above a simultaneous disinfection addionally is carried through in the first thermophilic stage using this technique. In this case the volume of the first stage indeed has to be split up in several tanks or it must be ensured by a so to speak continous or by an intermittent running that short circuit streams cannot occur.

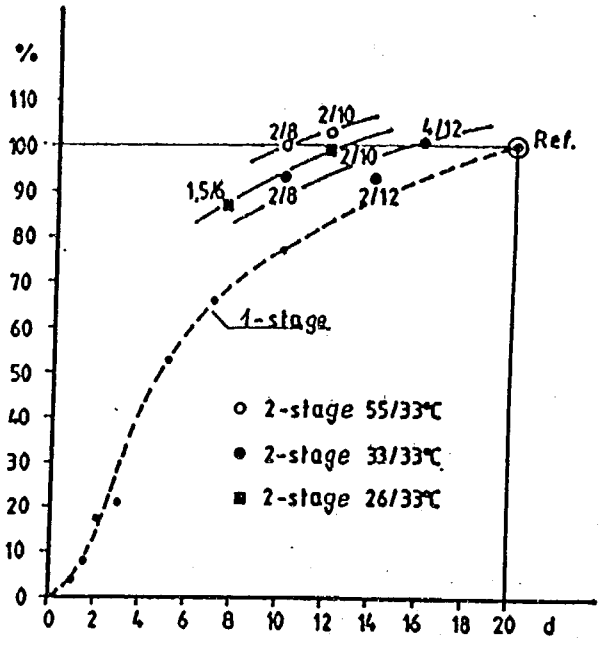


Fig. 13: Relative COD decomposition during the two-stage digestion in contrast to a one-stage mesophilic digestion process with 20 days retention time /9/

So far this technique has not been realized in a full-scale test. It is to be expected however, that this will be made good in short (several full-scale plants of this technique are being projected at the moment) so that in about 1 or 2 years sufficient experiences of the full-scale operation will be under discussion.

All in all there are the following advantages and disadvantages of the explained technique in regard to a one-stage mesophilic sludge digestion:

Advantages

- a better stabilization result with the same total volume or
- a smaller total volume with a constant stabilization result
- with a constant volume a higher gas output
- equalization of concentration in the first stage
- improved possibilities of reaction to toxic shocks
- very suitable for the enlargement of overloaded plants
- simultaneous disinfection without additional costs of a technical disinfection plant

Disadvantages

- higher expenditure of the mechanical equipment
- intensive circulation of the first stage necessary

This technique is especially favourable for operators of plants where subsequent installations for a disinfection are needed because of the ordinance on sewage sludge disposal. Another scope is to be seen in the reconstruction of overloaded digestion chambers resulting from additional connections or from a changed technique (for example two-stage activated sludge process).

4.2 Aerobic/anaerobic mode of method

The flow-diagram of a sludge stabilization plant with an aerobic/ anaerobic process control has been shown in Fig. 9. This technique can be used basically with air as well as with technical oxygen to gas the first stage. To what extent a meso- or a thermophilic temperature level is predominant in the first stage depends on different factors as there are:

- retention time in the first stage
- COD degradation in the first stage
- heat insulation of the reactor
- heat losses by gas outlet
- pre-heating by imported energy
- pre-heating by heat recovery

A safe running in the thermophilic range should be demanded from this technique too, in order to achieve a possible optimum of the process of sludge stabilization. Only by this the questions of disinfection can be considered sufficiently even if a mesophilic running would be adequate for a mere stabilization,

These circumstances are shown in Fig. 14. It is obvious that an aerobic pretreatment, independent of the temperature level of the pretreatment stage, leads to a significant discharge of the digestion, in this case compared with the stabilization result as the sum of organic acids in the outlet of the digester. On this occasion the same discharging effects occur by a mesophilic as well as by a thermophilic running and by using air- as well as technical oxygen. In the range between 0 and 2 days there is a distinct influence of the aerobic pretreatment (steep course of

the graph) whereas pretreatment periods of more than 3 days can only slightly ameliorate the stabilization grade to be achieved during the digestion. A further improved degradation then can only be obtained by lengthening the digestion time.

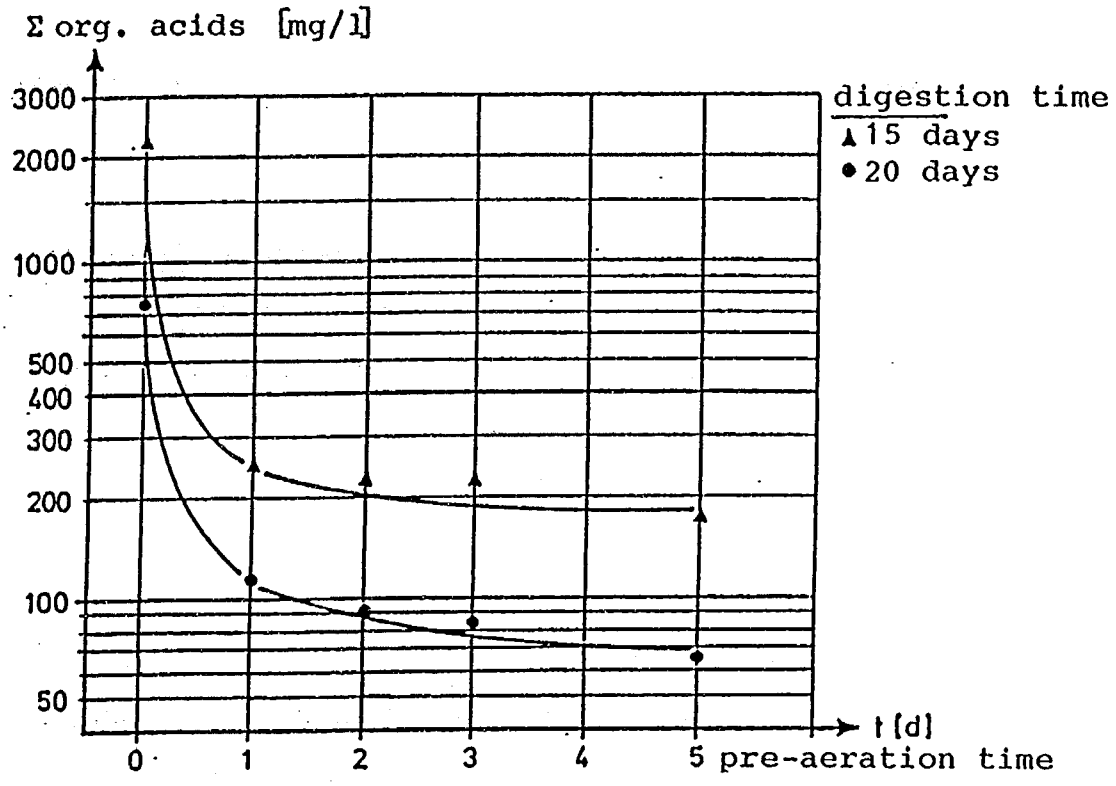


Fig. 14: Content of organic acids in the outlet of the digester as a function of the duration of the aerobic pretreatment

All in all the advantages and disadvantages of this technique can be summed up as follows:

Advantages

- smaller total volume necessary
- self-heating of the raw sludge by thermophilic microorganisms
- simultaneous pasteurization in the aerobic stage
- increase of the pH-rate up to factors between 7 and 8 in the aerobic stage
- higher pH-rates in the digestion
- equalization of volumes and concentrations in the aerobic pre-stage
- excellent stability of the total process
- total costs lower compared with the one-stage digestion
- sensible solution for a reconstruction of overloaded digesters
- smart possibility for a subsequent equipment of existing plants if a disinfection is asked for.

Disadvantages:

- slightly lower gas output
- higher expenditures to control the process (for example additional aggregates such as pumps, heat exchangers, etc.)
- possibly advanced pre-thickening necessary
- energy supply for the aeration of the first stage

Next to the mentioned primary aim of the sludge stabilization the secondary effects are of extraordinary importance to the operators of sewage treatment plants especially as far as the costs are concerned. Therefore the efficiency of a new technique in principle has to be compared with these factors because this is the only way to ensure its acceptance in practise. At the moment the following can be stated in detail regarding these secondary effects:

a) Reduction of the amount of sludge solids

For years Möller /3/ has examined scientifically the reduction of solid matter during the sludge digestion but there don't exist results about the aerobic stabilization. It can be expected that the reduction of the amount af sludge solids of a two-stage aerobic/anaerobic technique is of a similar dimension as of the sludge digestion. Exact results are expected from the full-scale use of this technique through the next years.

b) Amelioration of the dewaterability

At the moment the improvement of the dewaterability of this technique is disputed. While the mentioned scientific investigations /10/, /11/ could not prove an amelioration in this regard, Keller /5/ reports a significant improvement of the dewaterability. Statements of producers of plants indicate this effect as well. Investigations by Siekmann point out that the CST (used as a parameter for the dewaterability) even got slightly worse using the two-stage technique. Discussing this question one has to consider that the results of Keller as well as those of the mentioned producers of plants are based on operating results from Swiss sewage treatment plants. An inspection of these plants showed that these results cannot be transferred directly to German plants. None of these plants runs a mechanical dewatering of sewage sludge as it is common in Germany, the sludges are on the contrary post-thickened. However even the post-thickening cannot be compared with the post-thickening usual in Germany (run-through thickeners or batch thickeners with retention times of a few hours or days). On the contrary the post-thickening time of the mentioned Swiss plants reaches several weeks or months. Additionally all plants were equipped with a phosphate precipitation which influences the characteristics of sludge referring to the sedimentation and dewatering reaction as is well known. For these reasons

the so achieved results are surely not comparable to the results which one can expect from the German circumstances.

c) Reduction of pathogenic germs

According to Strauch /17/ a safe disinfection is ensured by a thermopilic process control in the pre-stage and a corresponding mode of method (prevention of short circuit streams) at sufficient retention times. One can say that the thermophilic pretreatment stage achieves an equally good result compared with a thermic pre-pasteurization. Here as well one can refer to Swiss experiences as the Swiss ordinance on sewage sludge orders a limit of less than 100 enteric bacteriums as a reference rate of disinfection. For German standards this tchnique can be classified as an orderly treatment according to the ordinance on sewage sludge.

d) Gaining of digester gas

The gaining of digester gas is of special interest for many operators of plants as by this a partly recovery of the energy used for the sewage purification and sludge treatment is possible. With the here presented technique the energy becomes available by two different kinds of process, that is:

1. Self-heating of raw sludge by aerobic-thermophilic degradation processes. The self-heating of sewage sludge by aerobic degradation has been known for long. Theoretical considerations as well as the interpretation of the mentioned tests have shown that one can proceed from a medium heat evolution of 2,4 up to 2,6 °C per g of decomposed COD and l. For a view on the total energy balances the achieved heat evolution then has to be contrasted with the loss of digester gas.

2. Production of digester gas

At the moment here exist contradictory opinions about the mode of action of an aerobic-thermophilic pretreatment. It should be remarked that the scientific investigations carried out so far have each stated a loss of gas (related to a 20 days digestion) whereas the aforementioned Swiss operating results assume a surplus of gas. A more precise investigation has shown that here often the running of a technically natured two-stage technique has been compared with a before existing unheated one-stage digestion. So the now achieved surplus of gas certainly can be explained. Fig. 15 shows the gas output implicating the loss of gas by aerobic pretreatment. An aerobic pretreatment is always connected with a degradation of organic matter (measured as BOD and COD) and a shifting of the BOD_5/COD ratio consequently. Therefore it becomes clear that with an aerobic predegradation, that is a reduction of the BOD_5/COD ratio in the raw sludge, the gas output to be expected decreases. These facts are obvious because biomass respirated aerobically is not available for an anaerobic metabolism.

It is incomprehensible that the proponents of this technique predict a gas surplus, because the advantages of the method (self-heating, simultaneous disinfection) predominate as well with a minor gas output. The total efficiency of this technique is guaranteed even with a 20 % deficit of gas output.

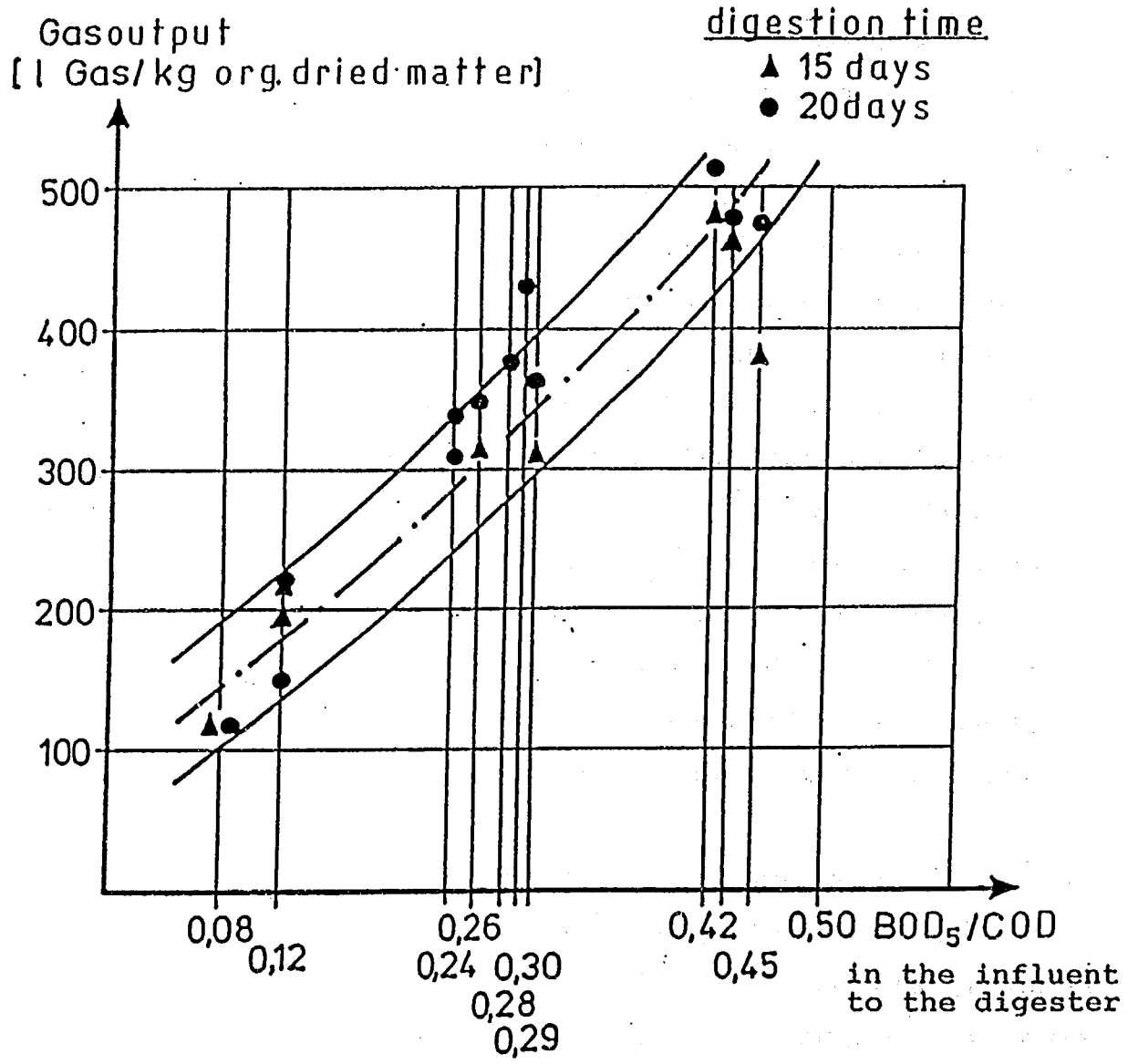


Fig. 15: Specific gas output as a function of the BOD_5/COD ration in the influent of the digester

5. Summary

The here presented results of investigation as well as already existing experiences from large-scale plants show that a two-stage technique of sludge stabilization is of distinct advantages in contrast to a one-stage mesophilic digestion or a one-stage aerobic stabilization. An aerobic as well as an anaerobic thermophilic pretreatment seems to be a promising partial step to reach not only the primary aim of stabilization. Altogether the advantages of a two-stage thermophilic/mesophilic mode of process compared with a one-stage mesophilic one can be summed up as follows:

1. smaller total volume to achieve an equivalent stabilization result
2. System-immanent disinfection without additional process-stage
3. very high process stability
4. reduced total costs compared with the one-stage technique
5. smart possibility for a subsequent equipment or reconstruction of existing plants

It is to be expected that after previous careful examination and calculation of the efficiency to be expected the two-stage technique will be of increasing importance in the near future.

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EVALUATION OF ENERGY SAVING INCINERATION USING HEAT BALANCE MODEL

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Evaluation of Energy Saving Incineration Using Heat Balance Model

INTRODUCTION

This paper describes the examination process of energy saving measures to reduce the required volume of auxiliary fuel consumed by the two types of furnaces for sewage sludge cake, i.e. stoker furnaces and fluidized bed furnaces, and gives the acutal method used, as below.

(1) A heat balance model was prepared on the basis of the mass and heat balance of the furnaces.

(2) The consumption of auxiliary fuel was estimated by changing such parameters as the air-to-sludge ratio and the moisture and organic contents of the cake of the above model.

(3) The differences between the theoretical values obtained by applying concrete data from the current operation to the model, and the measured results were examined and analysis made of the factors necessary for energy saving incineration.

1. EVALUATION OF ENERGY SAVING INCINERATION BY STOKER FURNACES

1.1 Characteristics of Stoker Furnaces

In the late 1960's, a heat treatment method using stoker furnaces was introduced in several cities, including Sapporo, for the treatment of sewage sludge. The cake obtained through this heat treatment method had a lower moisture content and a higher calorific value than that in the lime case using lime and ferric chloride as coagulants (polymer cake was almost non-existent at that time). These characteristics were similar to those of low grade coal and as a result, stoker furnaces began to be generally used

for the heat treatment of dewatered sludge. One main reason for the use of stoker furnaces was the effective use of the waste heat boiler to collect the heat from the high temperature exhaust gas and use it as a heat source for the treatment.

Recent sewage sludge treatment shows the following trends.

- o Increase in the calorific value of the cake due to the increase in the organic content.
- o Increase in the use of polymers as coagulants, as a result of which, the calorific value of the cake is higher than that of lime cake.
- o Installment of a dryer at the early stages of incineration, resulting in a lowering of the cake moisture content and stabilization of the cake characteristics.

In view of the above, an effective energy utilization plan is suggested where the autogenous combustion of sewage sludge cake in the furnace is envisaged along with the active heat recovery by the waste heat boiler. Against this background, stoker furnaces, which have achieved positive heat recovery results with respect to the heat treatment of dewatered cake, are now attracting a lot of attention. Figure 1 gives the outline of a stoker furnace.

The functions of stoker furnaces have the following advantages and disadvantages in comparison with other types of furnaces such as multiple hearth furnaces.

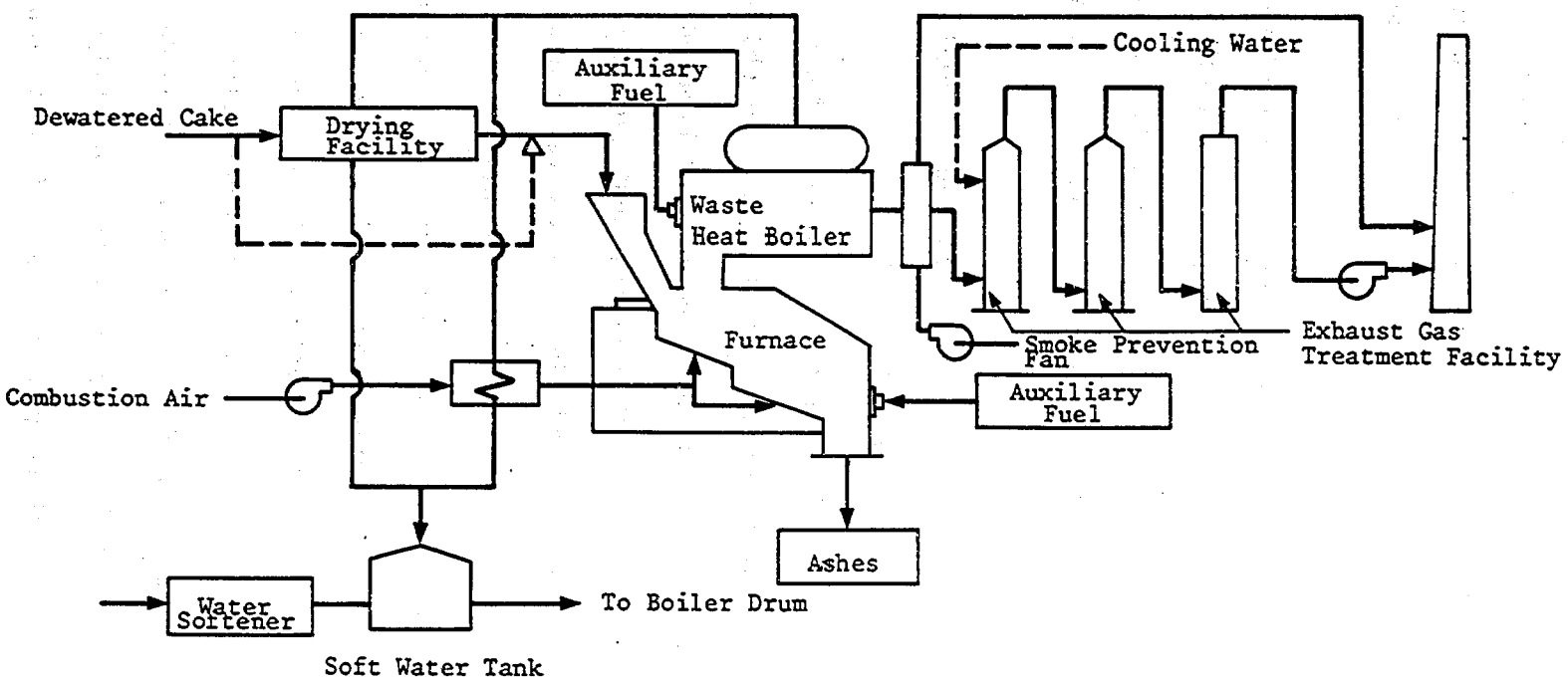


Fig. 1 Standard flow chart of incineration facility

(1) As the ash takes the form of clinker, it is easy to handle and dispose of.

(2) While multiple hearth furnaces produce a large volume of dust because of the cake and ash falling at each hearth, stoker furnaces have less dust in the exhaust gas due to the slow movement of the stokers and the lack of violent churning.

(3) In the case of stoker furnaces, the velocity and residual time of the cake and ash at drying, combustion or post-combustion (for residual carbon combustion) stoker stages can be individually regulated.

(4) The air for combustion which flows under the stokers, cools the stokers to prevent damage.

(5) Deodorization measures for the exhaust gas can be employed due to the high combustion temperature (around 800°C).

(6) When the heat recovery device employs the heat exchange method between the gases, the temperature of the heating surface which contacts the high temperature gas becomes as high as several hundred degrees centigrade. In the case of waste heat boilers, heat exchange is carried out between the gas and water, and the surface temperature of the heat exchanger tube is therefore only 15° - 20°C higher than the saturation temperature corresponding to steam pressure (for example, when heat recovery is made from saturated steam of 8 kg/m^2g, the corresponding saturation temperature is approximately 175°C). The stability of the equipment is therefore superior to that of other methods.

(7) As there is no violent churning of the cake in the stoker furnaces, the radiant heat does not sufficiently penetrate the sludge in the case of the direct incineration of polymer sludge. In addition, it is difficult to achieve thorough contact of the sludge with the combustion air

and because of this, drying tends to be a slow process. The maximum moisture content of the cake for efficient combustion is around 65%. The air-to-sludge ratio, an important factor in energy saving incineration, is conventionally as high as 1.7 - 2.0 which is similar to other methods such as multiple hearth furnaces. The lower ratio of 1.3 - 1.5 is currently being used due to the advancement of energy saving measures such as the introduction of air-tight furnaces.

1.2 Heat Balance Model of Stoker Furnace

In treating cake with a moisture content of over 65% with coagulants (hereinafter referred to as "chemical added cake"), the drying process shown as Model 1 in Figure 2 was added to the primary stage of the furnace. In the case of the heat treatment method, Model 2 which includes the heat treatment process was used and the dissolution of organic matter into water due to the heat treatment reaction was taken into account in terms of the mass and heat balances. These subject processes are shown in boxes in Figure 2 with thin and thick arrows showing the movements of the energy and mass respectively.

1.2.1 Conditions for Combustion Operation

The conditions for the combustion operation of the present model were set up as follows in view of the current state of the operation.

(1) Air-to-Cake Ratio for Combustion (ϕ_S)

The conventional air-to-cake ratio for stoker furnaces is 1.7 - 2.0. However, due to furnace improvements in view of energy saving, the current operation employs a ratio of 1.3 - 1.5.

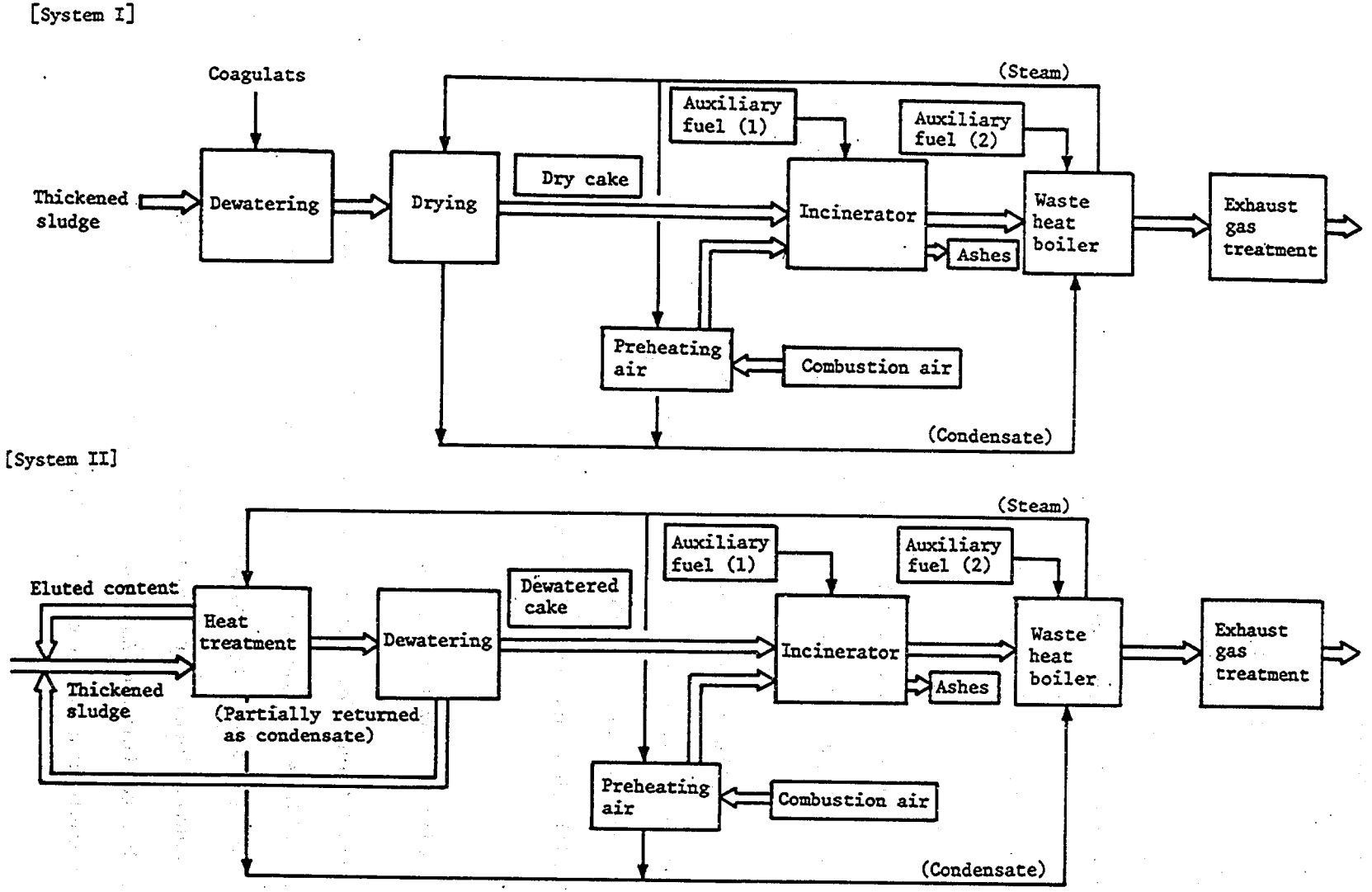


Fig. 2 Flow diagrams of stoker furnace incineration systems

(2) Auxiliary Combustion

Auxiliary combustion is introduced at the places indicated in Figure 2. Auxiliary combustion (2) is employed when the calorific value generated by the cake at the early incineration stage is inadequate as the required cake dryness is such that the cake commences autogenous combustion at the stoker furnace stage. Auxiliary combustion (1) is employed in the transitional period for the cake to commences autogenous combustion and to maintain the temperature of the stoker furnace.

(3) Heat Recovery

In the case of stoker furnaces, a waste heat boiler for gas-to-fluid exchange is installed instead of the gas-to-gas heat exchange method often used in conventional furnaces.

- o A waste heat boiler is more stable and easier to use in view of the material and mechanical strength required for heat recovery at high temperatures and also causes less dust problems.
- o The heat recovered by a waste heat boiler in the form of steam can be used for general purpose applications in terms of the effective use of recovered heat.
- o As the cake's fuel value can be expected to grow in the future due to the development of advanced dryers and other reasons, the effective use of surplus steam can also be anticipated.

When used as a heat source in the drying process in Model 1, the waste heat boiler recovers heat in the form of saturated steam of 8 kg/cm^2G with the temperature of the exhaust gas at the boiler outlet at around 250°C. In Model 2 where heat treatment facilities are included, the steam is used as a heat source for the heat treatment, heat is

recovered from saturated steam of 18 - 20 kg/cm^{2}G and the temperature of the exhaust gas at the boiler outlet is around 250° - 280°C.

1.2.2 Basic Numerical Values Used for Mass and Heat Balance Calculations

(1) Standard Used for Calculating Balance

In the model mass balance calculations, the volume of dry solids supplied to the furnace (DS) is given as a constant. Therefore, in view of the dissolution of organic matter into water during the heat treatment process in Model 2, the standard dissolution ratio is given as 20% of the DS. As a result, a model equation was prepared by increasing the volume of the cake supplied to the heat treatment process by 25%. The heat balance calculation is based on the calorific value, with the standard for the temperature (T) calculation being 0°C.

(2) Atmospheric Conditions

The air and sludge temperatures (T_2) are given as 20°C.

(3) Latent Heat of Vaporization

Vaporization latent heat of 586 Kcal/kg at 20°C is used in the model equation. However, 600 Kcal/kg is used for the calculation of the lower calorific value.

(4) Specific Heat

Table 1 shows the specific heat of the mass used in the balance equation.

Table 1 Specific heat of different mass

Mass	Air	Steam		Exhaust Gas at 250°C	DS in Sludge	Ash
		20 - 200°C	at 250°C			
Specific Heat	Ca	Cw1	Cw2	Cg	Cd	Cas
(Kcal/kg·°C)	0.24	0.445	0.453	0.252	0.4	0.3

(5) Characteristics of Cake and Auxiliary Fuel:

The characteristics of the cake and auxiliary fuel and the basic values used in the combustion calculation are given in Table 2 and Table 3. The lower calorific values of the cake and heavy oil A are calculated using the following equation and rounded to the nearest ten.

Lower Calorific Value of Combustible Content

$$H_L = H_h - (600) \times (18/2) \times (S_H/100 \text{ or } F_H/100)$$

Table 2 Characteristics of cake and auxiliary fuel

			Dewatered Cake	Heavy Oil A (Auxiliary Fuel)
High Calorific Value of Combustible Content		Kcal/kg	5,500	10,800
Low Calorific Value of Combustible Content		Kcal/kg	5,120	10,120
Elementary Composition of Combustible Content	C	%	S_C: 52	F_C:85.4
	H	%	S_H: 7	F_H:12.6
	N	%	S_N: 7	F_N: 0.4
	S	%	S_S: 1.5	F_S: 0.9
	O	%	S_O: 32.5	F_O: 0.7

Table 3 Basic values for combustion calculation

Combustible Matter / Item	Dewatered Cake		Heavy Oil A	
	Weight Standard (kg/kg)	Volume Standard (Nm^3/kg)	Weight Standard (kg/kg)	Volume Standard (Nm^3/kg)
Theoretically Required Volume of Dry Air for Combustion	$L_{SG} = 7.03$	$L_{SV} = 5.46$	$L_{FG} = 14.1$	$L_{FV} = 10.96$
Theoretical Volume of Dry Gas Generated by Combustion	$G_{SG} = 7.40$	$G_{SV} = 5.35$	$G_{FG} = 13.98$	$G_{FV} = 10.26$
Theoretical Volume of Steam Generated by Combustion	$G_{SSG} = 0.63$	$G_{SSV} = 0.784$	$G_{FSG} = 1.13$	$G_{FSV} = 1.41$

(6) Heat Loss

The following values are used to indicate the heat loss in a stoker furnace.

Heat loss during drying process (vis-a-vis total heat input) : $\ell d = 5\%$

Heat loss during treatment process (vis-a-vis required steam volume) : $\ell h = 10\%$

Heat loss in heat exchanger for preheating of combustion air : $\ell a = 5\%$

Heat loss during incineration process: $\ell i = 10\%$

Heat loss in waste heat boiler : $\ell b = 3\%$

1.2.3 Symbols Used for Mass and Heat Balance

S : Treated Sludge Volume (kg/hour)

Si: Dewatered cake input into dryer and sludge input to heat treatment process (kg/hour)

So: Sludge output from dryer and dewatered cake volume in Model 2. (kg/hour)

D : Volume of treated dry solids (DS) (kg/hour)

W : Moisture volume in sludge (if i or o are affixed, they have the same meaning as in S above) (kg/hour)

ω : Moisture content in sludge (if i or o are affixed, they have the same meaning as in S above) (%)

V_S: Volume of combustible contents in DS (kg/hour) $V_S = (D)(Vs/100)$

v_S: Ratio of combustible content in DS (%)

A_S: Ash content $As = D - V_S = D(1 - \frac{v_S}{100})$ (kg/hour)

ϕ_S: Air-to-sludge ratio

ϕ_F: Air-to-auxiliary fuel ratio ($\phi F = 1.2$)

F: Consumption volume of auxiliary fuel (kg/hour)

L: Volume of Combustion Air (kg/hour)

L_{SD}: Volume of dry air for sludge combustion

$$L_{SD} = (L_{SG})(\phi_S)(V_S)$$

L_{SW}: Volume of steam in sludge combustion air

$$L_{SW} = (X_A)(L_{SD})$$

L_{FD}: Volume of dry air for fuel combustion

$$L_{FD} = (L_{FG})(\phi_F)(F)$$

L_{FW}: Volume of steam in fuel combustion air

$$L_{FW} = (X_A)(L_{FD})$$

G: Volume of Gas (kg/hour)

G_D: Volume of dry gas generated by combustion

$$G_D = G_{SD} + G_{FD}$$

G_{SD}: Volume of dry gas generated by sludge combustion

$$G_{SD} = \{G_{SG} + L_{SG}(\phi_S - 1)\}(V_S)$$

G_{FD}: Volume of dry gas generated by fuel combustion

$$G_{FD} = \{G_{FG} + L_{FG}(\phi_F - 1)\}(F)$$

G_W: Volume of steam in gas

$$G_W = G_{SW} + G_{FW}$$

G_{SW}: Volume of steam in gas generated by sludge combustion

$$G_{SW} = G_{SSG} \cdot V_S + W_O + L_{SW}$$

G_{FW}: Volume of steam in gas generated by fuel combustion

$$G_{FW} = (G_{FSG} \cdot F + L_{FW}) = (G_{FSG} + X_A \cdot \phi_F \cdot L_{FG})\ (F)$$

G_C: Volume of dry carrier gas from dryer (kg/hour)

S_T: Volume of Steam (kg/hour)

S_{TE}: Volume of steam generated by waste heat boiler

S_{TR}: Volume of steam required for whole process

X: Absolute Humidity (kg: steam/kg: dry gas)

X_A: Absolute humidity of standard air $X_A = 0.01$

X_{C1}: Absolute humidity of carrier gas at dryer inlet

X_{C2}: Absolute humidity of carrier gas at dryer outlet

i: Enthalpy (Kcal/kg)

i": Enthalpy of steam

i': Enthalpy of condensed steam

1.2.4 Balance Equations for Models

1.2.4.1 Balance Equation for Model 1

(1) Mass Balance

(1-1) Drying Process

Figure 3 shows the drying process. The mass balance in this process is given below.

Dewatered cake input to dryer

$$S_i = (100)(D)/(100 - \omega_i)$$

DS input to dryer

$$D = (S_i)(1 - \frac{\omega i}{100})$$

Sludge moisture content at dryer inlet

$$W_i = (D)(\omega_i)/(100 - \omega_i)$$

Sludge volume at dryer outlet

$$So = (S_i)\{(100 - \omega_i)/(100 - \omega_o)\} = \frac{100\ (D)}{(100 - \omega_o)}$$

Evaporated water volume at dryer

$$\Delta W = S_i \cdot So = 100\ (D)\ (\omega_i - \omega_o)/\{(100 - \omega_i)(100 - \omega_o)\}$$

Volume of carrier gas (dry gas)

$$G_C = \Delta W/(X_{C2} - X_{C1}) = 100(D)(\omega_i - \omega_o)/\{(X_{C2} - X_{C1})(100 - \omega_i)(100 - \omega_o)\}$$

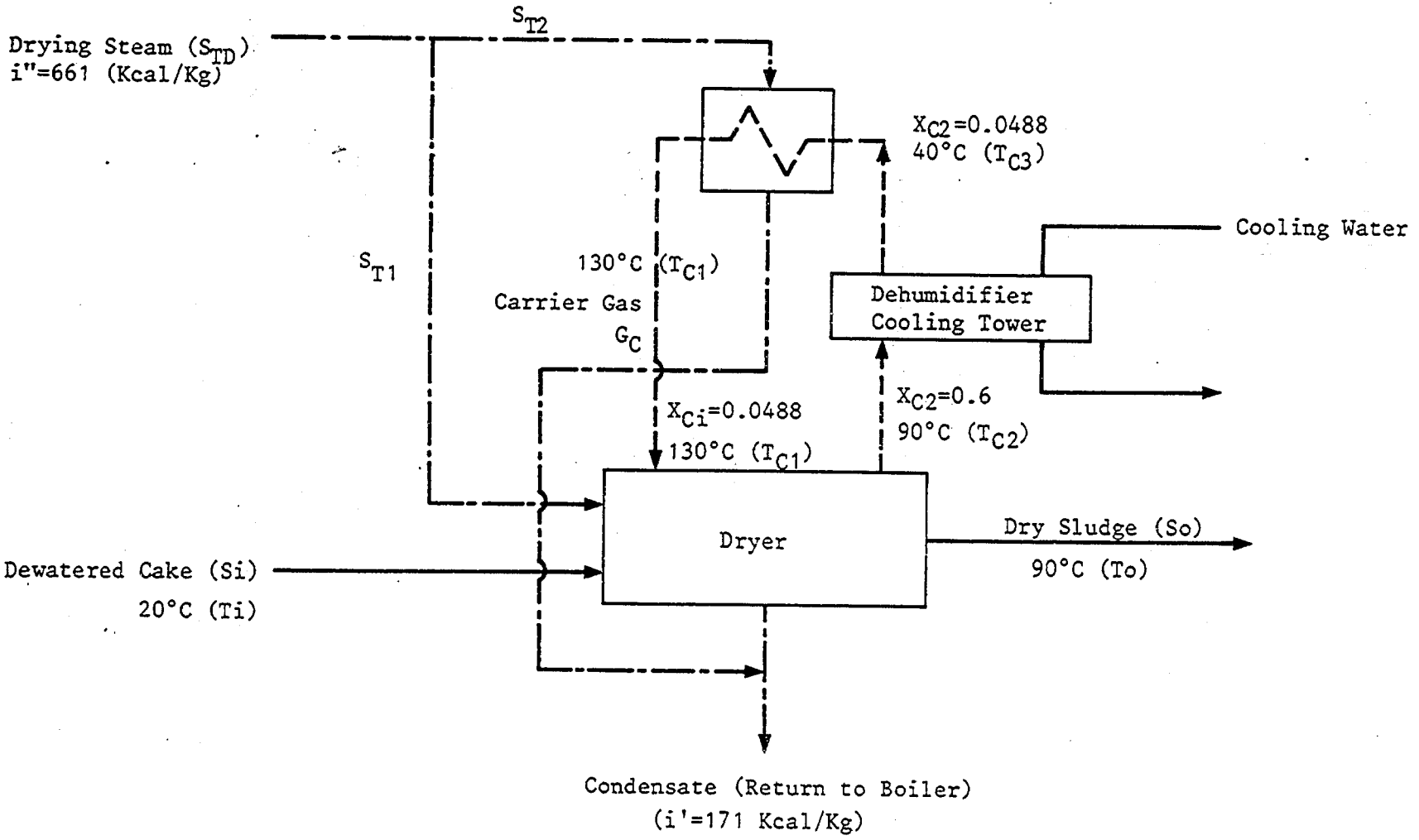


Fig. 3 Flow chart of drying process (Model I)

(1-2) Incineration Process

Figure 4 shows the incineration process. The mass balance in this process is given below.

Volume of incinerated sludge $So = 100(D)/(100 - \omega_o)$

Volume of sludge moisture $Wo = (So)(\omega_o/100)$

Volume of combustible contents $V_S = (D)(\upsilon_S/100)$

Volume of ash $A_S = D - V_S = (D)(1 - \frac{\upsilon_S}{100})$

Volume of air required for sludge combustion

$L_S = L_{SD} + L_{SW} = (L_{SG})(\phi_S)(V_S)(1 + X_A)$

Volume of air required for auxiliary fuel combustion

$L_F = L_{FD} + L_{FW} = (L_{FG})(\phi_F)(F)(1 + X_A)$

Volume of gas generated by sludge combustion

$G_S = G_{SD} + G_{SW} = \{(G_{SG}) + L_{SG}(\phi_S - 1)\}(V_S) + \{G_{SSG}\ V_S + Wo + L_{SW}\}$

Volume of gas generated by auxiliary fuel combustion

$G_F = G_{FD} + G_{FW} = [G_{FG} + G_{FSG} + L_{FG}\{\phi_F(1 + X_A) - 1\}](F)$

Total volume of gas generated by combustion

$G = G_S + G_F$

(2) Heat Balance

(2-1) Drying Process

Saturated steam of 7 kg/cm^{2}G is used as the heat source for the dryer. After the heat has been utilized at the dryer and the heat exchanger to preheat the carrier gas, the steam is returned to the waste heat boiler system as saturated condensed water of 7 kg/cm^{2}G.

[Heat Input]

Calorific value of cake input

$$Q_{i1} = (C_d \cdot D + Wi)(Ti) = (8 + \frac{20 \cdot \omega_i}{100 - \omega_i})(D)(Kcal/hour)$$

Calorific value of steam input

$$Q_{i2} = (i'')(S_{T1}) = 661\ (S_{T1})\ (Kcal/hour)$$

Calorific value of carrier gas input

$$Q_{i3} = G_C\{C_a \cdot T_{C1} + (C_{\omega i} \cdot T_{C1} + 586) \cdot (X_{C1})\} = 61.3\ (G_C)\ (Kcal/hour)$$

Total heat input

$$Q_{iT} = Q_{i1} + Q_{i2} + Q_{i3}\ (Kcal/hour)$$

[Heat Output]

Calorific value of heat output by dry cake

$$Q_{o1} = (C_d \cdot D + W_o)(T_o) = \{36 + 90\ (\omega_o)/(100 - \omega_o)\}(D)\ (Kcal/hour)$$

Calorific value of heat output by condensate

$$Q_{o2} = (i')(S_{T1}) = 171\ (S_{T1})\ (Kcal/hour)$$

Calorific value of heat output by carrier gas

$$Q_{o3} = G_c\{C_a \cdot T_{C2} + (C_{\omega i} \cdot T_{C2} + 586)\}(X_{C2}) = 396.1\ (Kcal/hour)$$

Calorific value of radiation loss by dryer

$$Q_{o4} = (1_d/100)(Q_{iT})\ (Kcal/hour)$$

Total heat output

$$Q_{oT} = Q_{o1} + Q_{o2} + Q_{o3} + Q_{o4}\ (Kcal/hour)$$

Supposing the total heat input (Q_{iT}) is equal to the total heat output (Q_{oT}), the volume of the steam required for the dryer (S_{T1}) is as given below.

$$S_{T1} = (D)\{128.96/(100 - \omega_i)\} - \{113.42/(100 - \omega_o)\} - 0.0932$$

(Kcal/hour) ..... (1)

The volume of steam required for preheating the carrier gas (S_{T2}) is obtained by the following equation.

$$S_{T2}(i'' - i')(1d/100) = G_C(C_a + C_{\omega i} \cdot X_{c1})(T_{c1} - T_{c3})$$

Therefore,

$$S_{T2} = (D)\{9/(100 - \omega_i)\} - \{9/(100 - \omega_o)\}(Kcal/hour) \quad ..... (2)$$

Based on equations (1) and (2) above, the total volume of steam required for the drying process (S_{TD}) is shown by the following equation.

$$S_{TD} = (D)\{137.96/(100 - \omega_i)\} - \{122.42/(100 - \omega_o)\} - 0.0932 \text{ (kg/hour)} \quad \ldots\ldots\ldots (3)$$

(2-2) Incineration Process

The next task is to calculate the heat balance for the furnace and waste heat boiler. The calorific value of the heat input by the cake to the furnace is neglected as it is negligible vis-a-vis the total calorific value. In calculating the heat balance for the furnace and waste heat boiler, the volume of steam required to preheat the cake combustion air is given by the following equation. Here, the temperature of the preheated air (T_2) is given as 170°C.

$$S_{T3}(i'' - i') = L_{SD}(C_a + C_{\omega 1} \cdot X_A)(T_2 - T_1)(1 + 1_a/100)$$

Therefore,

$$S_{T3} = L_{SD}(C_a + C_{\omega 1} \cdot X_A)(T_2 - T_1)(1 + 1_a/100)/(i'' - i')$$
$$= 5.524\ (\phi_S)(v_S)(D) \quad \text{(kg/hour)} \ \ldots\ldots\ldots (4)$$

Therefore, the total volume of steam required for Model 1 is given as follows using equations (3) and (4) above.

$$S_{TR} = S_{TD} + S_{T3}$$
$$= (D)\{137.96/(100\ \omega_i)\} - \{122.42/(100 - \omega_o)\}$$
$$+ 5.524\ (\phi_S)(V_S) - 0.0932 \quad \text{(kg/hour)} \ \ldots\ldots\ldots (5)$$

The heat balances for the furnace and waste heat boiler are given as follows.

Heat Input

Calorific value generated by sludge combustion

$$Q_{i1} = 5{,}120\ (V_S) - 600\ (\omega_o) \text{ (Kcal/hour)}$$

Calorific value of heat input by combustion air

$$Q_{i2} = L_{SD}(C_a + C_{\omega 1} \cdot X_A)(T_2) \text{ (Kcal/hour)}$$

Calorific value generated by auxiliary fuel combustion

$$Q_{i3} = 10{,}120 \text{ (F) (Kcal/hour)}$$

Total heat input

$$Q_{iT} = Q_{i1} + Q_{i2} + Q_{i3} \text{ (Kcal/hour)}$$

It is assumed here that the cake or pretreated dewatered sludge combusts autogenously in the furnace and that only the auxiliary combustion (2) shown in Figure 2 is employed.

[Heat Output]

Calorific value of heat output by ash

$$Q_{o1} = (C_{aS})(T_4)(A_S) \text{ (Kcal/hour)}$$

Heat loss by furnace

$$Q_{o2} = (Q_{i1} + Q_{i2})(1i/100) \text{ (Kcal/hour)}$$

Heat loss by boiler

$$Q_{o3} + (Q_{iT} - Q_{o1} - Q_{o2})(1_b/100) \text{ (Kcal/hour)}$$

Calorific value of heat output by combustion gas

$$Q_{o4} = (C_g \cdot G_o + C_{\omega 2} \cdot G_\omega)(T_3) \text{ (Kcal/hour)}$$

Calorific value of heat output by steam

$$Q_{o5} = (S_{TE})(i'' - i'_b) \text{ (Kcal/hour)}$$

Note: Here, i'b is the calorific value of the heat input by the condensate. The enthalpy of saturated water of 5 kg/cm^{2}G is given as 159 Kcal/kg.

Total heat output

$$Q_{oT} = Q_{o1} + Q_{o2} + Q_{o3} + Q_{o4} + Q_{o5} \quad \text{(kg/hour)} \quad \ldots\ldots\ldots \quad (7)$$

The volume of evaporation at the boiler (S_{TE}) is given by the following equation based on equations (6) and (7) above, where Q_{iT} is equal to Q_{oT}.

$$S_{TE} = [\{0.0889 - 3.9024\ (\phi_S)\}(v_S) + \{1.095 - 126.9024/(100 - \omega_o)\}](D) + 17.153\ (F) \quad \text{(kg/hour)} \quad \ldots\ldots\ldots \quad (8)$$

The required volume of auxiliary fuel is given as follows from equations (5) and (8).

$$F = [\{8.043/(100 - \omega_i)\} + \{0.2613/(100 - \omega_o)\} + \{5.4995 \times 10^{-4} (\phi_S) - 5.1828 \times 10^{-3}\}(v_S) - 0.0693](D) \quad \text{(kg/hour)} \quad \ldots\ldots\ldots (9)$$

When the value of the auxiliary fuel in equation (9) is given as zero, the relationship between the region for autogenous combustion and the moisture content of the dewatered cake is given by the following equation in connection with the changes of the ratio of the combustible content in the DS and air-to-cake ratio.

$$\omega_i = 100 - 8.043(100 - \omega_o)/\{[\{5.1828 \times 10^{-3} - 5.4955 \times 10^{-4} (\phi_S)\}(v_S) + 0.0693] \times (100 - \omega_o) - 0.2613\} \quad (\%) \quad \ldots\ldots\ldots (10)$$

1.2.4.2 Balance Equation for Model 2

The heat treatment Model 2 shown in Figure 4 has two different methods, i.e. high temperature method (anaerobic method) and low temperature method (aerobic method). Their treatment conditions differ in the following ways.

a) The high temperature method treats the sludge in the anaerobic condition without oxygen at 200°C while the low temperature method treats the sludge in the heat treatment reactor in the aerobic condition at 165 - 175°C by blowing air into it. As a result, the treatment pressure in the heat treatment reactor is below 17 kg/cm^{2}G in the case of the high temperature method and below 10 kg/cm^{2}G in the case of the low temperature method. Consequently, the enthalpy of the saturated steam for the heat treatment differs according to the method employed. In addition, the volume of the steam output by the reactor exhaust gas generated by the air blow cannot be ignored in the case of the low temperature method.

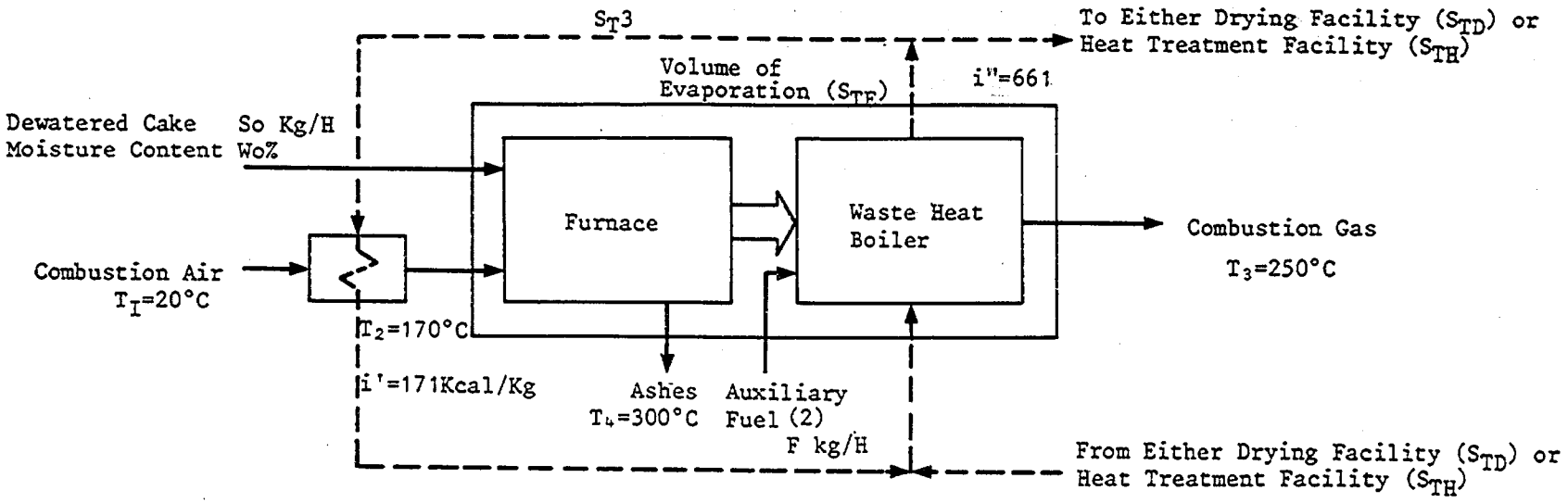


Fig. 4 Flow chart of incineration process

b) In the low temperature method, a steam type heat exchanger for heating the sludge is installed to prevent heat loss before the sludge goes into the reactor.

The low temperature method, therefore, requires a smaller volume of steam to be directly blown into the reactor for heating, as compared to the high temperature method. The recovered heat from the condensed steam flowing out of the steam type heat exchanger is used for the water supply to the boiler. (In the case of the high temperature method, the heat recovered from the heat treated sludge is used for preheating the raw sludge where the deficient calorific value is supplied by a direct infusion of steam to the sludge inside the reactor in order for the reactor temperature to reach 200°C.

Although the above-mentioned differences exist between the high and low temperature methods, the low temperature method with the steam type heat exchanger was used to prepare the equation for Model 2. A flow chart of the heat treatment for Model 2 is given in Figure 5.

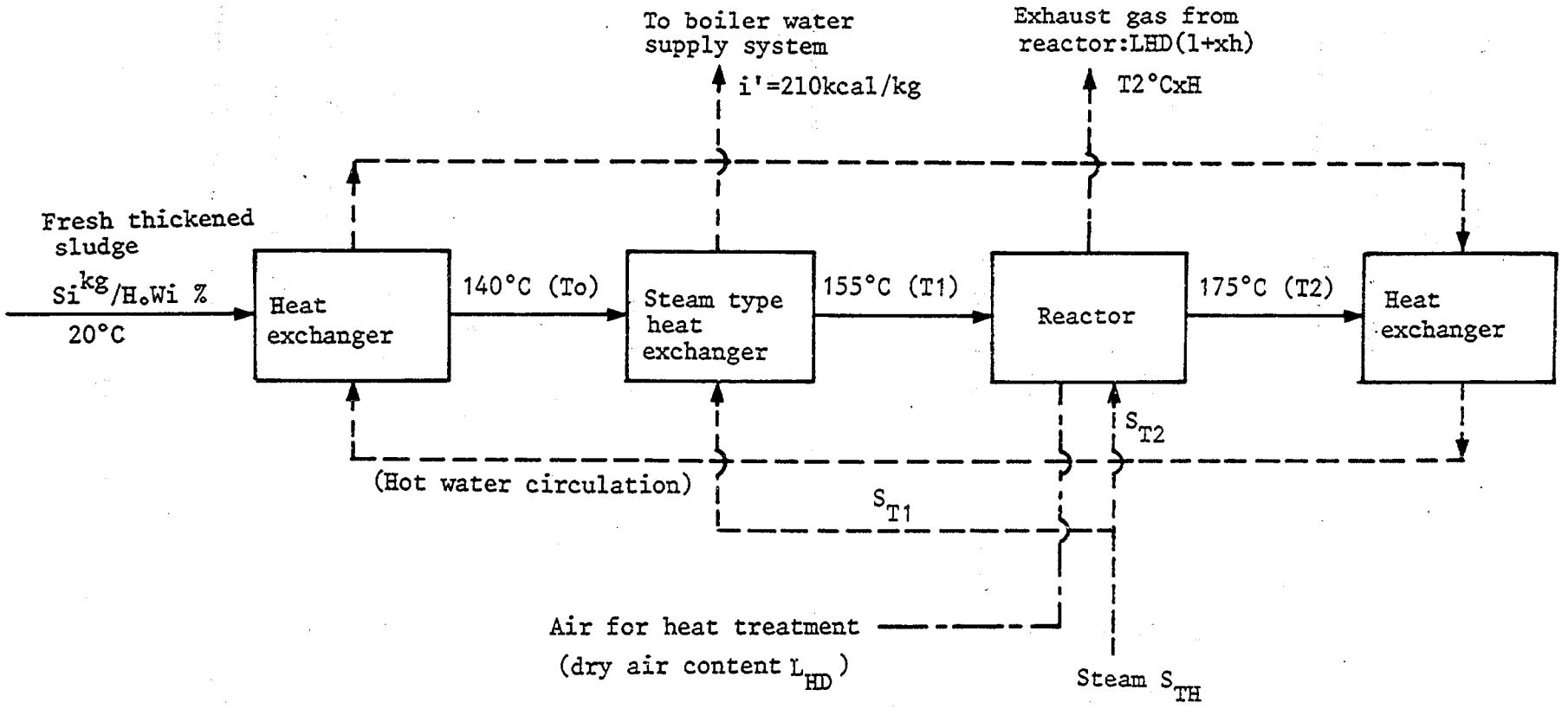


Fig. 5 Heat treatment flow sheet

(1) Mass Balance

In Model 2, the equation was prepared so that the volume of the DS based on the heat treated dewatered cake equals that of Model 1. Therefore, when the volume of the elution of organic contents, to fluid during the treatment is set at 20% of the total DS, the balance equation is given as follows.

Sludge input to heat treatment process

$$S_i = \frac{100\ (D)}{(1 - \frac{20}{100})(100 - \omega_i)} = \frac{125\ (D)}{(100 - \omega_i)} \quad \text{(kg/hour)}$$

Volume of solid after heat treatment D (kg/hour)

Volume of dewatered cake after heat treatment

$$S_o = 100\ (D)/(100 - \omega_o) \quad \text{(kg/hour)}$$

Air input to reactor (the air volume is given as 5 times the sludge volume subject to treatment and the specific volume of the sludge is given as 1 (m^3/T).

Volume of dry air

$$L_{HD} = 5 \times S_i \times 10^{-3} \times \frac{29}{22.4} = 0.809\ (D)/(100 - \omega_i) \quad \text{(kg/hour)}$$

Ratio of steam output by air to dry air (X_h)

$$Xh = \frac{18}{29} \times \frac{P_S}{(P - P_2)}\ \text{(kg - steam/kg - dry air)}$$

Here, P is the operation pressure of the reactor and P_S is the absolute pressure of the saturated steam pressure vis-a-vis the heat treatment temperature of the reactor. Therefore, under the conditions that P is 11 atm and the treatment temperature is 175°C, the saturation pressure P_S is 9.1 atm. Therefore,

$$X_1 = 2.976\ (\text{kg} - H_2O/\text{kg} - \text{dry air})$$

(2) Heat Balance

In the calculation of the heat balance, the following relation is obtained when the specific heat of the sludge is given as 1 Kcal/kg°C.

(2-1) Heat Treatment Process

Required steam volume for steam type heat exchanger (S_{T1})

$$S_{T1}(i'' - i') = S_i(T_1 - T_0) \times 1$$

$$S_{T1} = S_i(T_1 - T_0)/(i'' - i') = 4.094\ (D)/(100 - \omega_i) \quad \text{(kg/hour)} \quad \ldots\ (11)$$

The heat balance for the reactor is as follows.

[Heat Input]

Calorific value input by sludge

$$Q_{i1} = 1 \times (T_1)(S_i) = 19375\ (D)/(100 - \omega_i) \quad \text{(Kcal/hour)}$$

Calorific value input by steam

$$Q_{i2} = (i'')(S_{T2}) = 668\ (S_{T2}) \quad \text{(Kcal/hour)}$$

Calorific value input by air

$$Q_{i3} = (C_a)(T_a)L_{HD} = 9.71\ (D)/(100 - \omega_i) \quad \text{(Kcal/hour)}$$

(The steam volume in the air is neglected in the above equation.)

Total heat input

$$Q_{iT} = Q_{i1} + Q_{i2} + Q_{i3} = 19384.71\ (D)/(100 - \omega_i) + 668\ (S_{T2})$$

(Kcal/hour) ..... (12)

[Heat Output]

Calorific value output by exhaust gas from reactor

$$Q_{o1} = L_{HD}(C_a \cdot T_2 + i'' \cdot X_h) = 1632.614\ (D)/(100 - \omega_i) \quad \text{(Kcal/hour)}$$

Calorific value output by sludge

$$Q_{o2} = \{S_i + (S_{T2} - X_h \cdot L_{HD})\} \times 1 \times T_2$$

$$= 21453.6\ (D)/(100 - \omega_i) + 175(S_{T2}) \quad \text{(Kcal/hour)}$$

Total heat output

$$Q_{oT} = Q_{o1} + Q_{o2} + Q_{o3} = 23086.213\ (D)/(100 - \omega_i) + 175\ (S_{T2})$$

(Kcal/hour) ..... (13)

The required steam volume for the reactor (S_{T2}) is given by equations (12) and (13) above as follows.

$$S_{T2} = 7.508\ (D)/(100 - \omega_i) \quad \text{(kg/hour)} \ldots\ldots (14)$$

Therefore, the required total steam volume for the heat treatment process (S_{TH}) is as follows.

$$S_{TH} = (S_{T1} + S_{T2})(1 + l_h/100)$$

$$= 12.762\ (D)/(100 - \omega_i) \quad \text{(kg/hour)} \ldots\ldots (15)$$

(2-2) Incineration Process

The heat balance for the Model 2 incineration process differs from that of Model 1 in that the heat recovery at the waste heat boiler is made from the high steam pressure of 18 kg/cm^2. This is caused by the requirement for a stable air supply with a pressure below 10 kg/cm^2 to the reactor using the ejector effect of the high pressure steam in order that the input air pressure to the reactor is not subject to high pressure gas regulation. As a result, when the enthalpy of the steam and condensate in Model 1 are given as i" = 668 Kcal/kg and i' = 210 Kcal/kg respectively, and the temperature of the supply water to the boiler is given at 80°C, the following heat balance equation is obtained as in the case of Model 1.

Required steam volume for preheating combustion air (S_{T3})

$$S_{T3} = 5.91 \times 10^{-3}\ (\phi_S)(\upsilon_S)(D) \quad \text{(kg/hour)} \ldots\ldots (16)$$

Therefore, the total steam volume (S_{TR}) requirement in the Model 2 system is given as follows on the basis of equation (15) and (16).

$$S_{TR} = S_{TH} + S_{T3} = \{12.712/(100 - \omega_i) + 5.91 \times 10^{-3}(\phi_S)(\upsilon_S)\}(D)$$

(kg/hour) ..... (17)

In addition, the evaporation volume at the waste heat boiler is given by equation (18) below.

$$S_{TE} = 14.644\ (F) + [\{0.0759 - 3.332 \times 10^{-3}\ (\phi_S)\}\ (\upsilon_S) - 108.342/(100 - \omega_o) + 1.241\ (D)] \quad \text{(kg/hour)} \quad \ldots\ldots\ (18)$$

Using the above equations, the equations to determine the required volume of auxiliary fuel to make Model 2 viable and the relative expression between the cake characteristics (moisture content: ω_o, combustible content: υ_S) where no auxiliary fuel is required and the combustion air ratio (ϕS) and the concentration of sludge which is supplied to the heat treatment process (ω_i) are given below as equations (19) and (20).

$$F = [0.8715/(100 - \omega_i) + 7.3984/(100 - \omega_o) + \{6.311 \times 10^{-4}\ (\phi_S) - 5.183 \times 10^{-3}\}(\upsilon_S) - 0.0848](D) \quad \text{(kg/hour)} \quad \ldots\ldots\ (19)$$

$$(100 - \omega_i) = 0.8715\ (100 - \omega_i)/[[\{5.183 \times 10^{-3} - 6.311 \times 10^{-4}\ (\phi_S)\}(\upsilon_S) + 0.0848] \times (100 - \omega_o) - 7.3984] \ (\%) \quad \ldots\ldots\ (20)$$

1.3 Thermal Characteristics of Stoker Furnaces

Using the model equations described above, the changes in the cake moisture content (ω_i) for Model 1, in the concentration of the raw thickened sludge for Model 2 ($100 - \omega_i$) and in the combustible contents in the DS (υ_S) and the combustion air ratio (ϕ_S) can be calculated on the condition that the input volume of the dry solids (D) to the furnace is constant at 1,000 kg and that the moisture content of the sludge input (ω_o) to the furnace is also constant at 50%. The values of the parameters, i.e. the ratio of combustible contents in the DS and the combustion air ratio are set as follows.

Ratio of Combustible Contents (v_S)% 40, 50, 60, 70, 80

Combustion Air Ratio (ϕ_S) 1.3, 1.5, 1.7, 2.0

Table 4 shows the relationship between the fuel value (lower calorific value) of the sludge in accordance with the change in the ratio of the combustible contents in the DS vis-a-vis the sludge characteristics shown in Table 1.

Table 4 Relationship between combustible content ratio in dry solids and fuel value (low standard) of dewatered cake

Ratio of combustible content (%)	Lower calorific value of combustible portion (Kcal/kg)	Fuel value of dewatered cake (Kcal/kg)	
		Chemical added dewatered cake (moisture Content: 70 - 80%)	Chemical added sludge dried or heat treated (moisture content: 50%)
40	5,120	-70 - 190	700
50		30 - 350	980
60		130 - 500	1,230
70		230 - 650	1,490
80		340 - 810	1,750

1.4 Examination of Energy Saving Incineration Based on Two Models

Figure 6 examines the following items in view of the heat balances of the models when chemical added cake is dried to a moisture content of 50% and then incinerated.

(1) The auxiliary fuel consumption for one ton of DS in order for the heat balance of the system to balance against the moisture content of the dewatered cake which changes in accordance with the air-to-cake ratio. Here, the combustible contents in the DS are considered as the parameter.

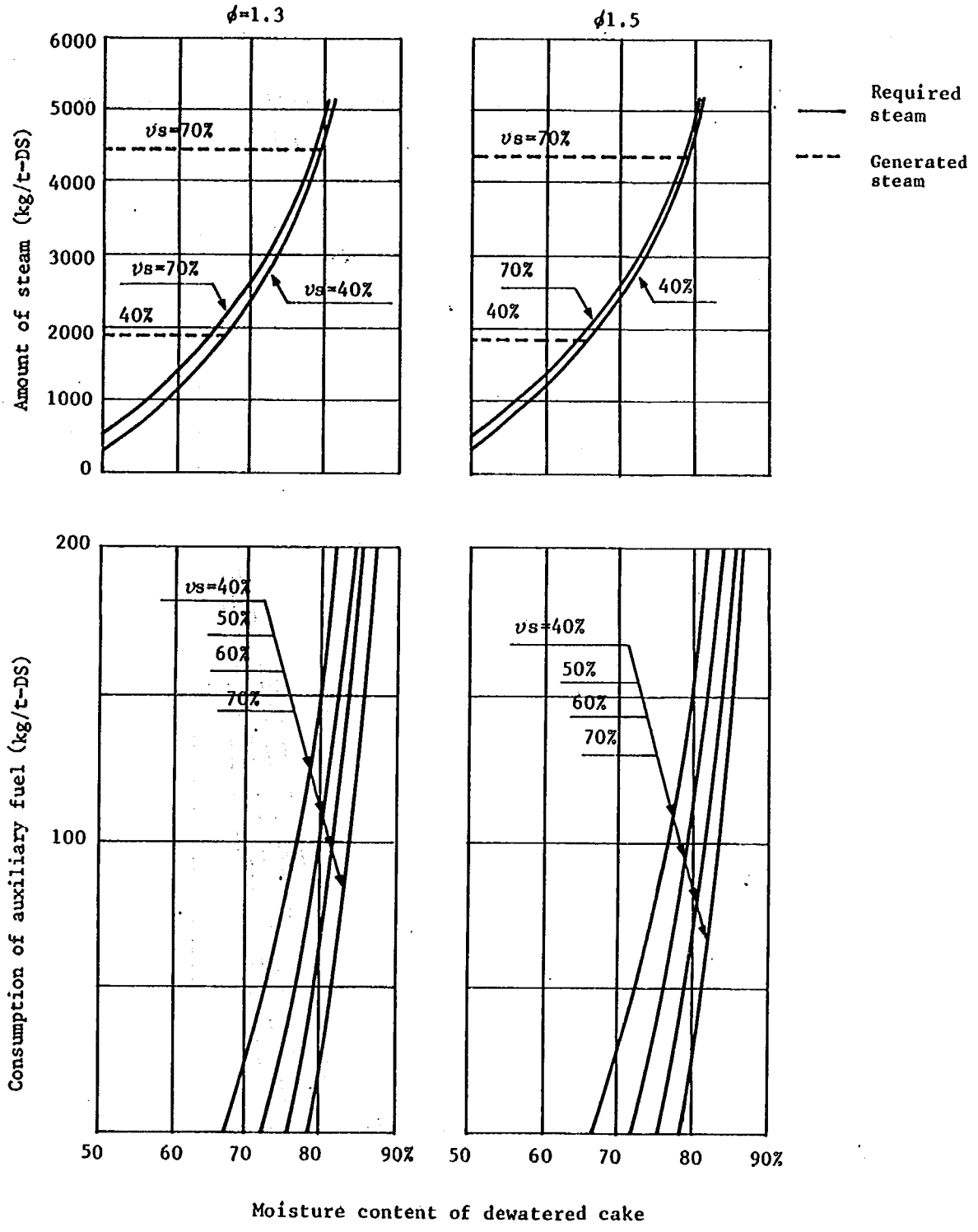


Fig. 6 Moisture content of cake and consumption of auxiliary fuel, required steam and amount of steam generated (Model I)

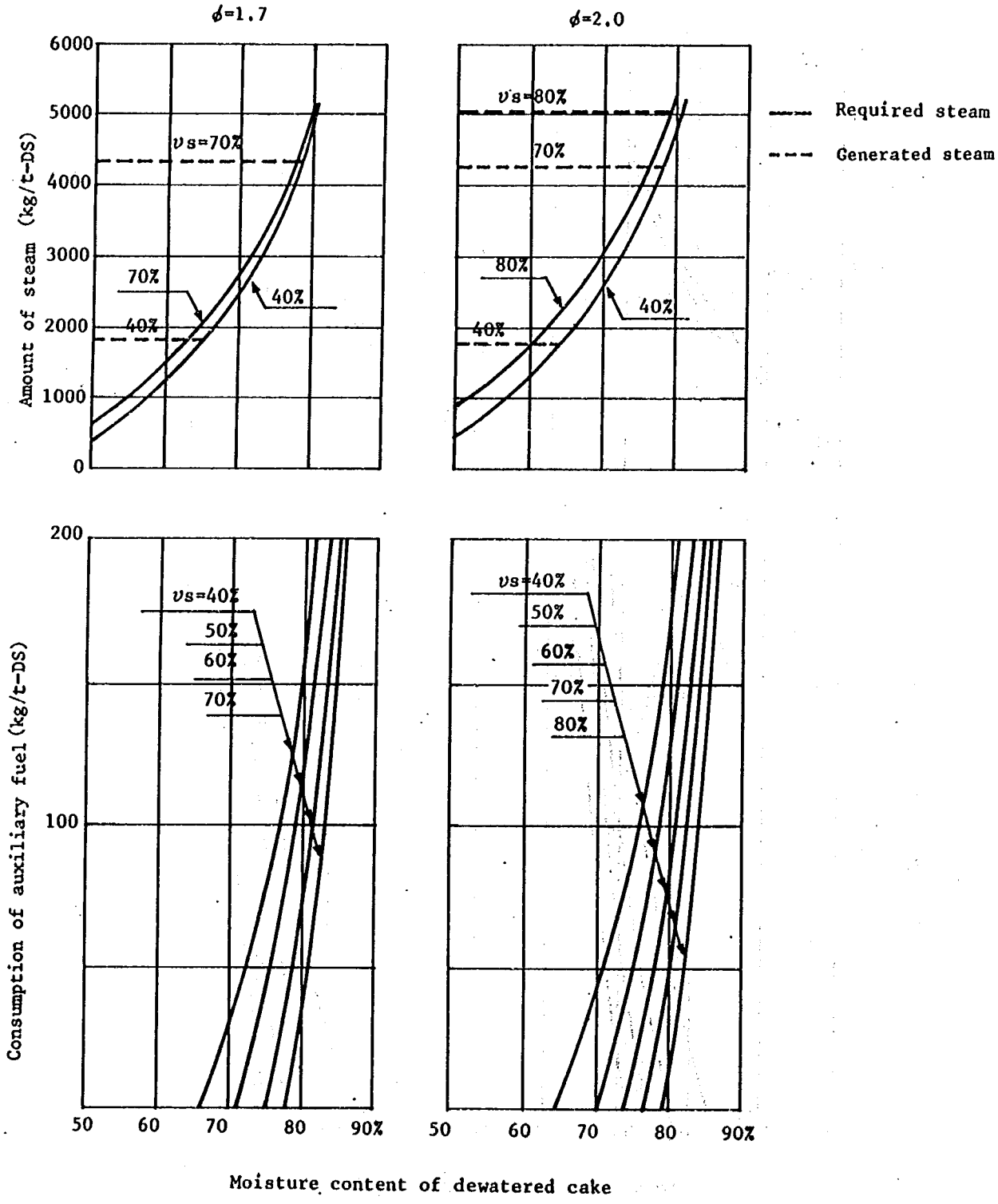


Fig. 6 - continued

(2) The volume of steam generated by heat recovery under the conditions described in (1) above.

(3) The total volume of steam required for the system under the conditions described in (1) above.

The current cake moisture content for the heat treatment method is around 50% and the corresponding auxiliary fuel consumption is shown in Figure 7. In the case of Model 2, however, the concentration of raw thickened sludge corresponding to the cake moisture content is used.

1.4.1 Model 1

When the cake moisture content is lower than the point on the abscissa axis where the auxiliary fuel is zero in Figure 8, the volume of steam required is lower than the generated steam volume achieved by autogenous combustion. If the cake moisture content is above that point, however, autogenous combustion of the cake in the furnace is possible. As deficiencies in the calorific value in the total system occur, however, auxiliary fuel is required at the waste heat boiler inlet (auxiliary fuel 2 in Figure 2).

For example, with an air ratio of (ϕ_S) of 1.3 and a combustible content ratio in the DS of 70%, the generated steam volume exceeds the volume of steam required (shown by the solid line), as some 4,400 kg/T-DS of steam generated with a cake moisture content of 78% or less. With an air ratio of 1.7 or less and a combustible content ratio of 80%, it is possible to have autogenous combustion with a moisture content of 80% or less. Generally speaking, the volume of auxiliary fuel required increases by some 3 kg in accordance with the increase in the air ratio (ϕ_S) by 0.1. In the high moisture content region, a small change in the cake moisture content region,

a small change in the cake moisture content largely affects the consumption volume of the auxiliary fuel.

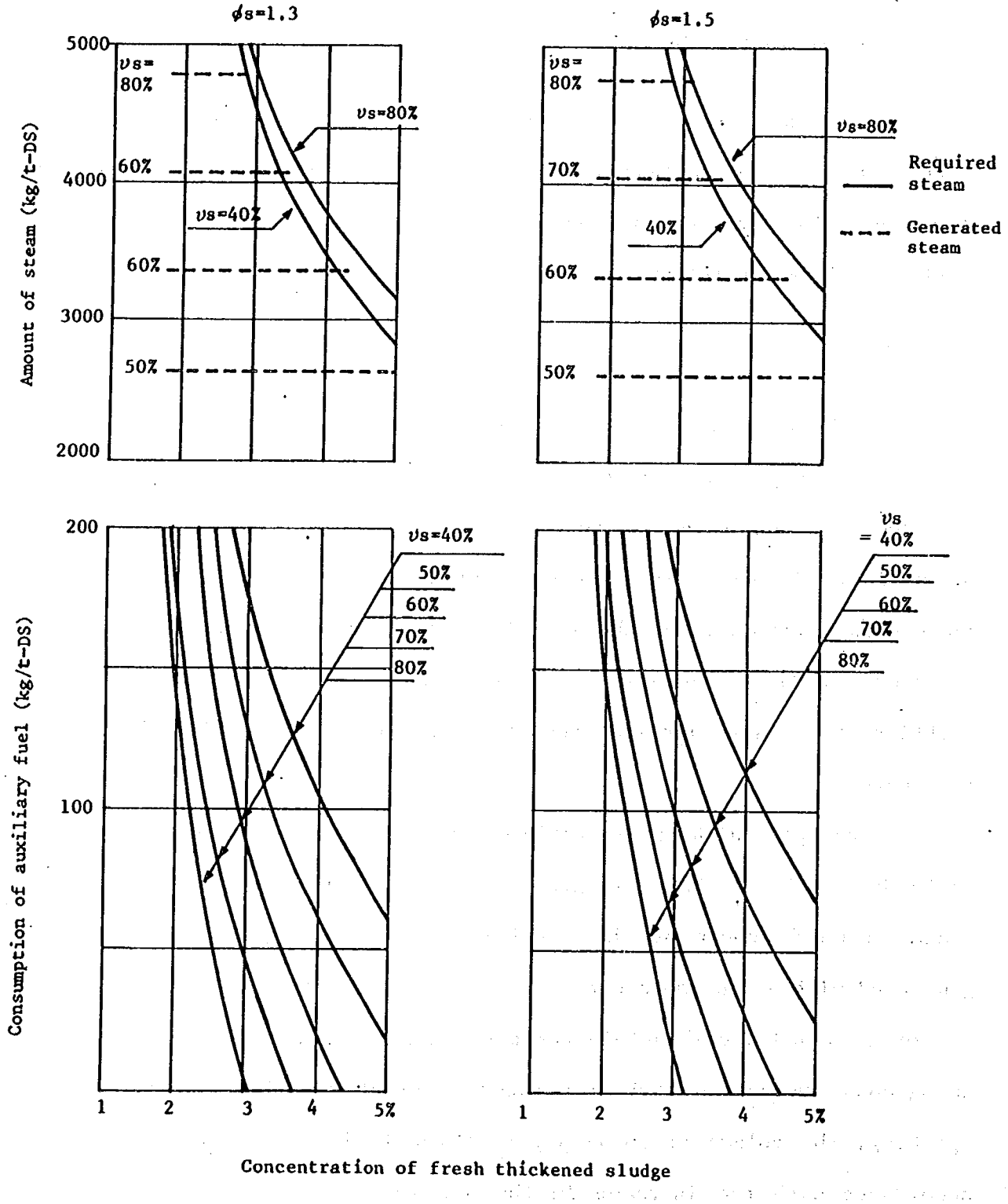


Fig. 7 Concentration of fresh thickened sludge and consumption of auxiliary fuel, required steam and amount of steam generated (Model II)

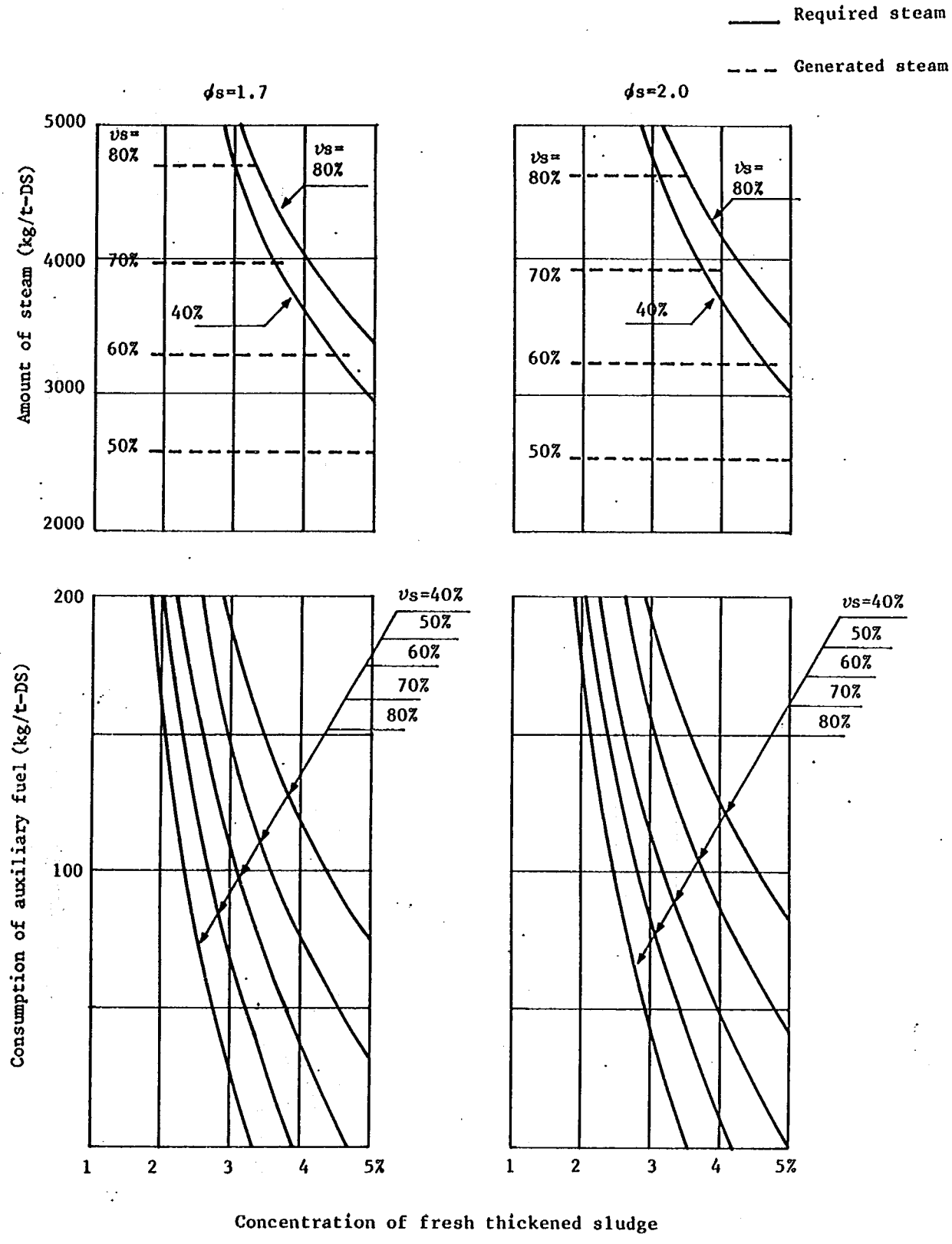
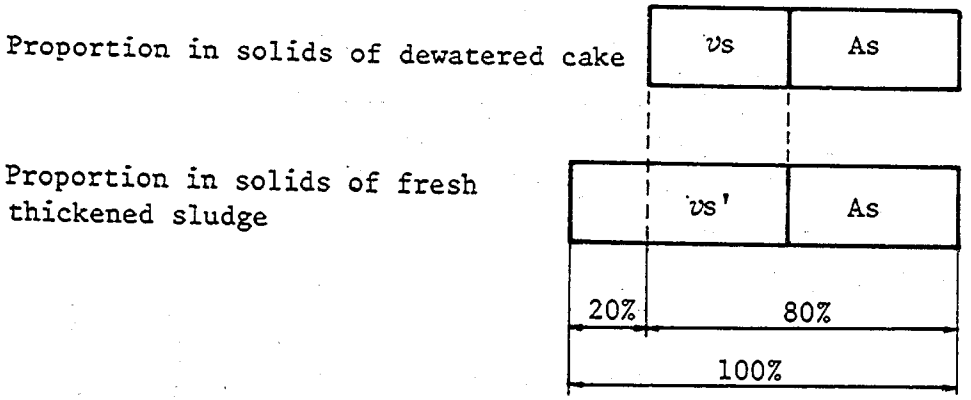


Fig. 7 - continued



vs : Proportion of volatile solids
vs': Proportion of volatile solids before heat treatment
As : Proportion of ashes

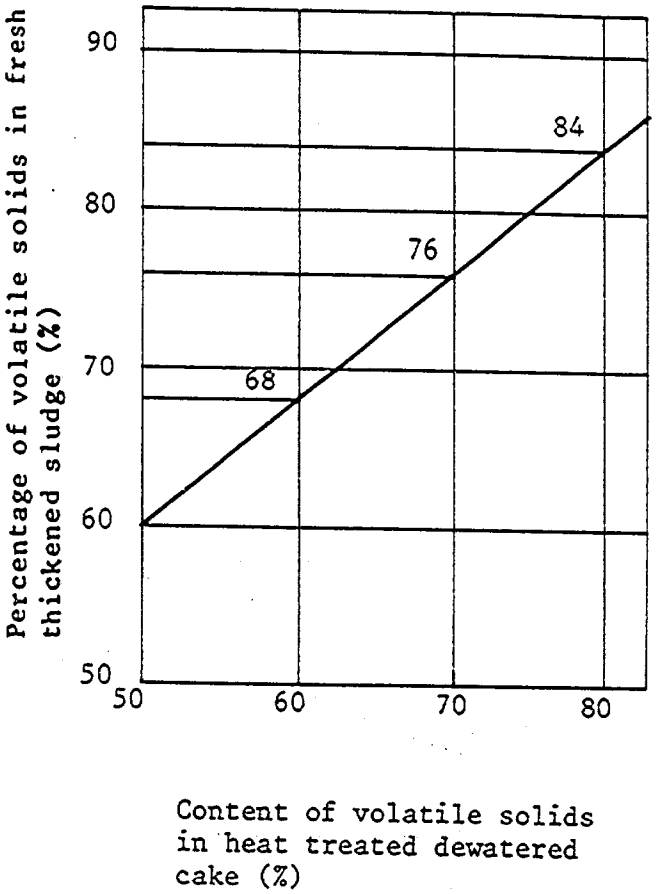


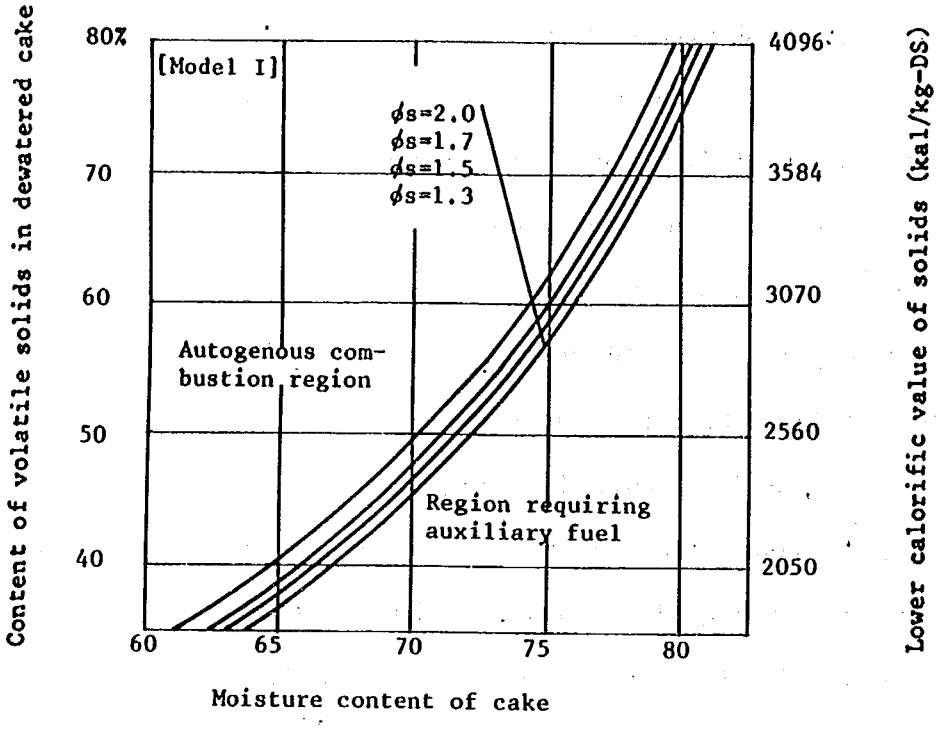
Fig. 8 Change in volatile solids due to heat treatment

1.4.2 Model 2

Taking into consideration the elution of organic matter into fluid during heat treatment, if the ratio of organic matter in the raw sludge is used to examine the heat balance on the same grounds as Model 1 instead of the ratio of the solid combustible content in dewatered cake as employed by the present model, the corrected ratio in relation to the ratio of organic matter shown in Figure 7 should be used. If 20% elution on a solid basis occurs during the heat treatment process, this is converted to an organic matter basis as shown in Figure 8. Here, the ash content, which is inorganic matter, is considered to be unaffected by the heat treatment. As shown in Figure 8, an organic content of 70% in the solid content of heat treated dewatered cake is equivalent to an organic content of 76% in the solid content of raw sludge.

Incineration operation in the furnace using the heat treatment system is currently planned on the basis of an air ratio of around 1.7. Under this condition, therefore, if the combustible content of the heat treated dewatered cake is 60% (the ratio of organic matter in the solid content of raw sludge is some 68%), a heat balance requiring no auxiliary fuel will be achieved with a concentration of raw thickened sludge of 4 - 5%. This means that if a 4 - 5% concentration of raw thickened sludge is achieved by mechanical thickening or other measures, a self-supply of energy can be obtained with the calorific value inherent in the sludge. The present model shows that the concentration of sludge plays an important role in energy saving.

Figure 9 shows the demarcation line where no auxiliary fuel is required for autogenous combustion for Models 1 and 2 based on equations (9) and (19).



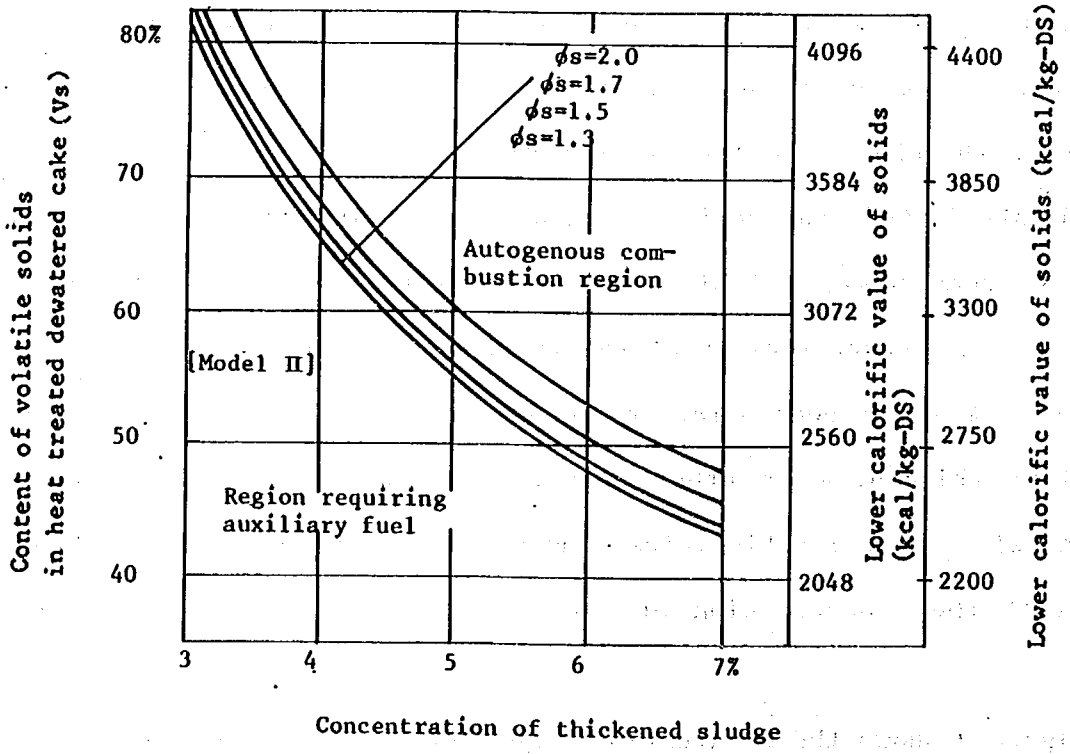


Fig. 9 Autogenous combustion regions

1.5 Evaluation of Existing Stoker Furnaces

There are currently 11 stoker furnaces in operation, and 5 of these have data usable for evaluation. Table 5 shows the operation data of these furnaces.

Plant A is a facility where "steam dry and incineration" are employed using a mixture of pressurized dewatering and vacuum watering. When there is a shortage of drying steam, the required volume is supplied from an independent boiler.

Other plants combine the heat treatment system with cake moisture contents varying from 40% to 50%. The recovered steam is used as a heat source for the pretreatment process.

The required volume of steam, therefore, largely depends on the concentration of the sludge supplied to the heat treatment process. Even with sludge capable of autogenous combustion in the furnace, auxiliary fuel is required for the waste heat boiler if the concentration of the thickened sludge is low.

Figure 10 shows the data of Plant A plotted on the characteristic chart of Model 1 with an air ratio (ϕ_S) of 1.5. Figure 11 shows the data of Plants B, C, D and E plotted on to the similar chart for Model 2 with an air ratio of 1.7. Figure 12 shows the data from Table 5 plotted on the model chart of the auxiliary fuel consumption and generated steam volume with an air ratio ranging from 1.5 to 2.0 and a cake moisture content of 50%, using the ratio of organic matter as the parameter. When the relationship between the theoretical values of Model 1 and the operational values of Plant A in Figure 10 is examined, the measured values are found to be almost identical to the theoretical values except for data A-3. When data A-3 in Figure 12 are examined, the measured values are slightly better than the theoretical

Table 5 Operating data for existing stoker furnaces

				Plant "A" (100T/day)					Plant "B" (40 T/day)			Plant "C" (30 T/day)			Plant "D" (30 T/day)		Plant "E" (30 T/day)	
System				System I					System II			System II			System II		System II	
Dewatering unit				Filter press and vacuum filter					Filter press			Filter press			Filter press		Filter press	
Planned incineration capacity (kg/hr)				4160					1600			1250 (W=48%)			1250		1250 (w=50%)	
Run				A-1	A-2	A-3	A-4	A-5	B-1	B-2	B-3	C-1	C-2	C-3	D-1	D-2	E-1	E-2
Properties of cake	Moisture content of cake		%	67.3	71.2	73.4	71.0	75.4	42.3	-	-	41.2	48.0	50.6	47.2	46.0	-	-
	Ratio of volatile solids		%	50.3	51.5	51.9	46.8	46.4	63.0	-	-	-	-	61.1	49.5	37.0	-	-
	Heating value	Higher heating value	Kcal/kg	5210	5135	5195	5020	5150	6542	-	-	6470		6380	6000	6290	5946	6370
		Lower heating value	Kcal/kg	4880	4990	5080	-	-	6120	-	-	5970		-	5490	5850	-	-
		Fuel value of cake	Kcal/kg	400	310	260	-	-	1970	-	-	1740		-	1150	830	987	1440
	Element composition	Carbon (C)	%	51.51	47.5	48.9	-	-	55.8	-	-	61.9	62.6	-	60.9	-	61.0	-
		Hydrogen (H)	%	6.16	2.7	2.2	-	-	7.7	-	-	9.2	9.35	-	8.75	-	9.7	-
		Oxygen (O)	%	36.76	43.8	42.9	-	-	29.7	-	-	23.3	22.36	-	25.5	-	20.8	-
		Nigrogen (N)	%	4.59	5.0	4.7	-	-	4.9	-	-	4.1	4.2	-	3.12	-	7.1	-
		Sulfur (S)	%	0.78	1.0	1.1	-	-	1.9	-	-	1.5	1.49	-	1.53	-	1.4	-
Operating conditions	Air surplus ratio		-	1.28	1.41	1.36	-	-	1.5	1.2	1.5	-	2.3	2.2	2.1	2.2	1.97	1.87
	Incineration temperature		°C	920	880	893	-	-	900 ~ 910	960 ~ 1040	810 ~ 1030	910	810	750 ~ 800	835	820	836	917
	Gas temperature at boiler outlet		°C	228	224	252	-	-	244	270	-	285	270	230 ~ 270	231	237	218	222
	Unburned substances in ashes		%	0.25 ~ 1.08			0.63	-	1.7 ~ 3.9	0.7	3.6	5.7		8.9	3.5	4.0	-	-
	Steam generated	Pressure	kg/cm^2	11.1	11.2	10.3	11.0	10.5	13.0 ~ 13.5			18.0	18.0	17.0	18.0	18.3	17.5	18.3
		Amount of steam	kg/hr	3260	3230	5070	3190	3990	3410	5360	4064	3600	2733	2200	2390	2420	2554	3844
	Consumption of auxiliary fuel		kg/hr	0	0	134	0	71	0	55	0	12	153	104	60	91	78.2	95.2
	Burned gas	Wet gas standard	m^3/hr	7330	6460	7535	-	-	7760	9600	-	-	-	-	-	-	7040	6280
		Dry gas standard	m^3/hr	6000	4800	5860	-	-	6570	7554	-	7770	-	-	-	-	-	-
	Concentration of heat treated sludge		%	-	-	-	-	-	3.0	3.0	3.0	2.9	2.5	2.0	4.2	4.2	3.7	3.7
	Amount of incineration		kg/hr	4180	3886	4050	4563	4538	1627	2280	1941	1425	975	605	1121	1340	1024	1275
	Amount of steam generated		kg/ ton-DS	2388	2886	4703	2415	3575	2910	3260	2903	2497	4310	5882	3230	2675	4300	4560
	Consumption of auxiliary fuel		kg/ ton-DS	0	0	124	0	63.6	0	33.4	0	11.5	240	278	81	100	132	113

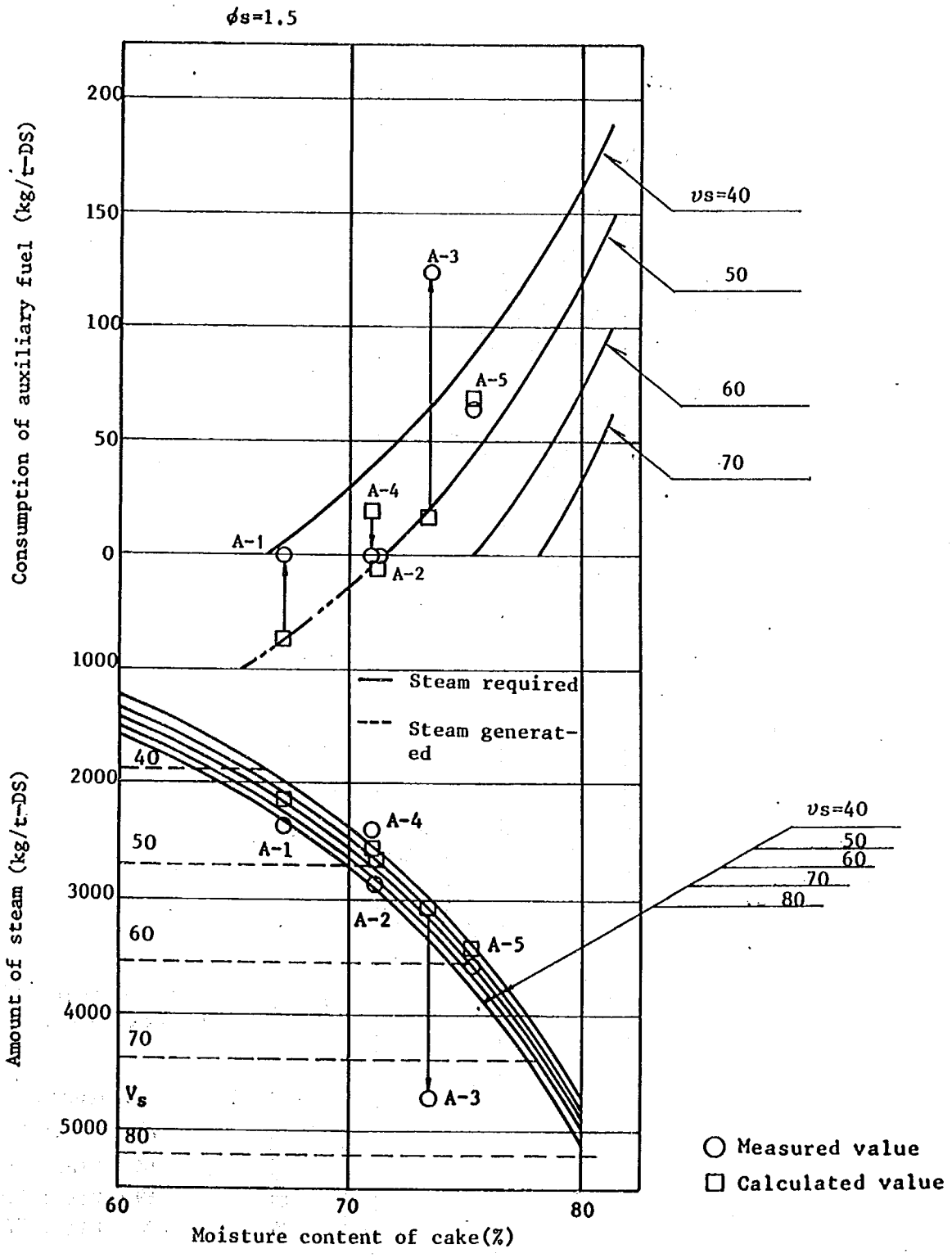


Fig. 10 Operating data of Model I (plant "A")

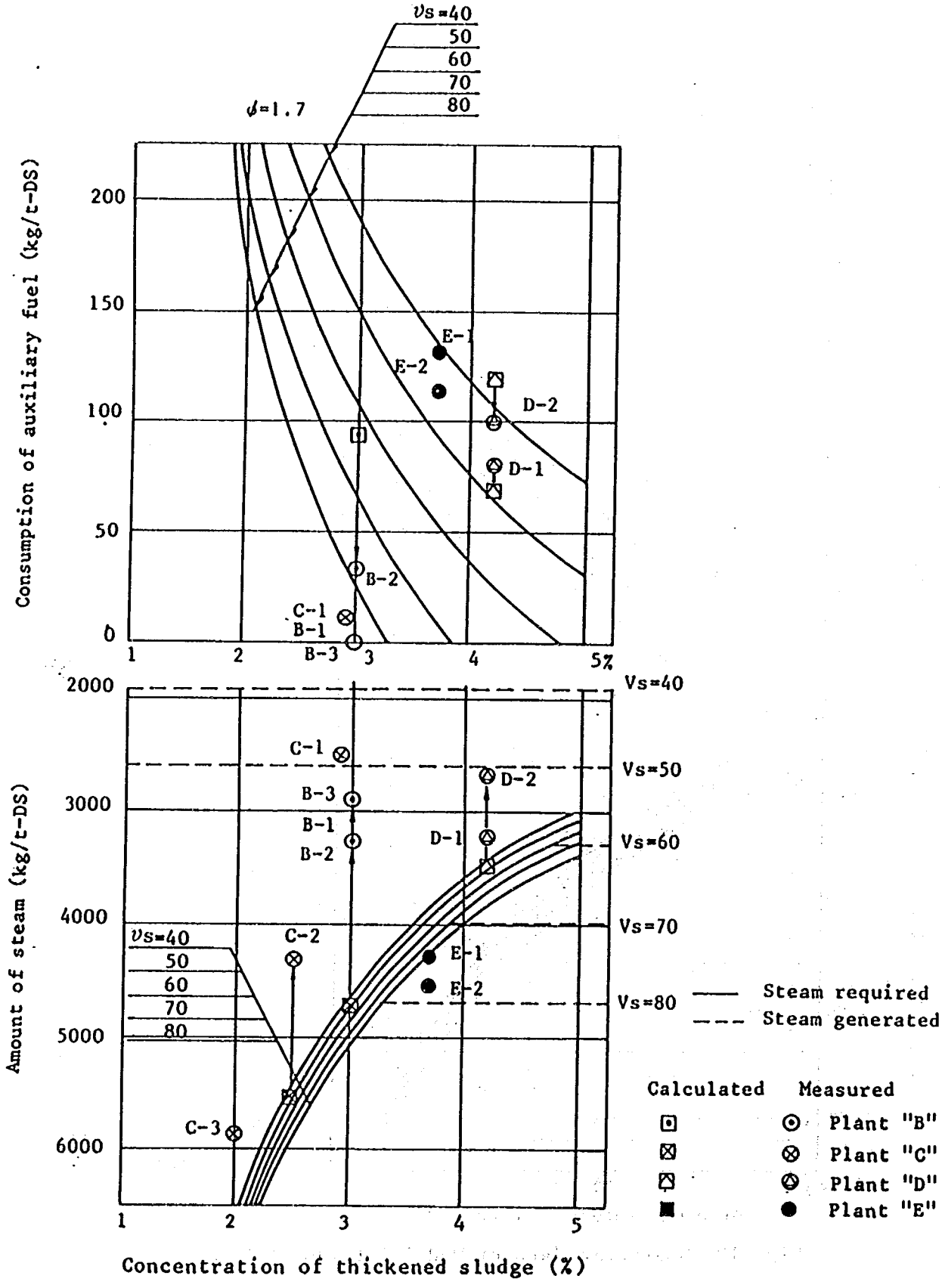


Fig. 11 Operating data of Model II (plants "B", "C", "D" and "E")

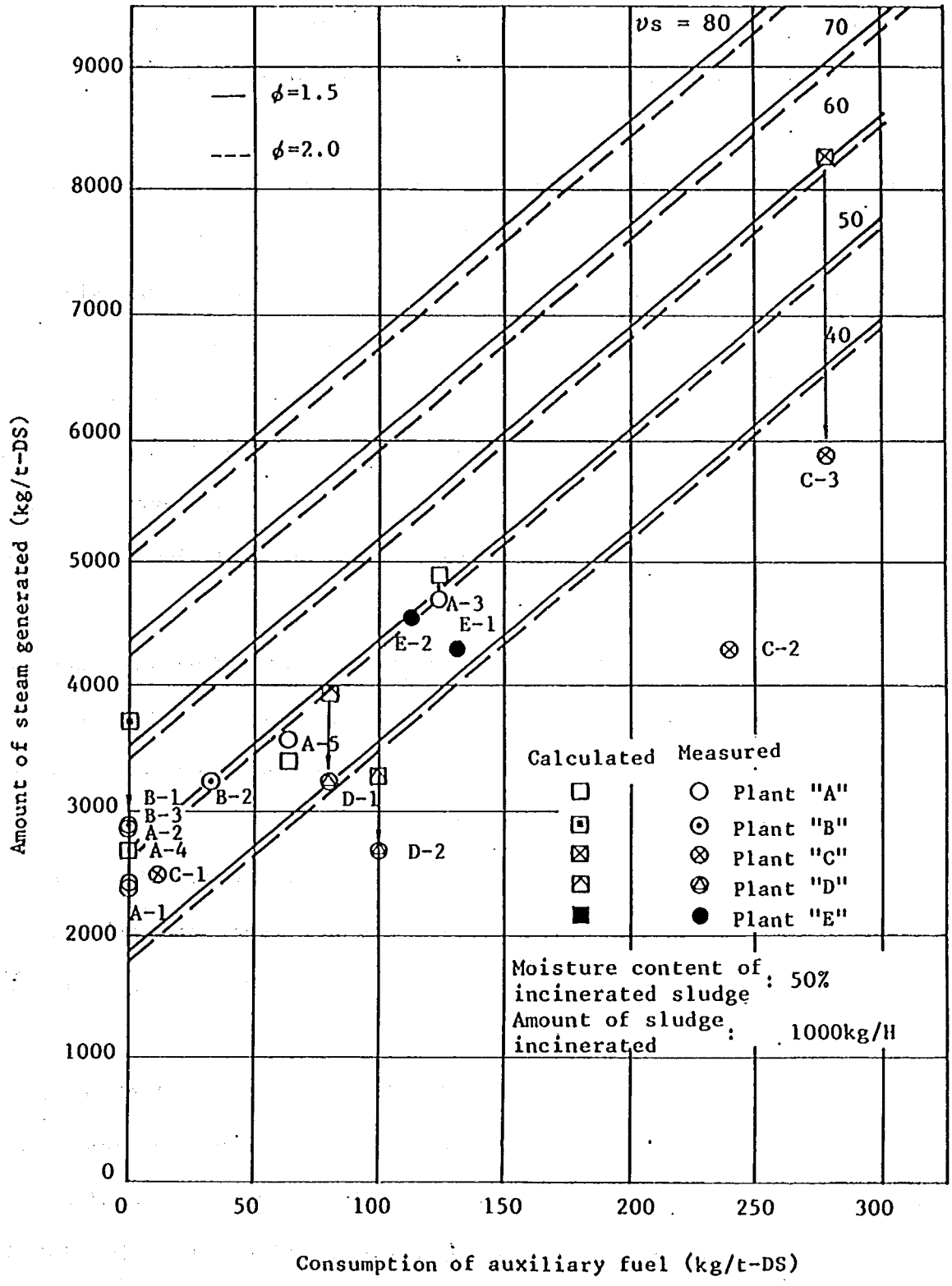


Fig. 12 Amount of auxiliary fuel consumption

values. In view of other data, however, these A-3 values can be considered to be within the tolerances of the model. Plant A's good performance presumably results from the low moisture content of the dry sludge which is approximately 40%.

In the case of Plant B, there is a problem of uncoordinated operation between the incineration process and the heat treatment process. Therefore, a comparison of its values in terms of the concentration of raw sludge with the theoretical values is inappropriate. (Despite the measured values being much higher than the theoretical values in the case of fuel consumption, the measured value of generated steam is much lower than the theoretical value.) As shown in Figure 12, the operational values appear to correspond to the some 50% organic matter content in spite of the fact that the actual organic content is over 60%.

In regard to Plant D, while the concentration of thickened sludge is as high as 4.2%, the actual operation is less sufficient than the theoretical values suggest due to the low ratio of the combustible content of 37 - 50% and, therefore, the high air ratio of 2.1 - 2.2.

In the case of Plant E, coordinated operation between the heat treatment process and the incineration process has not been achieved due to the small volume of sludge generated, which is turn due to the recent commencement of operation.

Although evaluation of Plant E is difficult due to the lack of measured values for the combustible content which is comparable to the present model, it can be assumed that the ratio of combustible content is not high in view of its data being taken from actual operation. (The extremely high lower calorific value of the cake in E-2 is caused by the low cake moisture content.) Therefore, the actual values of Plant E are considered to be almost the same to the theoretical values.

Based on the above, the theoretical values of Model 1 and the operation data are similar to one another, and so Model 1 is considered to be applicable. In the case of the relationship between the theoretical values of Model 2 and the operation data, the theoretical data appear to be reasonably approximate to the operation data except for in the case of Plant C.

1.6 Evaluation of Energy Saving Based on Two Models

The possibility of energy saving is considered below based on the present models and the operation data.

(1) Model 1 (Coagulant - Dewatering - Incineration)

1) In the high cake moisture content region, changes in the moisture content largely affect the auxiliary fuel consumption. Therefore, the lowering of the cake moisture content makes a substantial contribution to energy saving.

2) The increase in the air ratio of 0.1 results in an increase in the fuel consumption of 2 - 3 kg per one ton of treated solid. The lowering of the air ratio from the conventional 2.0 to 1.5 or under, achieves a reduction in the auxiliary fuel consumption of 10 - 15 kg per one ton of treated solid.

3) Heat recovery by the heat exchange method using steam has wider applications than the gas-to-gas heat exchange method.

(2) Model 2 (Heat Treatment - Incineration)

Lowering of the current dewatered cake moisture content cannot be expected in the heat treatment process. Therefore, the increase in the concentration of thickened sludge saves energy by reducing the steam consumption during the heat treatment process.

2. EVALUATION OF FLUIDIZED BED FURNACE FOR ENERGY CONSERVATION

Technical reviews have being carried forward for energy conserving the fluidized bed incineration systems in recent years. A typical system is the drying-fluidized bed incinerator system, for which heat is recovered from exhaust gases of high temperature and is used not only for heating combustion air but also for drying dewatered cake before it is input into the furnace. Energy saving effects will be reviewed based on mass balance and heat balance for the typical fluidized bed incineration systems which are recently proposed or put to practical use.

2.1 Fluidized Bed Furnaces Subject to Examination

Five types of fluidized bed furnaces (R-1 - R-5) were considered in the preparation of the heat balance model, and their rough flow charts are shown in Figure 13 to Figure 16. The characteristics of each model are determined by the heat recovery method from the exhaust gas at the furnace outlet, the degree of heat recovery and the use of the recovered heat. In terms of the facilities used, these five models differ in their combinations of heat exchangers, waste heat boilers, sludge dryers, etc.

The characteristics of the assumed facilities for each model are described below.

(1) R-1 Model (Conventional Type)

The heat of the exhaust gas at the furnace outlet is partially recovered by the heat exchanger to preheat the combustion air to 400 - 500°C.

(2) R-2 Model (High Temperature Heat Recovery Type)

While the flow is the same as for the conventional type, the recovery of heat by the heat exchanger is improved and the combustion air is preheated to some 600°C.

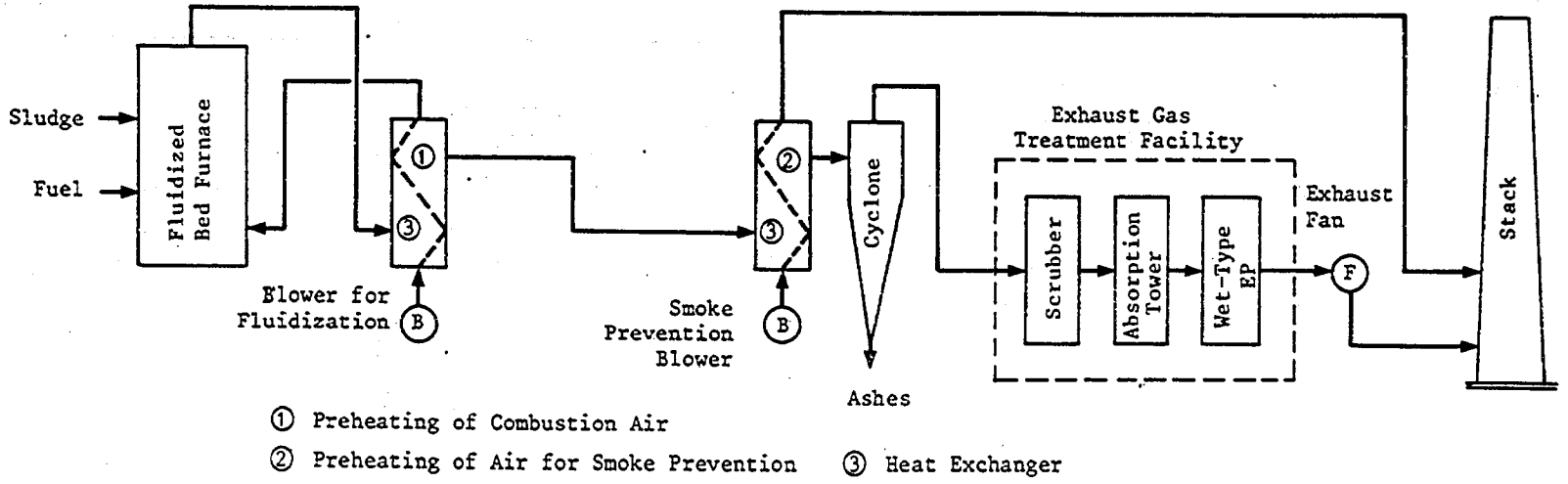


Fig. 13 Flow chart of conventional type (R-1) and high temperature heat recovery type (R-2) fluidized bed furnace models

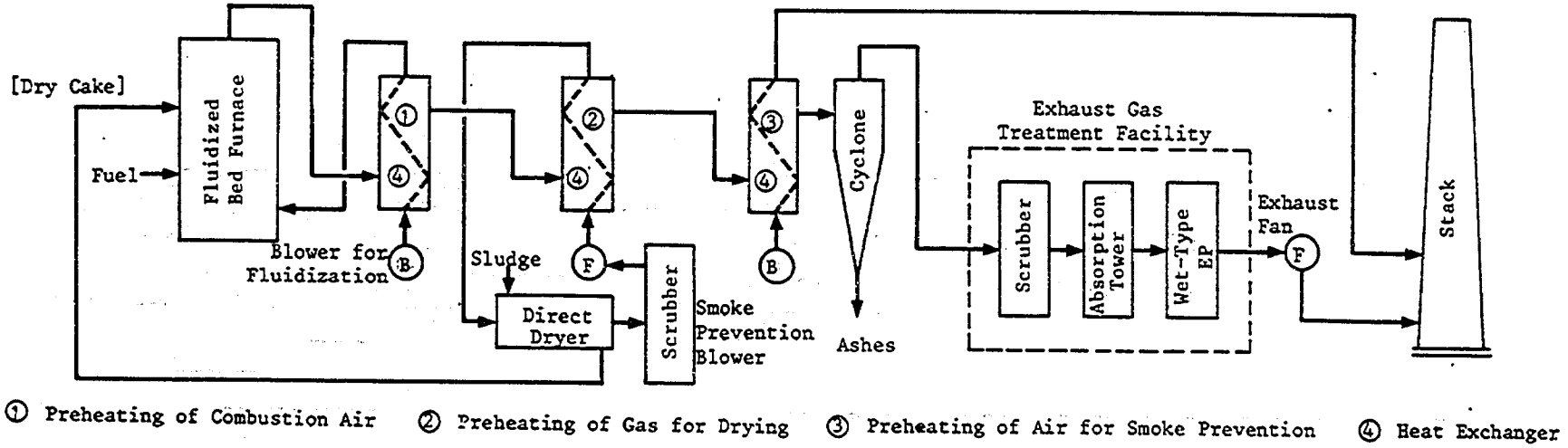


Fig. 14 Flow chart of preliminary drying with preheated direct hot air type (R-3) fluidized bed furnace model

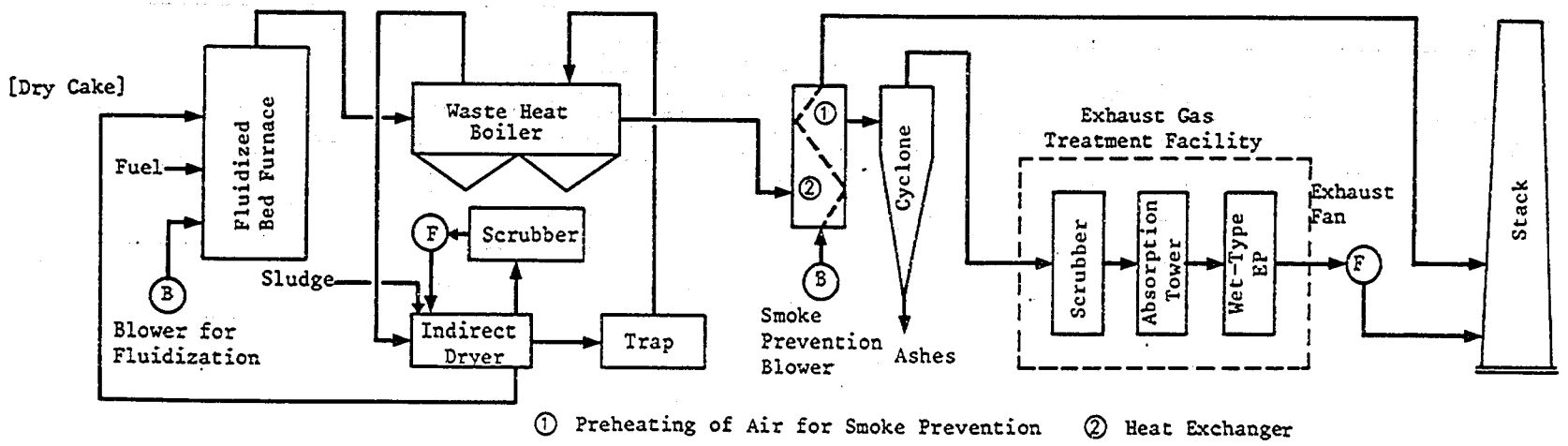
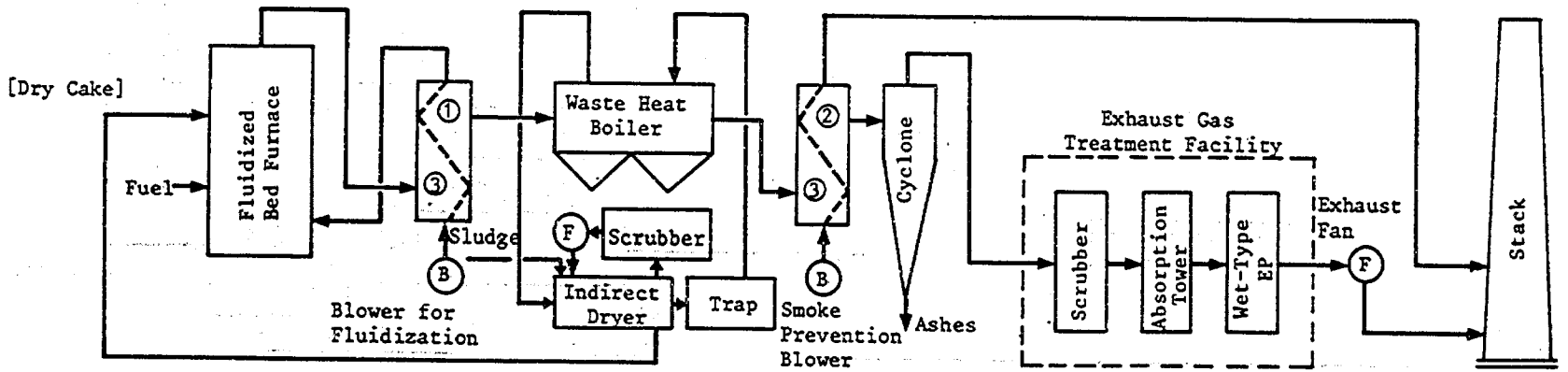


Fig. 15 Flow chart of indirect preliminary drying type (R-4) fluidized bed furnace model



① Preheating of Combustion Air ② Preheating of Air for Smoke Prevention

Fig. 16 Flow chart of indirect preliminary drying with preheating type (R-5) fluidized bed furnace model

(3) R-3 Model (Preliminary Drying with Preheated Direct Hot Air Type)

This model also recovers the heat from the exhaust gas at the furnace outlet and uses it for preliminary drying of the dewatered cake. The heating medium for drying the cake is gas, consisting mostly of air, and the drying facility employs the direct hot air method. The dry cake is thrown into the furnace.

(4) R-4 Model (Indirect Preliminary Drying Type)

In this model, the waste heat boiler is installed at the after stage of the furnace and part of the exhaust gas heat at the furnace outlet is recovered in the form of saturated steam. The latter is then used for the preliminary drying of the dewatered cake. The drying facility employs the indirect heating method using saturated steam at 7 kg/cm^2G as the heating medium. The dry cake is thrown into the furnace.

(5) R-5 Model (Indirect Preliminary Drying with Preheating Type)

The heat exchanger and the waste heat boiler are installed in that order at the after stage of the furnace in this model and the heat recovered from the exhaust gas at the furnace outlet is used for preheating the combustion air as well as for the preliminary drying of the dewatered cake. The facilities for heat recovery and heat utilization are similar to a combination of the corresponding facilities in the R-1 and R-4 models.

2.2 Conditions of Operation

The assumed values of the air and exhaust gas temperatures at various points in the R-1 to R-5 models and the heat loss at the heat exchanger and the waste heat boiler, etc. are given in Table 6 and Table 7.

The atmospheric conditions are as follows.

Temperature : 20°C

Air pressure : mb

Relative humdity: 70%

The heat balance is calculated using the higher calorific values based on 20°C.

Table 6 Set values for operation related items for each model

Item \ Model	R-1	R-2	R-3	R-4	R-5
Comprehensive air ratio in furnace (-)	1.3	1.3	1.3	1.3	1.3
Exhaust gas temperature at furnace outlet: T_1 (°C)	800	800	800	800	800
Preheated combustion air temperature: T_2 (°C)	500 or below	650 or below	530 or below	(20)	400 or below
Exhaust gas temperature at heat exchanger outlet for preheating combustion air: T_3 (°C)	To be calculated	To be calculated	To be calculated	(800)	To be calculated
Exhaust gas temperature at either waste heat boiler outlet or heat exchanger outlet for preheating heat medium: T_4 (°C)	-	-	250 or below	250 or below	250 or below
Heat medium temperature at dryer inlet: T_5 (°C)	-	-	520	180	180
Exhaust gas temperature at dryer outlet: T_6 (°C)	-	-	180	110	110
Dry cake temperature at dryer outlet: T_7 (°C)	-	-	80	80	80

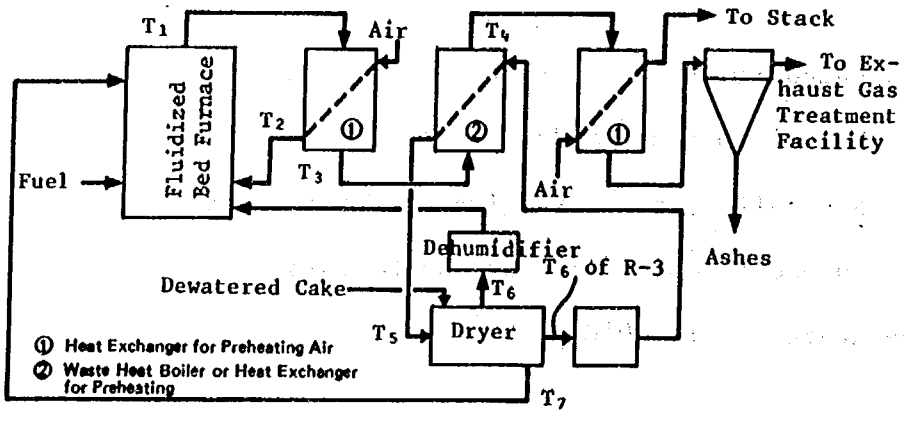


Table 7 Set values for heat losses and thermal efficiencies of heat recovery and heat utilization units

Item		Set value
Heat loss ratio at furnace:	η_1 (%)	30t/day: 9.5, 50t/day: 7.9 100t/day: 6.2
Heat loss ratio at heat exchanger for preheating combustion air:	η_2 (%)	10% of exchanged calorific value
Heat loss ratio at heat exchanger for preheating drying gas:	η_3 (%)	10% of exchanged calorific value
Heat loss ratio at waste heat boiler:	η_4 (%)	10% of exchanged calorific value
Heat efficiency of direct dryer:	β_1 (%)	80%*
Heat efficiency of indirect dryer:	β_2 (%)	90%*

* Definition of heat efficiency of dryer (ß)

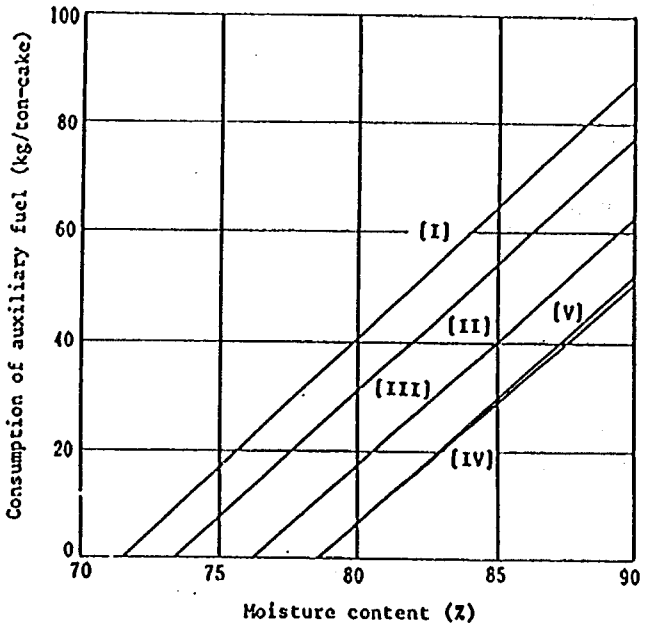
$$\beta = \frac{\text{Sensible heat of dry cake} + \text{Sensible and latent heat of evaporated water}}{\text{Heat input to dryer} - \text{Heat output from dryer}} \times 100\ (\%)$$

2.3 Evaluation of Fluidized Bed Incineration Systems

Explained below are the results of sensitivity analysis of various parameters based on the calculations of mass and heat balances for the consumption of auxiliary fuel by the fluidized bed incineration systems.

(1) Moisture content of cake and the consumption of auxiliary fuel per ton of cake

Shown in Figure 17 is the relationship between the moisture content of cake and the consumption of auxiliary fuel in the furnace under the condition of air ratio of 1.3, furnace capacity of 50 tons/day and the content of volatile solids Vs, of 70% of input cake. A decrease in the consumption of fuel as a result of 1% decrease in the moisture content of cake is shown in Table 8.



Furnace capacity: 50 ton/day
Volatile solids contents: 70%

Fig. 17 Moisture content of cake versus consumption of auxiliary fuel

In each model, the consumption of auxiliary fuel decreases as the moisture content of cake becomes smaller. This decreasing tendency is the same in all the systems.

When the moisture content is the same, the Run 1 has the highest consumption of auxiliary fuel, followed by Run 2, Run 3, Run 5 and Run 4. Difference in the consumption of auxiliary fuel between Run 4 and 5 at the moisture content of 80% is small or 0.7 kg/ton-cake Proportion of the consumption of auxiliary fuel in each model to the consumption of auxiliary fuel by Run 1 at moisture content of 80% is shown in Table 9.

Table 8 Decrease in auxiliary fuel due to 1% decrease in moisture content

(Unit kg/ton-cake)

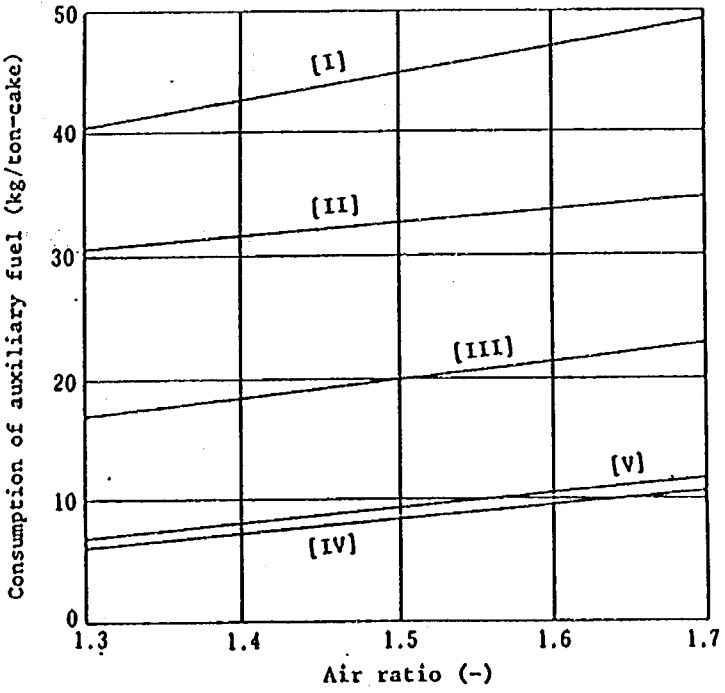
System / Item	I	II	III	IV	V
Decrease in auxiliary fuel	4.8	4.7	4.6	4.5	4.5

Table 9 Consumption of auxiliary fuel compared to 1.0 as standard of Model I

Item · System	I	II	III	IV	V
Consumption of auxiliary fuel	1.00	0.75	0.42	0.15	0.17

(2) Air ratio and consumption of auxiliary fuel per ton of cake

Shown in Figure 18 is the relationship between the air ratio and consumption of auxiliary fuel in the incinerator under the conditions where the moisture content of cake is 80%, content of volatile solids of cake is 70%,



Furnace capacity: 50 ton/day
Moisture content: 80%
Volatile solids content: 70%

Fig. 18 Air ratio versus consumption of auxiliary fuel

and incinerator capacity is 50 tons/day. Shown in Table 10 is the relationship between the increase in air ratio and the corresponding increase in the consumption of auxiliary fuel, and Table 11 shows the difference in the consumption of auxiliary fuel between the models as the air ratio becomes larger.

Table 10 Consumption of auxiliary fuel and air ratio

(Unit: kg/ton-cake)

Air ratio / System	1.3∿1.4	1.4∿1.5	1.5∿1.6	1.6∿1.7
[I]	2.1	2.2	2.2	2.4
[II]	1.1	0.9	1.1	1.1
[III]	1.4	1.4	1.5	1.5
[IV]	1.1	1.2	1.1	1.2
[V]	1.2	1.2	1.3	1.2

Table 11 Difference in consumption of auxiliary fuel between Model I and other systems

(Unit: kg/ton-cake)

Air ratio / System	1.3	1.4	1.5	1.6	1.7
[I]	0	0	0	0	0
[II]	10.0	11.0	12.3	13.4	14.7
[III]	23.5	24.2	25.0	25.7	26.6
[IV]	34.5	35.5	36.5	37.6	38.8
[V]	33.7	34.6	35.6	36.5	37.7

The consumption of auxiliary fuel per ton of cake increases as the air ratio becomes larger, and this increase is almost constant in each model regardless of the value of air ratio. Also, the difference in the consumption of auxiliary fuel between the models becomes larger as the air ratio increases.

The difference in the consumption of auxiliary fuel between Run 4 and 5 is considerably smaller compared to the difference from other models, but the tendency is the same as that of other models in which the consumption of auxiliary fuel increases as the air ratio becomes larger.

(3) Content of bolatile solids and the consumption of auxiliary fuel per ton of cake

Shown in Figure 19 is the relationship between the content of volatile solids, and the consumption of auxiliary fuel under conditions where the moisture content of cake is 80%, furnace capacity is 50 tons/day and air ratio is 1.3. Also, Table 12 shows the relationship between an increase of

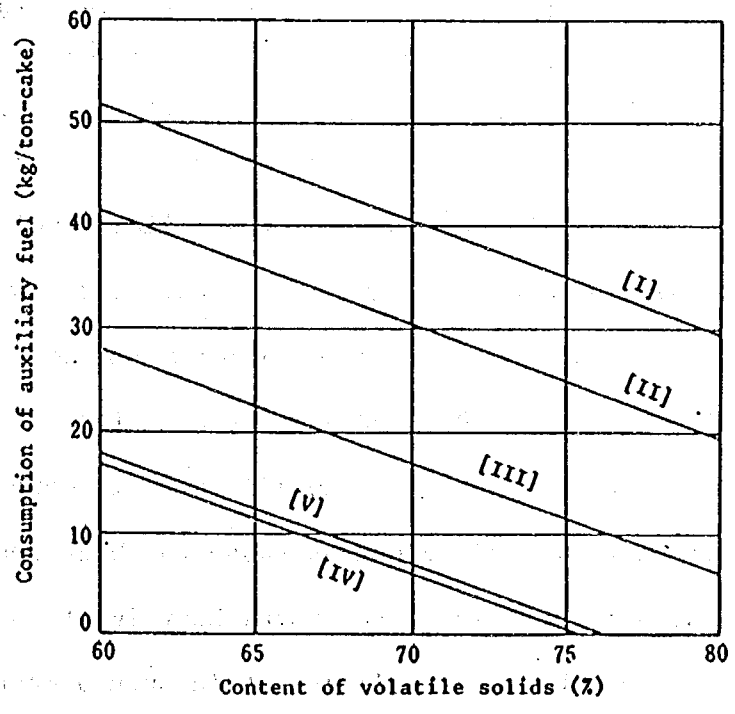


Fig. 19 Content of volatile solids and consumption of auxiliary fuel

1% in the content of volatile solids and the corresponding decrease in the consumption of auxiliary fuel.

The consumption of auxiliary fuel per ton of cake decreases as the content of volatile solids becomes larger. This tendency is also the same in each model, and the decrease in the consumption of auxiliary fuel is almost the same in each model when the content of volatile solids increases by 1%. When comparing the effect of the moisture content and the content of volatile solids on the consumption of auxiliary fuel, the decrease in auxiliary fuel is 4.5 to 4.8 kg/ton when the moisture content is decreased by 1%. However, the decrease in auxiliary fuel is 1.1 kg/ton when the content of volatile solids increases by 1% (refer to Table 8), so that it is more affected by the moisture content.

Table 12 Decrease in auxiliary fuel due to 1% increase in content of volatile solids

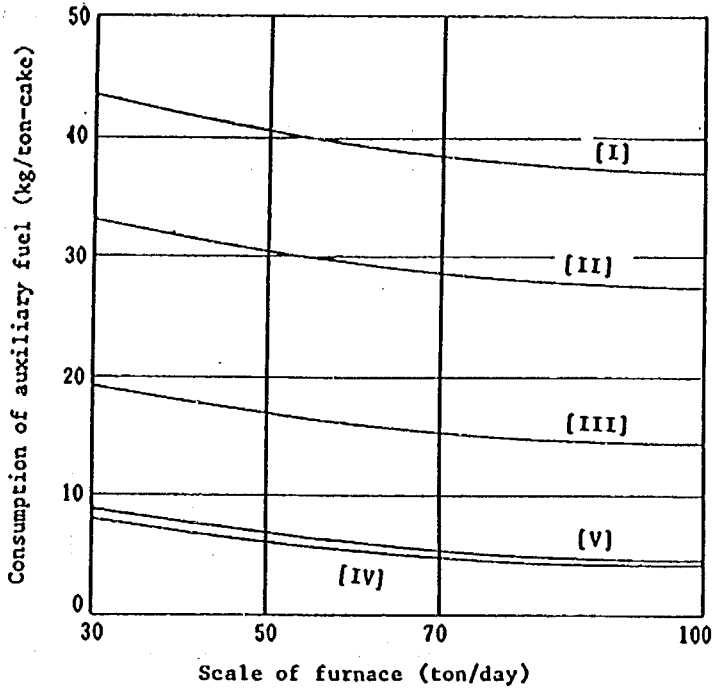
(Unit: Kg/ton-cake)

Item \ System	I	II	III	IV	V
Decrease in auxiliary fuel	1.1	1.1	1.1	1.1	1.1

(4) Scale of furnace and the consumption of auxiliary fuel per ton of cake

Figure 20 shows the relationship between the scale of furnace and the consumption of auxiliary fuel when the moisture content of cake is 80%, content of volatile solids is 70%, and air ratio is 1.3.

In each model, the consumption of auxiliary fuel decreases as the scale of furnace becomes larger. This occurs because the heat loss in the body of furnace is calculated as a ratio to the total heat input in the heat balance calculations. That is, the loss ratio is set in such a manner that the loss ratio decreases as the scale of furnace becomes larger. Loss ratio is shown in Table 13.



Moisture content: 80%
Volatile solids content: 70%
Air ratio: 1.3

Fig. 20 Scale of furnace and consumption of auxiliary fuel

Table 13 Set values of heat loss ratio of furnace

Scale of furnace (t/day)	30	50	70	100
Loss ratio (%)	9.5	7.9	7.0	6.2

The decrease in auxiliary fuel with an increase in the scale of furnace is shown in Table 14.

The decrease in auxiliary fuel with an increase in the scale of furnace is the largest in Run 1 and the smallest in Run 4. In Run 4, the consumption of auxiliary fuel is small and thus total heat input in the furnace is small, so that this model is less affected by the increase in the scale of furnace.

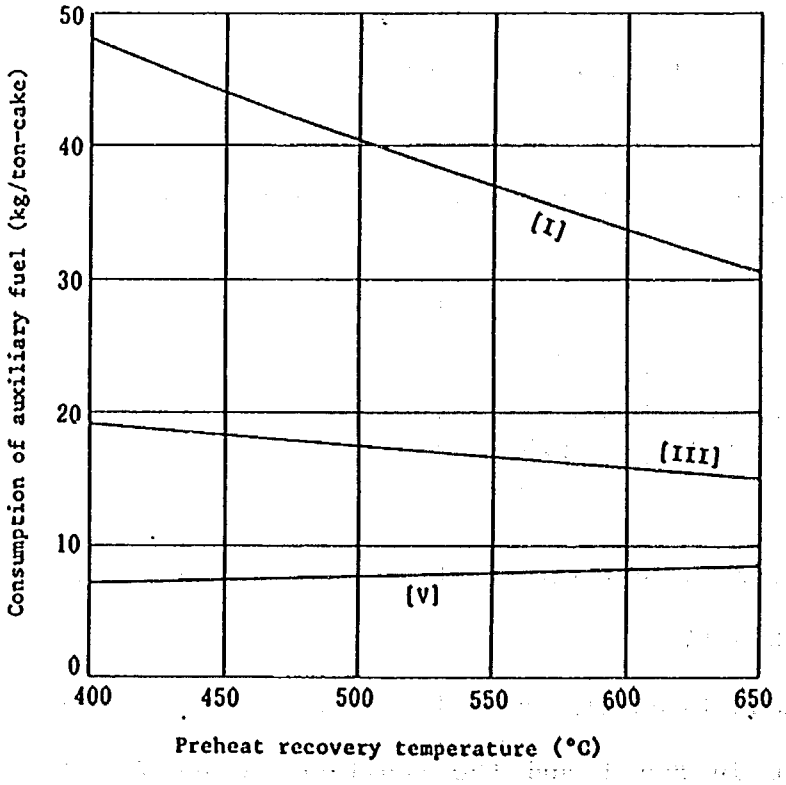
Table 14 Decrease in fuel for each scale of furnace compared with 30 ton/day furnace

(Kg/t)

Scale of furnace \ System	I	II	III	IV	V
30 (t/day)	-	-	-	-	-
50 (t/day)	3.4	3.0	2.7	2.0	2.3
70 (t/day)	5.2	4.6	4.1	3.4	3.5
100 (t/day)	6.8	5.9	5.3	4.3	4.5

(5) Preheat recovery temperature and the consumption of auxiliary fuel per ton of cake

Shown in Figure 21 is the relationship between the preheat recovery



Furnace capacity: 50 ton/day
Moisture content: 80%
Volatile solids content: 70%
Air ratio: 1.3

Fig. 21 Preheat recovery temperature and consumption of auxiliary fuel

temperature and the consumption of auxiliary fuel under the conditions where the moisture content of input cake is 80%, content of volatile solids is 70%, scale of furnace is 50 tons/day and air ratio is 1.3.

Heat exchangers for preheating air are provided for Run 1, 3 and 5. Shown in Table 15 is the change in the consumption of auxiliary fuel in response to every increase of 50°C in the preheat recovery temperature of each model.

Table 15 Change in consumption of auxiliary fuel due to a rise of 50°C of preheat recovery

(Unit: kg/ton-cake)

System \ Preheat recovery temperature	400∿450	450∿500	500∿550	550∿600	600∿650
[I]	- 4.0.	- 3.6	- 3.6	- 3.1	- 3.3
[III]	- 0.9.	- 0.7	- 0.9	- 0.8	- 0.8
[V]	+ 0.1.	+ 0.2	+ 0.1	+ 0.1	+ 0.1

In Run 1 and 3, the consumption of auxiliary fuel per ton of cake decreases as the preheat recovery temperature rises. Rate of decrease is the highest in Run 1, and the relative fuel consumption at 650°C is 0.65 compared to 1.0 of the consumption of auxiliary fuel at 400°C. In Run 3, the relative fuel consumption at 650°C is 0.79 compared to 1.0 of the consumption of auxiliary fuel at 400°C, so that the decrease is smaller than that of Run 1.

In Run 5, the consumption of auxiliary fuel slightly increases as the preheat recovery temperature rises. In this model, heat in exhaust gas is recovered by a heat exchanger to preheat combustion air and surplus heat is also recovered by the waste heat boiler to dry cake. Therefore, if the set efficiency of dryer is higher than the efficiency of heat recovery by the

preheat recovering equipment, then the rise in the preheat recovery temperature will result in the increase in the consumption of auxiliary fuel for the entire incineration facility.

(6) Natural limit

The results of predicting the cake water content ratio and the natural limit for each model are shown in Table-16, and the results of predicting the calorific value of the cake are shown in Table-17.

Table 16 Moisture content of cake as the limit of autogenous combustion (scale of furnace: 50 t/day)

(%)

System / Content of volatile solids	I	II	III	IV	V
60%	67.8	69.9	73.0	74.9	74.9
70%	71.5	73.5	76.3	78.6	78.4
80%	74.5	76.3	78.7	80.3	80.3

Table 17 Fuel value of cake as the limit of autogenous combustion (scale of furnace 50 t/day)

(Kcal/kg-cake)

System / Content of volatile (V)	I	II	III	IV	V
60%	590	510	400	330	330
70%	610	520	400	310	310
80%	620	530	410	340	340

In cases where the ratio (V) of organic matter contained in the cake and for each model was high, the natural limit became proportional to the high water content ratio, and in the case of Run 1, was approximately 68% when the water content ratio was 60%, and approximately 75% when the water content ratio was 80%.

The difference between the cake water content for each model, and the cake water ratio at the natural critical limit for Run 1 when the ratio (V) of organic matter in the cake was 70%, was predicted as being natural even when the contained water ratios were approximately 73.5%, 76.3%, 78.6% and 78.4% for Run 2, Run 3, Run 4 and Run 5 respectively.

Furthermore, the lowest cake calorific value the natural limit was in the range of 230 to 340 kcal/kg, using the furnace formula for Run 4.

The natural combustion region and the assisted combustion region for each system are shown in Figure 22.

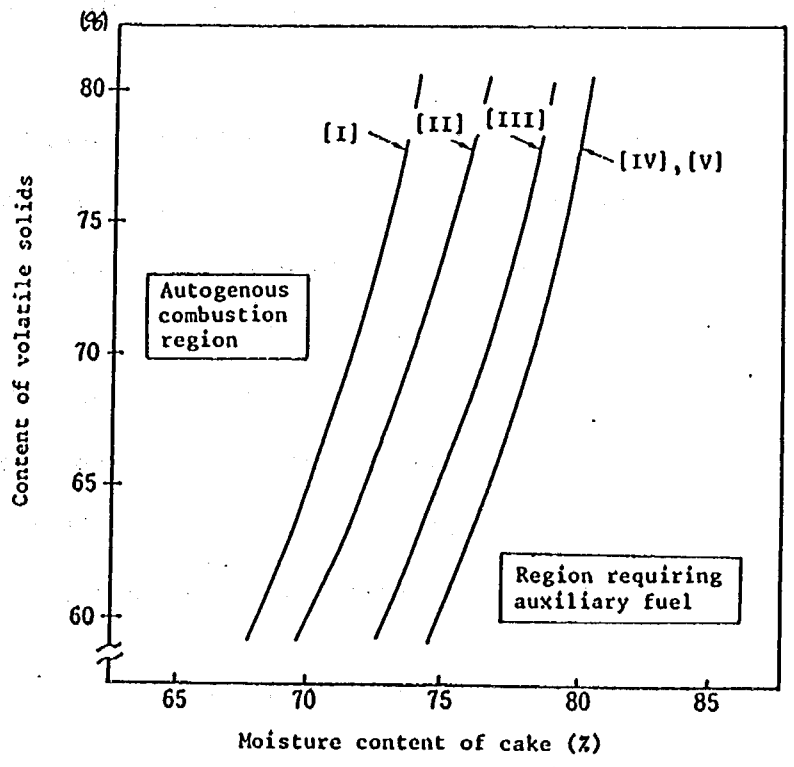


Fig. 22 Content of volatile solids versus moisture content for each system

(7) Conclusions

The results of a sensitivity analysis for the auxiliary combustion expenses for each of the parameters, per ton of moist cake, showed that the countermeasures to reduce these expenses were as listed below.

From item (1), the auxiliary combustion expense reduction associated with lowering the water content has a large influence upon the other parameters. Accordingly, the sludge input to the furnace should have a low water content.

From item (2), increasing the air ratio has the effect of increasing the expenses associated with auxiliary combustion.

From item (3), increasing the ratio of organic matter has the effect of decreasing the expenses associated with auxiliary combustion.

From item (4), increasing the scale of the furnace has the effect of decreasing the expenses associated with auxiliary combustion.

From item (5), increasing the preheat recovery temperature has the effect of decreasing the expenses associated with auxiliary combustion in all cases except that for models of the indirect drying type.

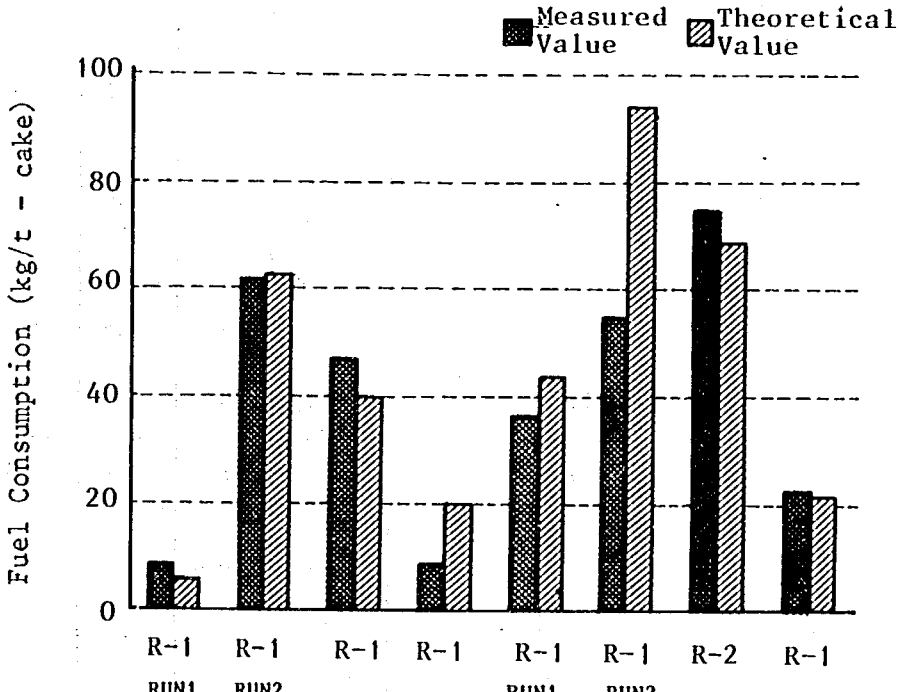
From the above, it can be seen that decreasing the water content has the largest effect. It is also desirabel as a factor relating to the operation, to reduce the air as much as possible, while it is also effective to raise the preheat recovery temperature according to the model.

2.4 Applicability of Models to Existing Furnaces

A comparison of the measured auxiliary fuel consumption for the operating furnaces and those of the models suggested in this paper is given

in Figure 23. The operating conditions and the cake characteristics which are used to obtain the theoretical fuel consumption values are given in Table 18. As Figure 23 clearly shows, the suggested models reflect the actual operation closely and therefore can be effectively used to evaluate energy saving measures in regard to furnaces.

Acknowledgement: The authors would like to express their thanks to Kazumi Suzuki, Kaoru Sasabe and Jiro Kubo of the Technical Development Division, Japan Sewerage Corporation for their cooperation in data collection and analysis. Thanks are also due to those public organizations which kindly cooperated with the survey on operating furnaces.



	Ohtsu	Ohtsu	Sagami	Kako	Sayama	Sayama	Suwa	Machida
Measured Value	9	62	47	9	37	55	75	23
Theoretical Value	6	63	40	20	44	94	69	22

Fig. 23 Examination of heat balance calculation

Table 19 Comparison between calculated values and measured values of fuel consumption and other items

		Ohtsu (Drying)	Ohtsu (Direct input)	Sagamigawa	Kakogawa	Sayama Run 1	Sayama Run 2	Suwako	Machida
Model applied		R-1	R-1	R-1 R-2	R-1	R-1	R-1	R-2	R-1 R-2
Measured exhaust gas temperature at furnace outlet (°C)			815	830	710	683	680	705	800
Fuel consumption (kg/h)	Measured value	10*(10)	47* (48)	67* (65)	20	70	80	119	42* (43)
	Theoretical value 1	13	51	68 52	32	76	105	105	52 33
	Theoretical value 2	13	49	64 48	52	109	143	118	52 33
Higher calorific value of cake (Kcal/kg)			2920	4069	2590	2010	1340	2077	4490
Preheated air temperature (°C)	Measured value		517	585	425	463	370	610	580
	Theoretical value 1	500	500	500 650	500	500	500	650	500 650
	Theoretical value 2								
Heat loss at furnace (%)	Measured value		8.8	6.2	2.8	10.2	4.5	5.2	3.6
	Theoretical value 1		11.0	9.0	8.0	8.0	9.0	9.0	9.0
	Theoretical value 2		4.9	49.5	13.3	19.6	28.8	3.7	4.2
Heat loss at heat exchanger for preheating (%)	Measured value		10.0	10.0	10.0	10.0	10.0	10.0	10.0
	Theoretical value 1								
	Theoretical value 2								

Notes: 1. Assumed ratio of chemical additive is cited from Sludge Treatment, (3) Extractor of Sewerage Statistics 1983.

2. Fuel with * is kerosene. Figures in ( ) are converted values to heavy oil.

SESSION 4

SMALL-SCALE WASTEWATER TREATMENT

"Treatment of Household Wastewater Treatment Tank Sludge in Japan"
Y. Magara 369
"Treatment of Domestic Wastewater from Dwellings and Small Communities in Special Treatment Plants"
P. Wilderer 387
"Performance Evaluation of Sequencing Batch Reactor for Municipal Wastewater Treatment"
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SESSION 4

SMALL-SCALE WASTEWATER TREATMENT

"Treatment of Domestic Wastewater Through Leach Bed in Japan"
Y. Nagao 283
"Treatment of Domestic Wastewater from Dwelling and Small Communities in Special Tourism Place"
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"Performance Evaluation of Separating Tank Reactor for Municipal Wastewater Treatment"
K. Tanaka 423

TREATMENT OF HOUSEHOLD WASTEWATER TREATMENT TANK SLUDGE IN JAPAN

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3. Proprties of Jyoukasoh sludge and collected human excreta 5
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1. INTRODUCTION

Domestic wastewaters in Japan are managed by four ways, i.e. public sewerage system, collective human excreta treatment system household wastewater treatment tank systm and small communal wastewater treatmnent tank system, in principle. The tanks of the latter two systems are called "Jyoukasoh" in Japanese or private wastewater treatment tanks because they are provided and managed by house owners privately. The collected human excreta treatment system which is a very unique system in the world means that human excreta in a voult toilet of each house is collected by a vacuum tanker and then treated at a collective human excreta treatment plant.

In one of each way above mentioned every Japanese house has an access to sanitary treatment of human excreta. The people who can use water flush toilet facilities connected with a public sewage system or Jyoukasoh are only limited to about 52 % of the total population in 1983, as shown in Figures 1 and 2. Even though the construction work of public sewerage system is implemented intensively, the annual increase of population covered by it remains at about one percent of the total population. Therefore the role played of human excreata

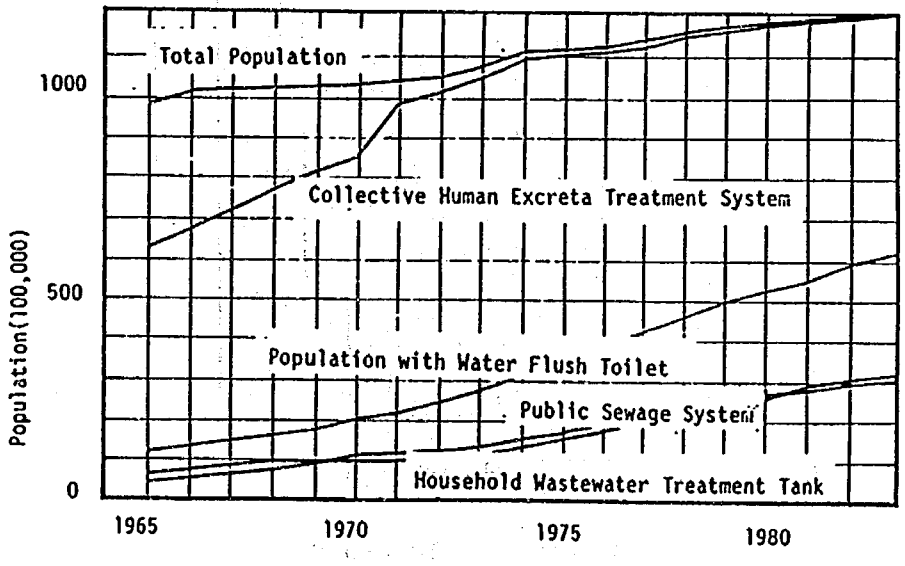


Figure 1.Progress of Sanitary Treatment of Human Excreta

treatment systems other than public sewerage system to provide a sanitary and aesthetic living environment will be continuingly high in some more decades.

The management of domestic wastes including human excreta as well as solid wastes is conducted under the responsibility of municipalities by the principle of Waste Disposal and Public Cleasing Law, and therefore the faicilities for sanitary treatment of human excreta as well as sludges genereated from Jyoukasoh are provided and operated by municipalities. Since the collective human excreta treatment system was firstly provided and Jyoukasoh system was not popular than other systems, therefore Jyoukasoh sludge has been mixed with collected human excreta and treated at collected human excreta treatment plant as shown in Figure 2,in principle.

The quality of Jyoukasoh sludge is different from that of collected human excreta. If the proportion of Jyoukasoh sludge to collected human excreta becomes higher in their combined treatment, care must be taken for it, and sometimes their separate treatment is necessary to meet the effluent quality regulations.

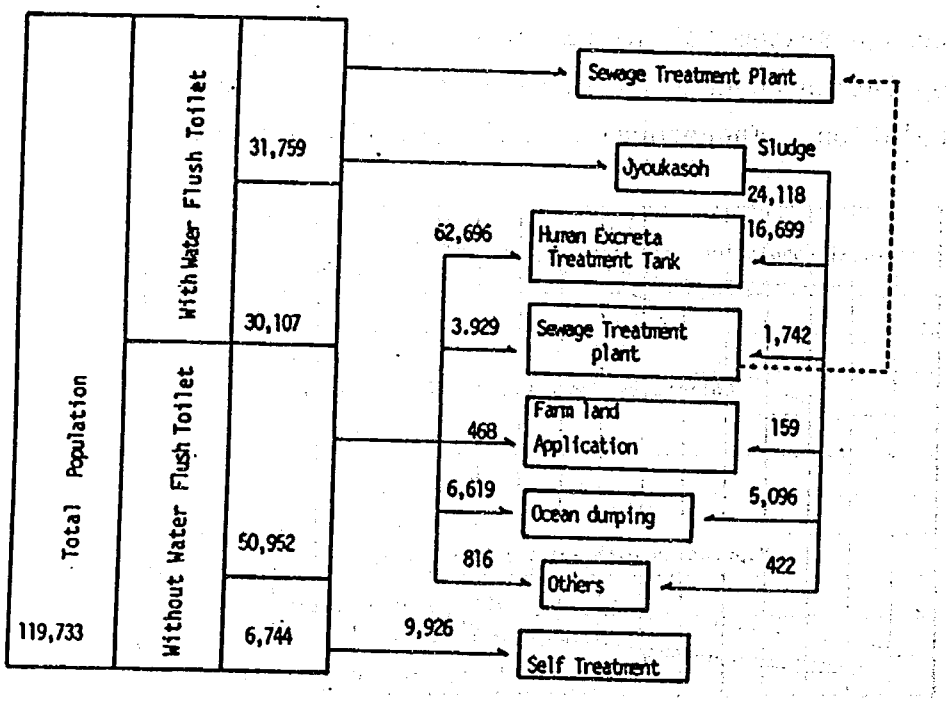


Figure 2. Treatment of Human Excreta in Japan

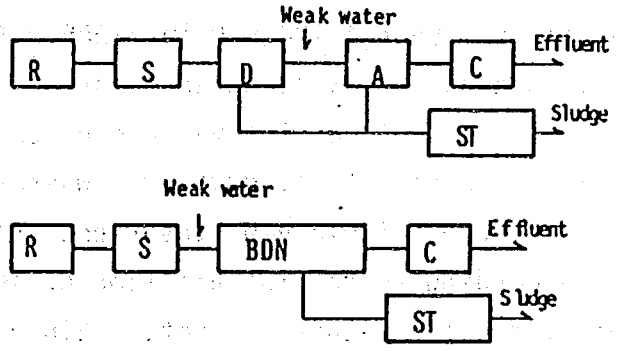
2. COLLECTIVE HUMAN EXCRETA TREATMENT PLANT AND JYOUKASOH SLUDGE

Collective human excreta treatment plants are classified into two types as sown in Figure 3 and Table 1. The one is a system which is mainly composed of primary aerobic or anaerobic digestion process and secondary treatmet of activated sludge process. The other one is a system of biological denitrification process.

Table 1. Basic Design Standards of Collective Human Excreta Treatment Plant

Type of Process	AND/AS	AD/AS	BDN
Digester	35-37 °C/30days	12days	
Supernatant(BODmg/l)	2500	2500	
Dilution rate	20 times	20 times	10times
BOD load(kg/kg/day)	0.15-0.25	0.15-0.25	0.05-0.15
Retention Time	6-8 hours	6-8 hours	60-70 hours
Overflow Rate of Clarifier (m/day)	18	18	9

AND/AS :Anaerobic digestion and activated sludge process
AD/AS:Aerobic Digestion and activated sludge process
BDN: Biological denitrification process



R: Receiving Tank S: Storage Tank D: Digester
A: Aeration Tank C: Clarifier ST: Sludge Treatment
BDN :Biological Denitrification Tank

Figure 3. Types of Collective Human Excreta Treatment Plant

Anaerobic digestion-activated sludge process is most popular and covers about 50% of total quantity of human excreta treated in collective human excreta treatment plant in 1985. As shown in Figure 3 and Table 1 collected human excreta is diverted from vacuum tanker into a receiving tank with a grit chamber and a screen and fed to a storage tank, and then pumped to a digestion tank, where human excreta is digested for about 30 days by heating up to about 35 to 37 C°. The supernatant of digester is diluted at 20 times by water in order to treat it by following conventional activated sludge process. The sludge settled at the bottom of digestion tank is extracted at a constant rate, and dewatered for disposal or farm land application. The digestion gas of which has an energy of 5000 to 6000 Kcal/m is generated at a rate of about 600 to 800L/kg of organic materials.

Biological denitrification process has been adopted in newly constructd human excreata treatment plants because of its low dilution ratio and effective removal of nitrogen as well as BOD. Human excreta after pretreatment is diluted at 10 times by water, and then treated in two stage (denitrification-nitrification/denitrification-aeration) biological denitrification process. Since the BOD/nitrogen ratio of collected human excreta is preferable for biological denitrification reaction, the effluent nitrogen concentration of 10 mg/l or less can be attained without any additional substrate such as methanol and alkali by the samllest size of facility among all of human excreta tratment process.

The total number of Jyoukasoh is about 4,100,000 in 1985. There are two types of Jyoukasoh,i.e. a household wastewater treatment tank in which only human excreta is treated(exclusive human excreta treatment tank) and the one in which all of the domestic wastewater including toilet wastewater is treated (combined treatment tank). Jyoukasoh are also classified according to the installation of septic tank prior to aerobic biological treatment tank or not. Since the effluent from Jyoukasoh did not always meet the effluent quality regulations, its design standard was ammended in 1984. Aaccording to the ammended design standard every exclusive human excreata treatment

tank and combined treatment tank with a design population of between 50 to 500 are obliged to install a septic tank. The number of of each type of Jyoukasoh is shown in Table 2. According to the regulation, the sludge in every Jyoukasoh which means all content of a septic tank and 50 to 80% of mixed liquor of an aeraion tank should be withdrawn at least once a year.

Table 2 Number of Jyoukasoh

Exclusive treatment type		Combined treatment type
With septic tank	Without septic tank	
2,087,482	2,857,009	58,500

3. PROPERTIES OF JYOUKASOH SLUDGE AND COLLECTED HUMAN EXCRETA

A Collected human excreta treatment plant is designed according to the instructions of the design standard provided by Ministry of Health and Welfare. In the design standard the general properties of collected human excreta and Jyoukasoh sludge as shown in Table 3 are given. Table 3 shows that the strength of Jyoukasoh sludge is weaker than that of collected human excreta , and the quantity of Jyoukasoh sludge generated per person is also smaller than that of collected human excreata.

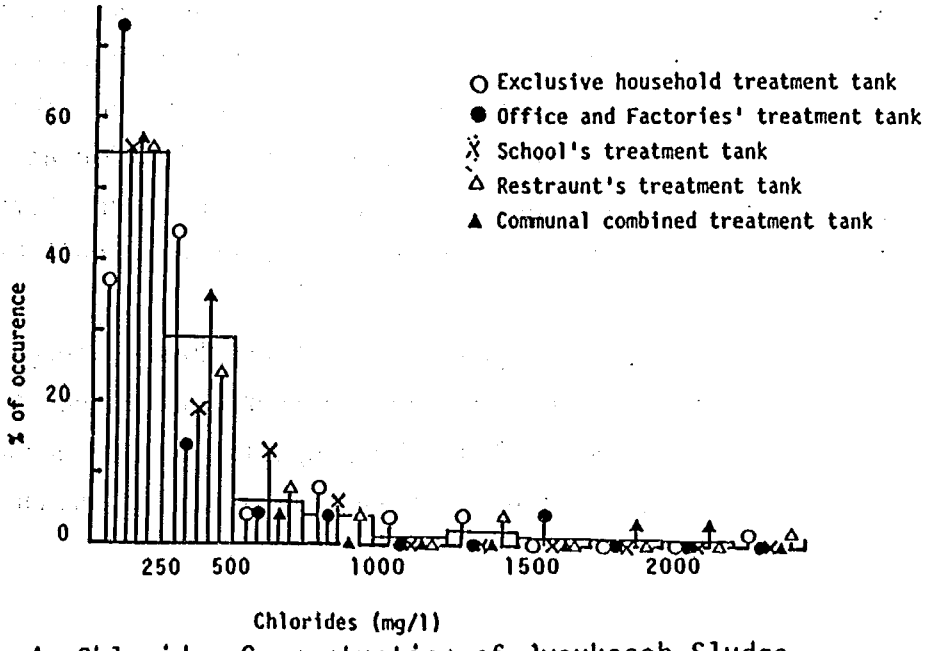


Figure 4. Chlorides Concentration of Jyoukasoh Sludge

Table 3 General Properties of Human Excreta and Jyoukasoh Sludge

	Human Excreta	Jyoukasoh Sludge
pH (-)	8	7
BOD (mg/l)	13,500	7,000
COD (mg/l)	7,000	4,000
SS (mg/l)	21,000	13,000
TS (mg/l)	30,000	17,000
T-N (mg/l)	5,000	1,000
Cl (mg/l)	5,500	200
Quantity(lcpd)	1.4	0.75 or 1.2

1) Exclusive type 2) Combined type

Table 4. Quantity of Jyoukasoh Sludge

	Population	Collected Sludge	Sludge (lcpd)
1975	17,532 000	13,170 kl/day	0.75
1976	19,040 000	14,367	0.75
1977	21,206 000	17,333	0.82
1978	23,620 000	18,055	0.76

The total quantity of collected Joukasoh sludge was about 24,100kl a day, and the population using Jyoukasoh was about 30 millions in 1984. Therefore the sludge generation per person was about 0.80 l a day. This estimated value is almost same as former data as shown in Table 4. But the generation of sludge depends on the type of Jyoukasoh, therefore the values of sludge quantity for an exclusive treatment tank and a combined treatment tank are given 0.75lcpd and 1.2 lcpd, respectively, by the design standard of a collective human excreta treatment plant.

Although the general properties of Jyoukasoh sludge are given by the design standard, there may be a difference between the type of Jyoukasoh. In order to make clear the properties of Jyoukasoh sludge about 130 samples were taken at certain collective human excreta

treatment plant and analyzed by the authors. The result of this study is shown in Table 5. Comparing this result with the general properties of collected human excreta, as shown in Table 3, where it is obvious that the strength of Jyoukasoh sludge is weaker than that of human excreta and widely distributed depending on the type of Jyoukasoh and it user. The average concentration of suspended solids in Jyoukasoh sludge is about 30 to 60 % of collected human excreta, and there is a big difference between samples, as shown in Figure 5.. BOD and COD concentration in Jyoukasoh sludge are more widely

Table 5 Properties of Jyoukasoh Sludge

	Household	Office	School	Restraunt	Community
pH	7.0	6.5	6.2	6.6	6.9
Cl (mg/l)	460	300	480	340	340
TS (%)	1.3	1.0	0.9	1.6	1.2
SS (%)	1.1	0.7	0.7	1.5	0.9
COD(mg/l)	2600	1400	1700	2400	2400
F-COD(mg/l)	230	120	160	120	160
BOD(mg/l)	3300	1900	2800	2900	2400
F-BOD(mg/l)	290	110	400	210	220
TK-N(mg/l)	830	600	460	850	630

	Exclusive treatment tank		Combined treatment
	with septic tank	without septic tank	tank
pH	7.0	6.7	6.7
Cl (mg/l)	330	460	440
TS(%)	1.1	1.3	1.8
SS(%)	0.8	1.0	1.6
COD(mg/l)	1800	2500	2500
F-COD(mg/l)	500	370	590
BOD(mg/l)	2200	3600	2500
F-BOD(mg/l)	230	370	320

distributed than its of suspended solids concentrations as shown in Figure 6. Soluble BOD and nitrogen concentration are lower than those of collected human excreta.

4. ADVERSE EFFECT OF JYOUKASOH SLUDGE ON TREATENT AND ITS COUNTERMEASURES

The demand for water flush toilets especially in the periphery of an urban area accelerate installation of Jyoukasoh. Consequently the proportion of Jyoukasoh sludge to the total quantity of human excreta and sludge to be treated in a plant increase the actual load to the

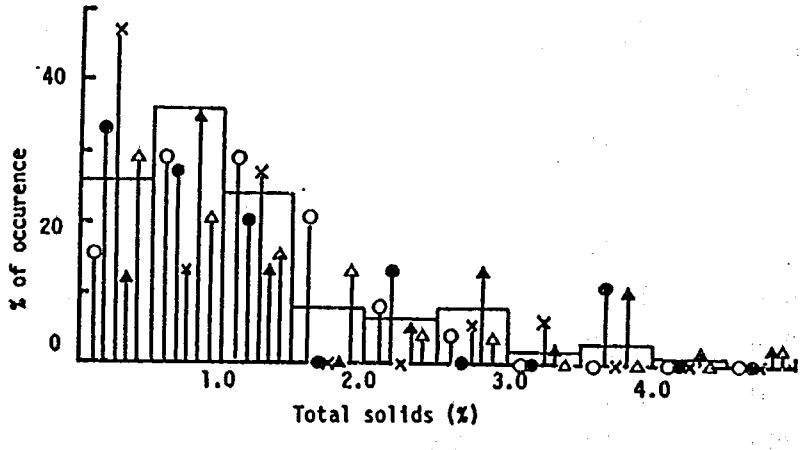


Figure 5. Total solid concentration of Jyoukasoh Sludge

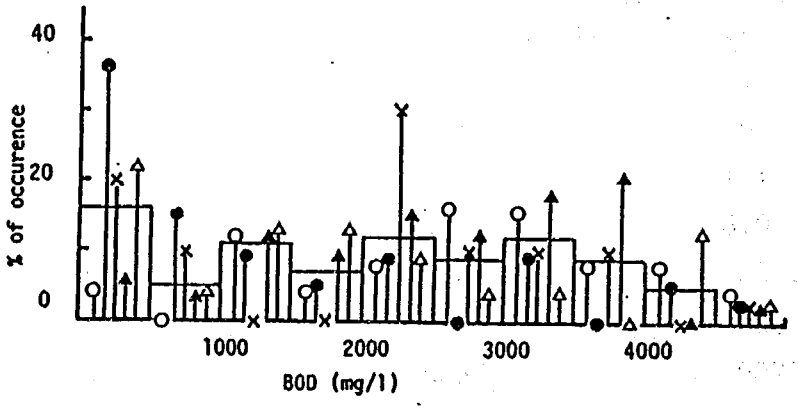


Figure 6. BOD concentration of Jyoukasoh Sludge

plant become changes from the design value, and the opreration and maintenance of the plant becomes more difficult. In some cases the effluent quality can not meet the effluent quality regulations.

It has been reported many adverse effects of which caused by under loading to human excreta treatment plant. Typical examples of adverse effects are as follows. Both of the production rate and energy potential of of digester gas are decreased and eventully auxiliary fuel is needed to keep the digester at an appropriate level. BOD concentration of the digester supernatant is decreased and the quality of final effluent becomes worse due to under loading to activated sludge process. Biological denitrification process is also faced with a more serious adverese effect on the activities of nitrifying/denitrifying bacteria due to shift of the BOD/nitrogen ratio of the influent from the design condition.

Since human excreta and Jyoukasoh sludge are collected separately, the manner of collection works leads the load fluctuation of a collective human excreta treatment plant. Supposing Jyoukasoh sludge is collected from a combined domestic wastewater treatment tank of which design populaton is 200, the quantity of sludge transfered to human excreta treatment plant become about 20kl per one time of withdrawing the sludge . If the design capacity of that plant is 100 kl a day which is equivallent to 100,000 designed populaion a day this amount of sludge will be 20% of the daily total capacity, which may cause adverse effect on the treatment.

Because the properties of Jyoukasoh sludge is different depending not only on the type but also on the user of Jyoukasoh, the manner

Table 6 Capacity of Sorage Tank of Jyoukasoh Sludge

Designed daily collected sludge (kl/day)	Capacity(m^3)
less than 10	5 * X
10 - 30	3 * X + 20
more than 30	2 * X + 50

X: Designed quantity of daily collected sludge

of collection works of Jyoukasoh sludge as above mentioned is one of the reasons of the technical problems encoutered at a human excreta treatment plant.

In order to equalize the load fluctuation of the collected Jyoukasoh sludge, a storage tank with a capacity of about 5 times its designed daily amount of collected Jyoukasoh sludge is installed in a human excreta treatment tank as shown in Table 6. After the storage of collected Jyoukasoh sludge is treated by one of the three methods as shown in Figure7..

The first method that is a joint treatment with Jyoukasoh sludge and collected human excreta has been usually applied in many collective human excreta treatment plants. Since the quality and quantity of Jyoukasoh sludge is equalized during storage in a storage tank and is able to mixed at any selected ratio to collected human excreta, only problems to remain are under loading and change in the composition of human wastes to be treated. It is reported that the function of treatment process is not disturbed by Jyouasoh sludge if

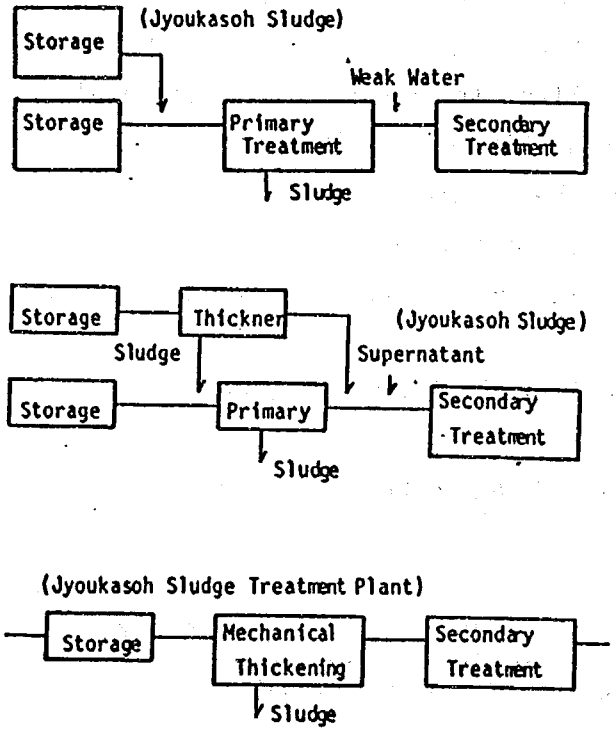


Figure 7. Types of Jyoukasoh Sludge Treatment Process

the proportion of Jyoukasoh sludge less than 30 % (v/v) of total quantity of human wastes to be treated. Based on this experiences the proportion of jyoukasoh sludge is recommended to be kept at lesser than 30% in the case of joint treatment by the design standard. Since the organic load to treatment process is slightly decreased depending of the proportion of Jyoukasoh sludge, however, the dilution ratio by water at the inlet of activated sludge process or biological denitrification process as a secondary treament is reduced so as to sustain the designed organic load.

Jyoukasoh is the major system for human excreta treatment system in the peripheries of big cities. The proportion of Jyoukasoh sludge to human wastes is becoming more at collective human excreta tretment plant in those area. The second and third methods are applied in the case when the proportion of Jyoukasoh sludge is more than 30 %. The strength of Jyoukasoh sludge is weaker than that of collected human excreta as already mentioned, and most of organic constituents of Jyoukasoh sludge are attributed to the solid fraction. Therefore if Jyoukasoh sludge is concentrated by an appropriate method, the concentrated sludge can be accepted by primary digestion process, and the supernatant of concentrator can be treated by the secondary biological treatment process with adjusting the dilution ratio so as to meet the designed organic load condition.

Table 7 shows the performance of gravity thickening of Jyoukasoh sludge which is aerated three days in a storage tank. Since the concentration of thickened slurry is almost the same as that of collected human excreta, primary digestion process will not be disturbed by mixing the slurry with collected human excreta. The quality of supernatant of a thickner also looks like the supernatant of a digester and can be accepted by the secondary activated sludge psrocess. Table 8 shows the performance of centrifugal thickening of Jyoukasoh sludge which is chemically conditioned with synthetic polymer. It is clear that Jyoukasoh sludge is effectively separated into solid and liquid fraction. Solids contents of thickened sludge is

Table 7 Gravity Thickening of Jyoukasoh Sludge

BOD Inlet(mg/l)	BOD Supernatant(mg/l)	TS Sludge(%)
6500	2000	3
4300	250	4
5800	1800	2.5
7500	2260	2
5510	1930	1.5
5060	1620	2.5
10800	3240	2.0
6700	2680	3.0

The sludge feeded to a thickner was aerated for three days in a storage tank

Table 8 Centrifugal Sludge Thickening With Chemical Conditioning

BOD Feeded (mg/l)	BOD Filtrate (mg/l)	TS (%)
4230	62	24.7
2080	19	22.1
3320	349	24.8
3650	13	20.0
3300	360	-
4800	592	-

so high that it can be disposed after an appropriate sterlization to landfill or farm land application , and the supernatant is also low in concentration of organic consitituents to be treated by conventional activated sludge process without any dilution. In the areas where the proportion of Jyoukasoh sludge is extremely high the third methods exclusive treatment of Jyoukasoh sludge is mainly adopted.

There are seventeen Jyoukasoh sludge treatment plants which are designed by the bases of the knowledge above mentioned in 1984. The total design capacity of these Jyoukasoh sludge treatment plant are 1350 kl/day, and the capacity of each plant ranges from 334kl/day of

which adopts the second method above mentioned to 20kl/day of the third method. The treatment cost excluding labor cost and investment cost at these treatment plants ranges from 162 to 2293(average 620) yen/kl. This cost for each user of Jyoukasoh becomes about 170 yen/year. It is said that by introducing these treatment plant the problems caused by increasing Jyouksoh sludge at collective human excreta treatment plant are overcomed with a reasonable additional cost.

5. Conclusions

Sanitary treatment and disposal of human excreta is essential from both of environmental and aesthetic points of view. In Japan, human excreta is treated in public sewarage system, household wastewater treatment tank and collective human excreta treatment system. Since public sewarage sytem can not provided for every peoples, people install household wastewater treatment tank, Jyoukasoh, privately in order to use water flush toilet.

Collective human excreta treatment plant accept Jyoukasoh sludge and reated it with collected human excreta. Since the properties of Jyoukasoh sludge is different from collected human excreta, and moreover the total quantity of Jyoukasoh sludge and collected human excreta is changed from the design value of a plant according to the prevailing Jyoukasoh, the operation of collected human excreta treatment plant is disturved with increasing of the quanity of Jyoukasoh sludge.

In order to satisfy the effluent regulations and to solve the problems arising by Jyoukasoh sludge, combined tretament and separate treatment of Jyouaksoh sludge are developed. Combined treatment is that process of which has pretreatment process for concentrating Jyoukasoh sludge prior to apply existing collected human excreta treatment palnt. Separate treatment is that process for exclusive treatment of Jyoukasoh sludge by gravity or mechanical thickening and biological treatment of supernatant. By both

of these methods about 1350 kl/day of Jyouaksoh sludge are treated at seventeen Jyoukasoh sludge treatment plant.

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TREATMENT OF DOMESTIC WASTEWATER FROM DWELLINGS AND SMALL COMMUNITIES IN SPECIAL TREATMENT PLANTS

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C O N T E N T

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ABSTRACT

Collection of wastewater by means of sewer systems is a difficult and most expensive task, when the catchment aerea to be served is low in population density. Nevertheless, collection and controlled treatment of domestic wastewater is, in many instances, required, despite of low population density, in order to protect the quality of the groundwater for drinking water purposes.

In several counties of the State of Schleswig-Holstein, West Germany the content of cesspools and multiple chamber septic tanks is regularly hauled by tank truck, and brought to special treatment plants established in the center of selected small size catchment areas. Since the rivers and creeks available as receiving waters for the treated effluent are susceptible to eutrification relatively high discharge standards have to be met.

Treatment of the collected wastewater at Steinhorst, Schleswig-Holstein includes bar screening, equalization and gravity sedimentation as pretreatment steps. Sludge is aerobically stabilized, dewatered, mixed with lime for ultimate desinfection, and finally given to the farmers as fertilizer and soil conditioner. The supernatant is biologically treated by means of a continuous flow activated sludge process , and discharged to a creek.

When the treatment plant was designed in 1981, information about composition, concentration and biodegradability of the wastewater to be expected was very limited. An entirely empirical approach had to be chosen. COD and the ammonia concentration of the wastewater decanted from the sedimentation tank were assumed to be high, and the biodegradability of the organic wastewater constituents was expected to be relatively poor as compared to normal

municipal wastewater. A hydraulic detention time of at least 3 days was considered to be necessary to meet the site specific discharge permits. Currently, the plant is operated at a hydraulic detention time of 6 days. The performance data are satisfying. Almost complete nitrification and denitrification is achieved.

Encouraged by the performance of the full scale plant a pilot study has been conducted to define proper design criteria. A Sequencing Batch Reactor (SBR) was chosen as a model system of the continuous flow activated sludge plant because of the convenience the SBR offers the investigator in getting time scale and process strategies systematically varied. A cylindrical tank, 10 m^3 in volume, was used for the studies. The reactor was subdivided into two compartments. Anoxic conditions were provided in an inner cylinder to achieve denitrification. In the outer, annular space aerobic conversion processes were established to achieve advanced carbon removal and nitrification. A novel decant system was used to drain the reactor.

From the result of the studies it can be concluded that at a temperature of above 10 ^{o}C a 2.5 days hydraulic detention time is appropriate. The dichromate-COD decreased from a level of about 2,ooo mg/l, on the average, to less than 80 mg/l, and the concentration of Total Kjeldahl Nitrogen (org.N plus NH_4-N) dropped from 150 mg/l, on the average, to less than 1 mg/l. During the studies the effluent concentration of nitrite and nitrate nitrogen was about 1 mg/l indicating accomplishment of complete denitrification.

EVALUATION OF THE PROBLEM

Germany is commonly known to be a densily populated and highly industrialized country. However, there are regions, mostly in the northern part of West Germany and in Bavaria, which are low in population density with small villages far apart from each other, and with dwellings and farm houses scattered over a wide area of land. Those regions are, in most cases, flat and it is, then, especially difficult, expensive and ecologically questionable to employ sewer systems for the transportation of wastewater to any central treatment plants far in the distance. On-site treatment of wastewater by means of septic tanks or small municipal treatment system may not be a tolerable alternative, either, since the actual performance data of those systems do often not satisfy the specific requirements of surface water and groundwater protection. Besides, sludges generated in those on-site systems must be taken care of, anyway.

According to a survey conducted by Bischofsberger (1982) the portion of people who, for economical reasons, cannot be served by sewers will probably remain close to 10 per cent of the total population, nation wide. For the State of Schleswig-Holstein that number may most probably stay above the 20 per cent level, however, because of the specific topographic situation there, and the low population density.

According to State Laws, houses not connected to sewers must be equipped with multi chamber septic tanks or with cesspools. The latter are required to be efficiently sealed to prevent percolation of polluted water into the underground. After the amendment of the Federal Water Management Act had passed the Parlament in 1976 the States of the Federal Republic agreed to enforce collection of the content of septic tanks and cesspools on a regular basis, and treatment of that material (septage) prior to

discharge. It should be noted that the term "septage", in this context, does not only refer to the sludge accumulated in the septic tanks and cesspool but also to the supernatant usually referred to as wastewater. The term septage is, in this paper, applied to the mixture of both wastewater and sludge.

Septage can be treated jointly with regularly sewered domestic or municipal wastewater provided a properly sized treatment plant is close by. A distance of 25 kilometers for the tank truck to carry on the septage is considered tolerable. Unfortunately, the distance to the closest treatment plant is often too long, or that treatment plant is too small, or it does not provide any further capacity.

As an alternative, the establishment and operation of a treatment plant spezialized to treat septage may be considered. Such a special treatment plant should be built in the center of a selected catchment area with the site chosen according to the results of a minimization study for the distances the tank trucks would have to cover.

Disscussion on how to deal with the septage to be treated in the County of Lauenburg, State of Schleswig-Holstein led eventually to the decision of establishing such a special treatment plant near the village of Steinhorst. The plant was built in 1982/1983 and has been in operation since September 1983. It serves an area of 400 square kilometers. The distance between the farthest dwelling and the treatment plant is 15 kilometers. Recently, pilot studies were conducted to investigate the potential of biological treatment to remove carbon and nitrogen.

The purpose of this paper is to describe the major design characteristics of that treatment plant, to summarize the treatment results achieved so far, and to inform about the results of the pilot studies.

TREATMENT CONCEPT

The wastewater (septage) pumped out of septic tanks and cesspools contains three major fractions of contaminants:

* coarse solids,
* settleable material, and
* suspended, colloidal and dissolved substances.

In principle, specific unit operations as they are normally used for wastewater treatment should be applicable for the treatment of septage also, i.e.

* bar screening,
* primary sedimentation,
  for the biodegradable portion of the pollutants:
* biological conversion followed by sedimentation,
  and for the sludges wastes from the sedimentation tanks:
* aerobic or anaerobic treatment followed by dewatering.

The overall concentration of organic substances in septage is reported to be very high (Table 1). From the difference between the parameter values for the filtered and the non-filtered samples, however, it appears that the major portion of the organic constituents is of particulate nature. Thus, the concentration of organic substances to be treated by means of biological wastewater treatment systems should decrease considerably, when effective solid-liquid separation methods were applied for primary treatment. Aerobic biological treatment processes can be assumed to be economically applicable for the removal of the organics remaining in the supernatant of a well designed gravity sedimentation/sludge thickening tank.

Besides the organic fraction ammonia appears to be a major pollutant of septage. As compared to municipal wastewater the values

listed in Table 1 for the parameter "ammonia nitrogen" are of an order of magnitude higher than normal. Discharge of an effluent containing that high ammonia concentration appears to be non-acceptable under any circumstances. Nitrification, in many cases followed by denitrification must be considered necessary to avoid fish toxicity, oxygen deficiency and eutrophication in receiving waters, and perturbation of the quality of groundwaters, respectively.

Table 1 Characteristics of septage according to a survey of Bischofsberger (1982)

parameter	range, mg/l	mean, mg/l
dry weight	200 - 75,000	14,000
total organics	160 - 50,000	10,100
BOD_5	700 - 25,000	4,700
filterable BOD_5	100 - 4,300	1,140
Dichromate COD	1,300 - 65,000	15,900
filterable COD	400 - 28,000	3,100
$KMNO_4$-COD	1,750 - 37,000	15,900
filterable $KMNO_4$-COD	400 - 2,400	1,600
total nitrogen	150 - 1,400	535
ammonia nitrogen	100 - 550	280
total phosphorus	25 - 400	150
organic acids	60 - 2,300	735
hydrogen sulfur	2 - 250	45

The solid phase separated from the septage must be expected to be high in concentration of organic substances but limited in volume. Since anaerobic conversion processes have, most probably, taken place in the septic tanks and cesspools, already and to some extent, the application of anaerobic treatment methods to

stabilize that sludge must be assessed critically. Alternatively, thermophilic, aerobic stabilization processes in combination with lime treatment can be taken into consideration.

Mineralization of the organic matter, elimination of pathogenics and destruction of seeds must be achieved. In this context, lime treatment is of particular interest. The generation of heat, and the lift of the pH-value after dosage of quick lime contributes to the progress in desinfection processes. Besides, the increase of the lime content of the treated sludge increases the value of the material as fertilizer and as soil conditioner. Agricultural application of the sludge is considered feasible since hazardous organics, and anorganic substances such as heavy metals must not be expected to be present in significant quantities.

Based on these considerations the treatment system sketched in Fig. 1 was proposed. The system consists of the following elements:

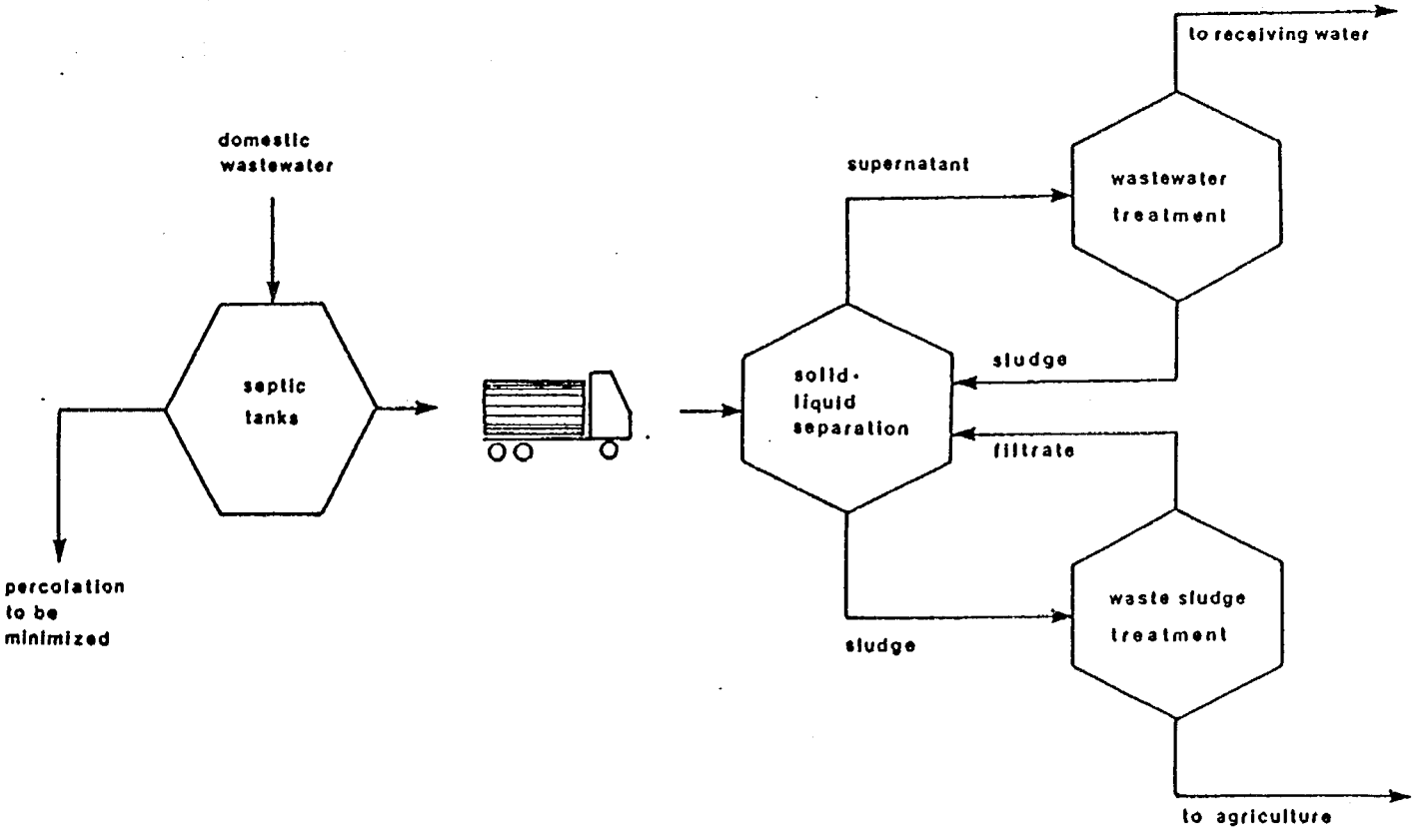


Fig. 1 Schematic of the proposed septage management system

The septic tanks and cesspools provide a certain degree of pretreatment, are, therefore, treated as a part of the overall treatment system.

Septage should be hauled by tank truck as often as possible to limit groundwater pollution.

The collected material is to be transfered through a bar screen and introduced into a gravity thickener which serves as a holding tank to equalize variations of flow and composition of the septage and, simultaneously, as a means to achieve solid-liquid separation.

The supernatant is to be decanted from that tank, introduced into an activated sludge treatment system for carbon removal, nitrification and denitrification, and gets finally discharged into any receiving water.

The solid phase is withdrawn from the bottom of the tank and pumped through an aerobic thermophilic stabilization system. The effluent of that system is dewatered, mixed with lime and finally given to farmers.

CASE STUDY: TREATMENT PLANT AT STEINHORST; SCHLESWIG-HOLSTEIN

SITUATION

The treatment plant which is described in the following is located in the County of Lauenburg, Schleswig-Holstein, about 3 kilometers far from the village of Steinhorst in midst of an agricultural field. The plant was designed to treat the septage collected and hauled from a distance as far as 15 kilometers. The catchment area is 400 square kilometers in size. About 14,000 pleople are living in that area, most of them in small communities, dwellings and single houses. Collection and treatment of the septage is managed by an association through which 35 small communities cooperate.

The plant was designed and constructed in 1982/1983, based on a detailed system management assessment. Since almost no experience on the achievable treatment efficiency was available at that time the State Water Authorities did not set specific effluent limits except for COD but insisted on the application of methods promising accomplishment of advanced carbon removal, nitrification and denitrification.

According to a survey conducted during the planning stage the quantity of septage to be treated was estimated to 36,000 cubic meter per year. The septage was assumed to be hauled on a 250 days per year schedule leading to an estimated flow of 145 cubic meter per day, on the average.

After more than two years of operation these assumption must be corrected to some extent. As slope and position of the linerized frequency distribution function (Fig. 2) reveal that the delivery of septage actually varies quantitatively in a wide range with

the actual flow being only about 70 cubic meter per day, on the average.

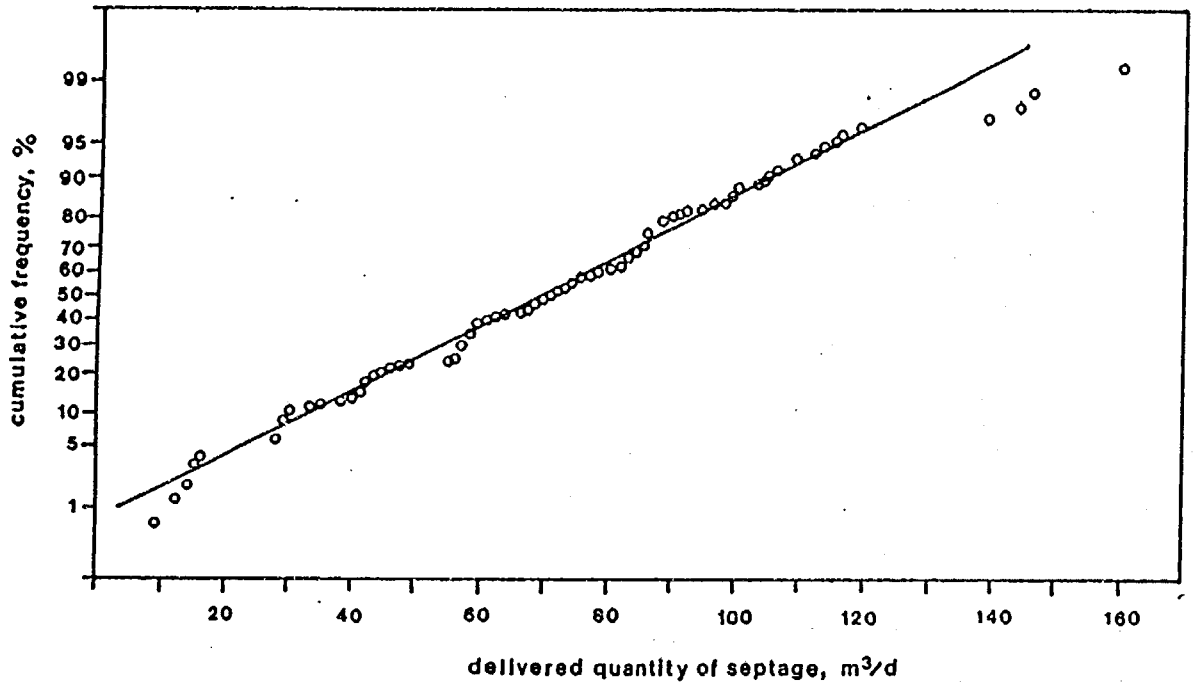


Fig. 2 Linearized frequency distribution function for the delivered septage. Situation at Steinhorst in 1986

The flow variations are caused by the differences in time the drivers actually need to fill the tank of the truck up, and to carry the material to the treatment plant site. The septic tanks and cesspools to be served turned out to be much more different in size than expected. In many cases the tank size was actually much smaller than initially reported by the owners of the houses. As a result, the quantity of septage which can be collected is well below the design value.

DESCRIPTION OF THE TREATMENT PLANT

The treatment plant was designed according to the concept discussed above. It consists of the following elements:

* screen,
* equalization tank
  simultaneously used for gravity separation, referred to in the following as "equalization/sludge thickening tank",
* activated sludge plant,
* polishing pond,
* three stage aerobic thermophilic sludge stabilization plant,
* belt filter press followed by mixing of the dewatered sludge with quick lime,
* storage compartment for the treated sludge.

The flowsheet of the plant is sketched in Fig. 3.

The septage is directly fed from the tank truck into a bar sreen chamber, and flows from there into a pump well of 12 m^3 capacity. The clear spacing between the bars is 25 mm. The pump well was designed to hold the entire load of the tank truck to allow quick drainage, and to minimize the layover time.

From the pump well the liquid is transfered to the equalization/sludge thickening tank. The volume of that tank is 250 m^3. It receives, besides the raw septage, surplus sludge wasted from the activated sludge treatment plant, filtrate of the sludge dewatering process, and sanitary wastewater. The hydraulic detention time is currently about 3 days on the average, but varies in a wide range as the delivered quantity of septage varies. Over the weekend, when the haul service does not operate, the tank gets actually drained to some extent. It should be realized that the flow rates noted in Fig. 3 reflect the design values rather than the situation on the average.

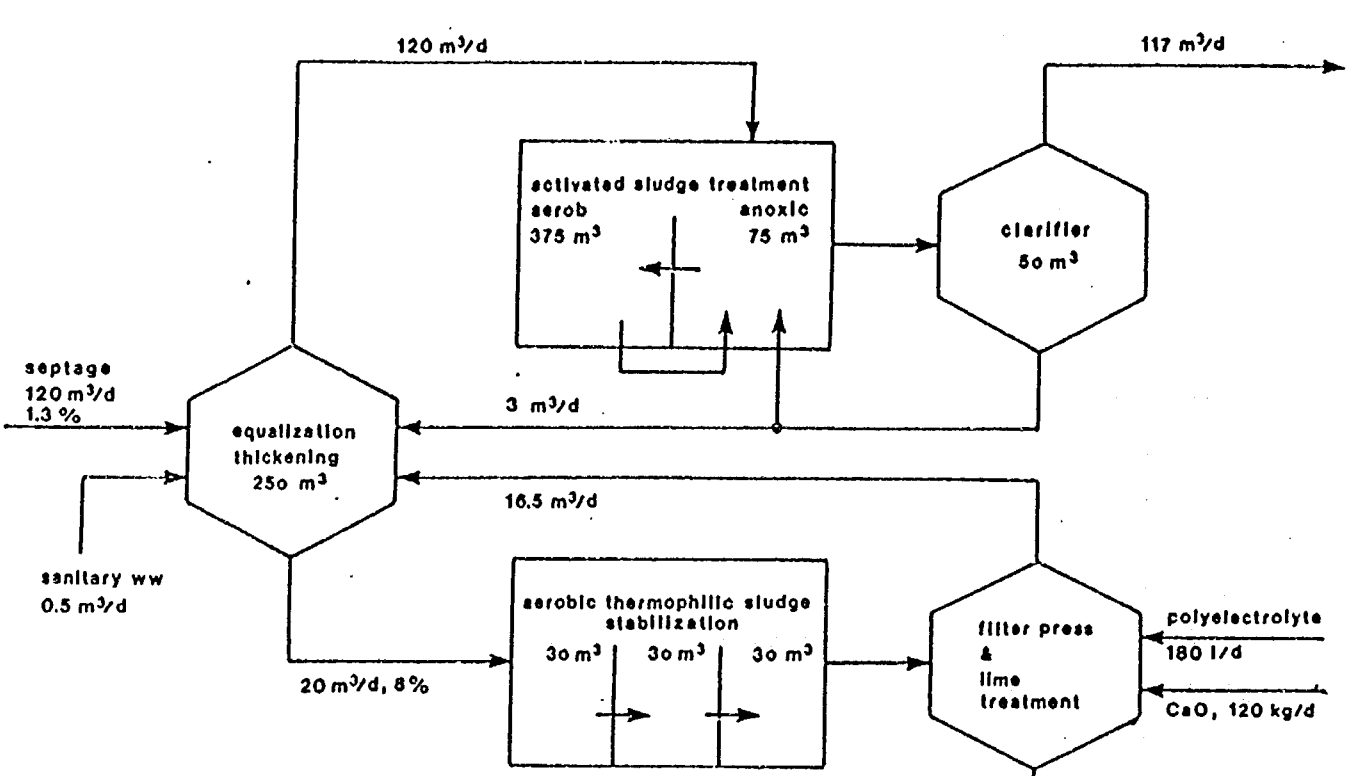


Fig. 2 Flowsheet of the treatment plant at Steinhorst

The equalization/sludge thickening tank was equipped with a rabbling machine to faciliate solid-liquid separation. Sludge which accumulates in the deeper zones of the tank is withdrawn continuously from the bottom of the tank, and is transfered to the sludge stabilization plant. The supernatant is pumped to the activated sludge plant at a rate of 3 to 4 m^3 per hour. To avoid loading of the activated sludge system with excess quantities of suspended solids, which would eventually lead to the washout of nitrifiers and other slow growers through the required increase of the sludge wasting rate, the decant pump is controlled by an ultra-sonic probe to monitor the sludge blanket.

The sludge taken out of the equalization/sludge thickening tank gets aerobically treated. The treatment system consists of three, well insulated, cylindrical reactors which are operated in series

(cascade). Each of the reactors is 30 m^3 in volume. The system was designed to handle a volumetric loading of 20 m^3 per day. The reactors were well insulated to allow recovery of the heat generated by mixing of the viscous fluid (sludge), and by the biological conversion processes. It was anticipated that the temperature would eventually increase to above 55 ^oC in the third tank. As a result, a considerable acceleration of the mineralization processes was expected, rapid die out of pathogenic organisms, and destruction of the viability of seeds.

A belt filter press is employed for dewatering of the sludge. Prior to filtration a cationic polyelectrolyte (Zetag 57, 1% solution) is mixed into the sludge to get a flocculation process initiated and to subsequently faciliate the filtration process. The filter cake is mixed with quick lime at a rate of 6 kg per m^3 of raw sludge. That high dosage was chosen to satisfy the requirements of the farmers interested in applying the treated sludge for agricultural purposes. As a result of lime treatment, the pH and the temperature was expected to increase rapidly leading to further desinfection of the sludge.

The supernatant of the equalization/sludge thickening tank is transferred to a compact activated sludge treatment plant. A cross section of the plant is shown in Fig. 4. It consists of an inner cylinder with a hopper bottom used as a clarifyer, an annular space in the middle used for carbon removal/denitrification, and an outer annular space used as an aeration basin for final carbon removal and for nitrification. The volume of the various compartments is recorded in Fig. 3.

The supernatant is introduced into the denitrification zone of the activated sludge plant. A recirculation pump takes care of the exchange of water and bacteria (mixed liquor) between the aerated and the non-aerated zone of the system. A propeller mixer is submerged in the denitrification reactor to keep the sludge in suspension. In the outer annular space an injector aerator is used to provide oxygen supply, mixing and a sufficient rotation

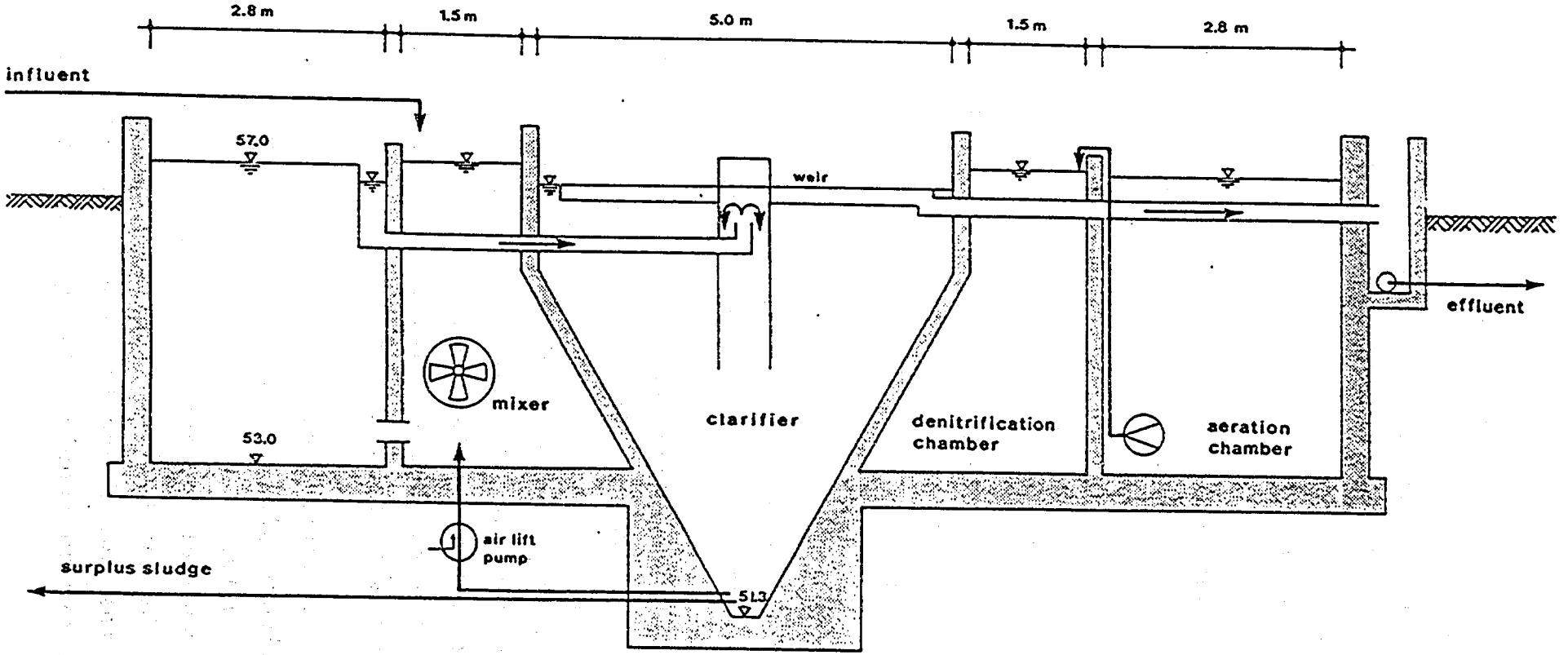


Fig. 4 Treatment plant at Steinhorst: cross section of the activated sludge treatment system

speed of the water body for suspension maintanance. The biomass concentration is maintained at a level of 4 to 5 g/l MLSS. Finally, the mixed liquor flows by gravity to the clarifyer, and from there to a small polishing pond. Sludge is returned to the anoxic zone by an air lift pump. Excess sludge is pumped back to the equalization/sludge thickening tank.

RESULTS

The septage management system described above including the special treatment plant has been in operation since September 1983. Table 2 summarizes the analytical result for the effluent decanted from the equalization/sludge thickening tank for two observation periods, one in 1984 and the second one in 1986.

Compared with the results of the survey summarized in Table 1 the parameter values, in particular the values for the filtered samples appear to be significantly reduced, particularly during the observation period in 1986. That effect can hardly be explained by the effectiveness of the solid-liquid separation process alone. On top of that it must be assumed that as a result of regular drainage of the septic tanks the composition of the collected material has been changed. Further more, it should be realized that the content of the equalization/sludge thickening tank gets regularly diluted to some extent by the filtrate of the dewatering system which is recovered from an aerobically and physicochemically treated sludge. In effect, not only the concentration of the sludge thickening tank supernatant, most remarkably the phosphorus concentration is actually much less than anticipated but also the composition of the supernatant, expressed in terms of the carbon to ammonia nitrogen ratio, for instance.

Table 2 Characteristics of the septage supernatant recovered at Steinhorst

observation period		suspended solids mg/l	COD mg/l	FCOD mg/l	TOC mg/l	FTOC mg/l	TKN mg/l	NH_4-N mg/l	total P mg/l	C : N
1984	min	400	960	692		260	113	81	41	2.1
	max	2,300	3,240	1,080		385	206	116	69	3.8
	mean	825	1,415	800		308	144	99	50	2.8
1986	min	193	1,156	329	196	122	97	65	0.5	1.7
	max	3,430	4,820	1,625	1,420	594	283	154	28.5	7.1
	mean	977	2,040	1,040	680	409	148	93	5.4	4.7

FCOD: filterable COD; FTOC: filterable TOC;

Solid-liquid separation by gravity sedimentation (sludge thickening) turned out to be a difficult task. The solids do actually not settle out readily unless the raw septage is significantly diluted with the filtrate of the filter press. The efficiency of the sedimentation process turned out to be dependent on the extent the capacity of the filtration process is exploited. This, in turn, depends on how efficiently the aerobic sludge stabilization process works.

The aerobic thermophilic sludge stabilization process proved to be the rate limiting step, in this context. Despite of the application of a sophisticated insulation the temperature in the reactors did often not increase as much as expected, especially during cold weather periods. As a result, the volumetric loading of the reactor cascade had to be reduced to achieve the desired stabilization state. In turn, sludge accumulated in the equalization/sludge stabilization tank, very much to the disadvantage of the quality of the supernatant to be treated in the activated sludge system. The suspended solid concentration increased to non-acceptable levels. Loss of the nitrifier population due to the subsequently required increase of the sludge wasting rate had to be suspected. Fortunately, lime treatment of the raw sludge turned out to be sufficient to meet the hygienic requirements of the agriculture. Thus, the system could actually be operated efficiently no matter whether or not the biological sludge stabilization process functioned properly.

Typical characteristics of the treated sludge are:

* solid content	35	to	40	%
* organic content	20	to	30	%
* nitrogen	4	to	6	kg/m^3
* phosphorus as $P_2 O_5$	7	to	10	kg/m^3
* lime as Ca O	80	to	130	kg/m^3

No problems were encountered associated with the activated sludge treatment process. The sludge settles very well. A sludge volume

index value of 94 ml/g MLSS characterizes the situation. The effluent of the clarifyer is mostly very clear but yellowish in color.

The treatment results obtained so far are summarized in Tabel 3. During summer, almost complete nitrification/denitrification is achieved. The nitrification capacity deteriorates gradually during winter, however. Nevertheless, the year round average values for both ammonia and nitrate nitrogen are satisfying.

Table 3 Characteristics of the treatment plant effluent

observation period		suspended solids mg/l	COD mg/l	NH_4-N mg/l	NO_2-N mg/l	NO_3-N mg/l
1985 (all year)	mean		80	5	< 0.1	5
1986 (summer)	min	6	54	< 1	< 0.1	< 1
	max	41	70	5.1	0.3	3.4
	mean	19	58	2.3	0.1	2.2

OPTIMIZATION STUDY USING SEQUENCING BATCH REACTOR TECHNOLOGY

OBJECTIVES

Encouraged by the results obtained from the full scale plant at Steinhorst it was decided to conduct a research project to determine the minimum hydraulic detention time requirements for the biological treatment of septage supernatant, and to develop proper process strategy recommendations. The need for a study on this topic resulted of fact that no systematic information was available on how to deal with a type of wastewater characterized by

* high concentration of suspended solids,
* high concentration of soluble and colloidal organics,
* high concentration of organics, natural in origin but poor in biodegradability such as humic acids,
* high concentration of ammonia nitrogen.

The Sequencing Batch Reactor (SBR) was chosen as a model system for these studies because of the convenience the SBR technology offers to the investigator. The duration of the various process phases of concern, and the intensity of the factors responsible for controlling the composition and the properties of the activated sludge biocommunity can be easily altered without the need of getting physical arrangements changed, in particular the volume of the various tanks of a continuous flow activated sludge treatment cascade. The system does not depend on clarifyer hydraulics, and on the efficiency of the return sludge transportation system.

Objective of the study was to determine not only the overall detention time requirements to achieve comparable treatment

results as with the full scale plant at Steinhorst but also to develop proper process strategies focused on the process components: carbon conversion, nitrification, denitrification, and sludge separation.

In the following the results obtained so far are summarized. The project has not entirely been accomplished yet so that only aspects can be discussed. The results presented below are preliminary in character.

DESIGN AND OPERATION CONSIDERATIONS

SBR technology is a modification of the activated sludge technology. It is characterized by planned filling and drawing of tanks in which first biological conversion takes place, then gravity sedimentation of the activated sludge. Biological conversion and sludge sedimentation take place in a time sequence rather than in a spatial sequence as in continuous flow systems.

The process phase dedicated to biological conversion has to be subdivided into several stages, when specific tasks such as anaerobic conversion, aerobic conversion including nitrification, and denitrification are to be accomplished. To establish proper process conditions continuous flow system must be designed as a reactor cascade. In Sequencing Batch Reactors the various processes are accomplished in a time sequence in one single tank, basically.

Both technologies have in common that microbial communities (activated sludge) are developed under periodically changing environmental conditions. The activated sludge organisms experience periodically very different environmental conditions

as they are recirculated in a continuous flow activated sludge system, or from cycle to cycle in the SBR, respectively.

One of the major advantages the SBR provides is that the time interval during which a particular environmental factor is effective can be chosen and maintained irrespectively the flow and loading variations in the treatment plant influent. It is easily possible to control the frequency of occurance of distinct environmental conditions as well as their intensity. Since the organism type distribution and the resulting metabolic capacity of the microbial community is directly affected by these factors the biological system developing in SBRs can be controlled in a much more straightforeward manner than in continuous flow systems.

When nitrogen removal is required, a strictly foreward oriented process schedule is problematic, however. Microorganisms employed to accomplish reduction of nitrate and nitrite to nitrogen gas (denitrification) depend on the availability of organic substances which can be used as electron donors. If the commencement of the denitrification process is postponed until ammonia is completely converted to nitrate, the extracellular and intracellular pool from which electron donors can be drawn has already been emptied, and the process proceeds at a very slow rate (Fig. 5).

To speed the denitification process up, a second fill strategy at Sequencing Batch Reactors may be considered advantageous, or introduction of raw wastewater into the ultimate denitrification zone of a continuous flow system, respectively. As demonstrated with Fig. 6 (Wilderer and Schroeder, 1986) this strategy - amplified by the temperature increase in this example - leads to a significant increase of the denitrification rate. Since raw wastewater does not only contain organic carbon compounds, however, but also nitrogen compounds the treatment process ends up with an non-tolerable high ammonia concentration. Internal recirculation techniques, on the other hand, or iterative

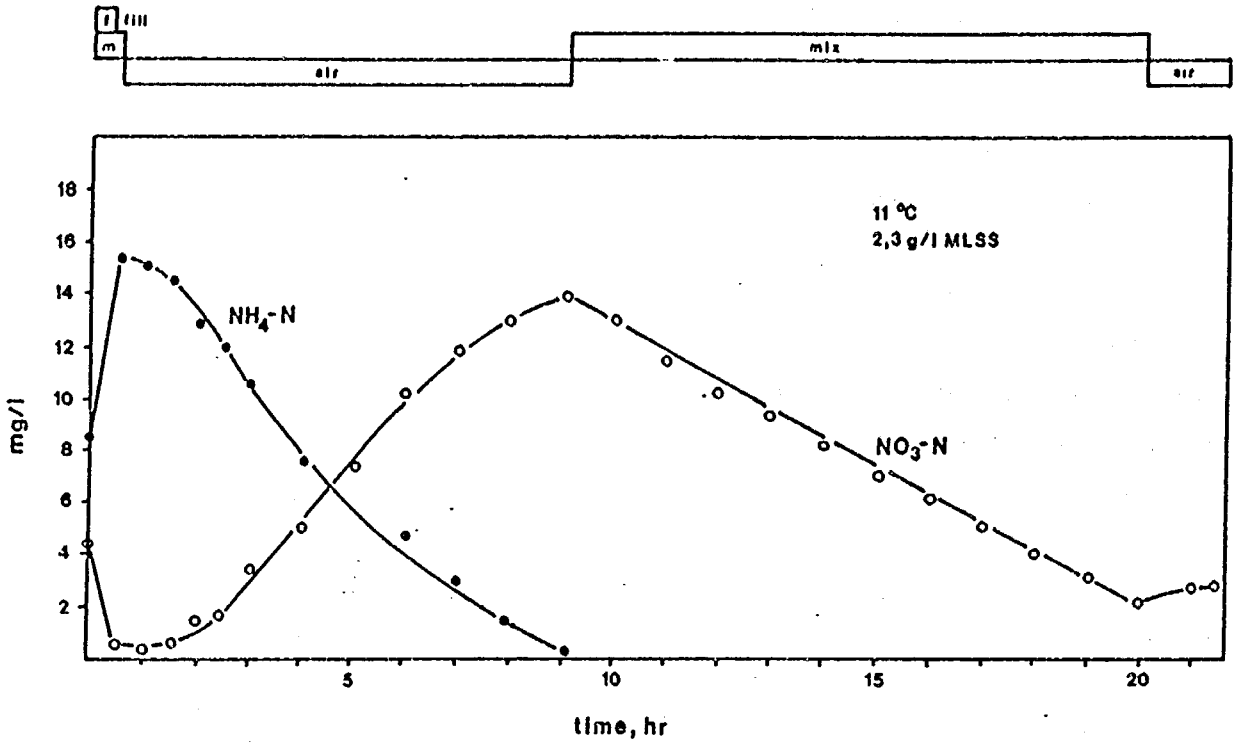


Fig. 5 SBR-experiment with municipal wastewater: effect of a strict sequencial process strategy on the denitrification rate. Concentration profiles for the parameter ammonia-nitrogen and nitrate-nitrogen. Source: Wilderer and Schroeder, 1986

aeration in Sequencing Batch Reactors (Fig. 7) hardly improve the system`s performance unless a well dosed portion of raw wastewater is introduced at the beginning of each denitrification phase. Exactly that is the technique which was applied at Steinhorst (Fig. 4), appearently with great success. It was, therefore, decided to translate that technique into a respective reactor design, and into a process schedule for Sequencing Batch Reactors.

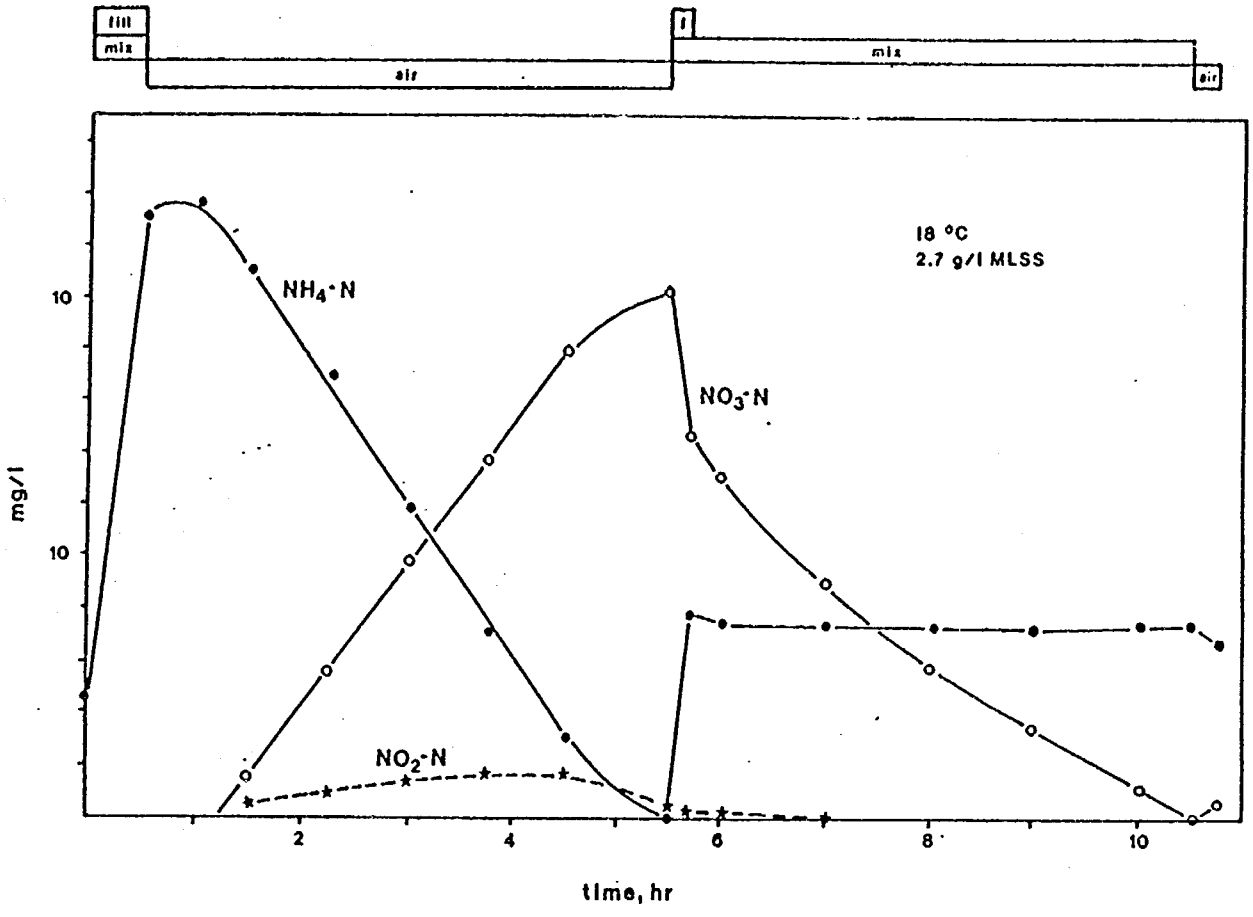


Fig. 6 SBR experiments with municipal wastewater: effect of a second fill strategy on the denitrification rate (first fill quantity: 80 %, second fill quantity: 20 %). Concentration profiles for the parameter ammonia-, nitrite- and nitrate-nitrogen. Source: Wilderer and Schroeder, 1986

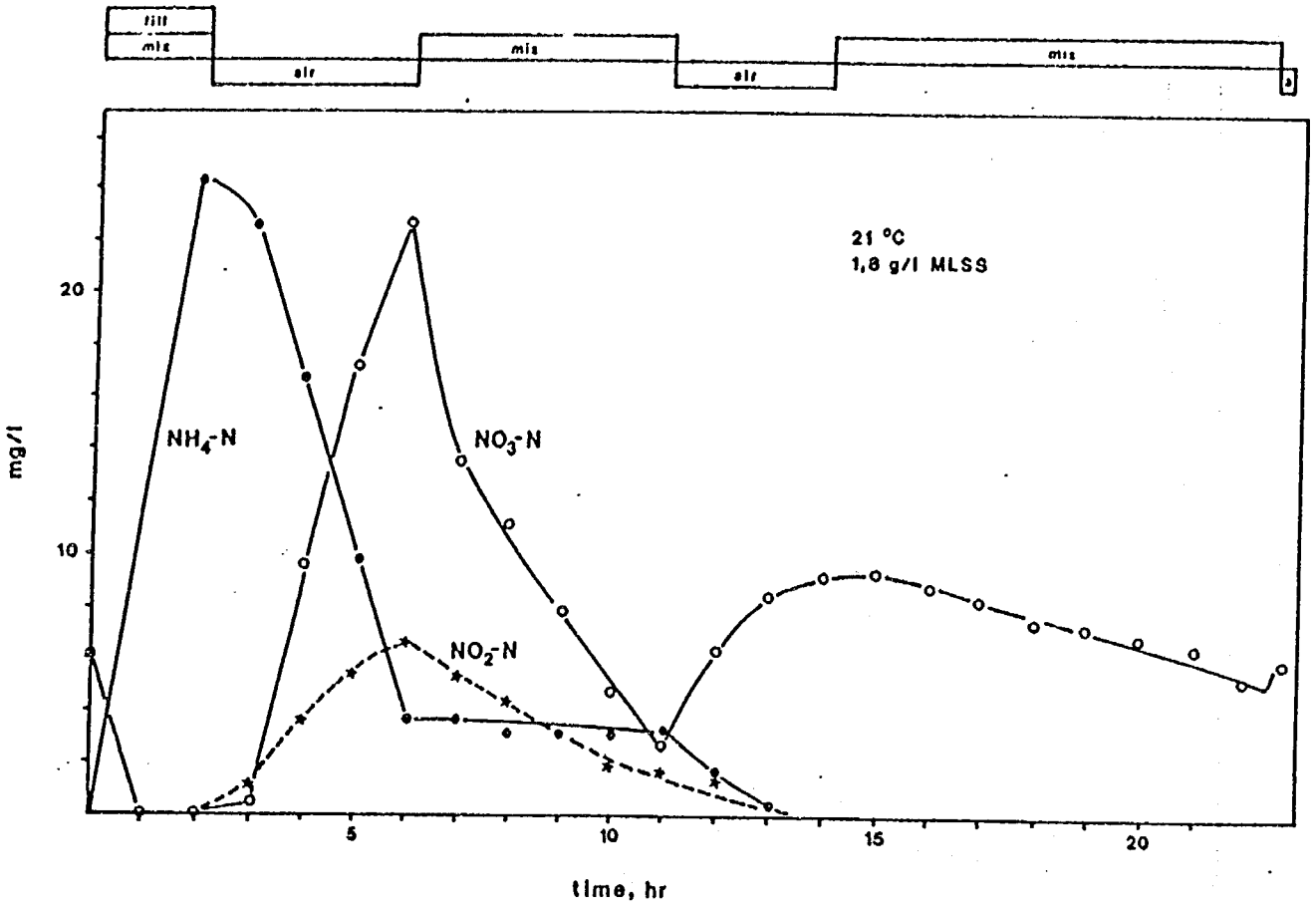


Fig. 7 SBR experiments with municipal wastewater: effect of an iterative aerobic/anoxic process strategy on the denitrification rate. Concentration profiles for the parameter ammonia- nitrite- and nitrate-nitrogen. Source: Wilderer and Schroeder, 1986

DESCRIPTION OF THE PILOT PLANT

The cross section of the reactor used for the pilot studies is shown in Fig. 8. Chosen was a cylindrical reactor form with a hopper bottom. The depth of the tank was 3 meters, the diameter 2 meters, and the working volume 10 m^3. The reactor was divided into an inner cylinder of 4 m^3 volume surrounded by an annular space of 6 m^3 volume. Both compartments were hydraulically connected by a number of openings arranged in different depths to allow, after sedimentation of the sludge, simultaneous drainage of the supernatant without getting the settled sludge resuspended. For recirculation, the mixed liquor was withdrawn from the bottom of the inner cylinder, and pumped into the annual space using a variable speed pump to control the mean hydraulic detention time in both sections of the reactor.

Oxygen supply of the microorganisms was accomplished by means of diffused air aerators. Fex-air diffusers, i.e. slitted tubes coated with plastic rubber sleeves, were mounted at the bottom of both reactor compartments. Normally, only the annular space was aerated. The aerator in the inner cylinder was only turned on prior to the commencement of the sedimentation phase to strip nitrogen gas bubbles. A submerged propeller mixer was used in the inner cylinder for mixing and for keeping the activated sludge in suspension.

Supernatant was withdrawn from the equalization/sludge thickening tank of the treatment plant at Steinhorst and pumped into the inner cylinder of the SBR. Based on the results of a mathematical simulation model, and based on experiments conducted in the laboratory the following operation strategy was chosen:

* duration of the various process phases

- entire cycle	12	hrs
- fill I (1 m^3) instantaneously	10	min

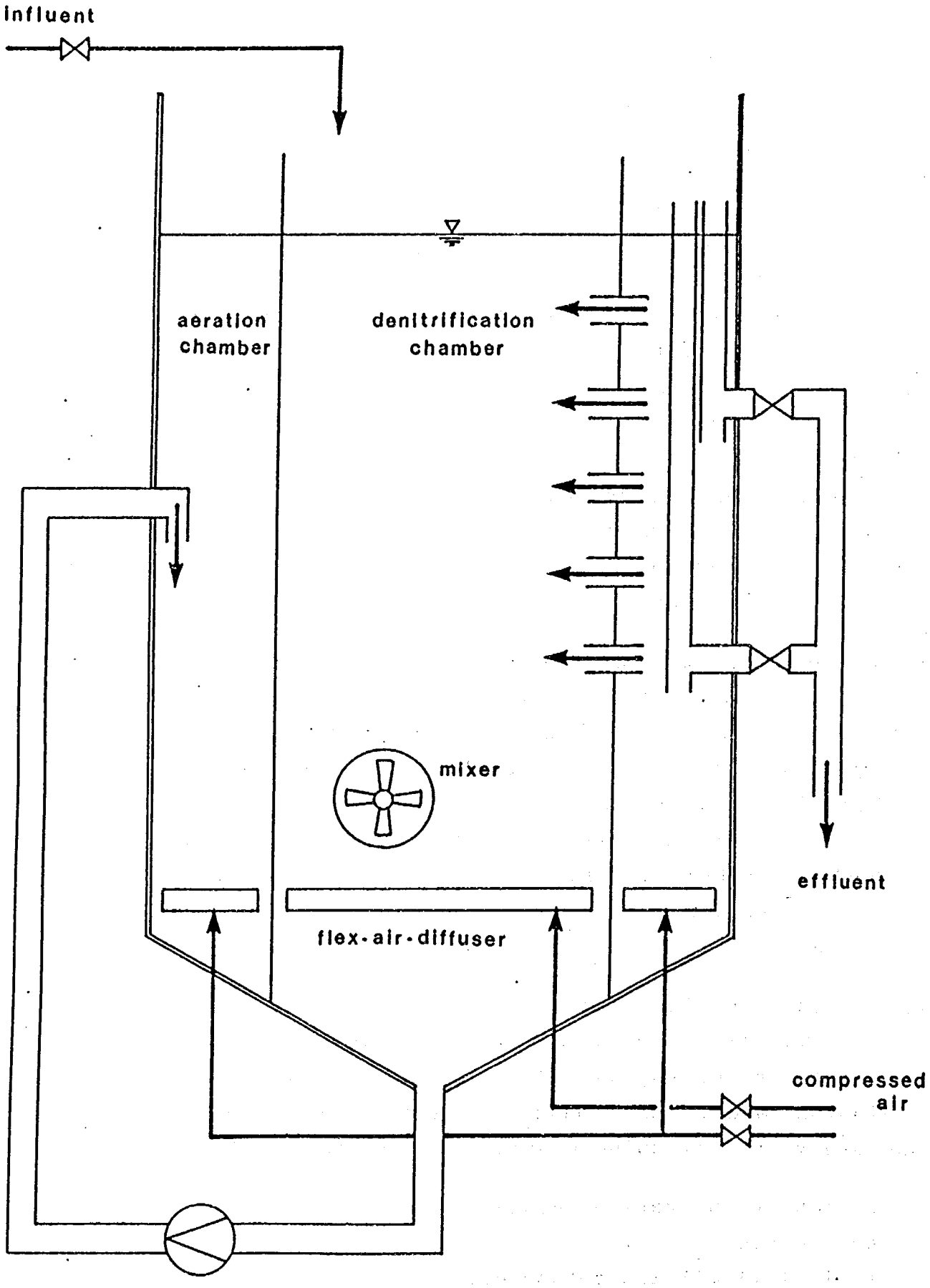


Fig. 8 Sketch of the SBR pilot plant

- fill II (1 m^3) intermittently, 0.5 hrs intervall	5.5	hrs
- aeration, annular space, medium rate	10	hrs
- aeration, annular space, high rate	2.3	hrs
- aeration, inner cylinder, medium rate	0.3	hrs
- mixing, inner cylinder	10	hrs
- sedimentation	1.5	hrs
- drainage (1.8 m^3)	0.5	hrs
* reactor volume	9.5	m^3
* fill volume per cycle	2	m^3
* recirculation rate	4	m^3/hr
* excess sludge withdrawal per cycle	0.2	m^3
* decant volume per cycle	1.8	m^3
* hydraulic detention time	2.4	days
* sludge age	25	days

The time schedule in detail is layed out at the top of Fig. 10. The simulation model and the result of the laboratory experiment will be presented elsewhere.

The pilot plant was automatically controlled by a computer timer. No operator assistance was required except for taking samples.

Because of the incorporation of an inner cylinder into the reactor there was no room left for a floating decanter to move. As an alternative, a fixed decanter was developed (Wilderer and Ohles, 1986) and established in the reactor (Fig. 8 and 9). The novel decanter offers the following advantages over floating decanters:

* it occupies only very little space;
* there are no moveable parts inside of the reactor to be serviced;
* flow can be controlled by stepwise opening of the valves;
* the effluent quality is not affected by scum withdrawal, since the supernatant is taken from underneath the water surface;
* the effluent is not affected by the entrappment of activated sludge inside of the pipings; the sludge settles automatically out during the sedimentation phase.

The decant system worked without any problem. The concentration of the effluent suspended solids listed in Table 4 are identical with the concentration of samples taken directly from the reactor supernatant.

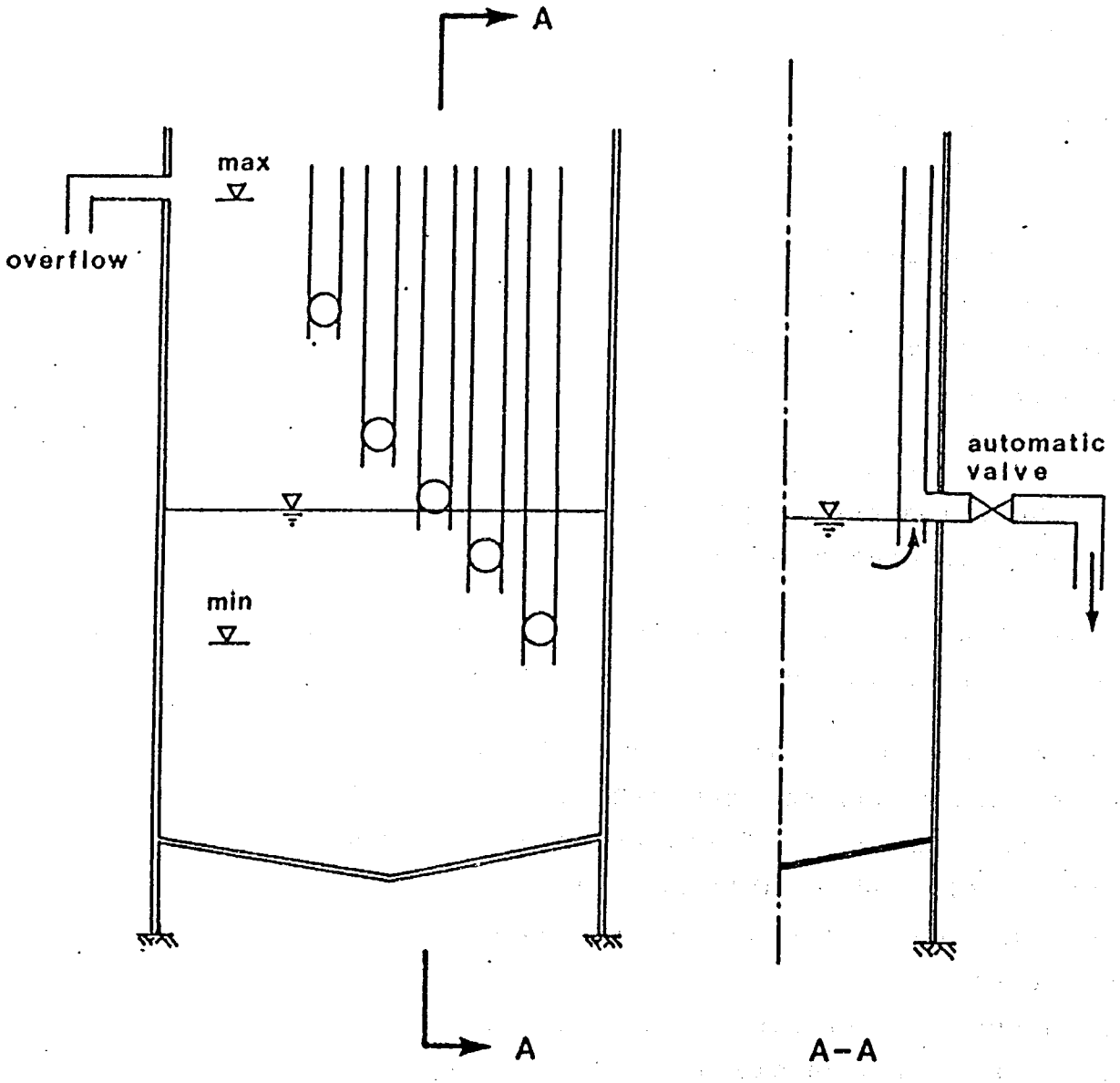


Fig. 9 Sketch of the decant system

RESULTS

During a one year test period various operation strategies were investigated. The one described above proved to be most effective with regard to both effluent results and reactor performance. Prolongation of the hydraulic detention time did not lead to a further improvement of the treatment results. On the other hand, any substancial decrease of the hydraulic detention time by increasing the fill ratio was not feasible because of foam problems which developed but could not be handled without the risk of creating oxygen deficiency problems.

Fig. 10 and 11 contain profiles characterizing the performance of the reactor. The profiles were taken during any one of the process cycles. The temperature of the water was 23 ^{o}C, the biomass concentration at the end of fill phase II was 2.8 g/l MLSS, and the organic content of the activated sludge 84 per cent.

The treatment results are summarized in Table 4. Compared to the results obtained from the full scale continuous flow plant for the same time period (Table 3) almost identical treatment could be achieved. Only the nitrite concentration was significantly higher, occasionally.

Considering a Sequencing Batch Reactor would be chosen to biologically treat septage supernatant at Steinhorst only 170 m^3 of tank volume would be required. As compared to volume of the existing activated sludge treatment plant (450 m^3 tank volume for the biological reactor plus 50 m^3 for the clarifyer) that makes about one third of the tank volume actually being required. Respective savings of both investment and operation costs are possible.

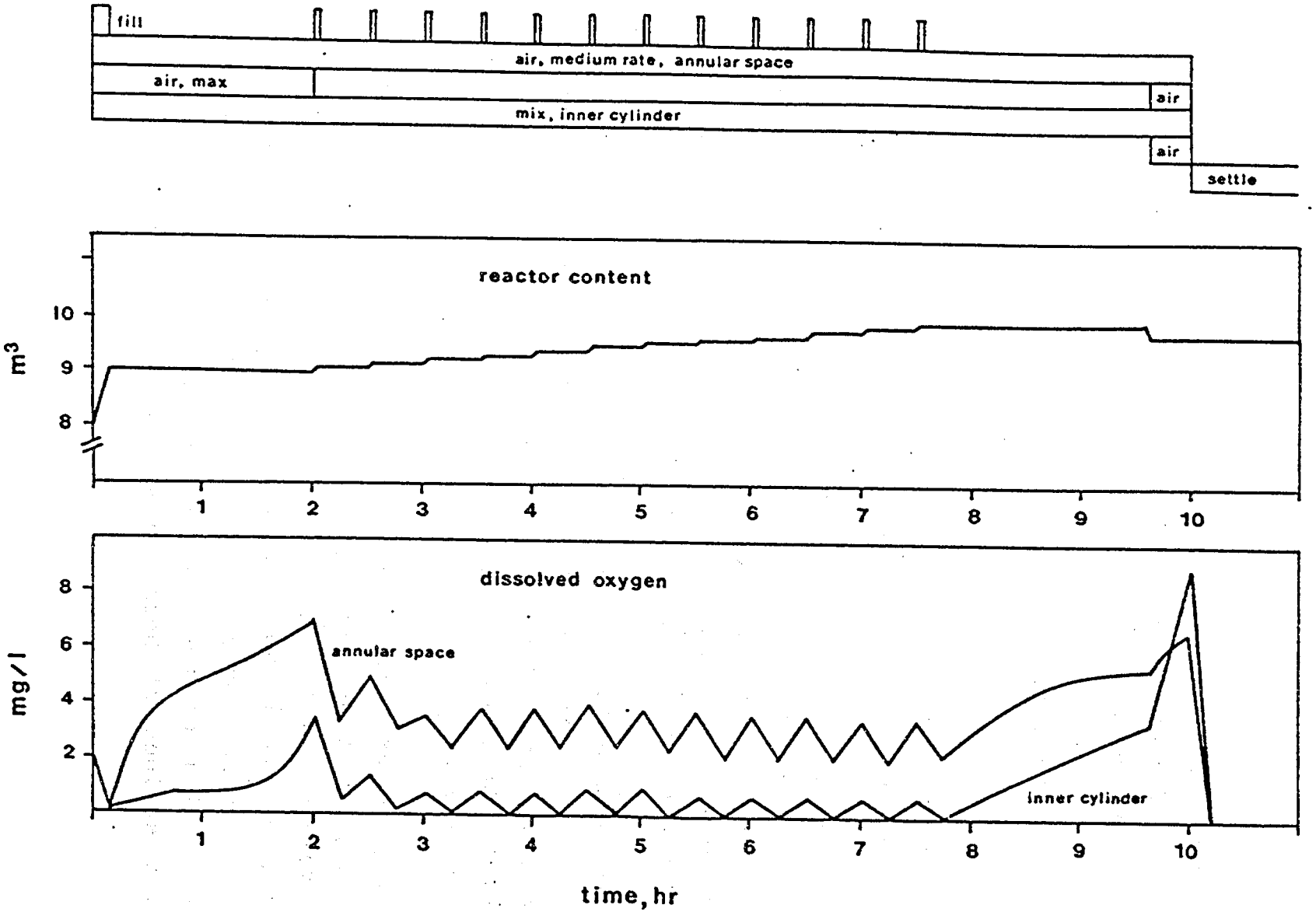


Fig. 10 SBR pilot studies: process schedule, and profiles for the parameter: volumetric content of the reactor and dissolved oxygen concentration in both reactor compartments.

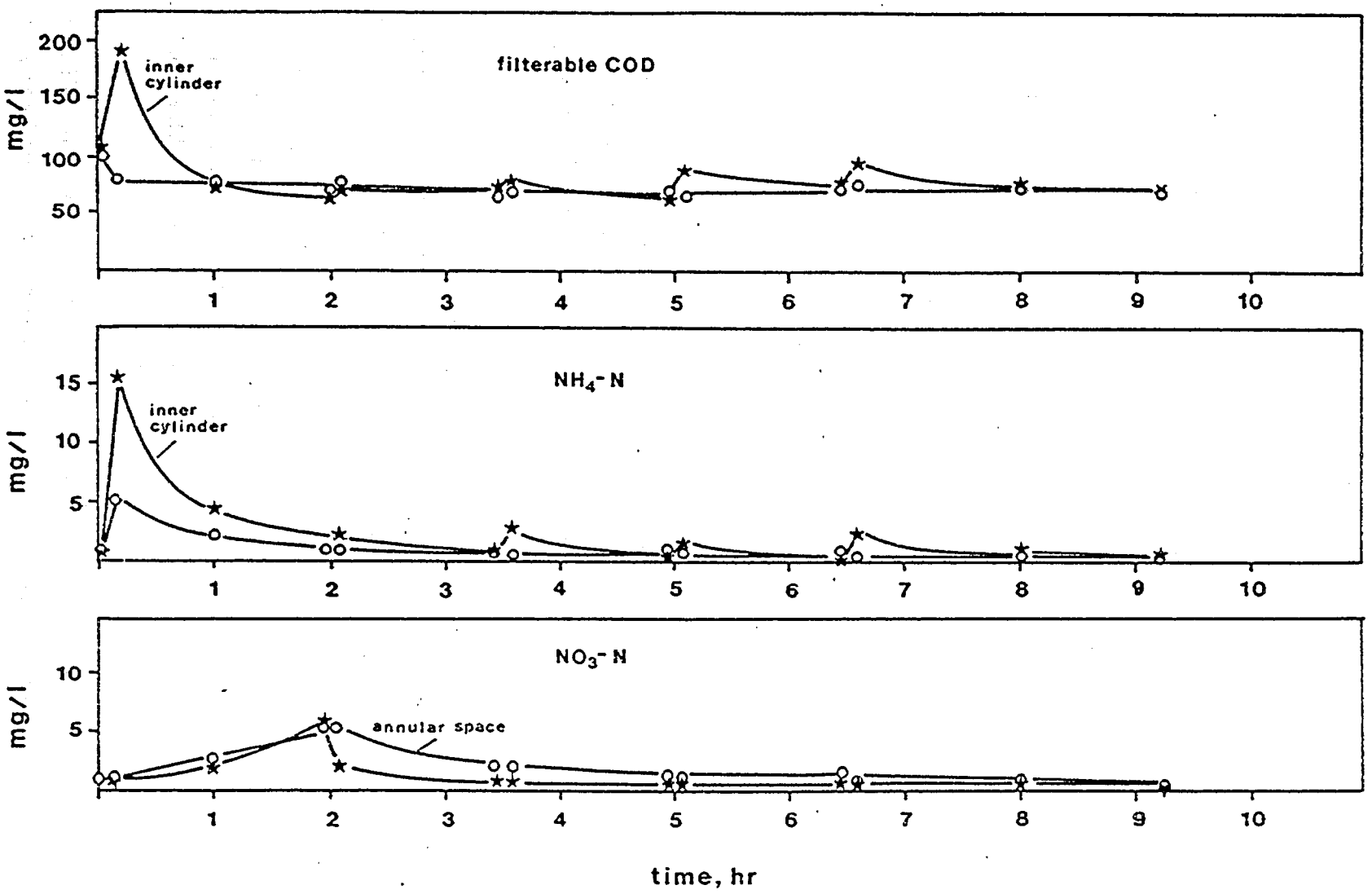


Fig. 11 SBR pilot studies: concentration profiles for the parameter COD, ammonia nitrogen and nitrate nitrogen. Influent FCOD: 980 mg/l; influent NH_4-N: 130 mg/l

Table 4 Characteristics of the pilot plant effluent

observation period		suspended solids mg/l	COD mg/l	NH_4-N mg/l	NO_2-N mg/l	NO_3-N mg/l
1986	min	8	51	< 1	< 0.1	< 1
(summer)	max	35	83	1.3	6.6	4.5
	mean	21	68	< 1	< 0.1	1.2

SUMMARY AND CONCLUSIONS

The results and observations which have been made in the context of the project described herein led to a number of conclusions regarding collection and treatment of wastewater from areas low in population density.

It could be successfully demonstrated that separate treatment of septage is effectively feasible. There is no need for dilution of septage with regular wastewater or waste sludge to get septage treatment accomplished. Collection of wastewater by tank truck, transportation to and treatment in special treatment plants appears to be an interesting alternative to construction and management of a long distance sewer systems, and operation of conventional wastewater treatment systems to treat the delivered wastewater.

Three major processes must be combined to get septage efficiently treated: solid-liquid separation/dewatering as a primary treat-

ment step, stabilization/hygienization of the solid phase, and biological treatment of the liquid phase. The treatment plant should provide plenty of storage capacity for the raw septage to equalize irregularities in the delivery of septage, and to allow operation of the various treatment processes consequently according to the process specific needs.

The effectiveness of solid-liquid separation proved to be crucial for the whole system. Gravity sedimentation is possible when operated in combination with mechanical dewatering, and recirculation of the filtrate for dilution. It is recommended to provide plenty of sludge dewatering capacity to allow effective dilution of the raw septage, and to avoid, subsequently, any restrictions to the entire system.

The treated sludge proved to be high in value for farmers, in particular because the sludge was supplemented with plenty of lime.

Lime treatment of the sludge proved to be very effective in getting the sludge ultimately desinfected. The aerobic thermophilic sludge stabilization process, on the other hand, was less reliable than expected. Both processes in combination provide satisfactory results.

Advanced biological treatment of septage supernatant including nitrification and denitrification proved to be easily possible, as long as the loading of the system with suspended solids was kept under control. Effluent values of less than 80 mg/l COD, less than 5 mg/l NH_4-N, and less than 5 mg/l NO_2/NO_3-N can be achieved without problem, year round. During summer almost complete nitrogen removal was demonstrated. Based on the results obtained from the Sequencing Batch Reactor experiments a 2.5 days hydraulic detention time, and a sludge age of 25 days appear to be fair design parameters.

ACKNOWLEDGEMENT

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PERFORMANCE EVALUATION OF SEQUENCING BATCH REACTOR FOR MUNICIPAL WASTEWATER TREATMENT

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SUMMARY

The sequencing batch reactor (SBR) is a fill and draw activated sludge system in which a cycle consisting of four discrete periods, i.e. Fill, React (Aeration), Settle, and Draw is repeated within a single or multiple reactors. The SBR system has been little utilized in Japan and only one plant with a capacity of 600 m^3/day is being operated. However, two new plants are currently under construction, and the use of the SBR system in small-scale wastewater treatment plants is expected to increase in the future.

Because of this, the Japan Sewage Works Agency has conducted both bench-scale and pilot-scale studies in order to clarify the design conditions and operation control method of the SBR system.

This report summarizes the removal performance of organic matter, nitrogen and phosphorus, and design conditions of the SBR system on the basis of the results of the bench-scale and pilot-scale survey.

Moreover, the characteristics and functions of newly developed supernatant discharge devices and a built-in sludge thickener in the reactor are also presented.

1. INTRODUCTION

The sequencing batch reactor (SBR) is a fill and draw activated sludge system in which a cycle consisting of four discrete periods, i.e. Fill, React (Aeration), Settle, and Draw is repeated within a single or multiple reactors. The configuration and operating method of the SBR system differ from those of the continuous flow activated sludge system widely used in wastewater treatment; so the SBR is said to have the following advantages:[1)]

(1) Fluctuations in the quantity and quality of influent wastewater with time are averaged out in the Fill Period.

(2) Both the anaerobic and aerobic conditions can be alternately established within a cycle in a reactor, attaining a biological removal of nitrogen and/or phosphorus depending on the design and operating conditions.

(3) Since the sedimentation of mixed liquor solids is made in a stationary state, stable solid-liquid separation can be obtained.

(4) Separate final clarifiers and return sludge equipments are not required and therefore the configuration of facilities can be simplified when compared to the continuous flow activated sludge system, with less effort being necessary for operation and maintenance.

(5) Settling characteristic of mixed liquor solids can be improved.

Recently in the area of municipal wastewater treatment, the SBR has been gaining much attention, and several full-scale facilities have been built and are in operation as a result of the research and development on the sequencing batch activated sludge system by Goronszy[2)] of Australia

(Intermittent Cycle Extended Aeration System) and Irvine et al.[3] of the U.S. (Sequencing Batch Reactor).[4]

In Japan, the SBR system is being widely utilized for the treatment of industrial wastewaters such as food processing wastes. On the other hand, this system has been little used for municipal wastewater treatment, except the 600 m^3/day SBR plant in Maizuru City. However, two new SBR plants are now under construction, and the use of the SBR system for small-scale municipal wastewater treatment plants is expected to increase in future.

Therefore, the Japan Sewage Works Agency has conducted both bench-scale and pilot-scale studies in order to clarify the design conditions and operation control methods of SBR system for municipal wastewater application.

In addition, the Agency has newly developed supernatant discharge devices and a built-in sludge thickener in reactors suited to the SBR system and has also performed the confirmation of their functions.

2. EXPERIMENTAL APPARATUS AND OPERATIONAL CONDITIONS

2-1 Outline of Experiments

There are several modifications of SBR system depending on the feed method of the wastewater, loading of organic matter, configuration of the reactor and so forth. For instance, SBR can be classified into a single tank configuration and a multiple tank configuration. This report is specifically for the two tank system in which the influent wastewater is alternately received by two reactors and where one-half of the time required for one cycle is generally spent in the Fill Period as shown in Fig. 1.

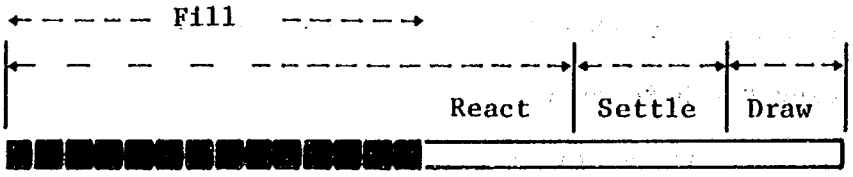


Fig. 1 SBR operation for one cycle

The time required for one cycle is the sum of the time required for the periods of Fill, React, Settle and Draw. These four discrete periods are determined as:

(1) Fill

Fill period is determined by the number of reactors per system.

(2) React (Aeration)

React period is determined by the concentration of MLSS, the necessary removal ratio of BOD_5 and the degree of nitrification.

(3) Settle

Settle period is determined by the concentration of MLSS, settling velocity of mixed liquor solids, and the withdraw ratio of the supernatant.

(4) Draw

Draw period is determined by the overflow rate of the supernatant discharge device, minimum clearance between water surface and sludge zone, and the capacities of the chlorine contact tank and effluent conduit.

In the continuous flow activated sludge system, the BOD-SS loading is defined by equation (1).

$$\text{BOD-SS loading} = \frac{Q_S \cdot C_S}{C_A \cdot V} \quad (\text{kg-BOD}_5/\text{kg-SS} \cdot \text{day}) \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (1)$$

where,

V : Volume of aeration tank (m^3)

Q_S: Quantity of influent wastewater (m^3/day)

C_A: Mean MLSS concentration in aeration tank (mg/l)

C_S: Mean BOD concentration of influent wastewater (mg/l)

For the SBR system, it is necessary to define the BOD-SS loading by considering the time of aeration as the reaction time. That is, if the aeration time ratio (the ratio of aeration time in one cycle to the time required for one cycle) is e ($0 < e < 1$), then the BOD-SS loading can be given by

$$\text{BOD-SS loading} = \frac{1}{e} \cdot \frac{Q \cdot C_S}{C_A \cdot V} \quad \cdots\cdots (2)$$

Further, if the withdraw ratio (the ratio in volume between the quantity of supernatant withdrawn in a cycle and the volume of mixed liquor after the end of the Fill Period) is 1/m, and the number of cycles per day is n, then the relation of $Q_S = nV/m$ stands good. Thus, equation (2) is rearranged as:

$$\text{BOD-SS loading} = \frac{1}{e} \cdot \frac{n}{m} \cdot \frac{C_S}{C_A} \quad \cdots\cdots (3)$$

The range of operational conditions defining BOD-SS loading in the bench-scale and pilot-scale reactor experiments is as shown in Table 1.

The experiments used sewage from the Ara River Wastewater Treatment Plant adjacent to the Research and Training Center of the Japan Sewage Works Agency.

The experiments were conducted from October 1983 to December 1984 for bench-scale experiments and from October 1984 to March 1986 for pilot-scale experiments.

Table 1 Range of operational conditions

Operational conditions	Bench-scale	Pilot-scale
Kind of Inflow	Primary clarifier effluent	Grit chamber effluent
BOD-SS loading (kg-BOD_5/kg-SS · day)	0.075∿0.65	0.055∿0.37
Aeration time (h/cycle)	0.5∿10.5	1.0∿9.0
Withdraw ratio (-)	1/4,1/2	1/4 1/2
Number of cycles (cycles/day)	1,2,3,4,6	2,3,4
MLSS concentration (mg/l)	1,000∿4,000	2,000∿4,000

2-2 Bench-Scale Experiments

Shown in Fig. 2 is the bench-scale experimental apparatus. The reactor is a transparent cylindrical vinyl chloride tank with a capacity of 60 liters and containing an agitator and air diffuser.

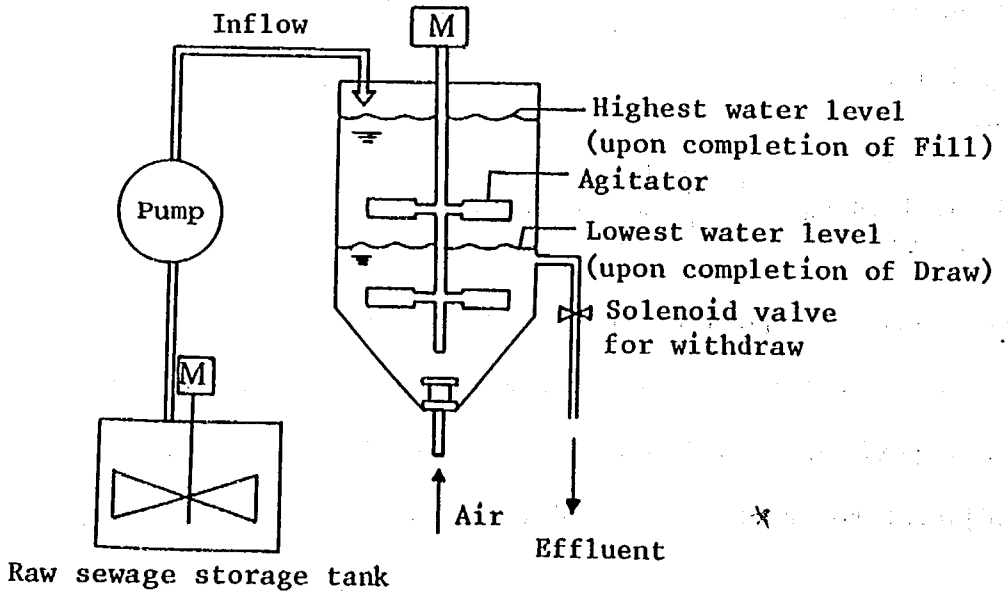


Fig. 2 Bench-scale experiments apparatus

The concentration of MLSS in the reactor was properly maintained by withdrawing the excess sludge once a day after the discharge of the supernatant. The experiment was performed in a thermostatic chamber at 20°C. The water temperature in the reactor during experiment was in the range of 15°C to 21°C. The operational conditions for the bench-scale experiments are shown in Table 2.

A settling time of 1 hour and a withdraw time of 30 minutes were used in all operation modes. The three kinds of reaction time as shown in Fig. 3 were set.

The mean BOD-SS loading in each run was in the range of 0.075 to 0.65 kg-BOD_5/kg-SS · day.

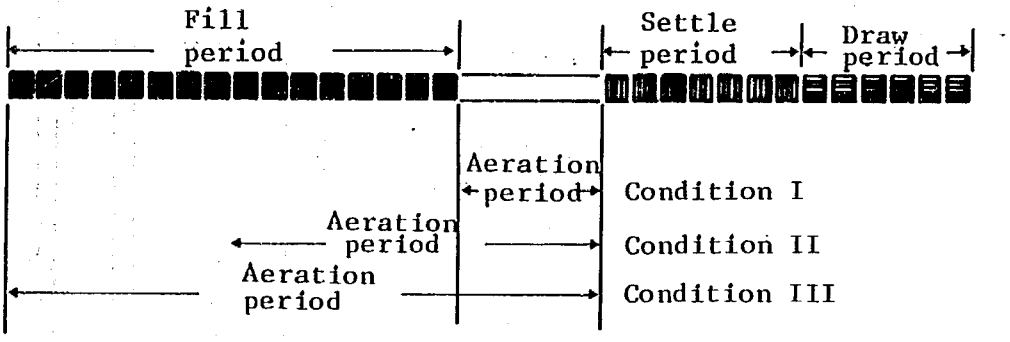


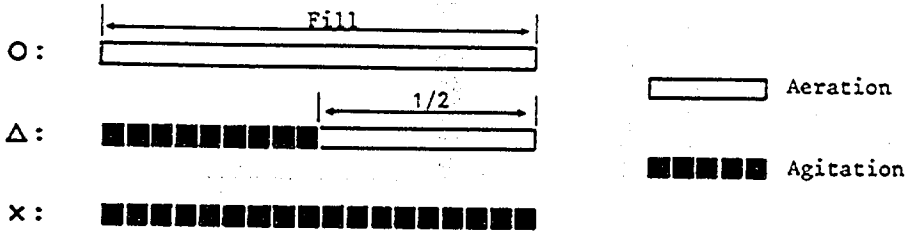
Fig. 3 Aeration Conditions

Table 2 Operational conditions of bench-scale experiment

Run	Operation mode					Quantity of influent (l/day)	Influent BOD_5 (mg/l)	MLSS (mg/l)	BOD-SS loading (kg-BOD_5 / kg-SS · day)	Air volume (l/min.)	Number of days of operation (days)
	MLSS (mg/l)	Number of cycles (cycles/day)	Withdraw ratio	a) Aeration conditions	Aeration time (h/cycle)						
A	1,000	1	1/2	×	10.5	30	72	1,100	0.075	0.35	40
B	1,000	2	1/2	×	4.5	60	99	1,160	0.23	0.81	38
C	3,000	3	1/2	×	2.5	90	72	3,380	0.10	1.5	40
D	3,000	4	1/2	×	1.5	120	95	3,420	0.22	2.5	38
E	3,000	4	1/2	×	1.5	120	127	3,230	0.31	2.4,3.7, 4.9	26
F	3,000	6	1/2	×	0.5	180	87	3,190	0.65	7.3	16
G	2,000	2	1/2	×	4.5	60	134	1,970	0.18	0.81	12
H	2,000	2	1/2	×	4.5	60	111	2,170	0.14	1.6,3.2	17
I	2,000	3	1/2	×	2.5	90	118	2,200	0.26	3.2	28
J	3,000	6	1/2	△	1.5	180	117	3,330	0.28	7.3	11
K	3,000	6	1/2	○	2.5	180	128	3,050	0.20	3.7	48
L	4,000	6	1/2.22	○	2.5	162	92	3.970	0.094	5.0	91
M	4,000	6	1/2.22	△	1.5	162	102	4,200	0.16	5.0	69
N	2,000	4	1/4	×	1.5	60	133	2,290	0.21	3.2	45
O	2,000	6	1/4	×	0.5	90	111	2,170	0.46	3.2	91
P	2,000	6	1/4	×	0.5	90	106	2,310	0.41	5.0	56
Q	2,000	6	1/4	△	1.5	90	128	2,180	0.21	5.0	38
R b)	2,000	6	1/2	×	1.2	180	134	2,530	0.48	7.5	65
S b)	2,000	4	1/2	×	2.5	120	126	2,020	0.31	5.0	28

Notes:

a) Conditions of aeration in Fill Period.



b) Fill period of Runs R and S is 1/3 of one cycle, and is 1/2 of one cycle in other Runs.

2-3 Pilot-Scale Experiments

Two kinds of pilot-scale experimental apparatus were run from October 1984 to March 1986.

Flow diagrams of pilot plants A and B are shown in Fig. 4 and Fig. 5. Two reactors used in each plant were operated independently from each other. Shown in Table 3 is the configuration of the main equipments used in each plant. The feed sewage used in the experiments was the effluent from the grit chamber in the Ara River Wastewater Treatment Center.

Shown in Tables 4 and 5 are the operational conditions for each run. A running-in period of about one month was provided before carrying out experiments on the treatment characteristics of each run. The experiments were continued for 4 weeks for each run, and the qualities of the influent and the effluent were analyzed at the rate of 2 to 3 times a week.

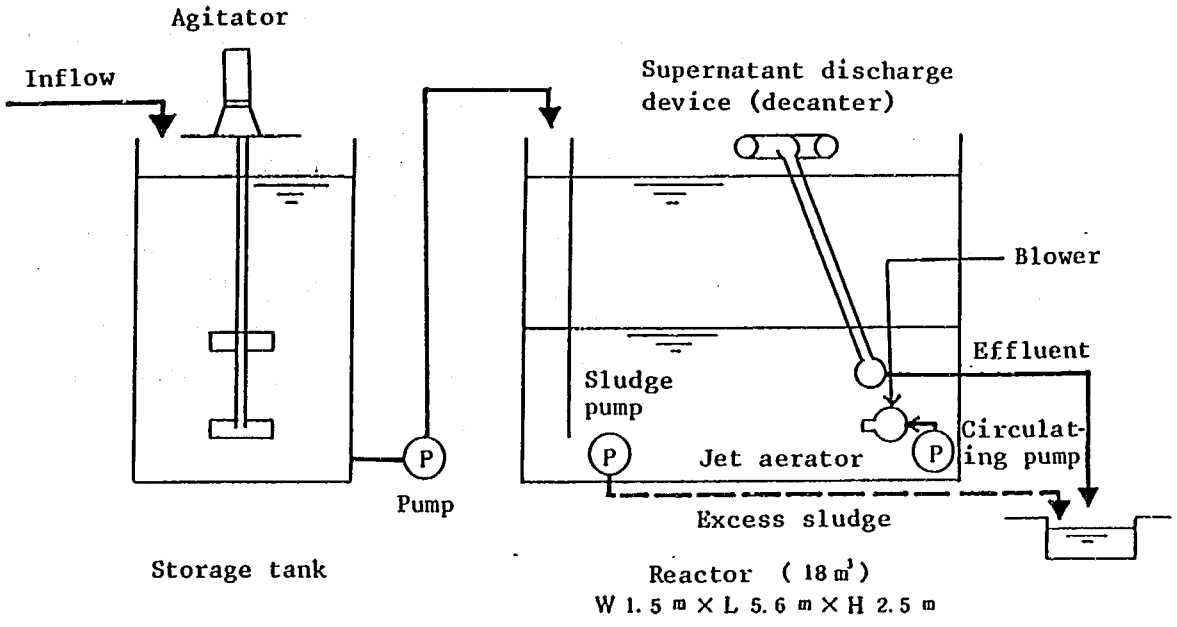


Fig. 4 Flow diagram of experimental apparatuses (plant A)

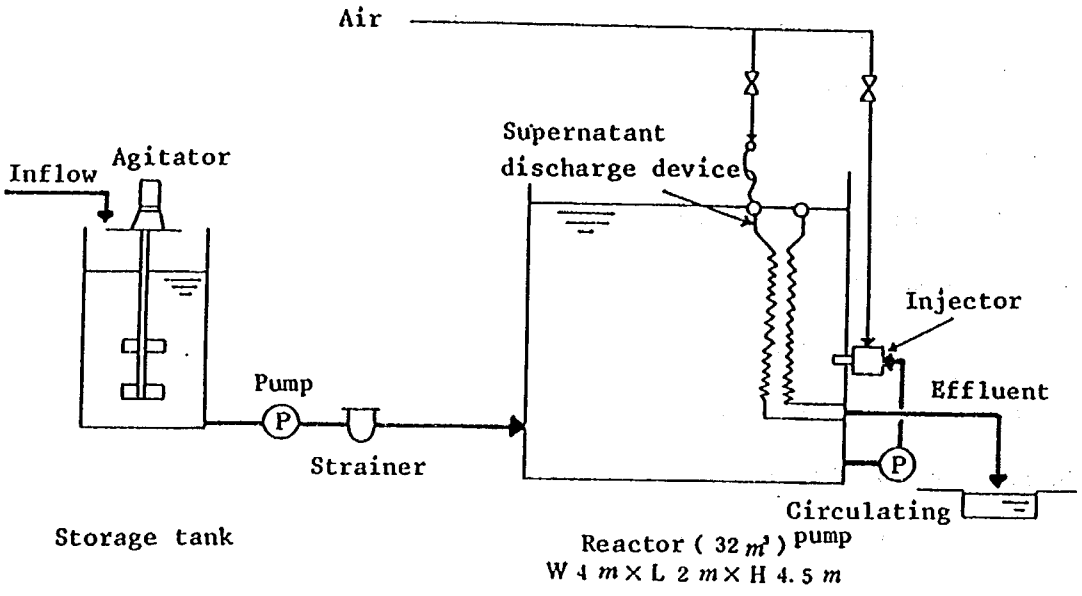


Fig. 5 Flow diagram of experimental apparatuses (plant B)

Table 3 Main experimental apparatuses (pilot scale)

Apparatus	Plant A	Plant B
Reactors	Dimensions: 1.5(W) x 5.6(L) x 2.5(D) m Quantity: 2 units	Dimensions: 2.0(W) x 4.0(L) x 4.5 (D) m Quantity: 2 units
Aeration device	Jet aerator (DM-1)	Bayer injector
Supernatant discharge device	Decanter type, self-suction pump type, submersible pump type	Weir-trough type, submerge weir-trough type
Excess sludge withdraw device	Submersible sludge pump	Buit-in sludge thickener

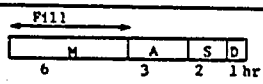
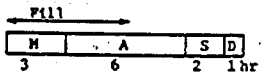
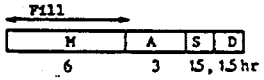
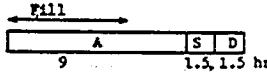
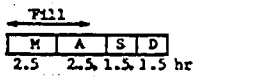
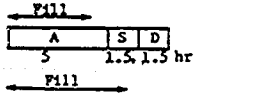
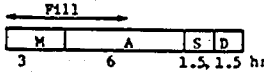
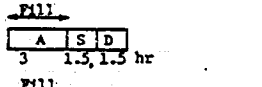
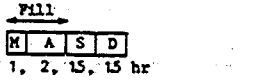
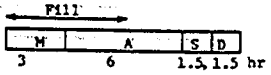
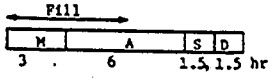
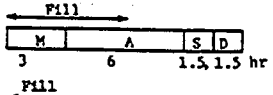
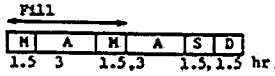
Table 4 Operational conditions (plant A)

Run	Operation mode					Quantity of influent (m^3/day)	Influent BOD_5 (mg/l)	MLSS concentration (mg/l)	BOD-SS loading (kg-BOD_5/ kg-SS · day)	Water temperature (°C)	Remarks
	MLSS concentration (mg/l)	Number of cycles	Withdraw ratio (-)	Aeration time (h/cycle)	Cycle						
1	2000	2	1/2	4	Fill → M 6 / A 4 / S D 1, 1 hr	18	111	1,740	0.19	12.1∿12.5	
2	2000	2	1/2	4	Fill → A 5 / A 4 / S D 1, 1 hr	18	111	2,000	0.17	10.1∿12.0	
3	2000	4	1/2	2	→ M 2 / A 2 / S D 1, 1 hr	36	143	2,630	0.33	20.5∿22.1	
4	2000	4	1/2	4	Fill → A 4 / S D 1, 1 hr	36	143	2,440	0.18	20.6∿22.2	
5-1	3000	4	1/4	1	Fill → M/A 4 / S D 1, 1 hr	18	95.4	3,460	0.17	22.5∿25.0	1) Intermittent aeration
6-1	3000	4	1/4	1	Fill → M 3 / A S D 1, 1, 1 hr	18	95.4	2,430	0.24	22.5∿25.0	
5-2	2000	4	1/4	1	Fill → M/A 4 / S D 1, 1 hr	18	94.0	2,080	0.27	20.7∿23.5	Intermittent aeration
6-2	2000	4	1/4	1	Fill → M 3 / A S D 1, 1, 1 hr	18	94.0	1,900	0.30	20.7∿23.5	
7	2000	4	1/2	3	Fill → A 3, 2, 1 hr / S D	36	134	2,540	0.21	13.6∿18.0	

☐ No agitation [M] Agitation [A] Aeration [S] Settle [D] Draw

Note 1: In the Aeration period of Run 5, 45 minutes of agitation and 15 minutes of aeration are repeated 4 times.

Table 5 Operational conditions (plant B)

Run	Operation mode: MLSS concentration (mg/l)	Number of cycles	Withdraw ratio (-)	Aeration time (h/cycle)	Cycle	Quantity of influent (m^3/day)	Influent BOD_5 (mg/l)	MLSS concentration (mg/l)	BOD-SS loading (kg-BOD_5/kg-SS · day)	Water temperature (°C)	Remarks
A	2000	2	0.475	3		30.4	131	2,060	0.24	11.0∿15.5	
B	2000	2	0.475	6		30.4	131	1,860	0.13	11.0∿15.5	
C	2000	2	0.475	3		30.4	88.7	2,170	0.16	16.5∿21.8	
D	2000	2	0.475	9		30.4	104	1,920	0.068	16.5∿21.8	
E	2000	3	0.475	2.5		45.6	192	2,360	0.37	23.5∿27.5	
F	2000	3	0.475	5		45.6	192	1,910	0.23	23.5∿27.5	
G	2000	2	0.475	6		30.4	138	2,220	0.12	23.5∿28.0	
H	2000	4	0.475	3		60.8	133	2,360	0.22	20.5∿21.5	
I	2000	4	0.475	2		60.8	128	2,180	0.34	20.5∿21.5	
J	4000	2	0.375	6		24.0	152	4,120	0.055	16.2∿20.0	
K	3000	2	0.375	6		24.0	152	3,110	0.073	16.1∿20.0	
L	2000	2	0.475	6		30.4	236	2,330	0.19	12.2∿13.5	
M	2000	2	0.475	6		30.4	236	2,170	0.21	12.2∿13.5	Intermittent aeration

M Agitation A Aeration S Settle D Draw

3. RESULTS OF OPERATION

3-1 Removal Performance

(1) BOD_5

[Relation Between BOD-SS loading and BOD_5 removal ratio]

The relationship between the mean BOD-SS loading in each run and the BOD_5 removal ratio in the bench-scale and pilot-scale experiments are shown in Figs. 6 and 7, respectively. Within the setting range of various operational factors shown in Table 1, the BOD_5 removal characteristics were related to the BOD-SS loading; as BOD-SS loading increases, the effluent BOD_5 tends to becomes higher. However, if the BOD-SS loading is lower than 0.4 to 0.5 kg-BOD_5/kg-SS · day, then the BOD_5 of the supernatant becomes less than 20 mg/1, meeting the secondary treatment standard in Japan.

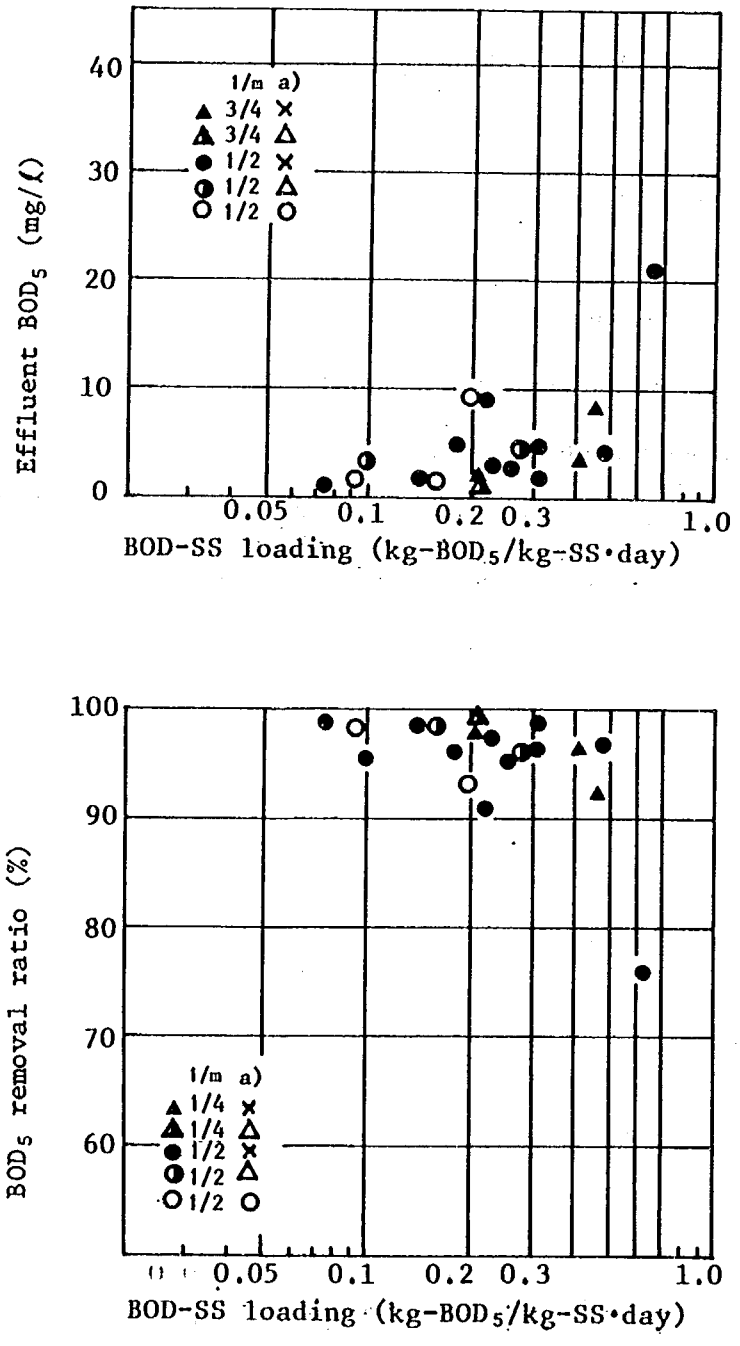


Fig. 6 Relationship between BOD-SS loading, and effluent BOD_5 and BOD_5 removal ratio (bench-scale), [a): refer to Table 2 for aeration conditions]

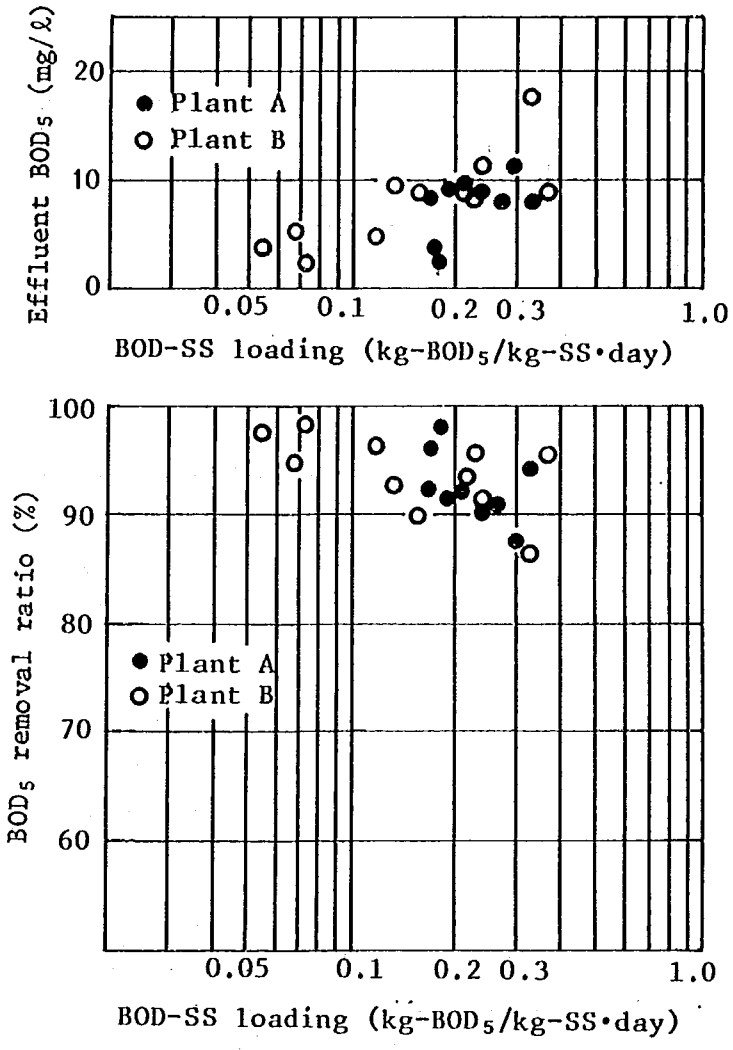
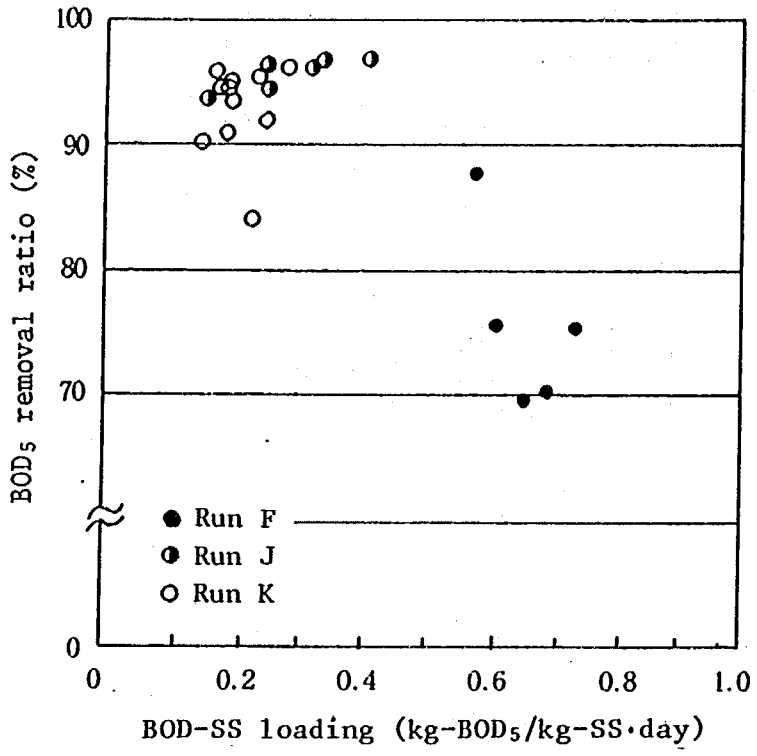


Fig. 7 Relationship between BOD-SS loading, and effluent BOD_5 and BOD_5 removal ratio (pilot-scale)

[Effect of aeration time on BOD_5 removal]

In order to check the effect of the length of aeration time per cycle on the BOD_5 removal characteristics, the relationship between the BOD_5 removal ratio and the BOD-SS loading in 3 kinds of operation modes (bench-scale, Runs F, J and K) was determined as shown in Fig. 8; operational conditions of Runs F, J, and K are the same except for aeration time. Fig. 9 shows the change in the quality of supernatant when the aeration time per cycle is sequentially increased from Runs F to J to K. From this, it will be known that the BOD_5 removal ratio can be improved as the aeration time per cycle increases.



Operational conditions (common to Runs F, J and K)

MLSS concentration: 3,000 mg/l
Number of cycles: 6 cycles/day
Withdraw ratio: 1/2
Fill period : 2 hours
Settle period: 1 hour
Draw period : 30 minutes

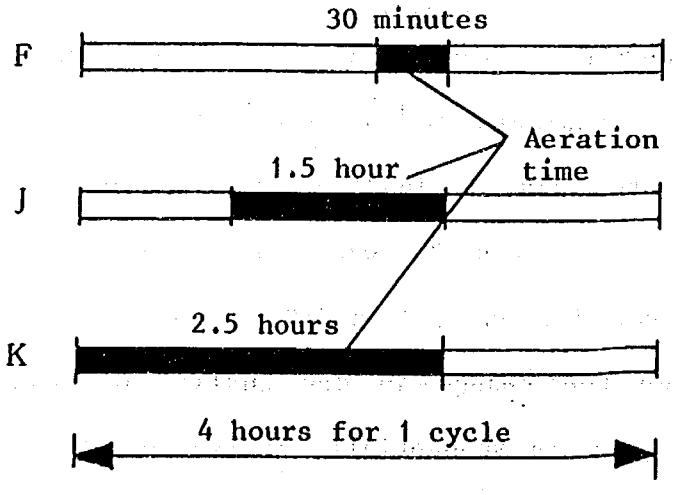
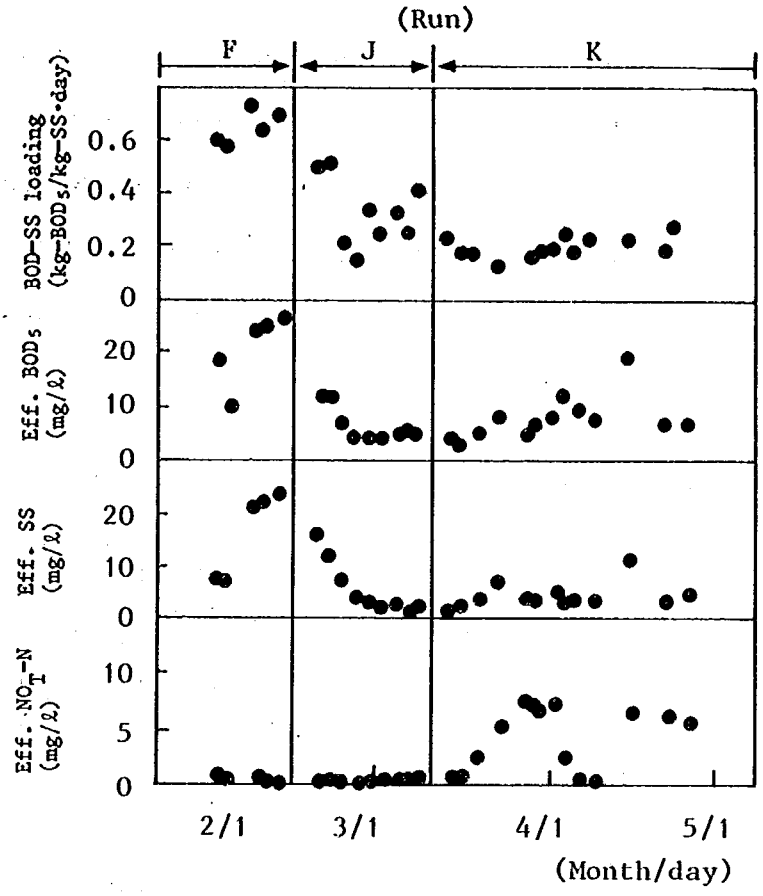


Fig. 8 Relationship between aeration conditions in Fill Period and BOD_5 removal ratio (bench-scale)



Note: NO_T^--N = NO_2^--N + NO_3^--N

Fig. 9 Change in treatment characteristics due to aeration conditions of Fill Period (bench-scale)

Fig. 10 compares the BOD_5 removal performance between Run 3 and Run 4 where two tanks of plant A were operated in parallel by varying only the aeration time.

In the same way as for the results of the bench-scale experiments, it will be known that the level of BOD_5 of supernatant can be improved by increasing the aeration time.

[Effect of withdraw ratio on BOD_5 removal]

Shown in Table 6 is the effect of the withdraw ratio on the BOD_5 removal performances in Runs F, O and P where the aeration time is the shortest with 0.5 h/cycle in the bench-scale experiments.

The effluent BOD_5 exceeds 20 mg/l in Run F where the withdraw ratio is 1/2 while the BOD_5 is 8.4 mg/l and 3.6 mg/l, respectively in Run O and Run P where the withdraw ratio is 1/4; it will be known that the BOD_5 removal ratio varies depending on the withdraw ratio. The effluent BOD_5 is different between Run O and Run P, which might be caused by the difference in air supply.

Table 6 Comparison of BOD_5 removal due to varying the withdraw ratio (bench-scale)

Run	Influent BOD_5 (mg/l)	Effluent BOD_5 (mg/l)	BOD_5 removal ratio (%)	BOD-SS loading (kg-BOD_5/ kg-SS · day)	Withdraw ratio (-)
F	86.6 (80.8∿98.1)	21.1 (10.3∿26.4)	75.6	0.65	1/2
O	111 (57.8∿165)	8.4 (1.5∿25.1)	92.4	0.46	1/4
P	106 (68.0∿163)	3.6 (0.5∿9.6)	96.6	0.41	1/4

Notes: 1. The smallest and largest values are shown in parentheses.
2. Run O and Run P have different air volumes.

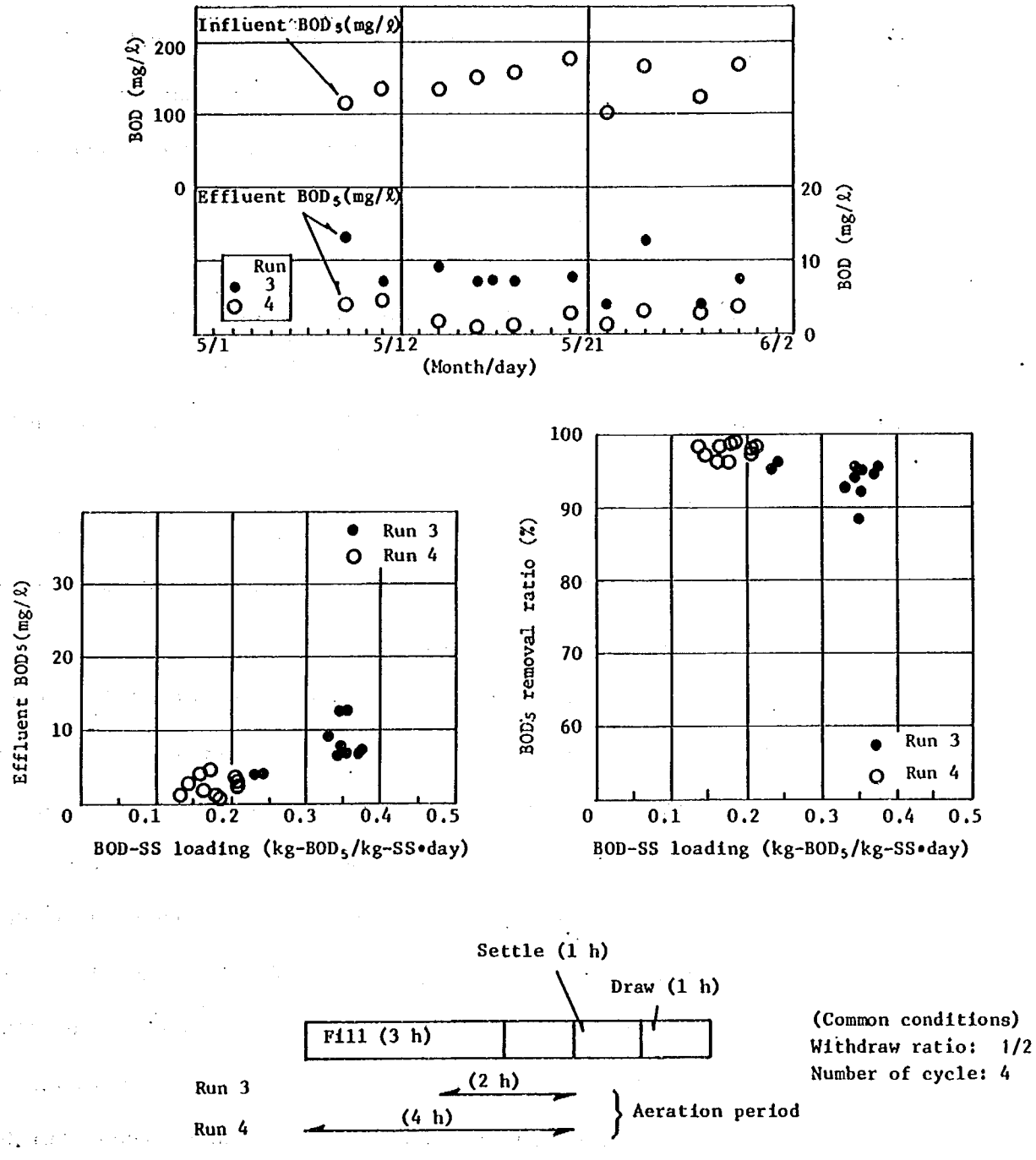


Fig. 10 Effect of aeration time on BOD_5 removal (plant A)

[Effect of water temperature on BOD_5 removal]

Shown in Fig. 11 is the relationship between the BOD-SS loading, the effluent BOD_5 and the BOD_5 removal ratio in Run A and Run C, and Run B and Run G for plant B which was operated during periods of low and high water temperature under the same conditions. No apparent difference in the BOD removal performance was recognized due to variation in water temperature within the range of 11.0 to 28.0°C.

(2) Suspended Solids

Shown in Figs. 12 and 13 are the relationships between the mean BOD-SS loading, the effluent SS and the SS removal ratio in each run in the bench-scale and pilot-scale experiments. The SS removal was satisfactory when the BOD-SS loading was lower than 0.5 kg-BOD_5/kg-SS • day for the bench-scale experiment, and when the BOD-SS loading was lower than 0.4 kg-BOD_5/kg-SS • day for the pilot scale experiments, regardless of the difference in other operational conditions.

In the SBR system, the solid-liquid separation is performed in the stationary state during the Settle Period so that a satisfactory SS removal can be expected by only preventing the rolling-up of settled sludge and the rolling-in of scum during the withdraw of supernatant. Shown in Fig. 14 is the change with time in the SS concentration for a water depth of 1 m (water level of 4 m) during the Settle Period of plant B. The SS concentration at this point becomes sufficiently small just after the surface of the sludge zone has passed this point due to settling, and then the remaining fine solids settle as time elapses, making the SS concentration decrease further.

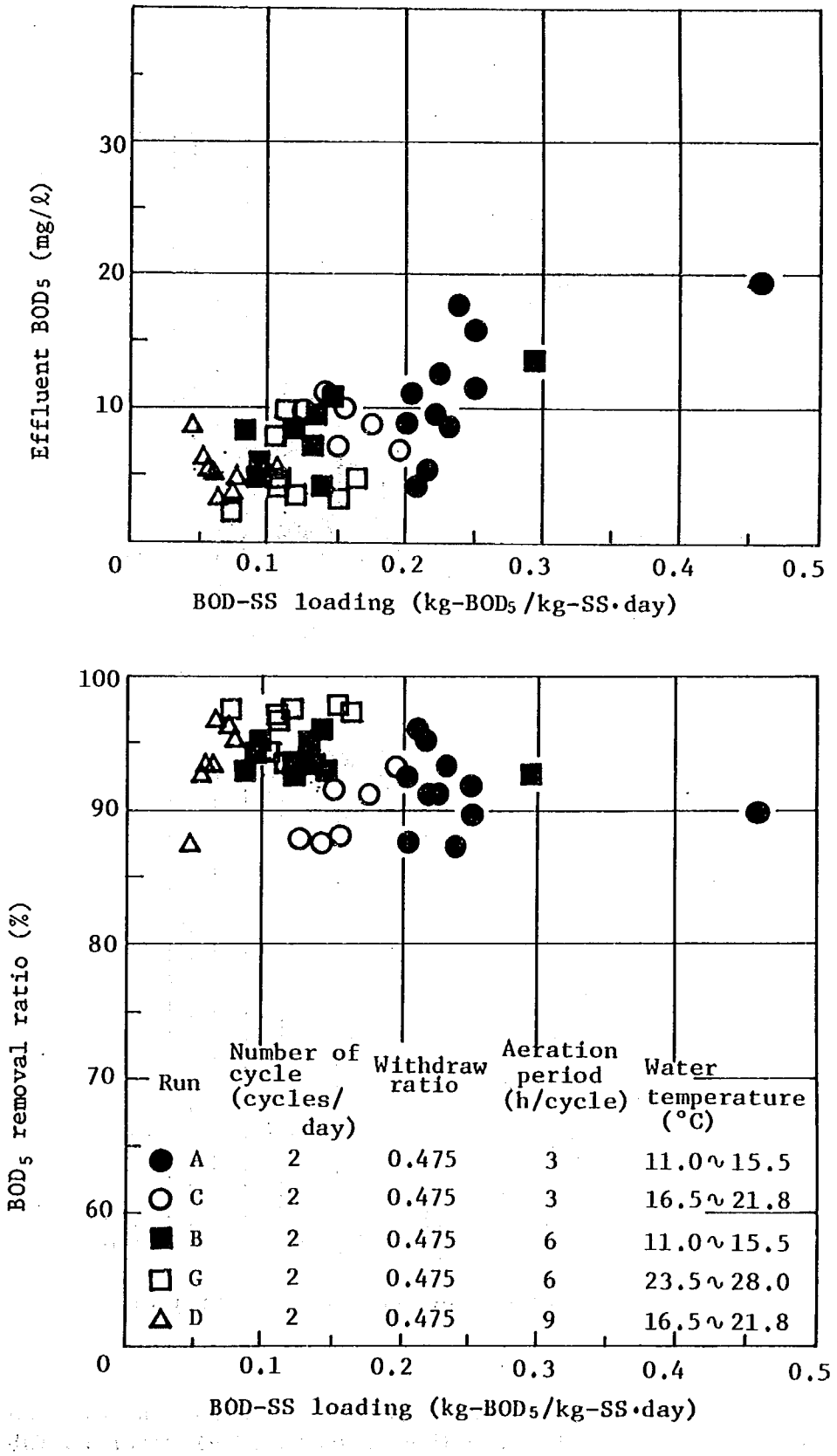
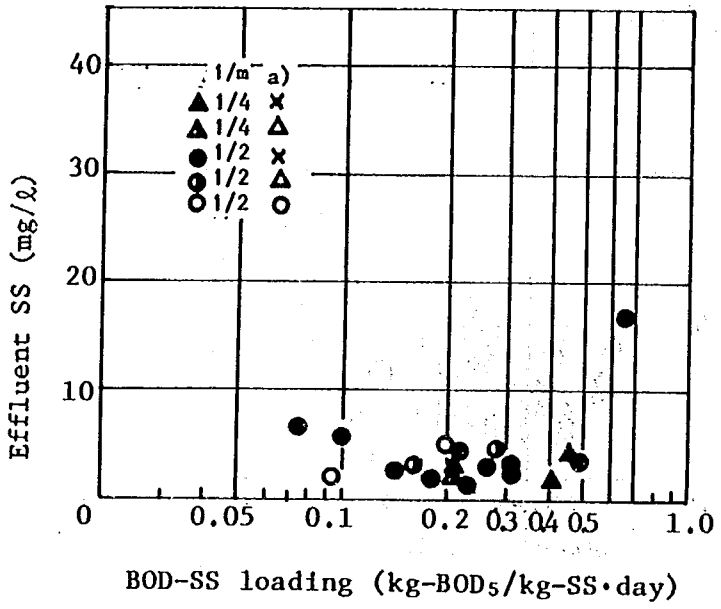


Fig. 11 Effect of water temperature on BOD_5 removal (plant B)



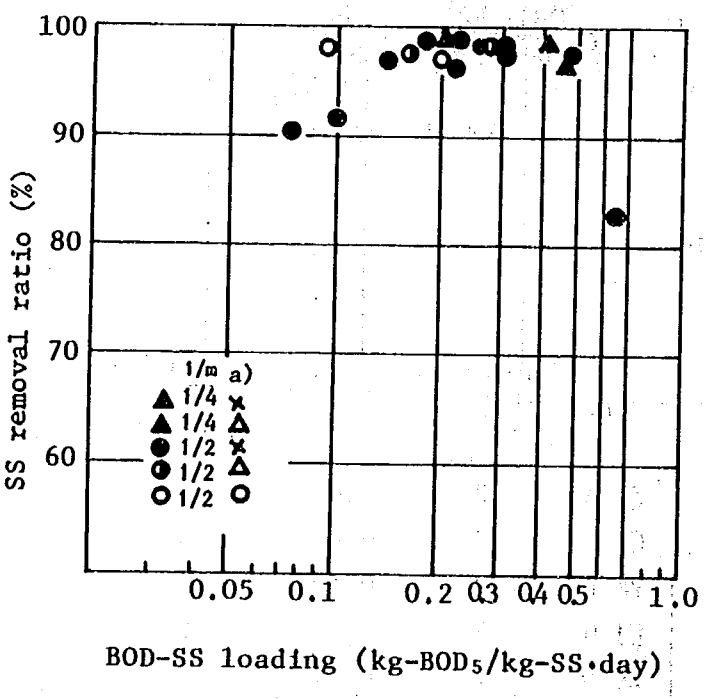
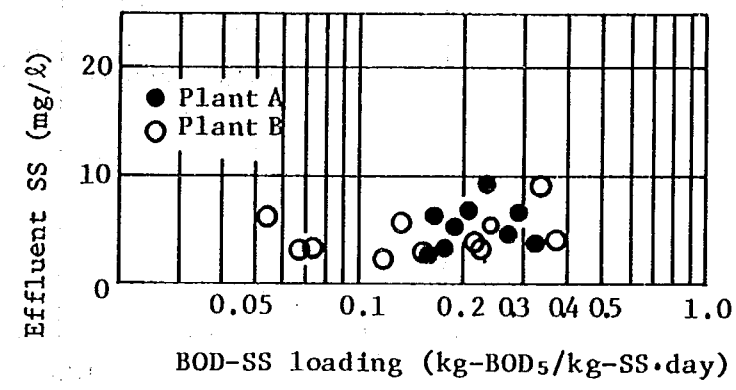


Fig. 12 Relationship between the BOD-SS loading, the effluent SS and the SS removal ratio (bench-scale), [a) refer to Table 2 for aeration conditions]



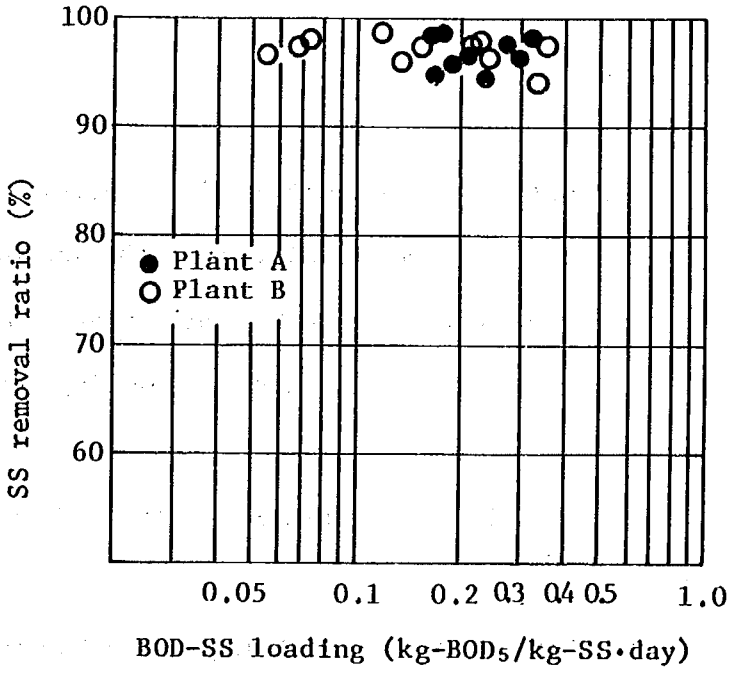


Fig. 13 Relationship between the BOD-SS loading, the effluent SS and the SS removal ratio (pilot-scale)

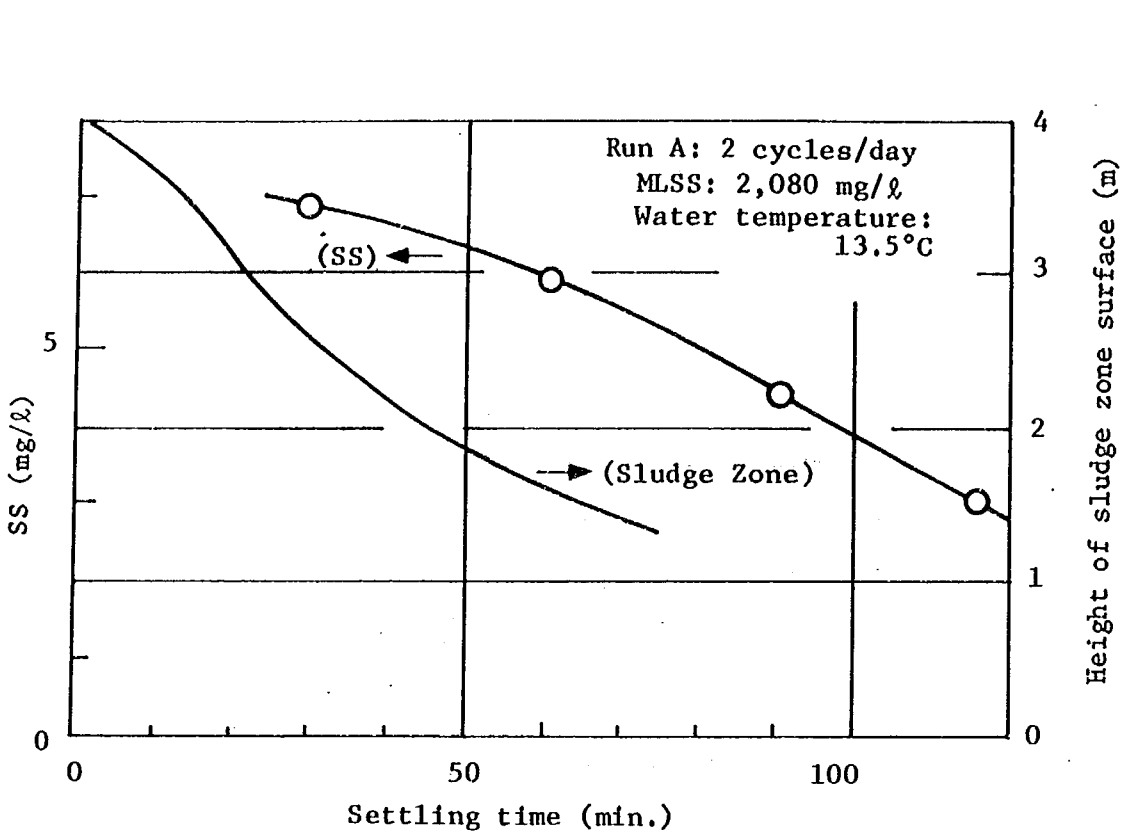


Fig. 14 Change with time in the SS concentration at the point of water depth of 1 m during Settle Period (plant B)

(3) Nitrogen

[Nitrification]

Fig. 15 shows the relationship between the removal ratio of Kjeldahl nitrogen (Kj-N) and the mean BOD-SS loading of each run in the bench-scale experiments. From this figure, it can be seen that the Kj-N is almost completely removed for as long as the BOD-SS loading is less than 0.15 kg/BOD_5/kg-SS · day.

The removal of Kj-N in the SBR system is considered to be caused by the following factors:

1) Biological nitrification, 2) settling and adsorption to activated sludge of colloidal and nonfilterable nitrogen, and 3) the use of soluble Kj-N for cell synthesis.

Sum of 2) and 3) of the above is the quantity of removed Kj-N due to the discharge of excess sludge to outside the system.

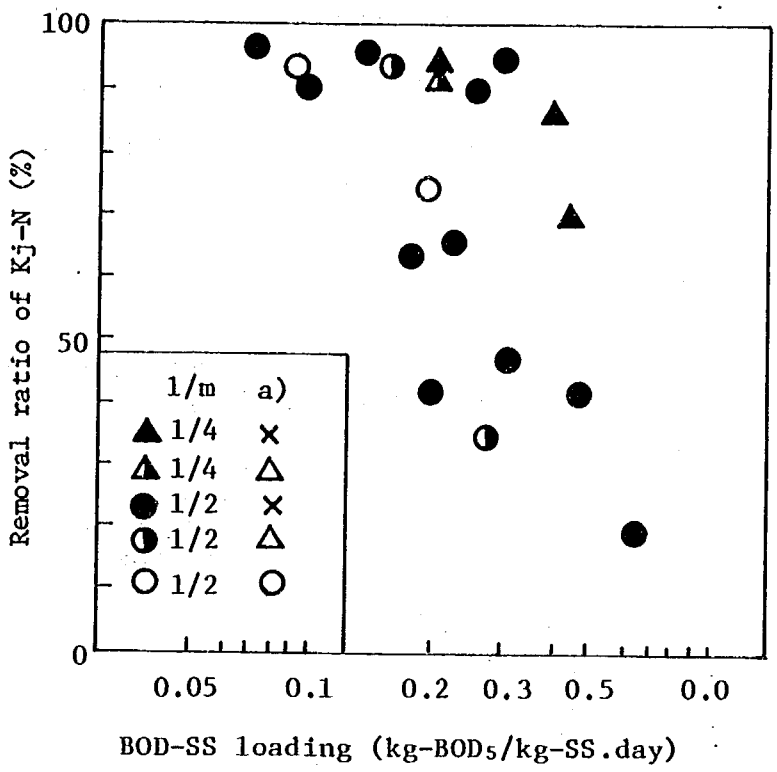


Fig. 15 Relationship between the BOD-SS loading and the removal ratio of Kj-N (bench-scale) [a): refer to Table 2 for aeration conditions]

Fig. 16 shows the ratio between the removed Kj-N due to nitrification and the removed Kj-N due to the discharge of excess sludge to the outside of the system, with the latter being calculated from the quantity of excess sludge generated and its nitrogen content. As the BOD-SS loading increases, the ratio of Kj-N removal due to nitrification decreases. The nitrogen content of activated sludge was about 7%, independent of the operational conditions.

Shown in Table 7 is the rate of nitrification K_N (mg-N/g-MLSS · h) calculated from the change with time in the NO_T^--N concentration (sum of NO_2-N and NO_3^--N) in the mixed liquor of the Aeration Period in bench-scale experiments.

Shown in Fig. 17 is the relationship between the BOD-SS loading and the Kj-N removal ratio obtained from the pilot-scale experiments, while Fig. 18 shows the relationship between the BOD-SS loading and the ratio of the total

quantity of removed Kj-N to the quantity of removed Kj-N due to the discharge of excess sludge to outside the system. The results shown in these figures are almost the same as those obtained from the bench-scale experiments.

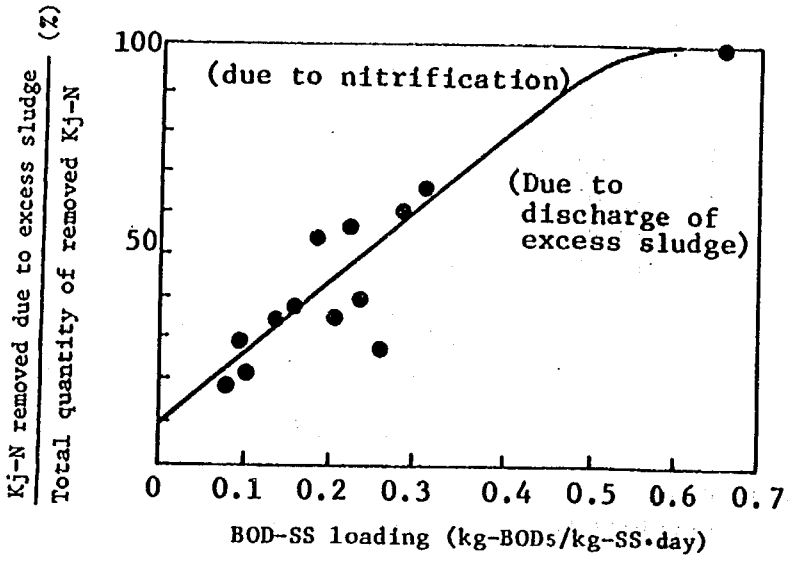
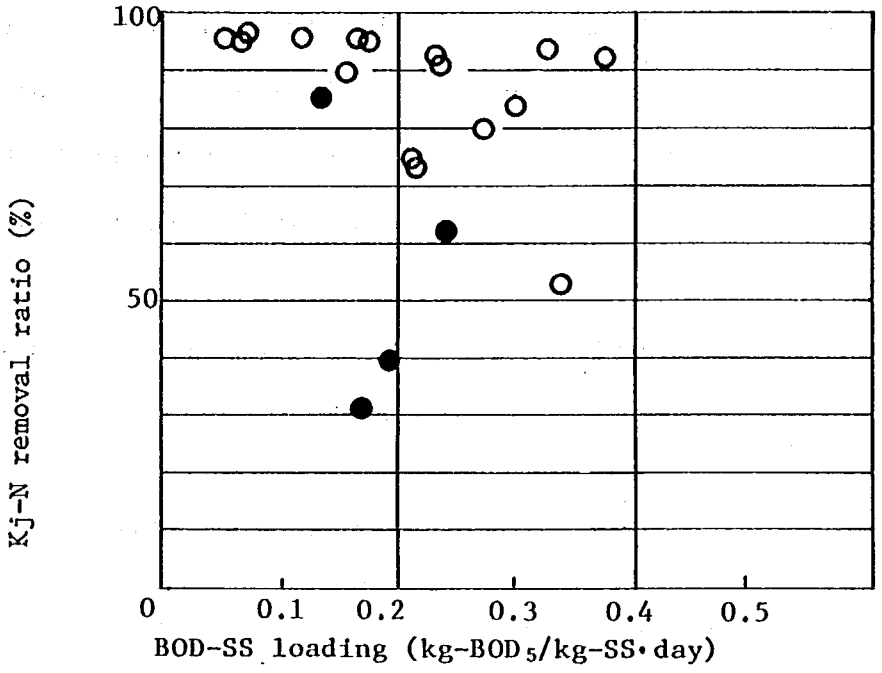


Fig. 16 Breakdown of Kj-N removal ratio (bench-scale)

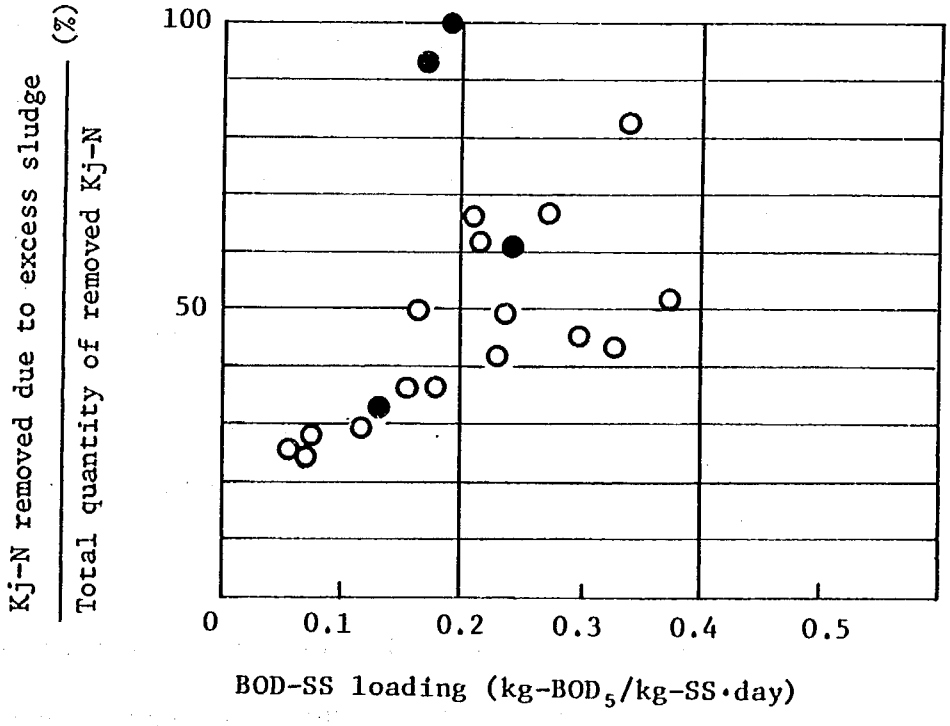
Table 7 Rate of nitrification and operational conditions in bench-scale experiments

Run	MLSS concentration (mg/l)	Number of cycles (cycles/day)	Withdraw ratio	Rate of nitrification (mgN/gMLSS/h)	SAT (day)
C	3 140	3	1/2	1.63	12
D	3 370	4	1/2	0.73	3
E	2 900	4	1/2	0.36	4
H	2 130	2	1/2	1.48	6
I	2 140	3	1/2	1/13	4



○: Temp. higher than 15°C; ●: Temp. lower than 15°C

Fig. 17 Relationship between the BOD-SS loading and the Kj-N removal ratio (pilot-scale)



○: Temp. higher than 15°C; ●: Temp. lower than 15°C

Fig. 18 Breakdown of Kj-N removal ratio (pilot-scale)

Shown in Fig. 19 is the relationship between the water temperature and the rate of nitrification calculated on the basis of the increment in NO_T^--N in mixed liquid in the Aeration Period of the pilot-scale experiments, in which the rate of nitrification becomes smaller as the water temperature decreases.

From the above results, Kj-N can be properly removed as long as the BOD_5-SS loading is less than 0.2 kg-BOD_5/kg-SS・day.

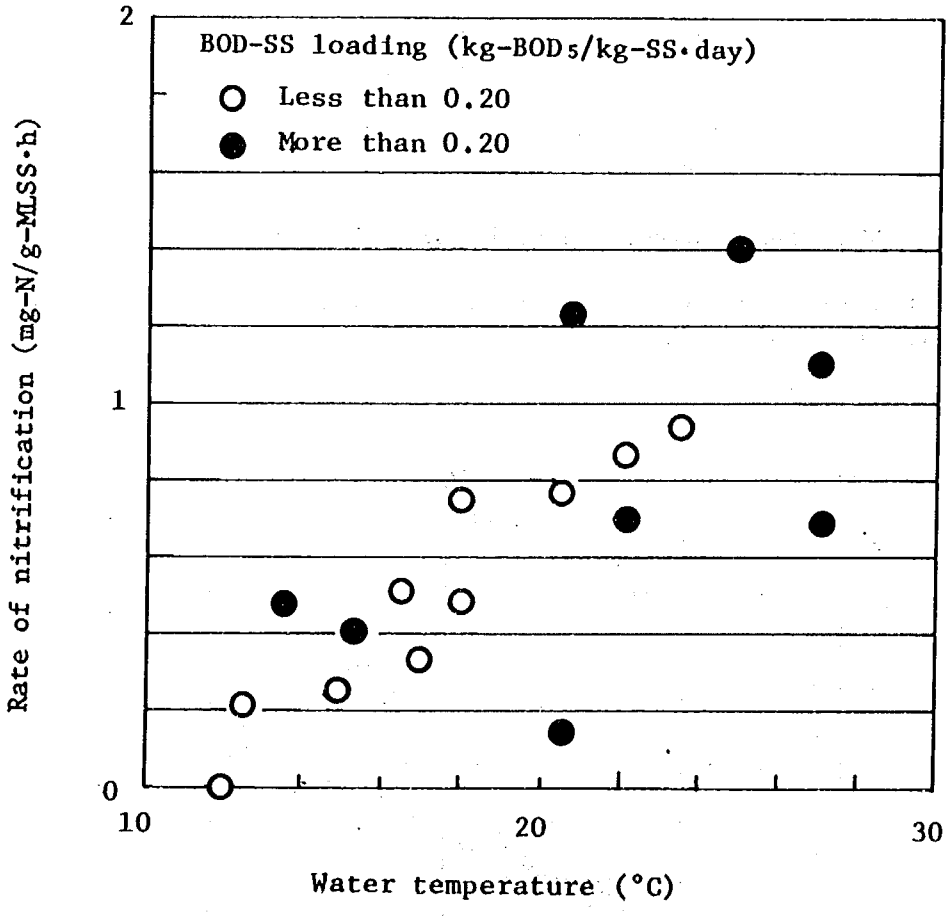


Fig. 19 Relationship between the water temperature and the rate of nitrification (pilot-scale)

[Denitrification]

The relationship between the removal ratio of total nitrogen (T-N) and mean BOD-SS loading in each run in the bench-scale experiments is shown in

Fig. 20, and the relationship between the removal ratio of Kj-N and the removal ratio of T-N is shown in Fig. 21.

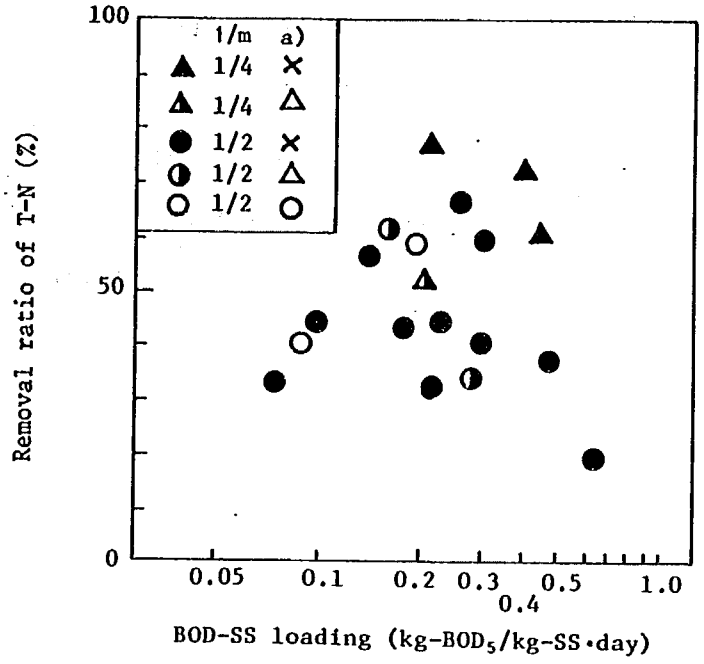


Fig. 20 Relationship between the BOD-SS loading and the removal ratio of T-N (bench-scale) [a): refer to Table 2 for aeration conditions]

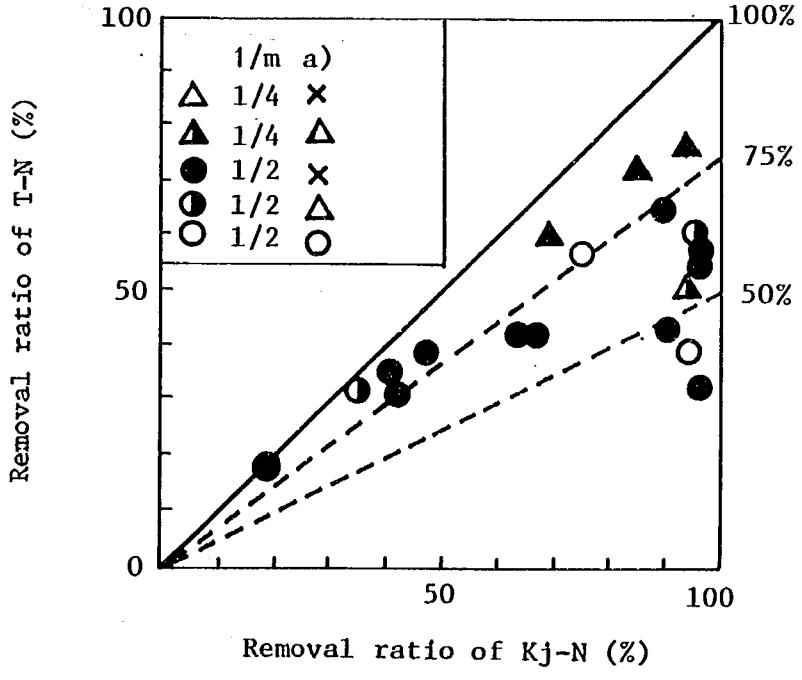


Fig. 21 Relationship between the removal ratio of Kj-N and the removal ratio fo T-N (bench-scale) [a): refer to Table 2 for aeration conditions]

If the removal of Kj-N is properly made and the denitrification is sufficient in the anoxic period, then the removal ratio of T-N increases as the withdraw ratio of supernatant becomes smaller. When the withdraw ratio was 1/4, the maximum nitrogen removal ratio of more than 75% was obtained.

The changes in the NO_T^--N concentration in the Fill Period (anoxic period) in bench-scale experiments are shown in Fig. 22.

This figure shows the denitrification process where the residual NO_T^--N of the prior cycle is being removed with the sewage inflows.

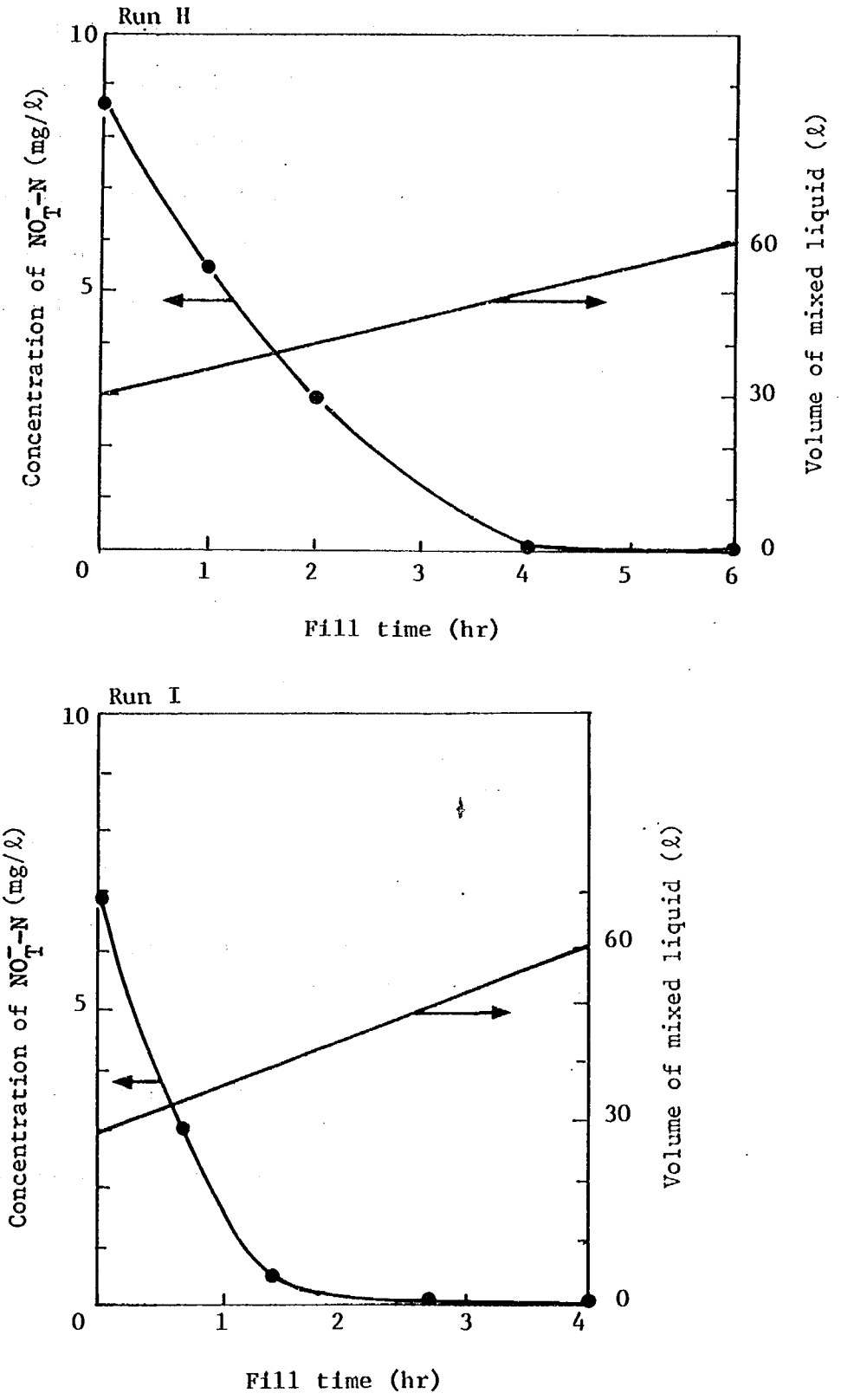
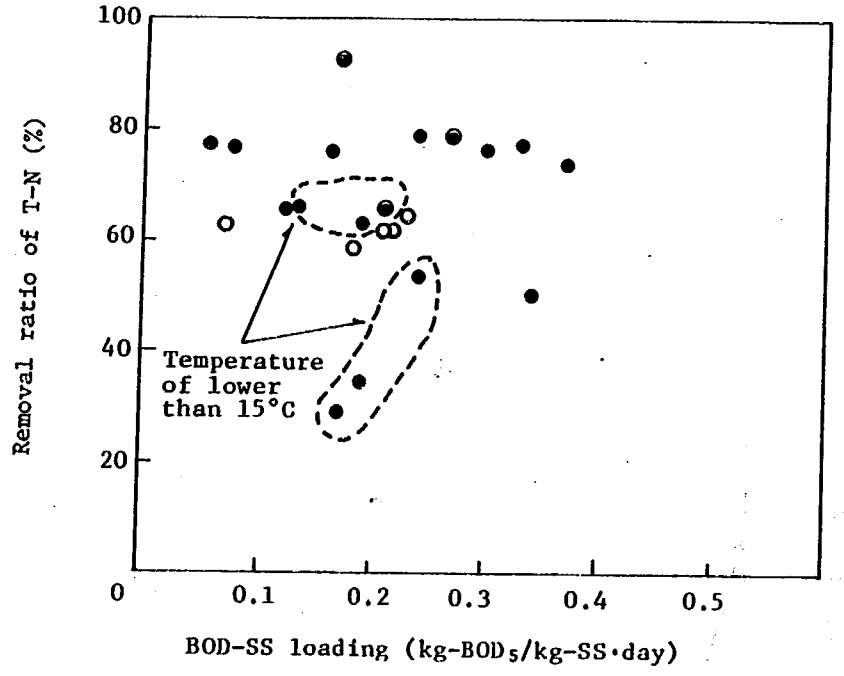


Fig. 22 Changes in the NO_T^--N concentration in Fill Period (anoxic period)

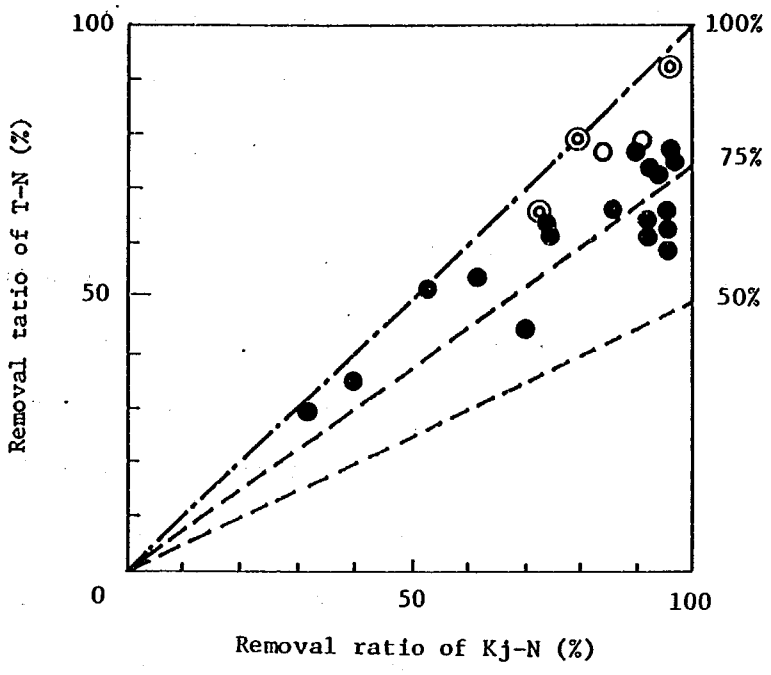
Fig. 23 shows the removal ratio of T-N and the mean BOD-SS loading in each run of pilot-scale experiments, while Fig. 24 shows the relationship between the removal ratio of Kj-N and the removal ratio of T-N.

It was possible to enhance the removal ratio of the total nitrogen by providing either intermittent aeration or anoxic period and by decreasing the withdraw ratio under the conditions where the nitrification can be made properly.



◉: Intermittent aeration; ○: Aeration is performed in periods other than Settle and Draw.; ●: Others

Fig. 23 Relationship between BOD-SS loading and removal ratio of T-N (pilot-scale)



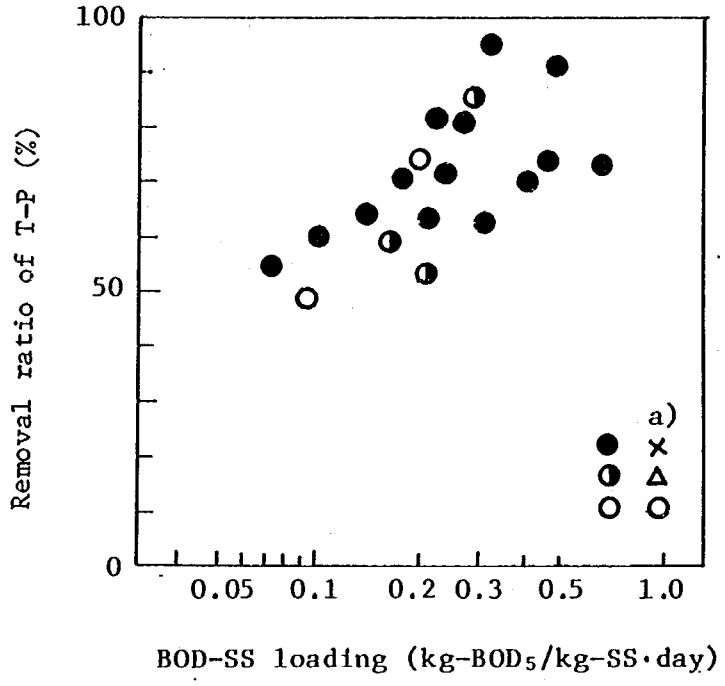
◎: Intermittent aeration; ○: Withdraw ratio of supernatant is 1/4.; ●: Others

Fig. 24 Relationship between the removal ratio of Kj-N and the removal ratio of T-N (pilot-scale)

(4) Phosphorus

Shown in Fig. 25 are the relationships between the mean BOD-SS loading, the removal ratio of T-P and the quantity of removed T-P per cycle (influent T-P concentration - effluent T-P concentration) in each run of the bench-scale experiments. The removal ratio of T-P and the quantity of removed T-P increase as the BOD-SS loading increases despite the T-P concentration of the influent wastewater varying by each run.

Examples of changes in the NO_T^--N concentration and in the concentration of soluble total phosphorus (S-T-P) per cycle are shown in Fig. 26, during which the anaerobic state due to denitrification is recognized in the reactor. Accompanied with this is the release of phosphorus from activated sludge and, thereafter, the uptake of phosphorus in aerobic state.



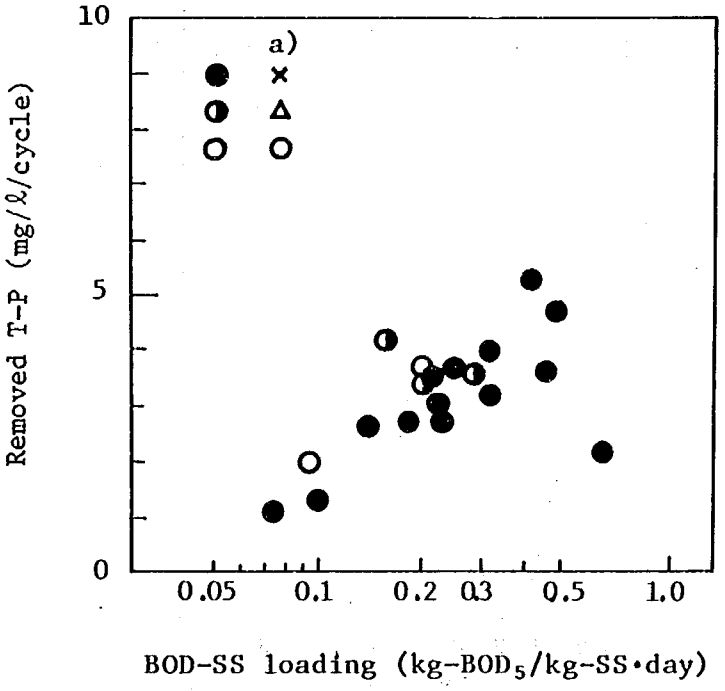


Fig. 25 Removal ratio of phosphorus (bench-scale) [a): refer to Table 2 for aeration conditions]

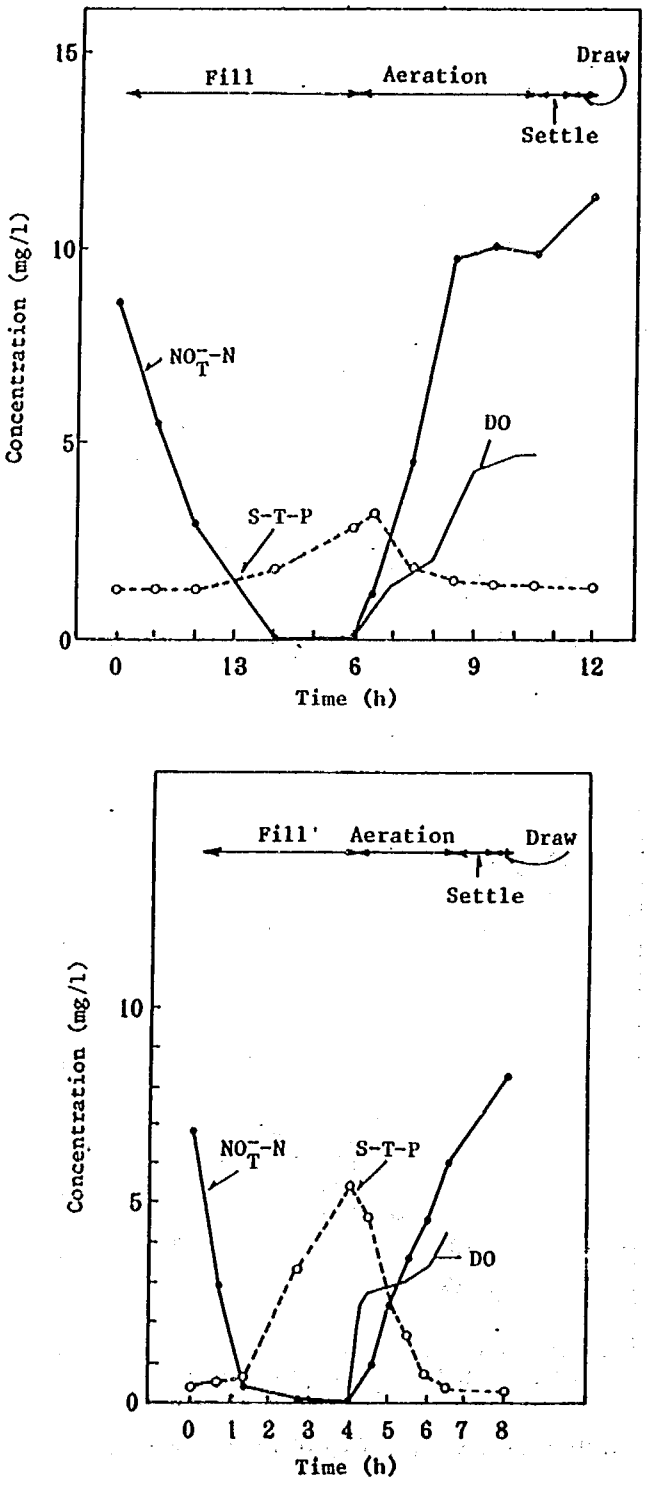


Fig. 26 Changes in the concentration of soluble T-P and NO_T^--N per cycle (bench-scale)

The biological removal of phosphorus is accomplished by discharging the phosphorus as excess sludge to outside the system; the phosphorus can be more effectively removed as the amount of excess sludge generated and phosphorus content in the excess sludge become larger.

The relationship between the phosphorus content in the excess sludge and the anaerobic time per cycle is shown in Fig. 27, and the relationship between the BOD-SS loading and the phosphorus content (%) in the excess sludge is shown in Fig. 28.

From Fig. 28, the increasing tendency of the phosphorus content in excess sludge accompanied by the increase in BOD-SS loading can be clearly known. With respect to the effect of the anaerobic time, it can be seen that an anaerobic time of about 1 hour is appropriate. However, it was not possible to fully clarify the change in the phosphorus content in excess sludge due to the presence of an anaerobic state or change in the anaerobic time since the decrease in BOD-SS loading corresponds to an increase in the anaerobic time.

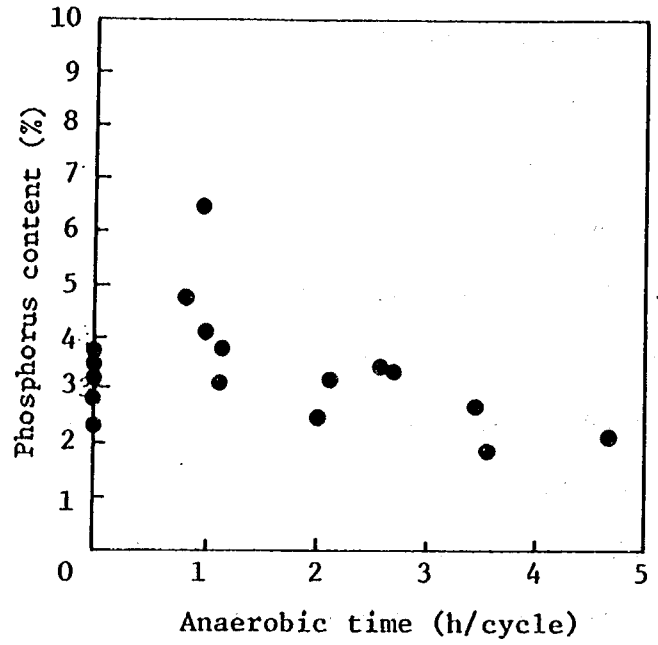


Fig. 27 Relationship between the anaerobic time and the phosphorus content in excess sludge (bench-scale)

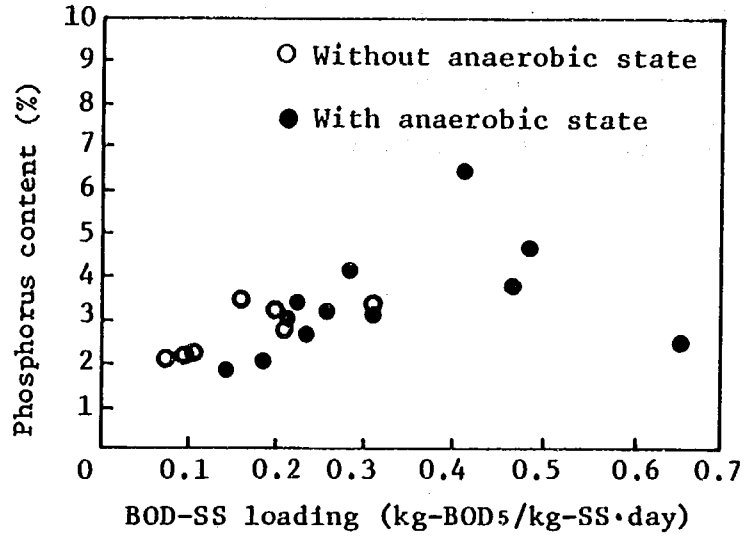
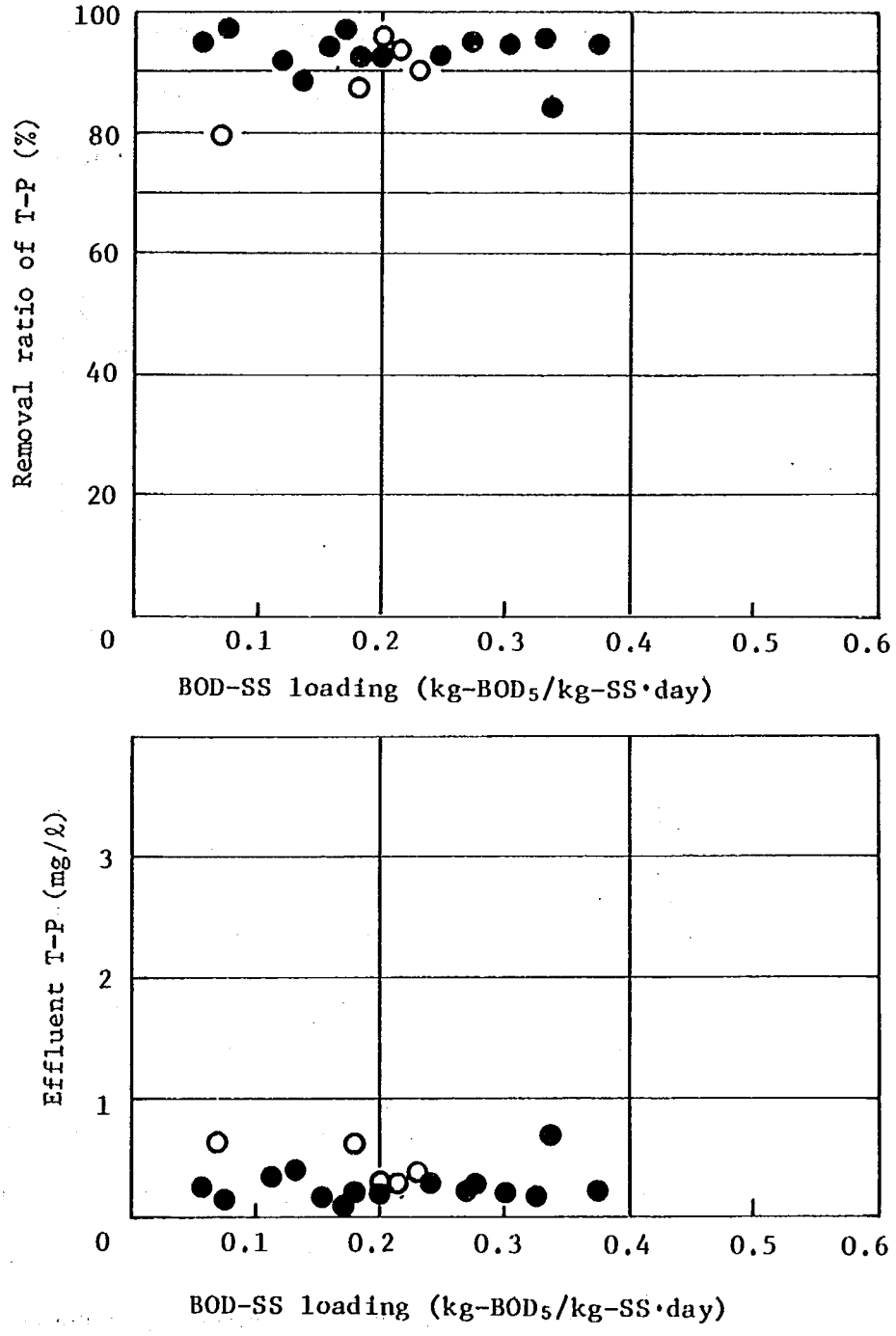


Fig. 28 Relationship between the BOD-SS loading and the phosphorus content in excess sludge (bench-scale)

Shown in Fig. 29 is the relationship between the BOD-SS loading, the T-P of supernatant and the removal ratio of T-P obtained from the mean values in each run in pilot-scale experiments.



● : With anoxic period; ○ : Without anoxic period

Fig. 29 Relationship between the BOD-SS loading, effluent water and the removal ratio of T-P (pilot-scale)

The pilot-scale experiments achieved a better removal of T-P than that of the bench-scale experiments, and the T-P was removed properly even in the runs not having an discrete anoxic period. Shown in Fig. 30 is the relationship between the quantity of removed T-P determined from the T-P concentrations of influent and effluent, and the quantity of removed T-P determined from the amount of excess sludge and its T-P content. It will be known that there is a close agreement between them.

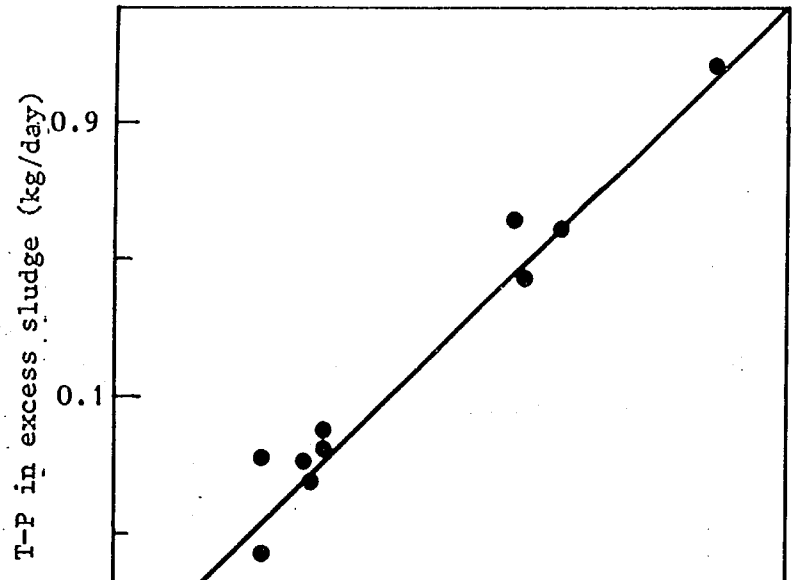


Fig. 30 Balance of removed T-P (pilot-scale)

Shown in Fig. 31 are the changes during a cycle in the concentrations of NO_T^--N and PO_4^--P in mixed liquor, the sludge zone and in supernatant in Run 7 of plant A. Run 7 has no anoxic period; there is the release of phosphorus from the activated sludge and biological phosphorus removal is effectively performed because an anaerobic state occurs due to denitrification in the sludge zone in the Settle and Draw Periods. A T-P removal ratio of higher than 90% was obtained in the BOD-SS loading range from 0.2 to 0.4 kg-BOD_5/kg-SS • day.

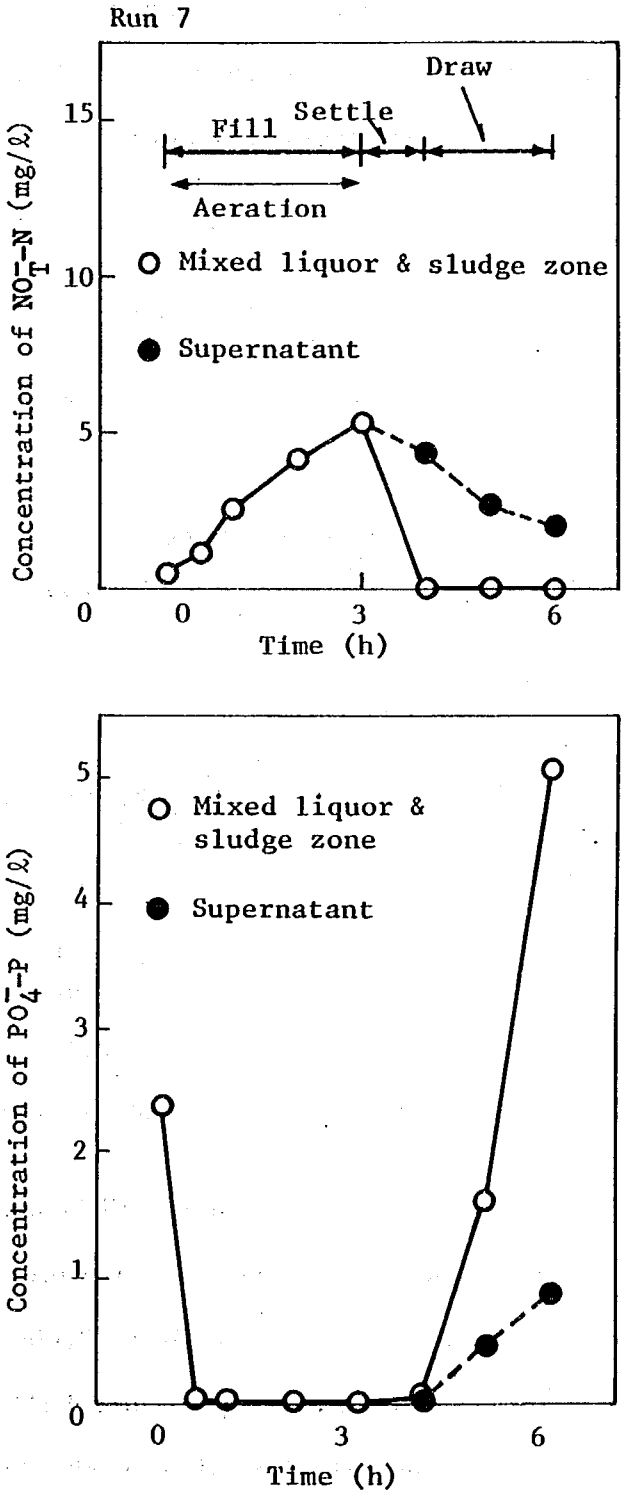


Fig. 31 Changes in the concentrations of NO_T^--N and PO_4^--P in the sludge zone (pilot-scale)

3-2 Settling Characteristics of Activated Sludge

The solid-liquid separation of the SBR system occurs under near ideal quiescent conditions. Fig. 32 shows the example of the settling curve of mixed liquor solids. The settling curve can be divided into an initial constant velocity section and compression section.

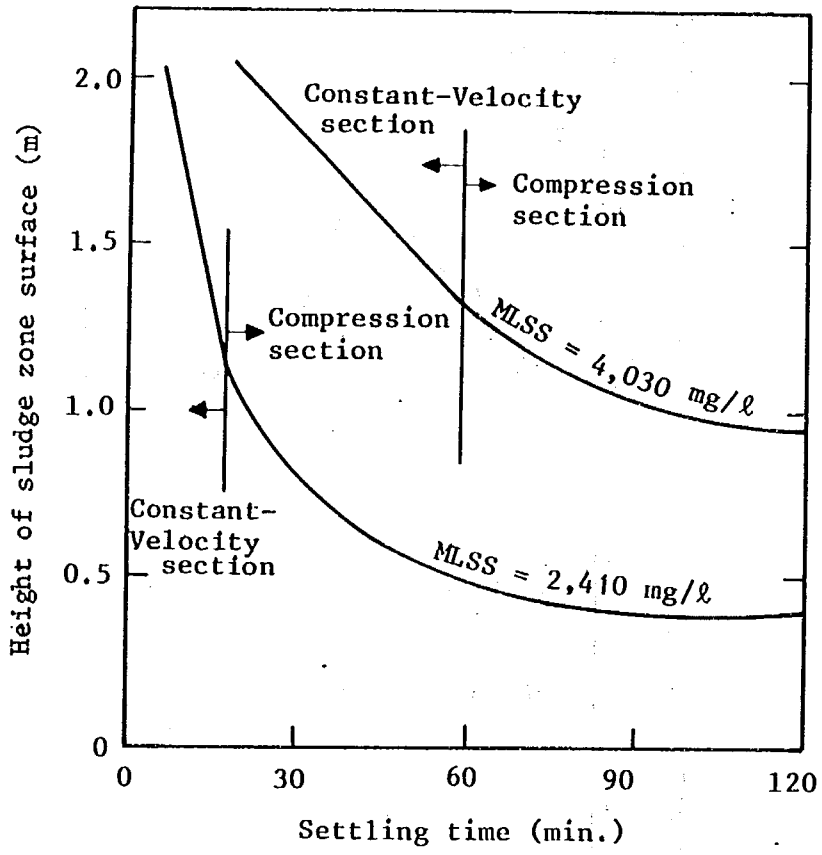


Fig. 32 Examples of settling curves of mixed liquor solids (plant A)

Fig. 33 shows the relationship between the MLSS concentration and the % of settled sludge volume when the initial constant velocity section is terminated. If the MLSS concentration is about 2,000 to 3,000 mg/l, then the constant velocity section continues up to one-half of the water depth. Therefore, the initial settling velocity of the mixed liquor solids can be used for setting the required Settle Period.

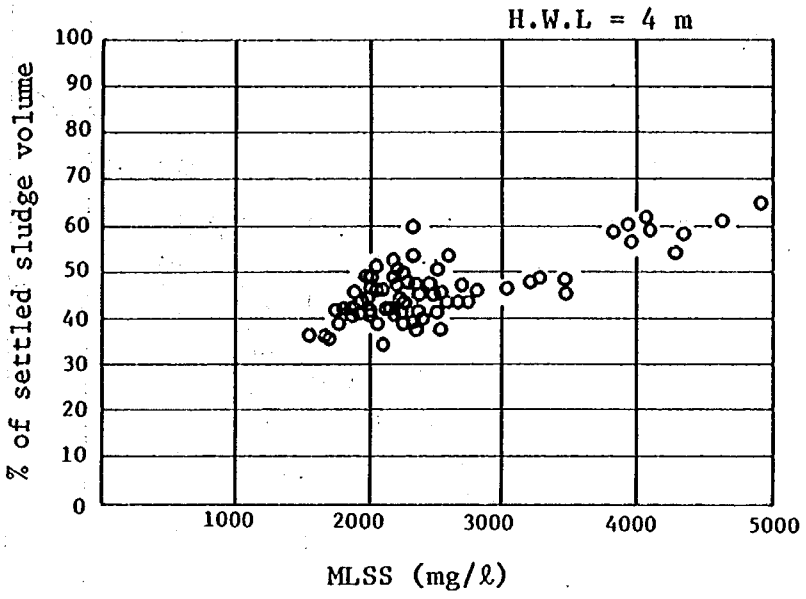


Fig. 33 Difference in % of settled sludge volume at the end of initial constant velocity section due to MLSS concentration

Shown in Fig. 34 is the relationship between water temperature and initial settling velocity of mixed liquor solids when the MLSS concentration is 1,800 to 2,200 mg/1.

As shown in equation (4), the initial settling velocity Vmax (m/h) is almost proportional to the water temperature T (°C).

$$Vmax = 0.185\ T^{0.97} \quad \cdots\cdots (4)$$

In order to see the effect of the MLSS concentration on the settling performance of mixed liquor solids, the relationship between Vmax/T and the MLSS concentration C_A (mg/1) is shown in Fig. 35. This relationship can be obtained as follows:

$$Vmax = 7.4 \times 10^4 \cdot T \cdot C_A^{-1.7} \quad \cdots\cdots (5)$$

As expressed in equation (5), the initial settling velocity of mixed liquid solids is governed by only the MLSS concentration and the water temperature.

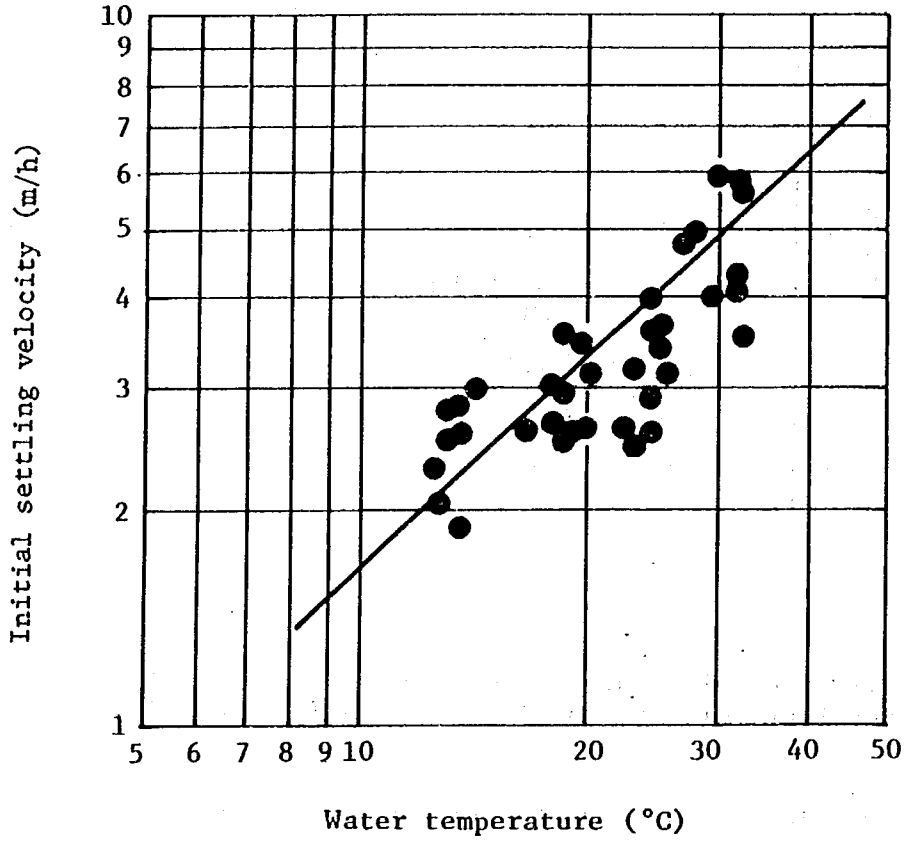


Fig. 34 Relationship between the water temperature and the initial setting velocity (plant B) MLSS: 1800 ∿ 2200 mg/ℓ

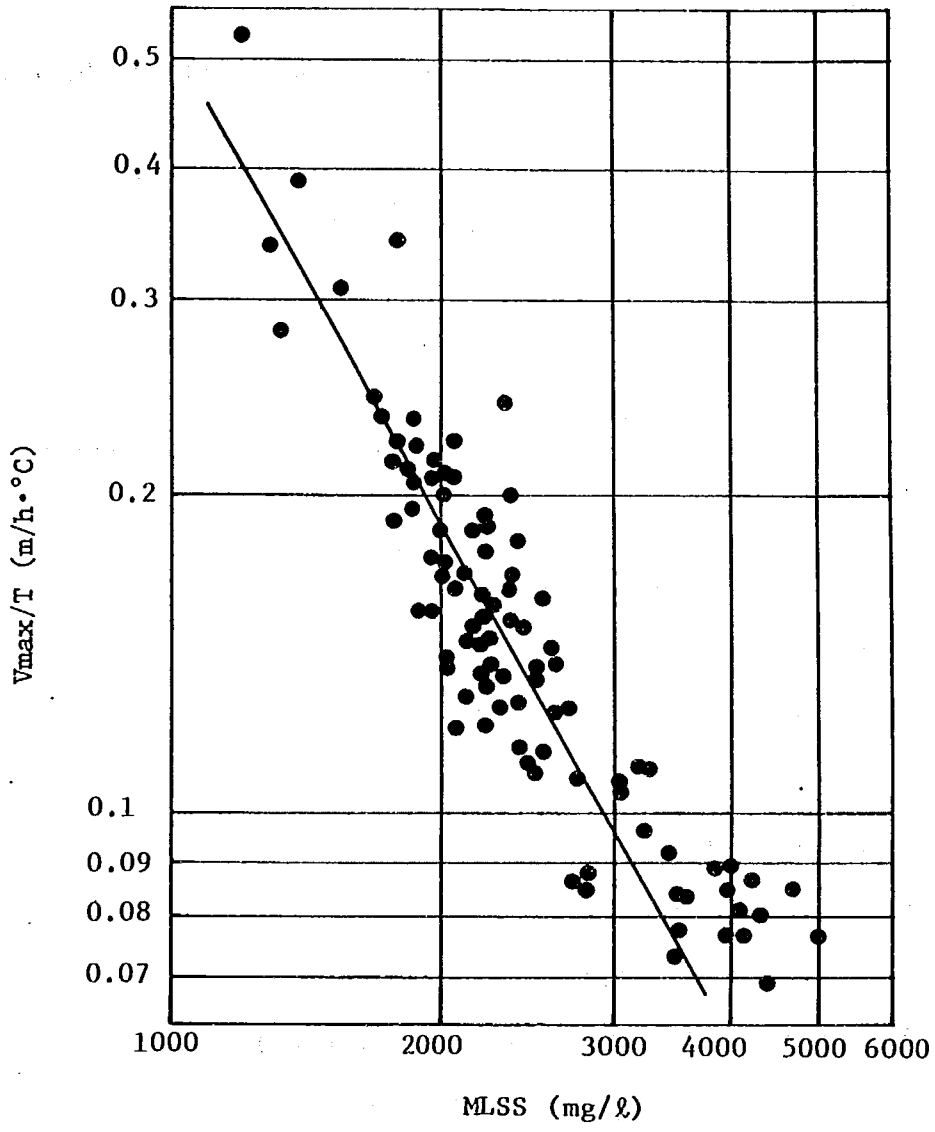


Fig. 35 Relationship between the MLSS concentration and the initial setting velocity (plant B)

3-3 Excess Sludge Production

In the SBR system, the Sludge Aeration Time (SAT) can be expressed as follows:

$$SAT = \frac{\text{Amount of sludge in reactor (kg)}}{\text{Amount of excess sludge (kg/day)}} \times e$$

$$= \frac{C_A \cdot V \cdot e}{n \cdot \frac{V}{m} \cdot C_S \cdot R_S \cdot \alpha_S}$$

$$= \frac{1}{L_S \cdot R_S \cdot \alpha_S} \quad \cdots\cdots (6)$$

where,

e : Aeration time ratio

C_A : MLSS (mg/1)

V : Volume of reactor (m^3)

n : Number of cycles (cycles/day)

1/m: Withdraw ratio

C_S : Influent BOD_5 (mg/1)

R_S : Removal ratio of BOD_5

α_S : Net sludge yield (kg-SS/kg-BOD_5 removed)

L_S : BOD-SS loading (kg-BOD_5/kg-SS · day)

Therefore, the net sludge yield α_S can be given by equation (7)

$$\alpha_S = \frac{1}{L_S \cdot R_S \cdot SAT} \quad \cdots\cdots (7)$$

Fig. 36 shows the relationship between the net sludge yield and the SAT in the pilot-scale experiments.

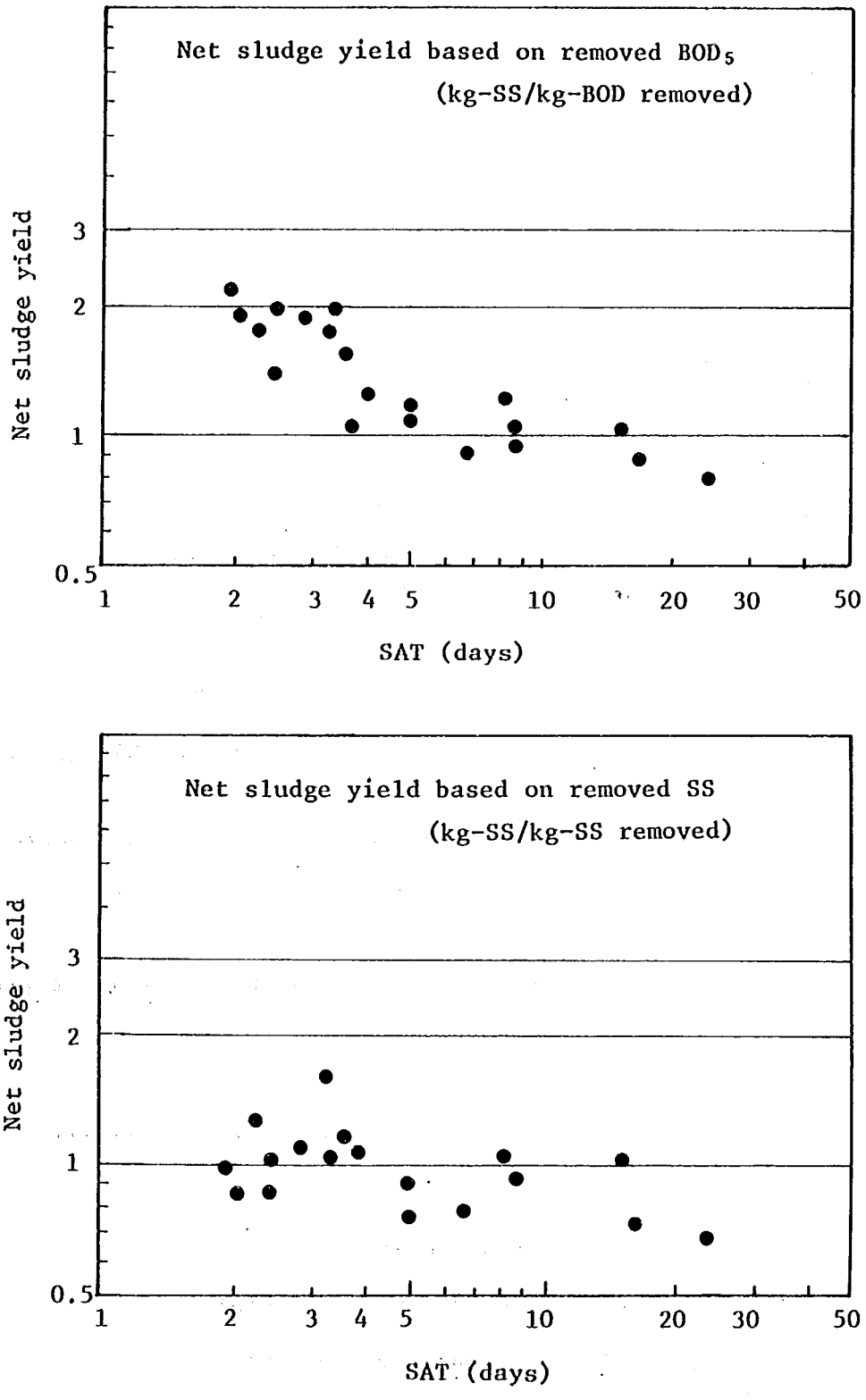


Fig. 36 Relationship between the SAT and the net sludge yield (pilot scale)

The relationship between the BOD-SS loading and the SAT is shown in Fig. 37.

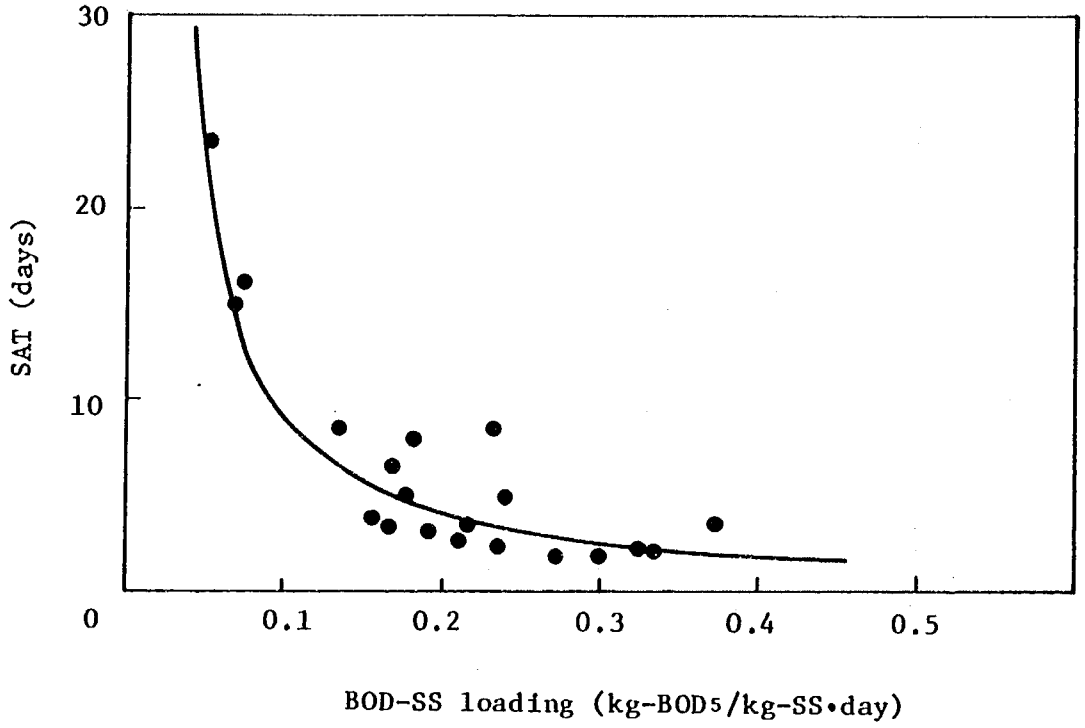


Fig. 37 Relationship between the BOD-SS loading and the SAT (pilot-scale)

The quantity of excess sludge generated increases as SAT decreases as a result of an increase in the BOD-SS loading.

The net sludge production in the SBR system can be expressed by equation (8).

$$\Delta M = a \cdot L_r - b \cdot \Sigma MLVSS \cdot T_A \quad \cdots\cdots (8)$$

Where,

ΔM : Net sludge production (kg-VSS/cycle)

L_r : Amount of BOD removed (kg-BOD_5/cycle)

Σ MLVSS: Amount of sludge in reactor (kg-VSS)

T_A : Aeration time (h/cycle)

a : Coefficient (kg-VSS/kg-BOD_5)

b : Coefficient (1/h)

The values for coefficients of a = 0.94 and b = 0.00081 are obtained on the basis of the pilot-scale experiments.

$$\Delta M = 0.94\ L_r - 0.00081\ \Sigma MLVSS \cdot T_A \quad \cdots\cdots\cdots (9)$$

Fig. 38 shows the comparison of the excess sludge production between measured and calculated by equation (9).

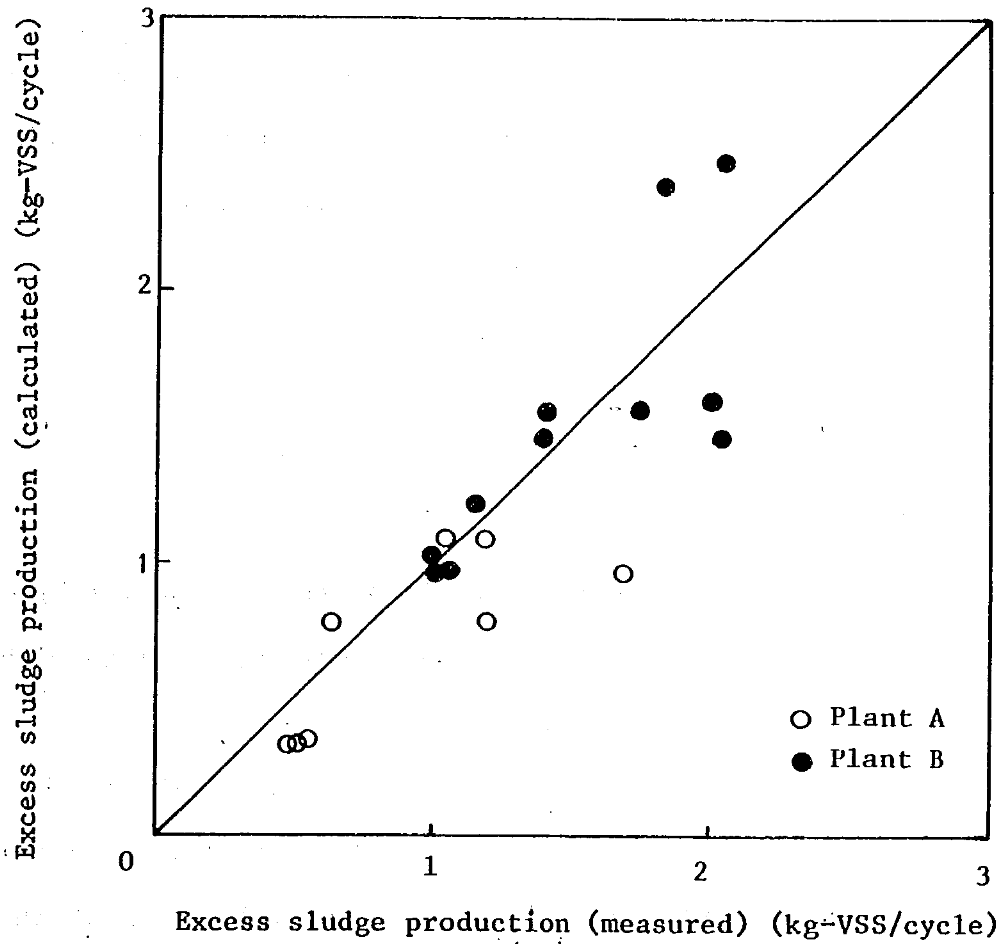


Fig. 38 Comparison of excess sludge production (pilot-scale)

4. SPECIFIC EQUIPMENT FOR SBR

4-1 Supernatant Discharge Devices

The following types of supernatant discharge devices are available:

Type- I: Discharging supernatant from an outlet fixed underwater.

Type-II: Discharging supernatant from an outlet fixed to a floating body. In this case, the floating body is lowered as the supernatant is discharged.

Type-III: Discharging supernatant from an outlet lowered mechanically.

Figs. 39 and 40 show the supernatant discharge devices produced on trial in the pilot-scale experiments, and can then be classified as follows:

Type-II: 1. Self-suction pump type (plant A)

2. Submersible pump type (plant A)

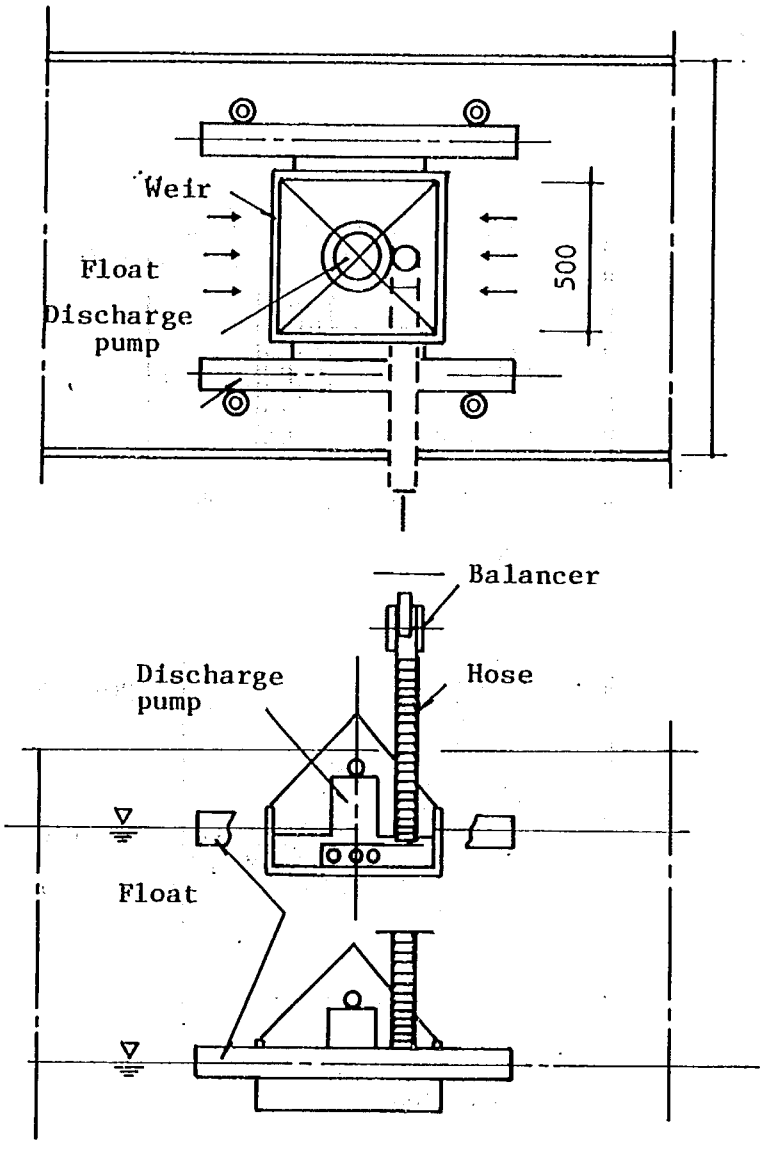
3. Weir-trough type (plant B)

4. Submerged weir-trough type (plant B)

Type-III: Decanter (plant A).

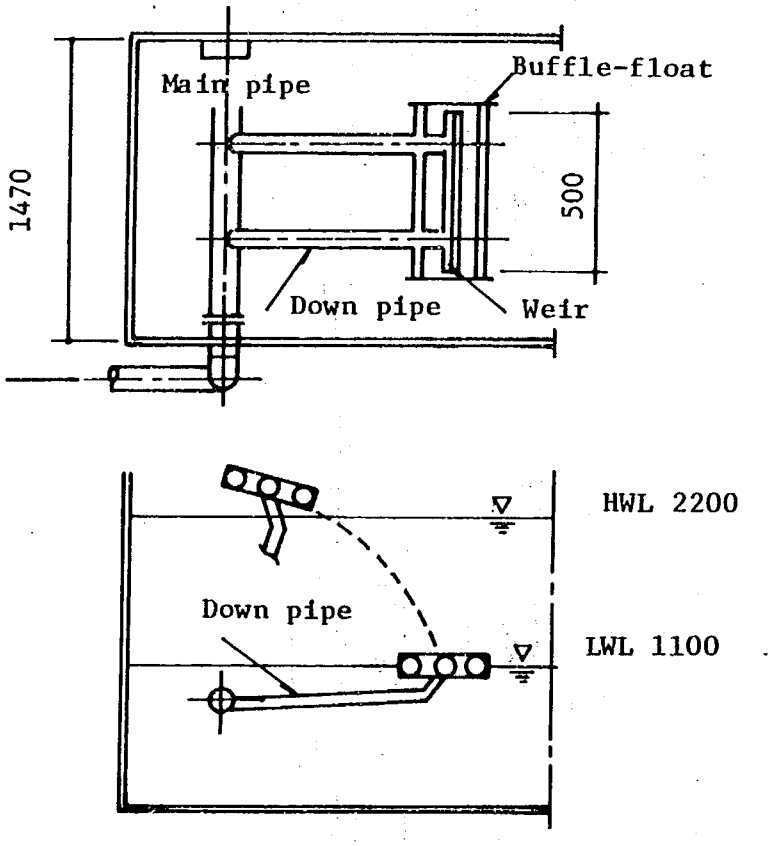
The decanter, patented by an Australian company, is available on the market. With respect to the Type-I discharge device, openings with a valve are installed on the side of the reactor.

In the design of the supernatant discharge devices, it is most important to determine the clearance between the lowest water level and the settled sludge surface which will not permit the rolling-up of the settled sludge at the predetermined overflow rate or discharge rate. It is also important to prevent the outflow of scum during discharge.



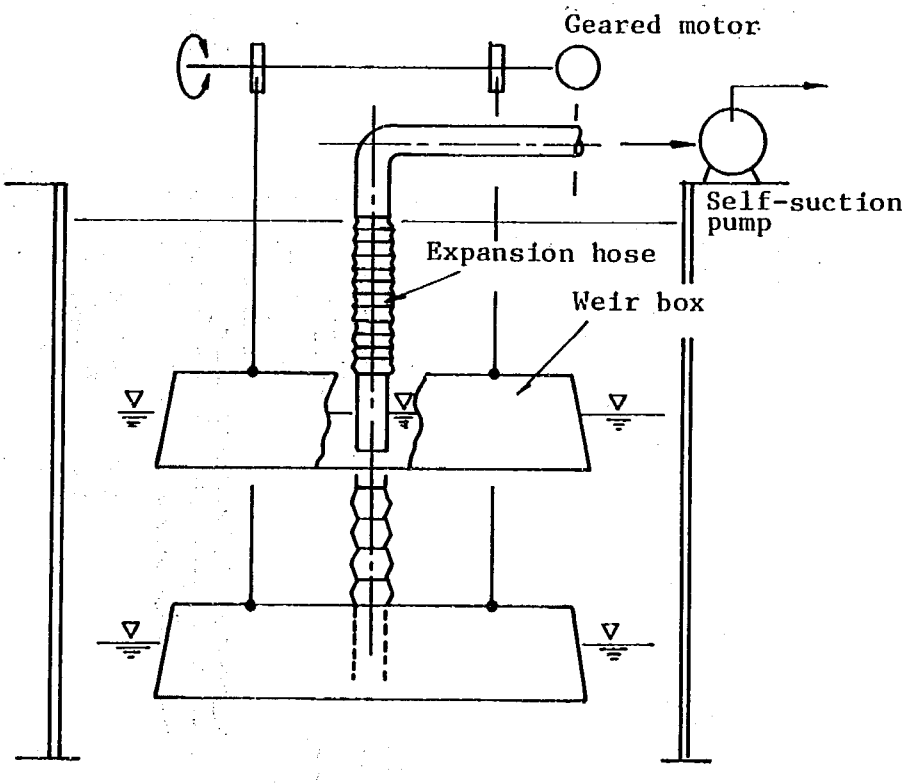
(Submersible pump type)

Fig. 39 Supernatant discharge devices (plant A)



(Decanter type)

Fig. 39 - Continued



(Self-suction pump type)

Fig. 39 - Continued

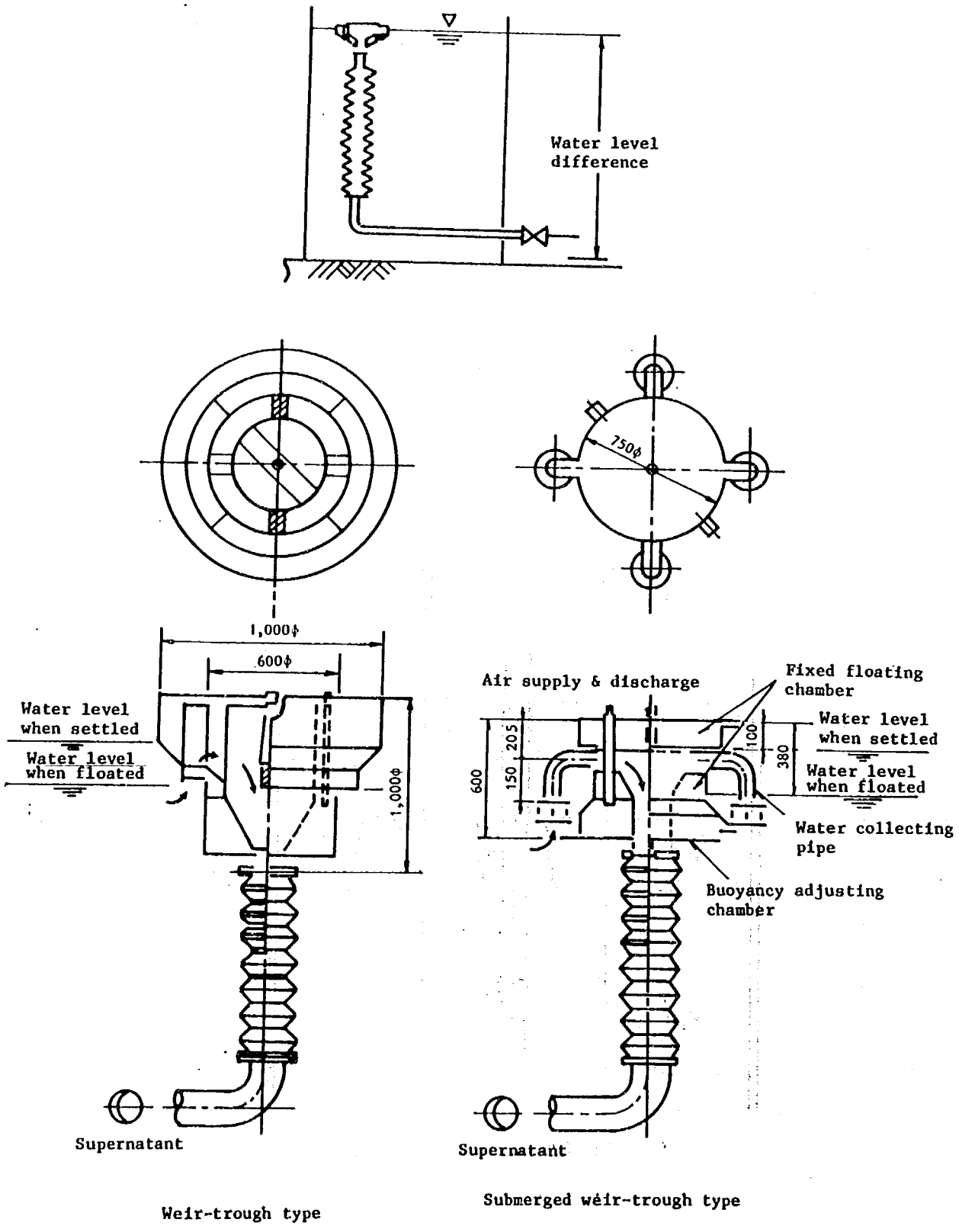
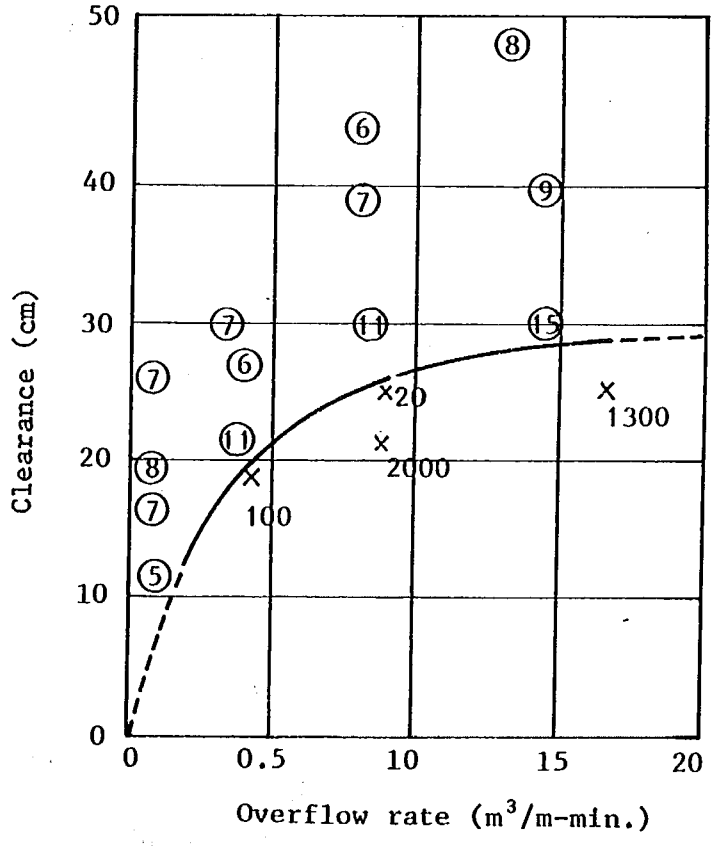


Fig. 40 Supernatant discharge devices (plant B)

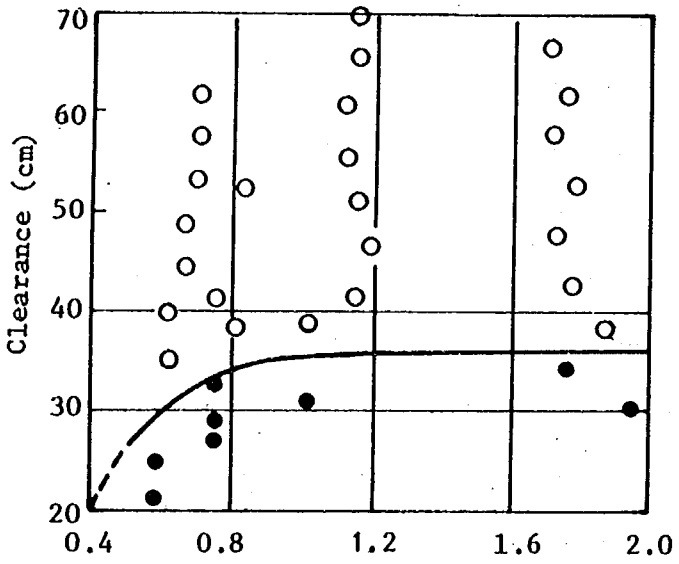
Figs. 41 to Fig. 45 show the relationships between the clearance and the SS in discharged supernatant at the various overflow (or discharge) rate for each type of discharge devices. In all cases, the clearance required for attaining the SS in overflow water below a certain level tends to increase as the overflow rate (or discharge rate) becomes larger.

The clearance between the settled sludge surface and the lowest water level that will not cause the rolling-up of the settled sludge varies depending on the overflow (discharge) rate and discharge devices. From these figures, the clearance required for making the effluent SS below 20 mg/l is summarized as shown in Table 8.



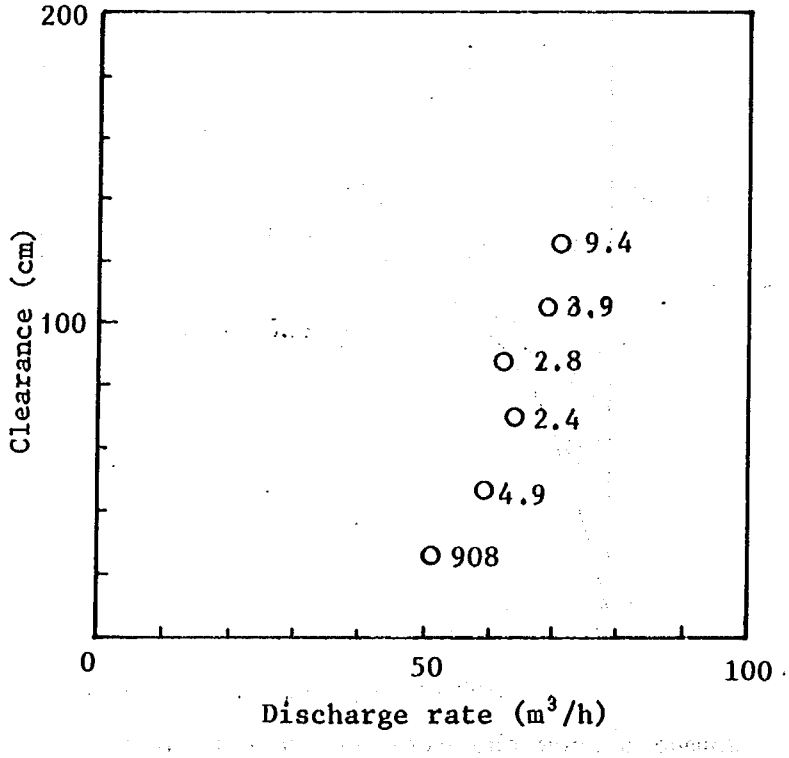
Numbers show the effluent SS concentration.

Fig. 41 Relationship between the overflow rate and clearance (decanter type, plant A).



○: SS > 20 mg/ℓ, ●: SS < 20 mg/ℓ.

Fig. 42 Relationship between the overflow rate and clearance (self-suction pump type, plant A)



Numbers show the effluent SS concentration

Fig. 43 Relationship between the discharge rate and clearance (weir-trough type, platn B)

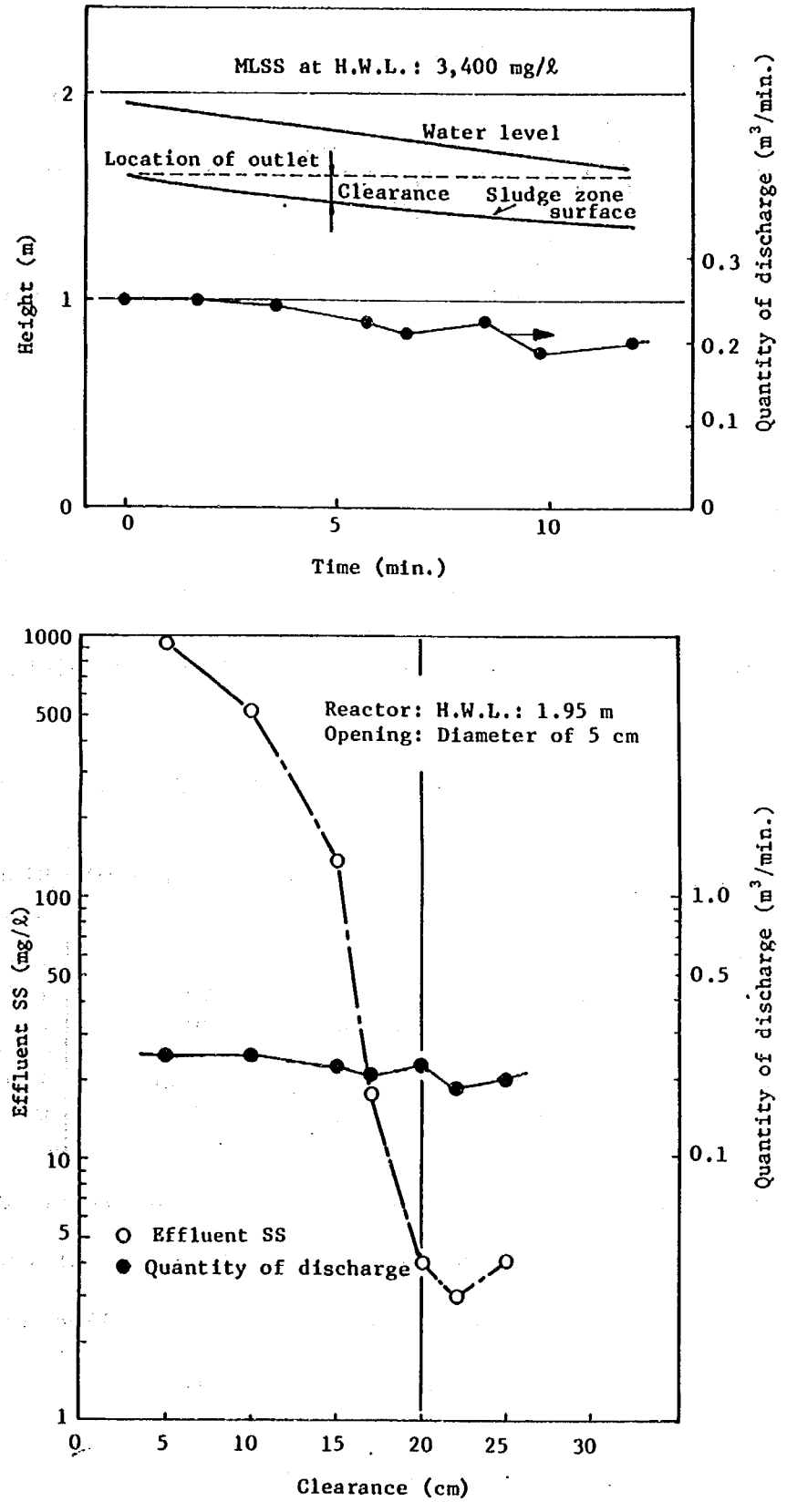


Fig. 44 Clearance in the case of discharge from side piping of reactor (plant A)

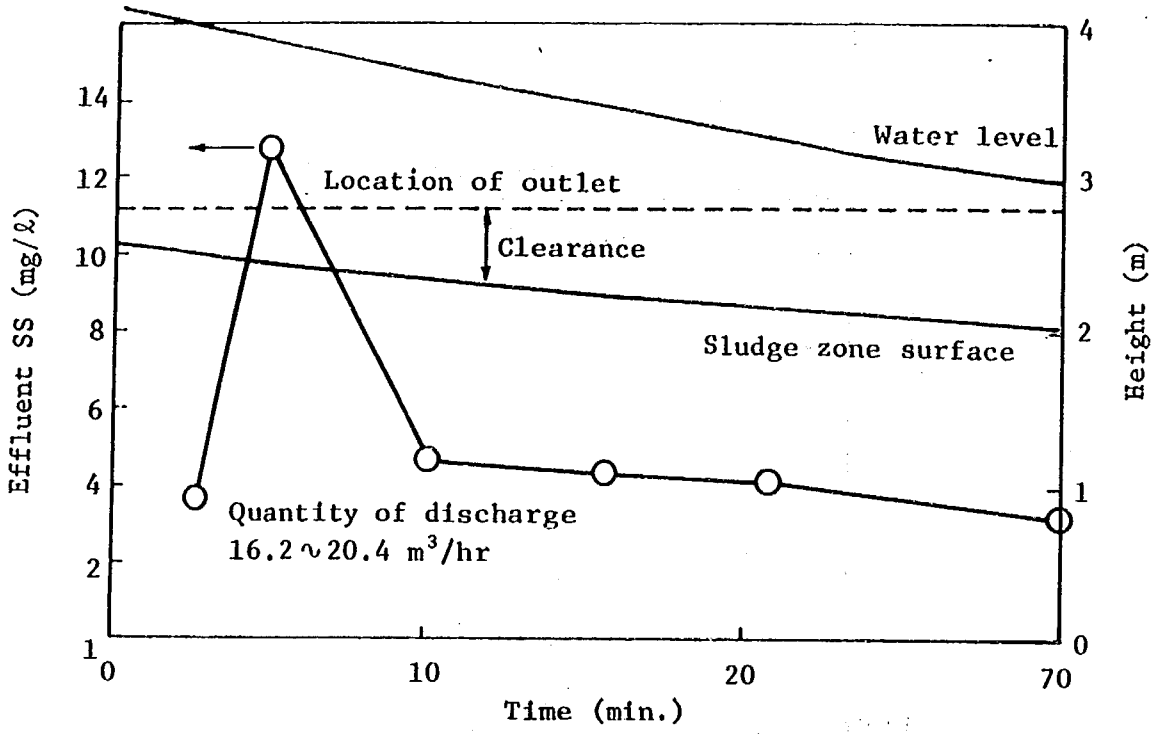


Fig. 45 Clearance in the case of discharge from side piping of reactor (plant B)

Table 8 Clearance required in various discharge methods

Discharge method		Clearance (cm)	Discharge rate
Plant A	Decanter type	30	1.5 m^3/m min
	Self-suction pump type	40	1.8 m^3/m min
	Discharge from side opening	20	0.2 m^3/min
Plant B	Weir-trough type	50∿60	1.0 m^3/min
	Discharge from side opening	50	0.3 m^3/min

Changes with time in the SS of discharge supernatant during the Draw Period is shown in Fig. 46. The value of SS increases at the time of start of discharge since part of the floating scum is rolled in, while the SS concentration of composite samples is low enough.

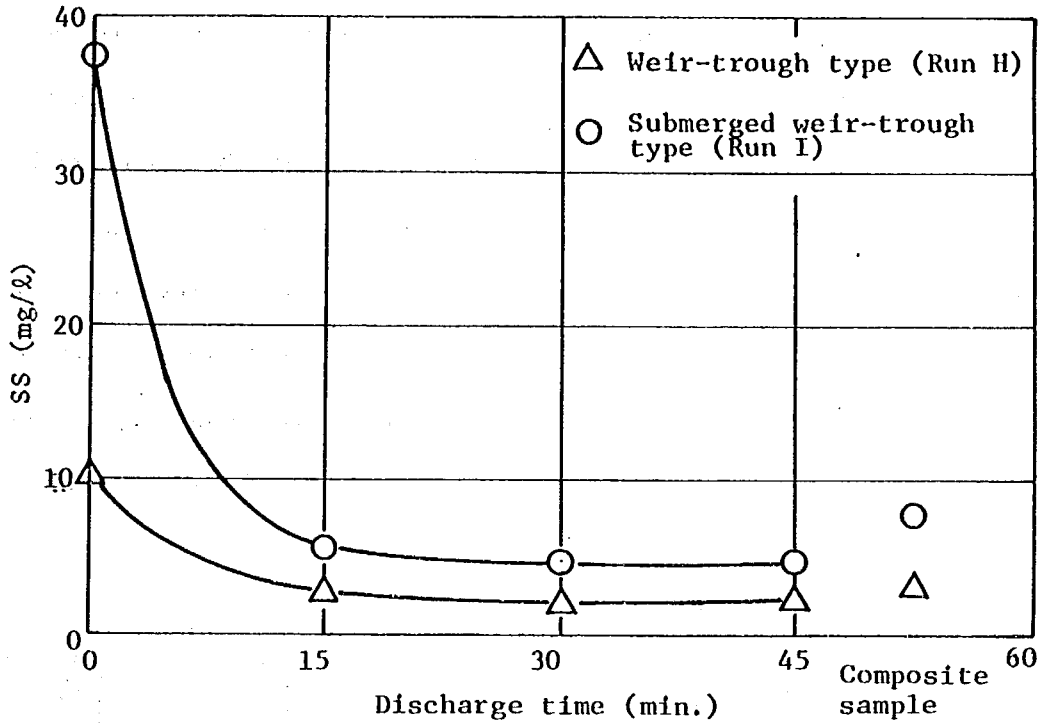


Fig. 46 Change with time in the SS of discharge water (plant B)

4-2 Built-in Sludge Thickener in the Reactor

Plant B has a newly developed built-in sludge thickener as shown in Fig. 47.

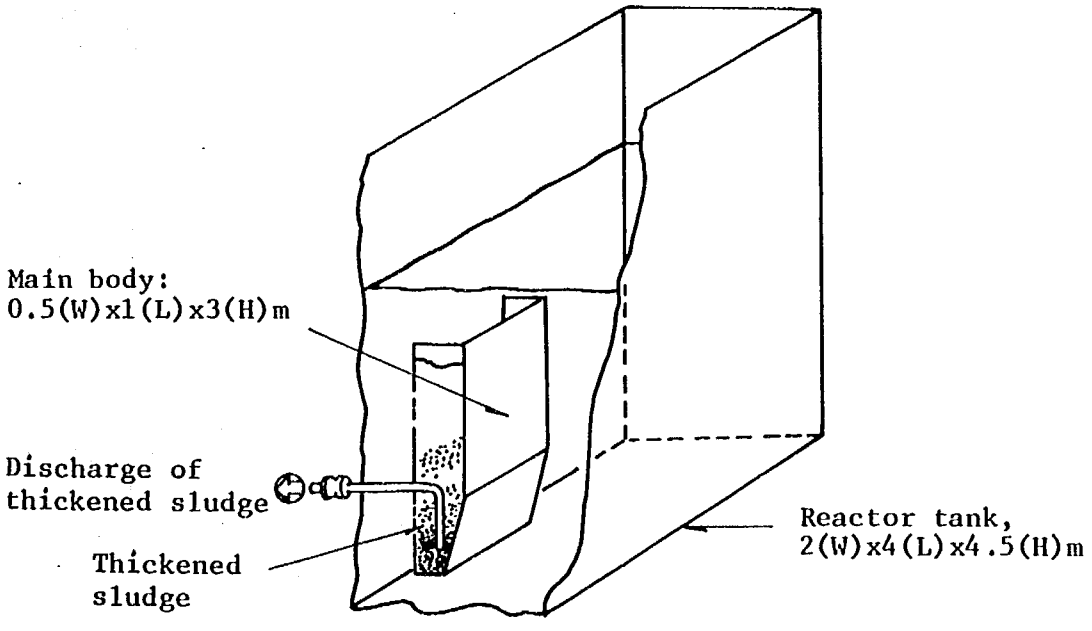


Fig. 47 Built-in sludge thickener in the reactor [equipment (1)]

In the sludge thickener, the mixed liquor solid is settled and thickened in a quiescent state during the Settle and Draw Periods. However, in the following Fill and Aeration Periods, part of the thickened sludge in the upper part of the thickener is affected by agitation and taken out of the thickener, by which the thickening efficiency is reduced. Because of this, as shown in Fig. 48, original equipment (1) was remodeled to the gallery type (equipment (2)) and then to the slit type (equipment (3)) in order to regulate the current in the upper portion of the thickener.

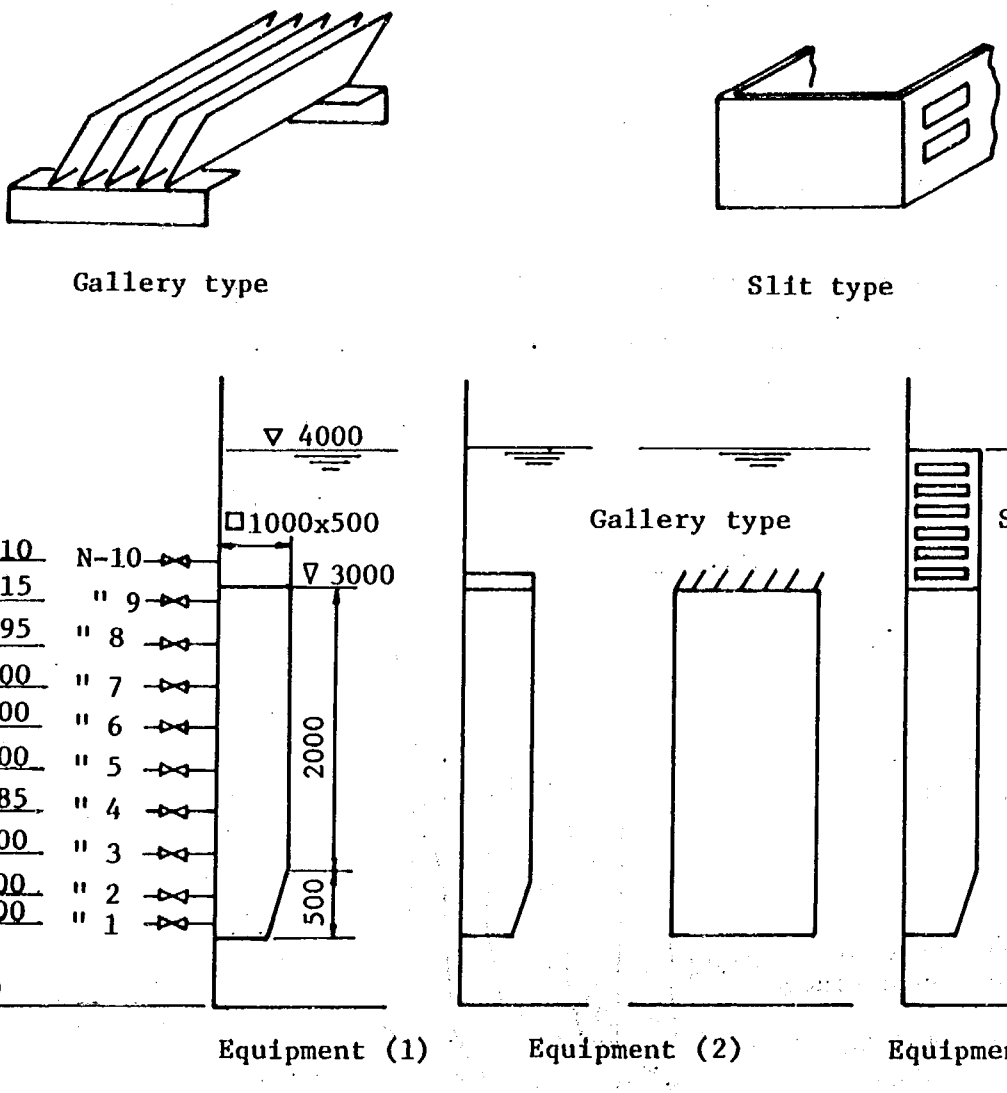


Fig. 48 Various types of built-in sludge thickener (plant B)

Thickened sludge was taken out manually about once a day. Table 9 shows the concentration of the thickened sludge and the quantity of the thickened sludge taken out at each time for equipment (1), (2) and (3). The slit type (equipment (3)) is found to be the most effective, and thickened sludge with a mean concentration of 2% was obtained.

Table 9 Comparison of performance of built-in sludge thickener (plant B)

Kind of equipment	Run	Concentration of thickened sludge (%)	Quantity of thickened sludge taken out (liters at each time)
(1)	C	1.41 (1.26∿1.57)	260∿1,040
	D	1.39 (1.06∿1.59)	260∿1,040
(2)	E	1.26 (1.03∿1.76)	130 ∿ 650
	G	1.33 (1.11∿1.73)	390 ∿ 520
	H	0.98 (0.80∿1.23)	390 ∿ 520
	J	1.42 (1.20∿1.87)	130 ∿ 650
	H	1.27 (1.01∿1.70)	260 ∿ 520
(3)	F	1.94 (1.35∿2.58)	130 ∿ 650
	I	2.07 (1.59∿2.56)	390 ∿ 650
	K	2.01 (1.53∿2.72)	260 ∿ 910
	L	2.02 (1.59∿2.46)	260 ∿ 650

Fig. 49 shows the change during one cycle in the distribution of sludge concentration in the height direction of the sludge thickener. In equipment (3), sludge thickening progressed even in the Aeration period and was least affected by agitation. From the above, it is confirmed that the slit type thickener in the reactor has a simple construction and maintenance and can be fully put to practical use.

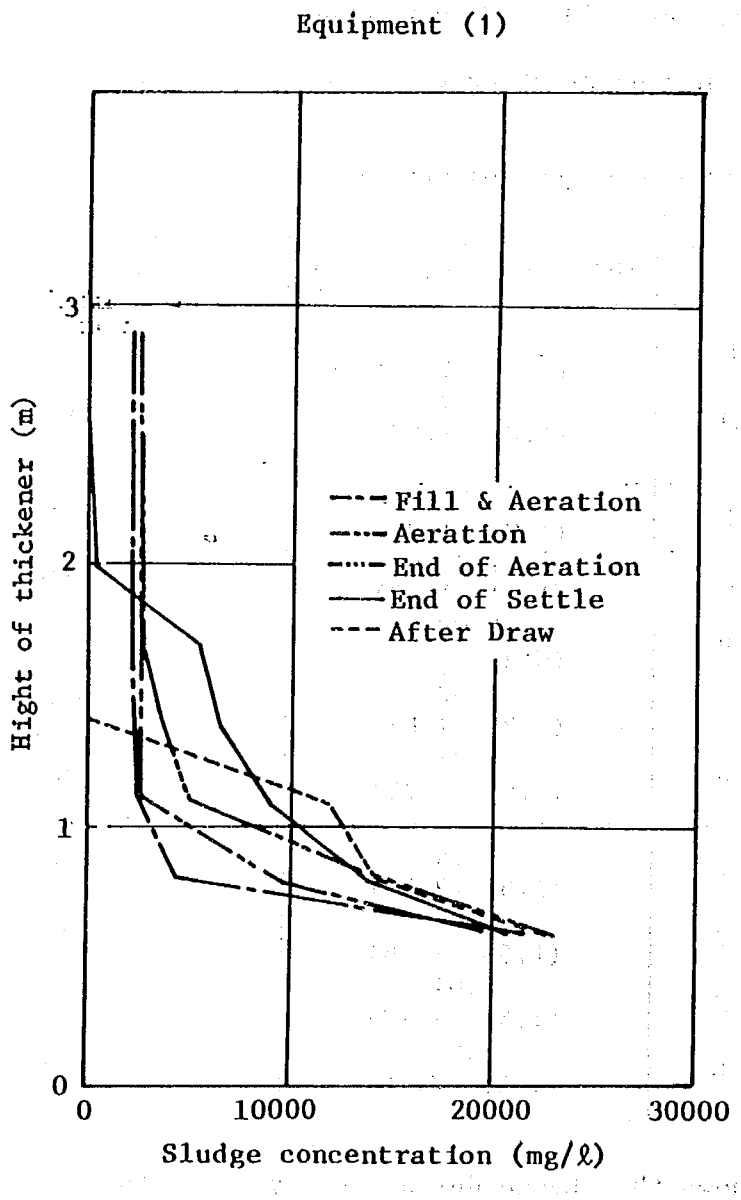


Fig. 49 Changes with time in sludge concentration in sludge thickener (plant B)

Equipment (2)

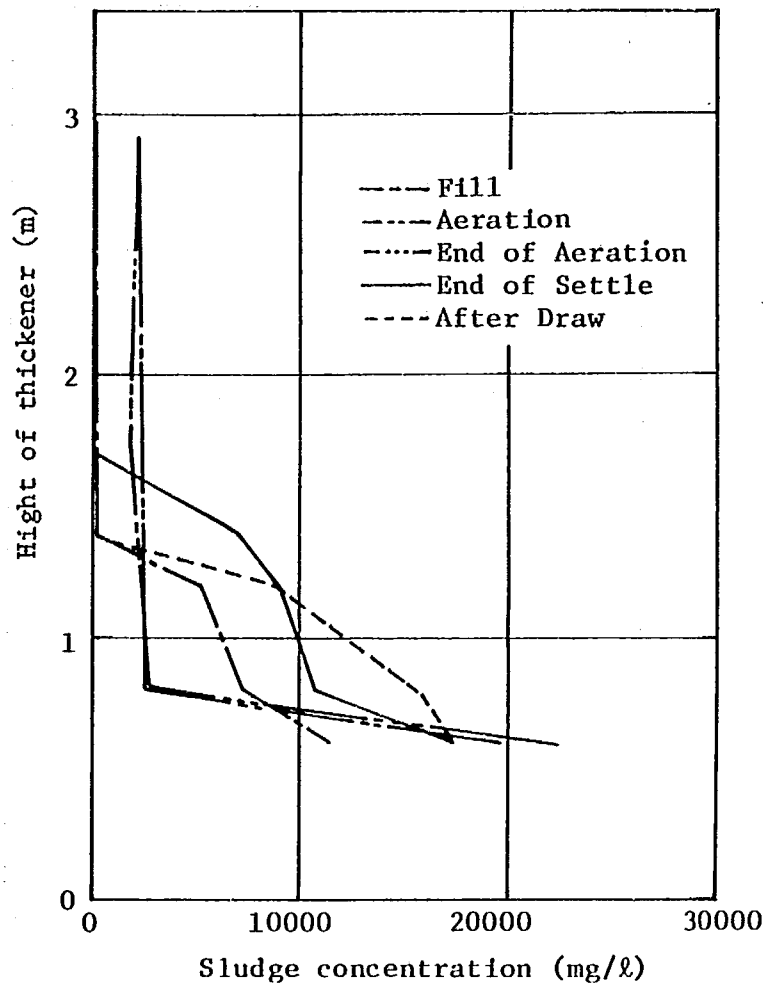


Fig. 49 - Continued

Equipment (3)

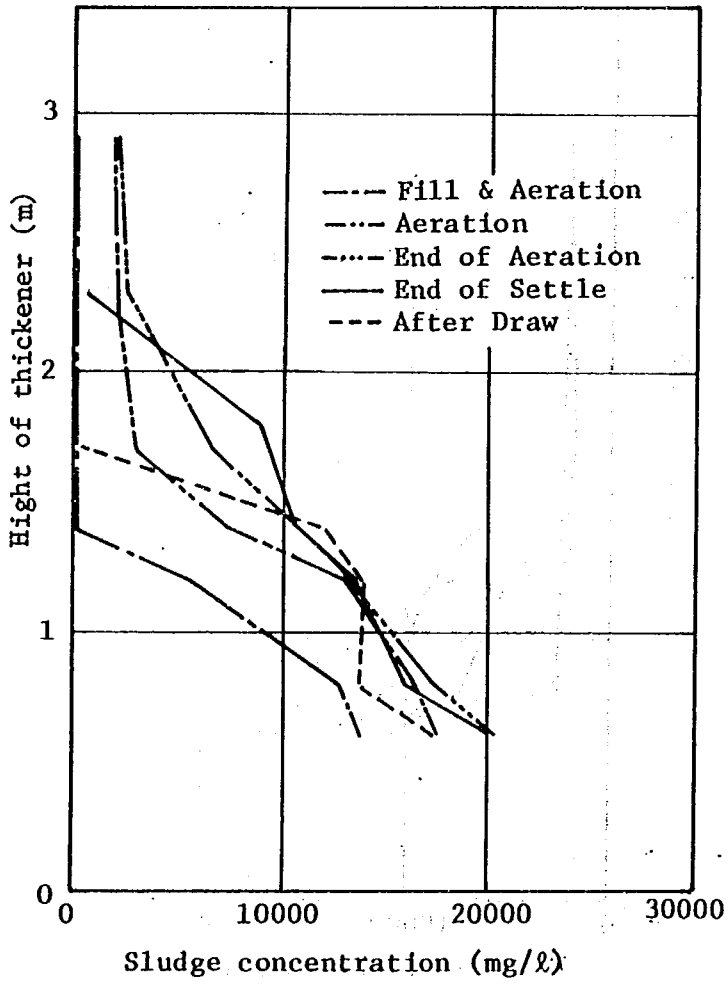


Fig. 49 - Continued

4-3 Oxygen Requirements

(1) Specific oxygen consumption rate

Shown in Figs. 50 and 51 are examples of changes in the substrate concentration and specitic oxygen consunpton rate Kr($mg\text{-}O_2/g \cdot MLSS \cdot h$) within the reactor in the pilot-scale experiments. In the SBR system, the substrate concentration changes with time within a reactor and, accordingly, Kr also changes. Kr at the end of the Aeration Period decreases as the organic loading becomes smaller.

On the other hand, if the organic loading is high or if the aeration time per cycle decreases, Kr at the end of the Aeration Period becomes larger, so that the amount of air supply has to be increased.

(2) Oxygen requirements

Fig. 52 shows the relationship between the quantity of oxygen required, which is determined from the measured oxygen consumption rate in the Aeration period, and the quantity of oxygen supply calculated from the oxygen transfer coefficient KLa, of the experimental apparatus. Both of them almost agree with each other.

The quantity of required oxygen (O_D ($kg\text{-}O_2/cycle$)) for the SBR system can be expressed by equation (10).

$$O_D = a \cdot L_r + b \cdot \Sigma MLVSS \cdot T_A + c \cdot N_o \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (10)$$

Where,

Lr : Amount of BOD_5 removed ($kg\text{-}BOD_5/cycle$)

Σ MLVSS: Amount of MLVSS in reactor (kg)

T_A : Aeration time (h/cycle)

N_O : Amount of nitrification (kg-N/cycle)

a : Coefficient ($kg\text{-}O_2/kg\text{-}BOD_5$)

b : Coefficient ($kg\text{-}O_2/kg\text{-}MLVSS \cdot h$)

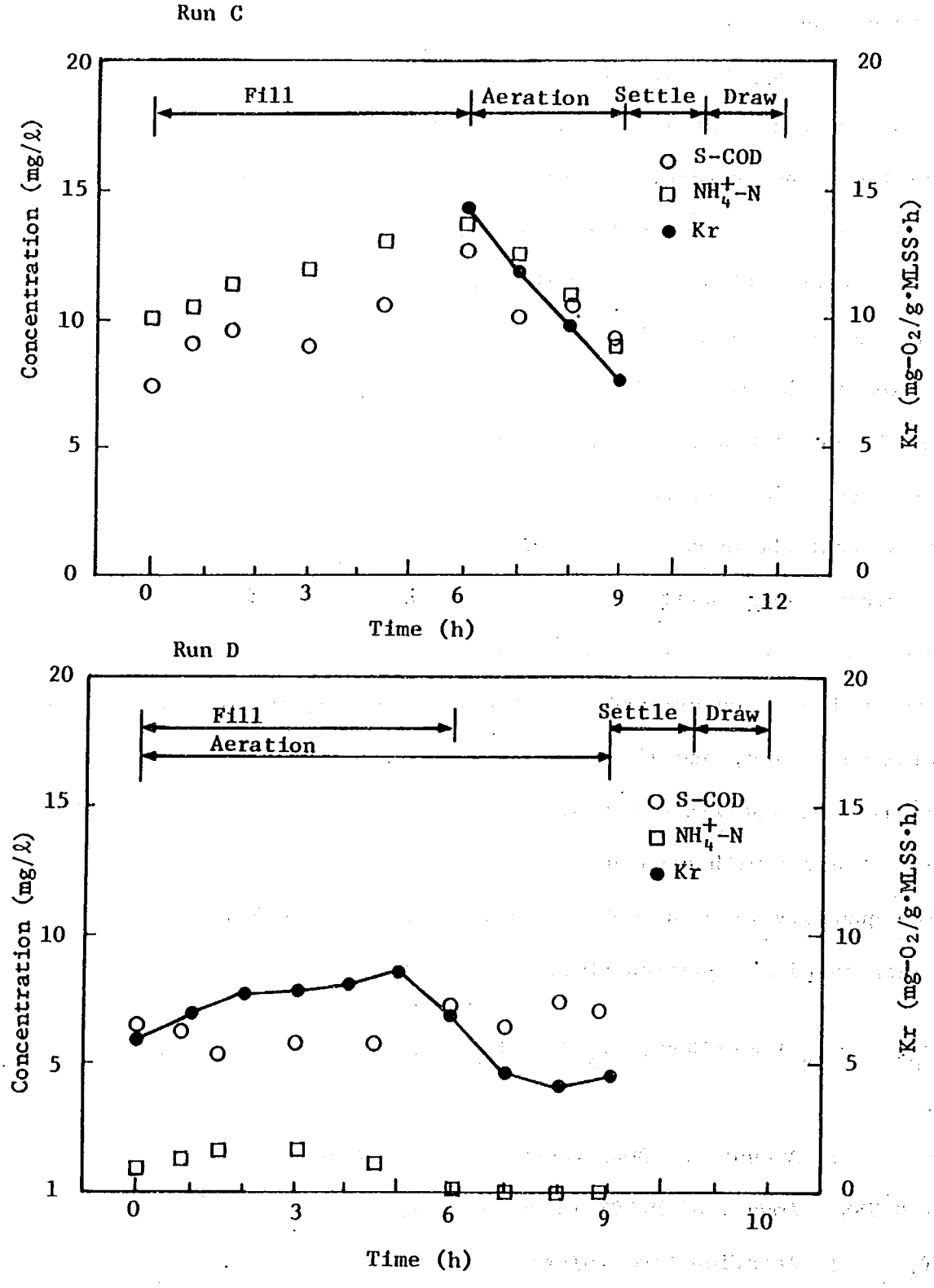


Fig. 50 Changes in substrate concentration and specific oxygen consumption rate (I) (plant B)

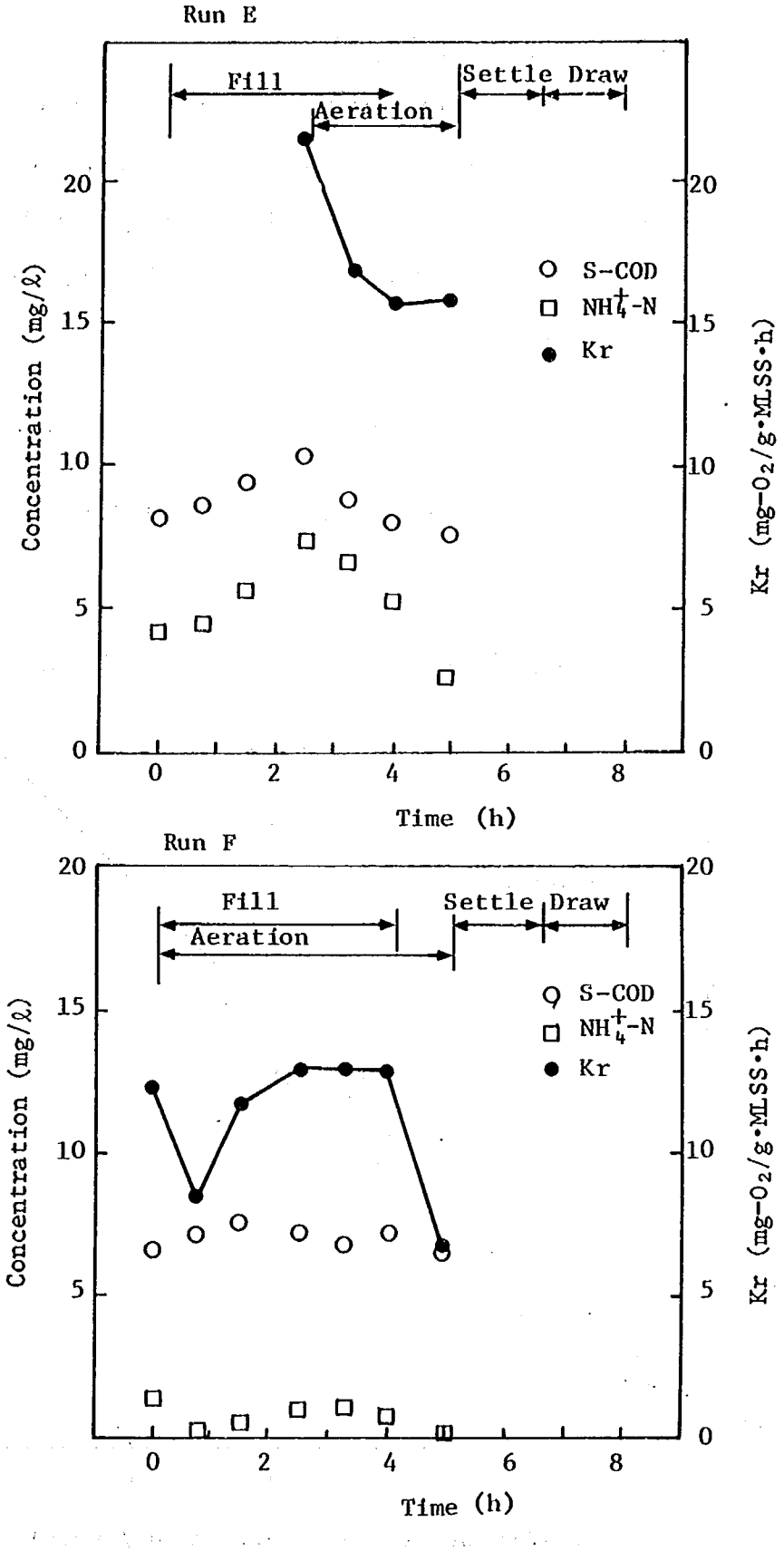


Fig. 51 Changes in substrate concentration and specific oxygen consumption rate (II) (plant B)

c : Coefficient ($kg\text{-}O_2/kg\text{-}NH_3^+\text{-}N$)

The coefficient c, nitrification oxygen demand, is theoretically 4.57 $kg\text{-}O_2/kg\text{-}NH_3^+\text{-}N$. Also, where the denitrification occurs, organic matter equivalent to the oxygen demand of 2.86 mg/l is required for 1 mg/l of NO_3^- -N. If the quantity of denitrification is N_D ($kg\text{-}NO_3^-$ -N/cycle), equation (10) can be modified as follows:

$$O_D = a(L_r - 2.86 \cdot N_D) + b \cdot \Sigma MLVSS \cdot T_A + 4.57 \cdot N_O \quad \cdots\cdots\cdots\cdots (11)$$

Using the results of pilot-scale experiments, coefficients a and b are determined as follows.

$$O_D = 0.38\ (L_r - 2.86 \cdot N_D) + 0.0048\ \Sigma MLVSS \cdot T_A + 4.57 \cdot N_O \quad \cdots\cdots\cdots (12)$$

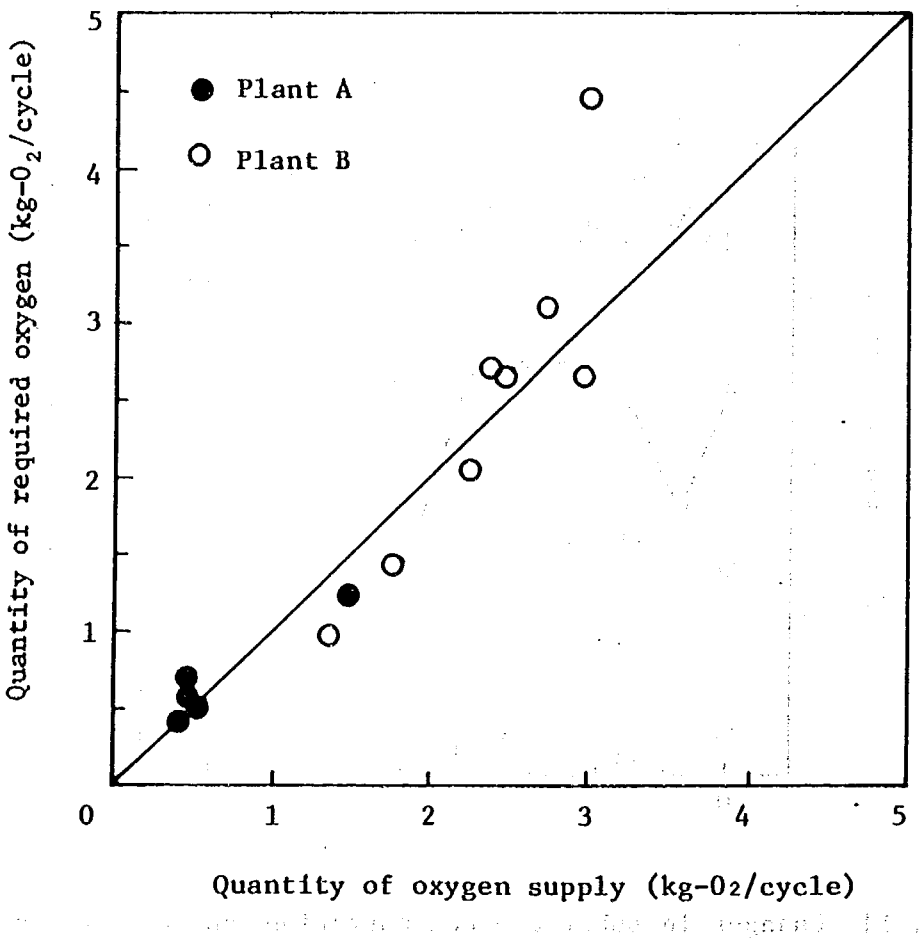


Fig. 52 Relationship between the quantity of required oxygen and the quantity of oxygen supply (pilot-scale)

Fig. 53 shows a comparison between the measured values and the calculated values of the oxygen requirements obtained from equation (12). Both of them agree relatively well with each other.

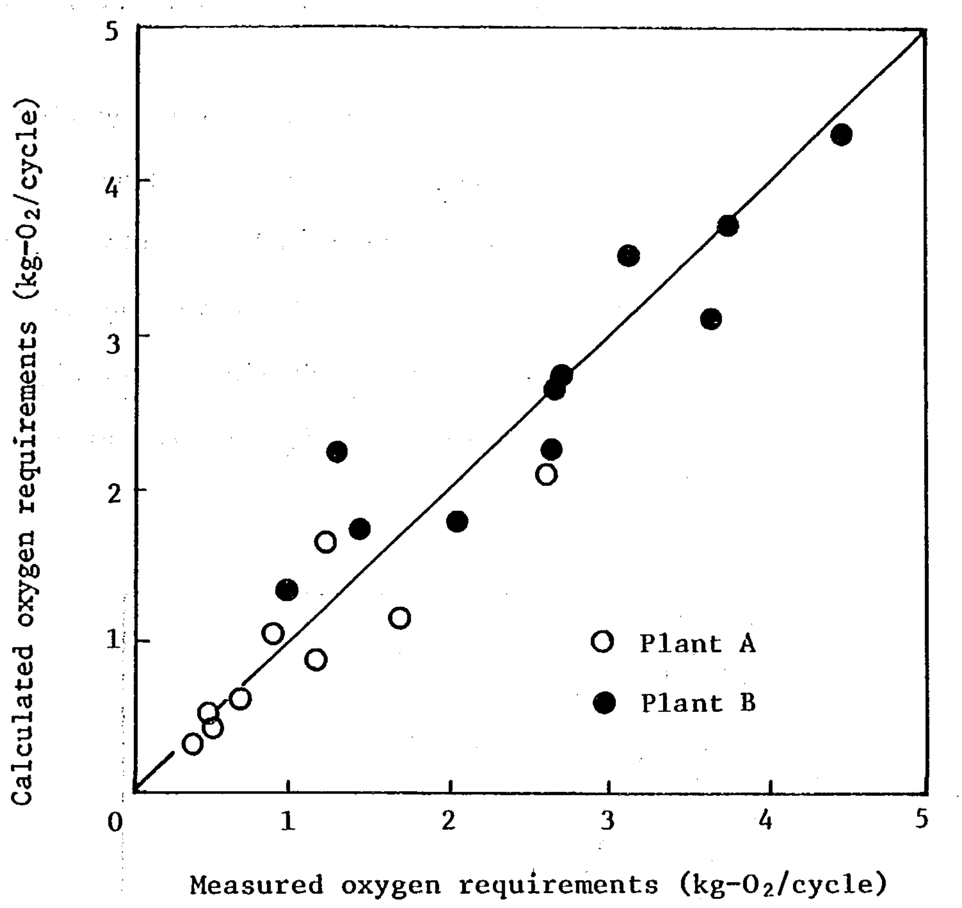


Fig. 53 Forecast of oxygen requirements (pilot-scale)

5. SUMMARY AND CONCLUSIONS

(1) The performance of organic matter removal can be summarized by the BOD-SS loading which is given by the followeing equation,

$$L_S = \frac{n \cdot C_S}{e \cdot m \cdot C_A} \quad \cdots\cdots (13)$$

Since the aeration time ratio $e = n \cdot T_A/24$, the aeration time T_A per cycle can be obtained from the equation (13):

$$T_A = \frac{24 \cdot C_S}{L_S \cdot m \cdot C_A} \quad \cdots\cdots (14)$$

Fig. 54 shows the required aeration time T_A by using the equation (14). In this case, the relationship between the BOD-SS loading and T_A is shown for four MLSS concentrations with the withdraw ratio of 1/2 and the influent BOD_5 of 140 mg/l. For the required removal ratios of BOD_5 and SS, the necessary BOD-SS loading can be obtained from Fig. 7 and Fig. 13, so that the required aeration time T_A can be obtained from Fig. 54.

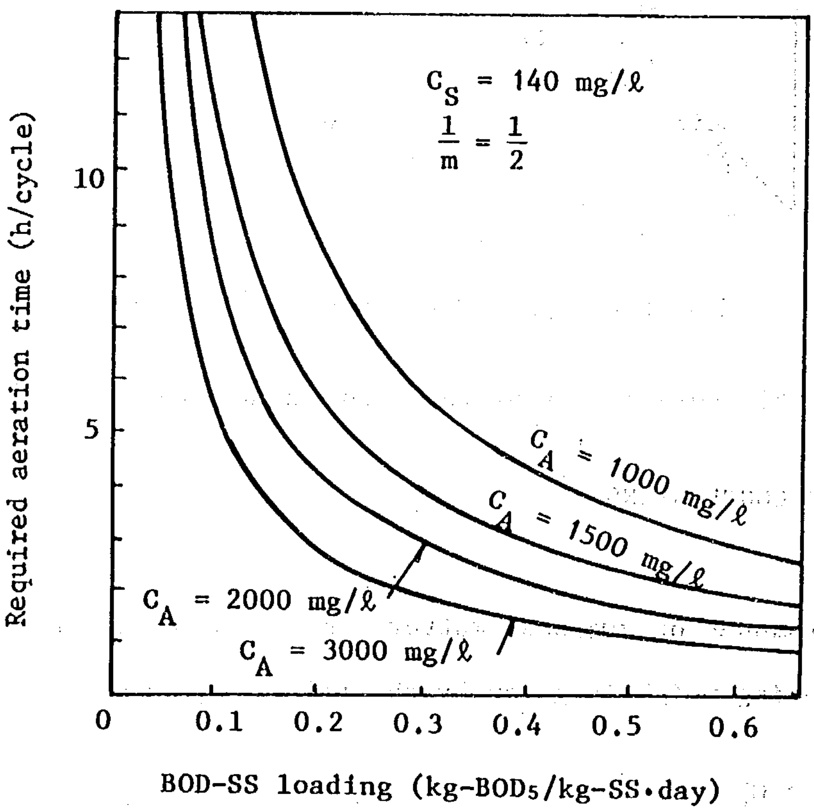


Fig. 54 Relationship between the BOD-SS loading and the required aeration time

(2) Removal of Kjeldahl nitrogen is made by both the nitrifying reaction and the discharge of excess sludge to outside the system. The quantity of removed Kj-N by nitrifying reaction increases when the BOD-SS loading is low. On the other hand, the net excess sludge production increases under conditions of high BOD-SS loading, so that the amount of Kj-N removed due to discharge of excess sludge also increases.

For obtaining a high removal ratio of T-N, it is necessary that denitrification be performed fully in the anaerobic period. Denitrification can be accelerated by reducing the withdraw ratio or by performing intermittent aeration.

(3) Phosphorus removal can be made more effectively as the excess sludge production increases and phosphorus content in sludge becomes larger. Therefore, conditions of a high BOD-SS loading are advantageous for biological phosphorus removal.

(4) The settling time required for one cycle can be determined by the intial settling velocity of mixed liquor solids. It is found that the initial settling velocity can be predicted from both the MLSS concentration and the water temperature as follows.

$$V_{max} = 7.4 \times 10^{4} \times T \times C_A^{-1.7} \quad \cdots\cdots (15)$$

The required settling time T_S can be determined by equation (16).

$$T_s = \frac{H \times (1/m) + \beta}{V_{max}} \quad \cdots\cdots (16)$$

where,

H : Water depth in reactor (m)

1/m: Withdraw ratio

β : Clearance between the lowest water level and the settled sludge surface (m)

(5) Discharge characteristics of several kinds of supernatant discharge devices produced for trial are clarified.

(6) By employing a newly developed simple built-in sludge thickener in the reactor, it is possible to obtain thickened sludge with a concentration of 2%. The maintenance for this sludge thickener is sufficiently simple for it to be used at small-scale treatment facilities.

6. REFERENCES

(1) The First Evaluation Report of the Sequencing Batch Reactor for Municipal Wastewater Treatment (draft), Technology Evaluation Committee, Japan Sewage Works Agency (March 1986).

(2) Goronszy, M.C., Intermittent Operation of the Extended Aeration Process for Small Systems, Jour. WPCF, Vol. 51, No. 2 (1979).

(3) Irvine, R.L., Fox, T.P., and Richter, R.O., Investigation of Fill and Batch Periods of Sequencing Batch Biological Reactors, Water Research Vol. 11, p. 713 (1977).

(4) Technology Evaluation of Sequencing Batch Reactors, James M. Montgomery Consulting Engineers, Inc. (1984).

SESSION 5

SLUDGE DISPOSAL AND UTILIZATION

"Study on Sludge Melting Process and Effective Utilization of Slag Generated"
Y. Yuki 503
"Sewage Sludge Utilization and Disposal Practices in the Federal Republic of Germany
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"Long-Term Continuous Application of Sewage Sludge in Test Farms"
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SUBJECT INDEX AND ANNOTATION

STUDY ON SLUDGE MELTING PROCESS AND EFFECTIVE UTILIZATION OF SLAG GENERATED

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ABSTRACT

The conventional sludge incineration system requires a large quantity of auxiliary fuel, and poses a problem with regard to disposal of incineration ash; therefore, a new technique is required to permit further stabilization and volume reduction of sludge as well as enhancing energy-saving effect. Cyclone melting furnace system combined with drying is expected to meet the requirements above; experiments were conducted by using a cyclone melting furnace with treatment capacity of 2.4t-sludge/day.

The results of the present experiment are summarized as follows:

(1) It was proven that sludge is easily molten to slag. Volumes of organic sludge (conditioned with polymer) and inorganic sludge (conditioned with lime) were reduced by slagging to approx. 1/30 and 1/16, respectively. The volume reduction rate was 1/2 to 1/3 of that with incineration ash obtained by multiple-hearth furnace conventionally employed.

(2) The melting furnace used in the present experiment, the treatment capacity of which is 2.4t-sludge/day, consumed heavy oil as an auxiliary fuel at the rate of 32.8ℓ/t-sludge in the case of organic sludge. In an actual system which can treat 200 ∿ 300 ton of sludge per day, the rate of auxiliary-fuel consumption is less than 10ℓ-heavy oil/t-sludge due to lower heat loss ratio; it is inferred that the system consumes only 1/2 ∿ 1/4 of the auxiliary-fuel comsumped in the conventional multiple-hearth incinerator.

(3) Both exhaust gas production and NO_x production were successfully reduced by using the two-staged combustion method at a low air ratio. Heavy metals in slag proved to be immobilized and no elution was found; it has been proven taht these heavy metals are stable in slag.

(4) Slag satisfied all requirements of the JIS (Japan Industrial Standards) for aggregate. In addition, concrete test gave evidence of the possibility of using slag as an aggregate for concrete.

(5) As the result of laboratory experiments, the minimum slag melting temperature was obtained by adjusting basicity (CaO/SiO_2) to approx. 1. Crystallized slag was successfully obtained either by maintaining slag at a temperature approx. 50°C lower than its melting point for a specified period or by cooling it gradually.

1. INTRODUCTION

Osaka's sewerage system accommodates almost the entire municipal area, service ratio reaching 99.4% with respect to population as of March, 1986, the highest level among the large cities in Japan.

Sewage generated in the municipal area is for the most part treated at the twelve sewage treatment plants using the activated sludge process. Sludge generated at those treatment plants amounts to 860 tons (of dewatered sludge) per day on average. As the sewerage system advances, the volume of sludge generated is expected to increase. A large volume of the sludge presently generated is incinerated in vertical multiple-hearth furnaces, the rest being solidified via cement and used together with municipal refuse for reclamation.

In the future, however, it seems it will be difficult to obtain disposal sites not only on land but at sea. The shortage of disposal sites is a common problem confronting municipalities operating sewage treatment plants. Osaka City, therefore, is planning to incinerate whole sludge in order to minimize its volume. In addition, development of sludge melting technology is currently underway, for further volume reduction and stabilization.

conventionally, Osaka City employs multiple-hearth furnaces, which permit stable incineration of a large volume of sludge in a limited space. Recently, however, as the vacuum filters mainly used in Osaka City are increasingly being replaced by centrifuges using organic coagulant (polymer), various problems have arisen, including difficulty in furnace temperature control due to the high calorific value of sludge, clinker generation at the combustion section and increased exhaust gas odor. Furthermore, 25 ∿ 40% of auxiliary fuel (calculated in terms of heavy oil) is required to incinerate 1t of sludge; technical problems also arise with respect to energy saving.

Recently, therefore, increasing numbers of large cities have begun employing large fluid-bed incinerators equipped with drying machines. This system seems as though it may predominate for a time; however, problems in terms of fly ash and the possibility of heavy metal elution remain unresolved.

To convert sludge into slag, arc furnaces and microwave melting furnaces have been under development for over ten years; recently it has been proven that slagging of sludge is feasible at a cost almost equivalent to that required in the ordinary incineration process. Osaka City considers slagging of sludge to be very promising as regards volume reduction and stabilization at the disposal site and the effective utilization of slag as construction material.

Since 1984, we have conducted drying-incineration/melting experiments using a pilot plant with daily treatment capacity of 2.4t sludge, and various other experiments for effective utilization of generated slag. Through such experiments we are being increasingly convinced of the promise of slagging technology.

2. OUTLINE OF SLUDGE SLAGGING

2-1 Advantages of Sludge Slagging

Generally, sludge slagging as thermal processing is defined as follows: When solid sludge is heated to 1,200 ∿ 1,500°C, organic matter is decomposed and burnt, remaining inorganic matter being molten to fluid. Molten inorganic matter, when cooled and solidified, is called slag. Thus thermal processing of sludge into slag is called slagging of sludge. However, slagging of incineration ash is also feasible and can be employed for the process subsequent to incineration.

Compared with incineration ash, slag has the following advantages in disposal:

(1) High bulk density and reduced volume

(2) High stabilization, without problems in terms of fly ash and elution of heavy metal at disposal site

(3) High feasibility of utilization as aggregate for construction

Slagging, therefore, if as economically efficient as incineration and if permitting easy plant maintenance and control, can replace conventional incineration as a promising sludge thermal process.

2-2 Outline of Melting Systems

At present, two types of melting plants are used in Japan: the arc furnace system and the coke-bed furnace system. There are also two additional systems under development, the surface melting furnace system and the cyclone melting furnace system, both having been studied at pilot plants, and now presumably reaching the stage of practical use. These four systems are outlined below.

(1) Arc furance system

This system, similar in principle to the arc furnace system generally used for steel production, introduces current to graphite electrodes located inside furnace, causing arc discharge across electrodes and base metal, melting inorganic matter with the heat thus generated.

There are two types of such furnaces available, for dried sludge melting and incineration ash melting, the former being employed at Kase Sewage Treatment Plant in Kawasaki City. In terms of running cost, the latter type is considered superior. As this arc furnace system uses electric power as heat source, it is economically more efficient to melt incineration ash already reduced in volume. (Fig. 2-1)

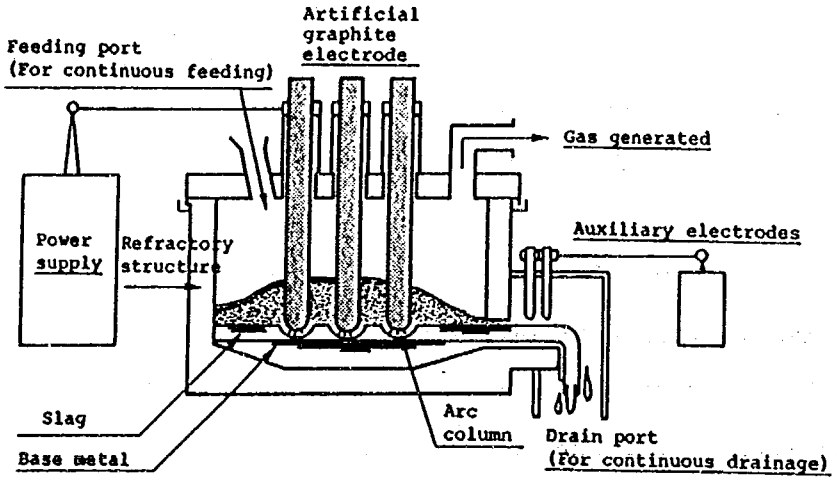


Fig. 2-1 Arc Melting Furnace

(2) Coke-bed furnace system

This system, employing a cupola for sludge melting, feeds granulated sludge dried to around 50% water content onto a coke bed, where it melts with the high heat released by coke combustion. This system requires a certain amount of coke to form a coke bed. If this amount exceeds that needed to produce the heat required in the process, the furnace discharges excess energy. It is therefore a key point in designing the system to use this excess energy as effectively as possible. At present, a plnat (40t/day) using this system is in operation at Chuo Sewage Treatment plant in Osaka Prefecture. (Fig. 2-2)

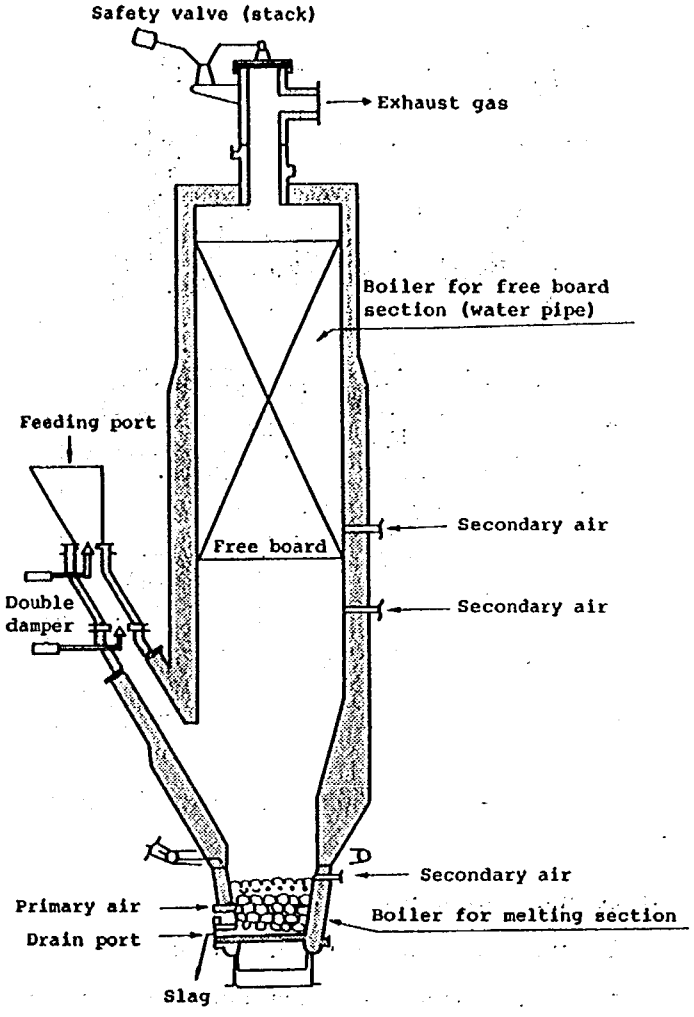


Fig. 2-2 Coke-bed Furnace

(3) Surface melting furnace system

This double cylinder furnace, with dewatered sludge placed around its core, forms a type of reverberatory furnace, via combustion of Auxiliary fuel and sludge, to melt sludge from the surface. As the water content of fed sludge should be as low as 20%, drying is required prior to melting. With this system, therefore, it is a key point in designing to effectively use waste heat discharged from the furnace in the drying process. Melting of incineration ash is also feasible in the furnace; the system can therefore be designed to permit melting after incineration. (Fig. 2-3)

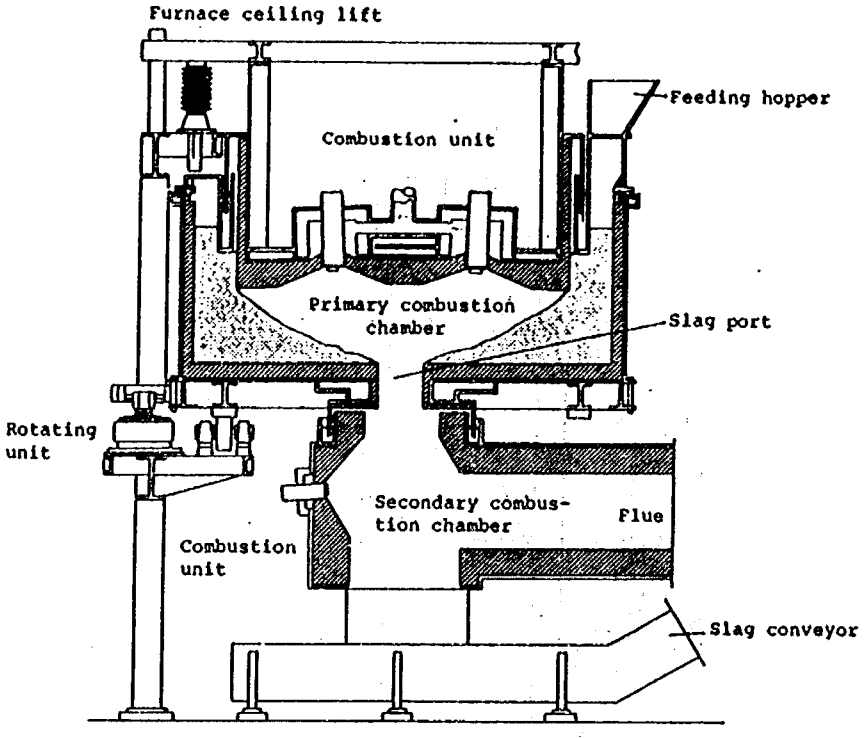


Fig. 2-3 Surface Melting Furance

(4) Cyclone melting furnace system

In this system, dewatered sludge is dried to water content below 10%, pulverized and then fed along with combustion air into a cylindrical furnace in such a manner as to generate a spiral air flow. Dried and pulverized sludge is burnt in the spiral air flow, inorganic matter being molten by combustion heat. This system features a compact furnace and permits melting using only heat released by burning sludge when sludge calorific value is sufficient. Therefore a key point in designing this system is effective utilization of furnace-generated heat. (Fig. 2-4)

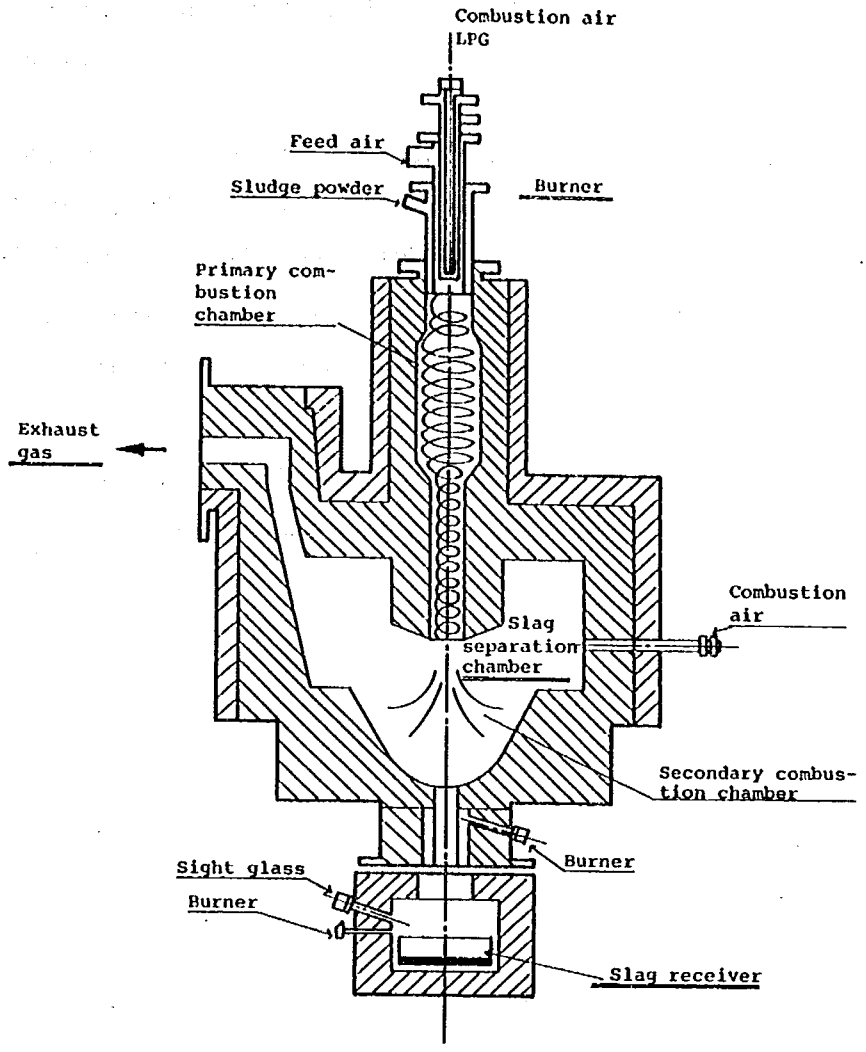


Fig. 2-4 Cyclone Melting Furnace

2-3 Selection of Melting Furnace

Given the various opinions as to the economical efficiency of melting furnaces, it is very difficult at present to determine the best furnace.

The sludge melting process requires a dryer for pretreatment of sludge to be fed into the furnace, in terms of water content and appropriate shape; the economical efficiency of the process depends on the drying method.

In view of the energy saving and high efficiency, we selected the cyclone melting furnace system, combined with a direct contact fluidized bed dryer using superheated steam.

The cyclone melting furnace system first dries and pulverizes sludge, then melts it under high heat load. This furnace, frequently used as a pulverized coal incinerator for thermal power generation, features:

- High heat load combustion, permitting compact furnace and ensuring high thermal efficiency via self-sustained combustion of high calorific value sludge
- Powder combustion, permitting quick response to ON-OFF switching and emergency stop, and ensuring continuous stable melting
- Simple mechanism; easy to enlarge

3. MELTING EXPERIMENT

Using a cyclone incineration/melting system with a daily sludge treatment capacity of 2.4t, a drying-incineration/melting experiment was conducted through long-term continuous system operation. Various data was obtained for system evaluation, as outlined below.

3-1 Study Items

The following items were studied:

(1) Relationship between sample sludge properties and melting characteristics

(2) Overall energy balance obtained by utility consumption, temperature, pressure and flow rate, measured at different points in the experimental system

(3) Properties of exhaust gas: SO_X, NO_X and dust concentration

(4) Volume reduction of slag in comparison with sample sludge and incineration ash

3-2 Time and Place of Experiment

Pilot plnat location: Tsumori Sewage Treatment Plnat, Osaka

Period : March, 1984 ∿ March, 1986

3-3 Outline of Experimental Setup

The experiment used the drying-incineration/melting system, which effectively utilizes the calorific value of sludge. Flow sheet and system specifications are shown in Fig. 3-1 and Table 3-1, respectively.

Table 3-1 System Specifications

(1) Design values	
• Treatment capacity	: 2.4 tons (sludge of 70 ∿ 80% water content)
• Dried sludge	: Water content: 5% max., Averange grain diameter: 200μm max.
• Melting temperature	: 1,300°C or higher
• Slag	: Water-cooling or gradual cooling

(2) Components	
① Sludge storage tank	600 × 600 × 1,700 (0.4m^3)
② Sludge feeding pump	100kg/h, 1.5kg
③ Fluidized bed dryer	Mixing fluidizea bed 350ϕ × 550, 0.75kw
④ Dust collector	Inertial dust collector and cyclone
⑤ Mill	350kg-DS/h, 2.2kw
⑥ Dried sludge feeder	30kg-DS/h, 0.4kw
⑦ Furnace burner	30kg-DS/h
⑧ Incineration/melting furnace	Vertical cyclone type with cylinders 240ϕ × 500
⑨ Slag separation chamber	——
⑩ Slag receiver	650ϕ × 250
⑪ High temperature cyclone	500ϕ × 1,550
⑫ Steam heater	Multi-tube heat exchanger, 490ϕ × 2,600
⑬ Steam circulation blower	Turbo blower, 500kg-STM/h, 15kw
⑭ Exhaust gas fan	280Nm^3/h, 3.7kw
⑮ Furnace blower	120Nm^3/h, 3.7kw
⑯ Deodorization furnace blower	100Nm^3/h, 2.3kw
⑰ Deodorization furnace	560ϕ × 1,800
⑱ Compressor	500ℓ/min, 5kg/cm^{2}G, 3.7kw
⑲ Control panel	0.5kw

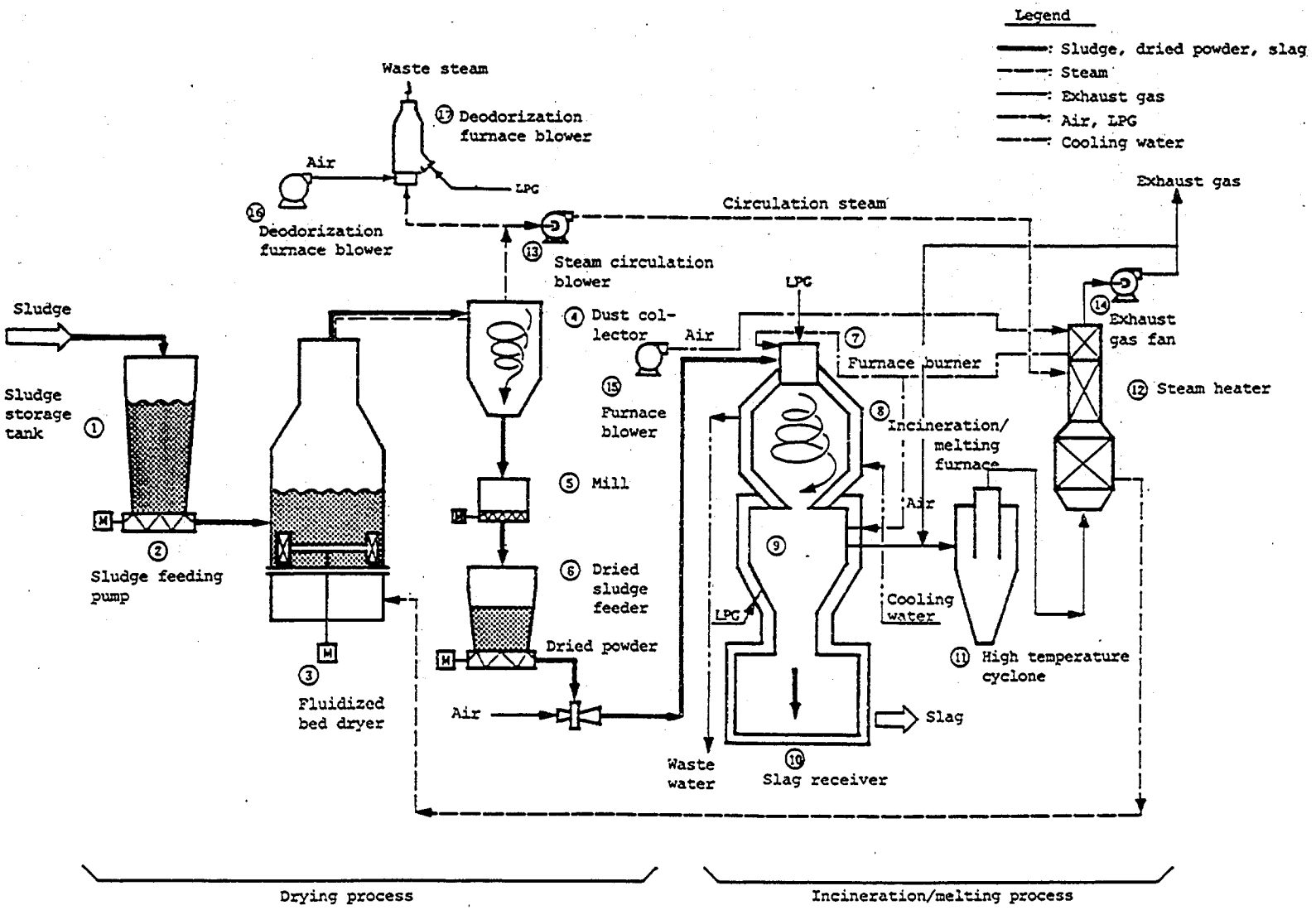


Fig. 3-1 Experimental System Flow Sheet

The system process is composed of steam-drying of dewatered sludge and incineration and melting of dried sludge.

(1) Drying

This process brings sludge of 70 ∿ 80% water content into direct contact with superheated steam to produce dried sludge with a water content of 5% or lower.

Conventionally, the indirect steam heating method has been used to dry sludge. This method, however, has various disadvantages, including scale development on heating surface, small overall heat transfer coefficient resulting in high heating surface area reuqirement for low water content sludge, and complex mechanism leading to high construction expenses.

Generally, therefore, sludge is dried via direct heating by hot blast (heated air). This method, however, poses the potential danger of ignition via direct contact with heated air, and the disadvantage of difficulty in employing the closed system.

To resolve those problems, thereby ensuring efficient and stable drying of sludge to low water content, superheated steam, as heating medium, was used to dry sludge by bringing it into contact with the sludge in the fluidized bed dryer.

The main features of this drying process are:

1) Simple mechanism, easy operation
2) Very large heat capacity, compact (Note)
3) High heat efficiency, excellent economy (Note)
4) Uniform drying permitting generation of low-water content sludge powder
5) Water content in sludge converted into superheated steam for recycling, available as heat source
6) Simple, explosion-proof measures
7) Closed system feasible, low exhaust gas generation and simple deodorization

Note) Superheated steam (above 170°C), in corparison with the heated air conventionally used for drying, permits high drying rate and higher treatment efficiency.

The process, comprising fluidized bed dryer ③, dust collector ④, mill ⑤, steam heater ⑫ and steam circulation blower ⑬, forms a superheated steam circulation system.

Dewatered sludge, fed from storage tank ① into fluidized bed dryer ③ via feeding pump ②, is dried to a water content of below 5% by means of superheated steam. The superheated steam, which drops in temperature from approx. 350°C to 150°C as heat is removed during drying, flows along with dried sludge out of fluidized bed dryer ③ into dust collector ④, where superheated steam is separated from dried sludge, drawn into steam circulation blower ⑬, and reheated to approx. 350°C by steam heater ⑫ for circulation.

Dried sludge discharged from dust collector ④ is pulverized to 200μm or smaller diameter by mill ⑤. Excess steam generated during sludge moisture evaporation is superheated to 800°C or higher by deordorization furnace ⑰ to decompose odor, and is then discharged.

(2) Incineration and melting

To slag inorganic matter contained in dried sludge for volume reduction and stabilization, this process burns dried sludge to maintain high furnace temperature.

The process comprises dried sludge feeder ⑥, furnace burner ⑦, sludge incineration/melting furnace ⑧, slag separation tank ⑨, slag receiver ⑩ and high temperature cyclone ⑪.

Dried sludge is transported by pressurized air from dried sludge feeder ⑥ into furnace burner ⑦. 80 to 90% of the theoretical volume of air is fed into incineration/melting furnace ⑧ as primary air to generate spiral flow. As dried sludge is burnt in this strong spiral air flow, furnace interior temperature promptly rises over 1,300°C. Inorganic matter in sludge melts, flowing down furnace wall into slag receiver ⑩ provided at the bottom of furnace ⑧, then is cooled and solidified.

Incomplete combustion gas in sludge incineration/melting furnace is completely burnt in slag separation tank ⑨ using secondary air amounting to 30 ∿ 40% of the theoretical volume of air. Two-stage

combustion with low air ratio (total air ratio: 1.1 ∿ 1.2) is employed to minimize NO_x concentration.

Structures of the fluidized bed dryer and sludge incineration/melting furnace are shown in Figs. 3-2 and 2-4 respectively.

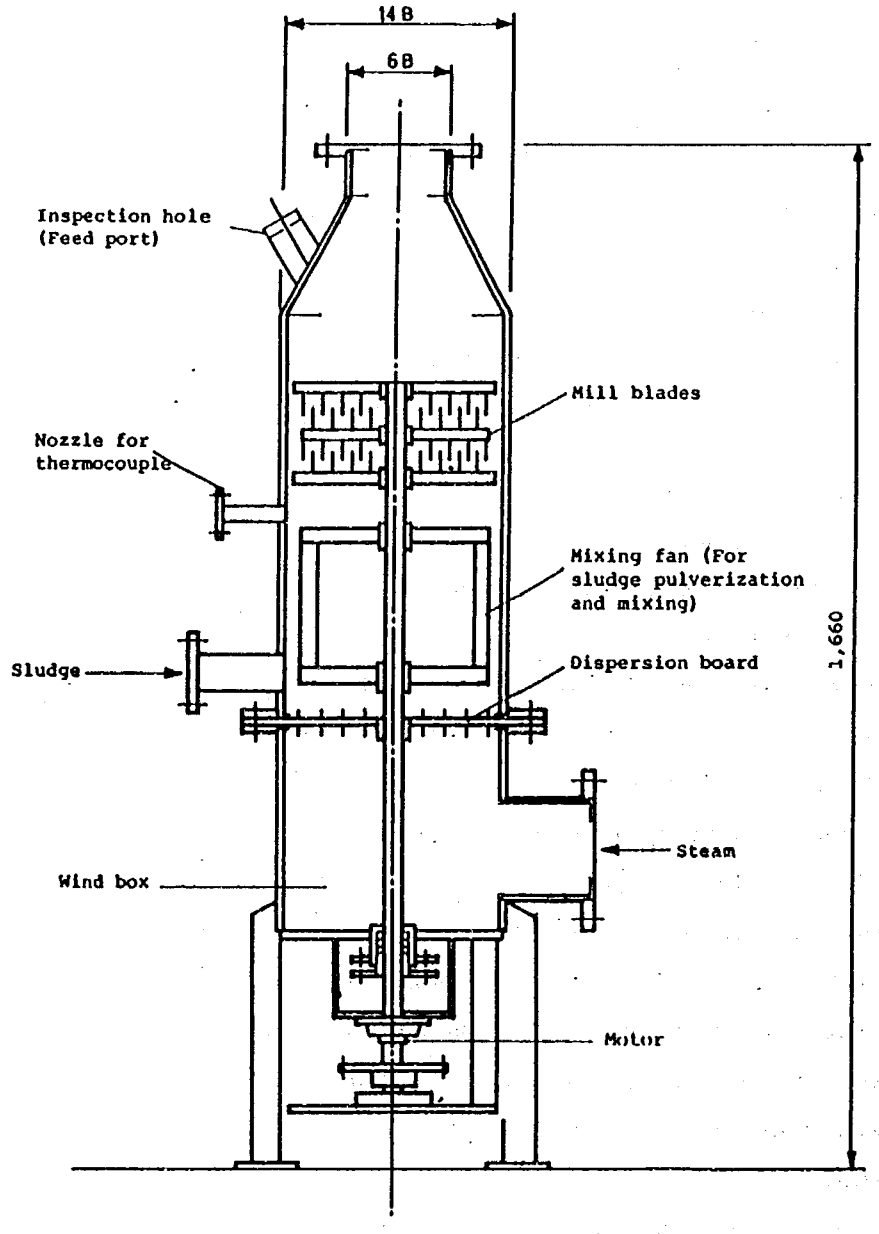


Fig. 3-2 Fluidized Bed Dryer

3-4 Description of Experiments

For experiments, around-the-clock plant operation was continued for 14 days per run. After heating the furnace to the specified temperature by LPG combustion, dried sludge was fed into the furnace with auxiliary fuel supply controlled to keep furnace temperature within the specified range. Various data was obtained after temperature inside furnace and dryer was stabilized. Two types of samples were used for the experiments: raw sludge dewatered by centrifuge with polymer added and digested sludge dewatered by vacuum filter with slacked lime and ferric chloride added. The former is called organic sludge, and the latter inorganic sludge hereinafter. Experiments employed the gradual cooling method, in which slag is first held in the receiver at a constant temperature, and then left to cool in the atmosphere.

Note) In this paper, "sludge" is used in the same meaning as "dewatered sludge"; "dried sludge" means sludge which has been dried.

4. EXPERIMENTAL RESULTS

4-1 Operating Conditions

In experiments, organic sludge was fed at 90 ∿ 120kg/h and inorganic sludge at 90 ∿ 150kg/h into an incineration/melting furnace with 100kg/h treatment capacity. To dry sludge, dryer outlet temperature was controlled to 130 ∿ 150°C for organic sludge and 150 ∿ 170°C for inorganic sludge. Dried sludge thus obtained was of flake or granular form with water content of 5% or below.

Stable discharge of slag requires controlling temperature inside furnace at 1,300 ∿ 1,400°C for organic sludge and 1,400 ∿ 1,500°C for inorganic sludge; therefore, auxiliary fuel (LPG) was supplied at 2 ∿ 3kg/h and 3 ∿ 4kg/h for organic and inorganic sludge, respectively.

Furnace temperature was raised at the rate of 50 ∿ 60°C/h; approx. 24 hours were required before starting sludge supply to melting furnace.

Table 4-1 Operating Conditions

Type of Sample Sludge	Sludge				Dried Sludge			Furnace Temperature °C	Auxiliary Fuel Supply rate kg/h	Slag Generation Rate kg/h	Slag Cooling Method
	Feed Rate kg/h	Ignition Loss %	Water Content %	Lower Calorific Value kcal/kg-DS	Feed Rate kg/h	Ignition Loss %	Water Content %				
Organic Sludge	90∿120	64.6	81.4	3,610	15∿20	58.0	1.92	1,300 ∿1,400	2∿3	6∿ 7	Gradual cooling (air cooling)
Inorganic Sludge	90∿150	36.9	78.5	1,680	10∿30	35.6	2.2	1,400 ∿1,500	3∿4	3∿15	Ditto

4-2 Sample Sludge Properties

Average values as to properties of organic and inorganic sludges used for the experiments are shown in Table 4-2. As this table indicates, average water content is 80.4% for organic sludge and 77.7% for inorganic sludge, while average ignition loss of sludge is 64.5% for organic sludge and 35.7% for inorganic sludge, the latter being approx. 30% lower than the former.

Average gross calorific values of organic and inorganic sludges are 3,780kcal/kg-DS (Dry Solids) and 1,830kcal/kg-DS, respectively. Average lower calorific value is 3,490kcal/kg-DS for organic sludge and 1,650kcal/kg-DS for inorganic sludge. Fig. 4-1 indicates the relationship between ignition loss and lower calorific value; good correlation is observed.

Table 4-3 compares drying characteristics between organic and inorganic. As the table shows, for inorganic sludge dried at higher temperature than that for organic sludge, the reduction in ignition loss (VTS), the quantity of dried sludge powder (dust) flying into waste steam and the value of COD in waste steam condensate are small in comparison with the figures for organic sludge. Thus drying characteristics differ between the two types of sludge; however, both could be dried to a water content of 5% or below.

As concerns dried sludge collection using a cyclone, measurement of dust in circulation steam revealed a dust collection rate of 90 ∿ 98%, as shown in Table 4-4.

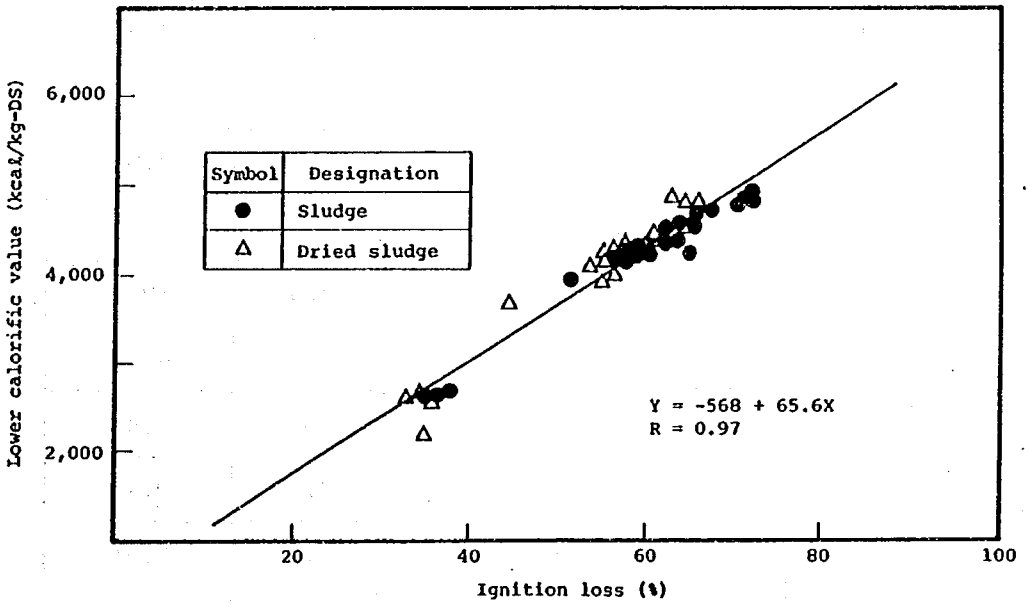


Fig. 4-1 Relationship between Ignition Loss and Lower Calorific Value

Table 4-2 Sludge Properties

Measurement Item			Organic Sludge	Inorganic Sludge
Water contetn (%)			80.4	77.7
Ignition loss (%)			64.5	35.7
Calorific value	Gross	kcal/kg-DS	3,780	1,830
		kcal/kg-ignition loss	6,010	5,060
	Lower	kcal/kg-DS	3,490	1,650
		kcal/kg-ignition loss	5,540	4,570
Elementary analysis (%) (In dried condition)	C		34.4	19.0
	H		5.32	3.21
	N		4.5	2.2
	S		1.74	2.01
Chemical analysis (In dried condition)	Fe_2O_3		4.65	11.1
	CaO		2.31	19.3
	P_2O_5		3.97	4.09
	SiO_2		13.2	7.42
	K_2O		0.30	0.18
	Al_2O_3		2.79	2.75
	MgO		0.72	0.94
Treatment plant				
Basicity (CaO/SiO_2)			0.21	2.85
Melting property	Melting temperature (°C)		1,243	1,338
	Flow temperature (°C)		1,278	1,370

Table 4-3 Comparison of Drying Characteristics between Organic and Inorganic Sludge (Example)

Item	Organic Sludge	Inorganic Sludge
1) Sludge properties		
Water content (%)	81.5 (Mean)	78.5 (Mean)
VTS (%)	64.5 (")	36.9 (")
Lower calorific value (kcal/kg-DS)	3,600 (")	1,680 (")
2) Drying conditions		
Sludge feed rate (kg/h)	80∿120	80∿140
Detention time (min)	15∿ 20	12∿ 20
Steam circulation rate (kg/h)	400∿500	350∿500
Steam inlet temperature (°C)	300∿400	350∿450
Steam outlet temperature (°C)	130∿150	150∿170
3) Dried sludge properties		
Water content (%)	2 max.	5 max.
VTS (%)	56.9 (Mean)	35.6 (Mean)
Lower calorific value (kcal/kg-DS)	3,340 (")	1,640 (")
4) Waste steam properties		
Dust (g/Nm^3)	4 (Mean)	0.6 (Mean)
Condensate COD (mg/ℓ)	4,600 (")	400 (")

Table 4-4 Dried Sludge Collection by Cyclone

Item	Organic Sludge			Inorganic Sludge	
Amount of solids in Sludge (kg/H)	18.7	20	19.7	27	31.5
Steam circulation rate (Nm /h)	570	570	460	810	730
Amount of dust in waste steam (g/Nm)	3.7	4.6	4.1	0.6	0.6
Dust collection rate (%)	88.7	86.9	90.5	98.2	98.6

Note) Cyclone dust collection rate = $(A - B)/A \times 100$ (%)

A: Amount of solids in sludge
= Sludge feed rate × (1 - waste content/100)

B: Amount of dust
= Amount of dust in waste steam × steam circulation rate

4-3 Melting Characteristics

Table 4-2 shows basicity, melting temperature and flow temperature of organic and inorganic sludge[Note). Average basicity is 0.21 and 2.85 for organic and inorganic sludge, respectively, melting temperature 1,243°C and 1,360°C, respectively. As concerns the relationship between basicity, melting temperature and flow temperature, the basic test revealed that melting temperature is lowest at around basicity 1, as shown in Fig. 4-2.

Note) Melting temperature, flow temperature

In melting sludge, furnace opearting conditions are influenced by melting conditions such as melting temperature and fluidity in each case. Sludge melting conditions are varied by composition and ratio of inorganic matter in sludge. Therefore, basicity indicated by CaO/SiO_2, $(CaO + Na_2O + K_2O + Fe_2O_3)/(SiO_2 + Al_2O_3)$ etc. is considered to be an important index for melting treatment.

Melting temperature: Temperature at which a slug cube halves in height

Flow temperature : Temperature at which a slug cube is reduced to one third in height and starts flowing

As concerns slag crystallization, it is observed taht vitreous slag is increased in strength by crystallization. X-ray diffraction was used to study slag crystallization characteristics. Results of crystallization tests performed under four different conditions are shown in Table 4-5. As this table indicates, slag held for two hours at 1,300°C (Case 1) is vitreous and is not crystallization, while with slag held for two hours at 1,205°C, 50°C lower than the melting temperature (Case 2), three peaks are observed at around diffraction angles 46°, 66° and 77°, indicating that slag crystallization occurs. When slag is held for three hours at 1,205°C (Case 3), higher peaks are observed, showing that slag is further crystallized. When slag is molten for an hour at 1,400°C and then cooled in the furnace at a cooling rate of 3°C/min. (Case 4), satisfactorily crystallized slag is obtained.

According to Table 4-5, as slag crystallization advances, compressive strength tends to increase; thus slag crystallization is considered to serve as an index for slag quality enhancement.

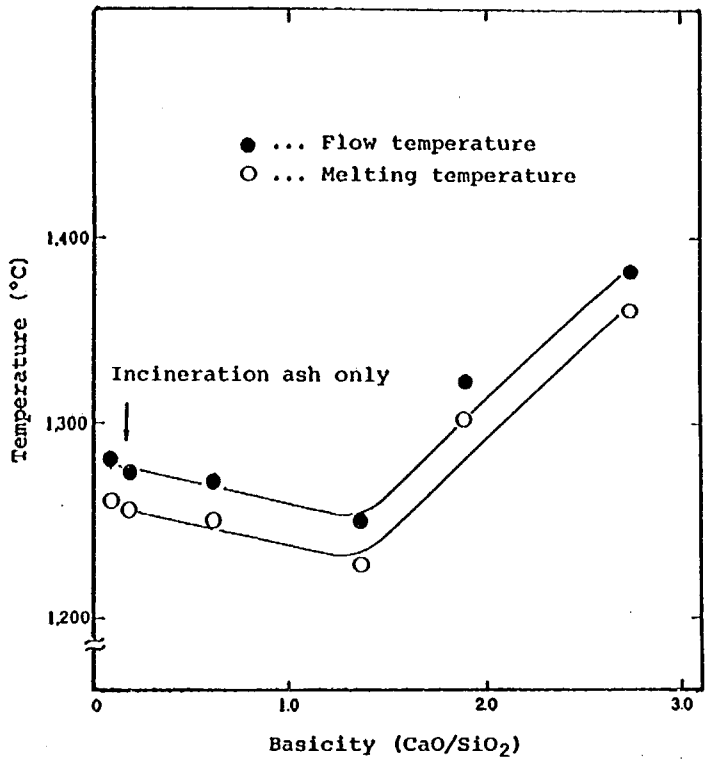


Fig. 4-2 Melting Temperature and Flow Temperature of Slag Conditioned in Basicity

Table 4-5 Slag Crystallization Test Results

Item \ Sampel No.		Case 1	Case 2	Case 3	Case 4
Melting conditions	Temperature (°C)	1,402	1,402	1,402	1,400
	Time (hr)	1	1	1	1
Crystallization conditions	Temperature (°C)	1,003	1,205	1,205	Cooling in furnace at cooling rate of 3°C/min
	Time (hr)	2	2	3	
Slag properties	Vickers hardness	551	583	563	634
	Compressive strength (kg/cm^2)	420	1,218	1,630	—
	X-ray diffraction results	Vitreous slag	Slag under crystalliztion	Crystallized slag	Crystallized slag

Note) Samples (Cases 1 ∿ 4): Organic sludge (September, 1984)
Melting temperature : 1,225°C

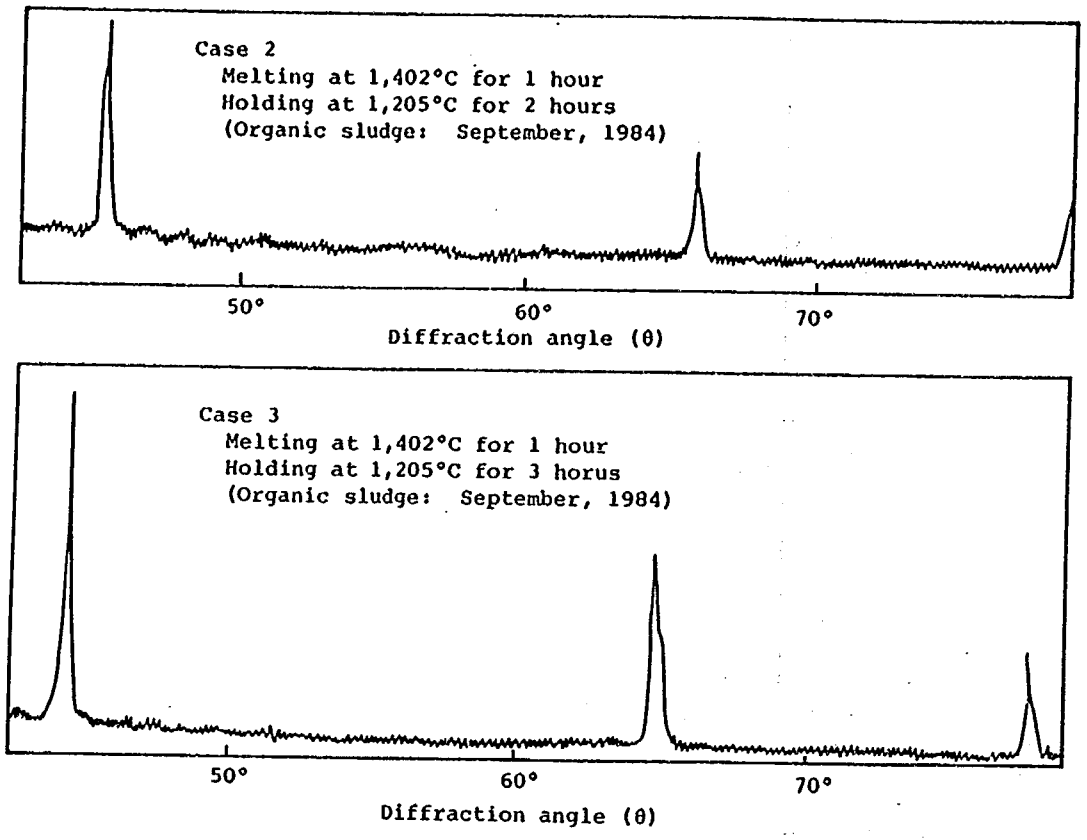


Fig. 4-3 X-ray Diffraction of Crystallized Slag (Cases 2 and 3)

Note) X-ray diffraction

X-rays, of short wavelength, have the same properties as light. One such property is diffraction. Light diffraction is a phenomenon in which when light of uniform wavelength enters a diffraction grating (composed of many parallel lines regularly spaced on a flat surface), reflected light strength differs depanding on the angle of incidence. This diffraction phenomenon can be generated also using X-ray and crystal instead of light and diffraction grating. It is generally said taht the further the crystallization, that is, the more regular the atomic arrangement, the higher the strength of the reflected X-rays.

4-4 Material Balance and Heat Balance

Examples of material balance and heat balance obtained from the operation using organic and inorganic sludges are shown in Table 4-6. The quantity of heat of sludge and fuel were calculated based on the lower calorific value.

In the operation using organic sludge, the percentage of dried sludge heat input and auxiliary fuel heat input in proportion to the total heat input, are approx. 57% and 32%, respectively; 32.8ℓ heavy oil per ton of sludge on the average was required as auxiliary fuel. As the experimental system is small in scale and heat loss around the slag gradual cooling box is high, heat loss over the whole system is approx. 30% of the total heat input. for the actual plant, on the other hand, the ratio of heat loss to total heat output is expected to be significantly lower as the ratio of heat loss by instrumentation, accessories, flanges and valves is presumed to be low.

In the operation using inorganic sludge, as dried sludge is of lower calorific value, the ratio of the heat input by sludge to the total heat input is as low as 21%, the ratio of the heat input by auxiliary fuel marking the higher level, around 65%. The auxiliary fuel requirement is 64.4ℓ heavy oil/t-dried sludge on average, higher than that for organic sludge. This is because inorganic sludge is of lower calorific value than organic sludge and also because furnace temperature must be maintained at around 1,400°C, approx. 100°C higher than for organic sludge.

Table 4-6 Heat Balance and Material Balance

Heat Balance

		Organic Sludge		Inorganic Sludge	
		kcal/h	%	kcal/h	%
Input	Sludge sensible heat	2,342	2.2	2,200	1.8
	Dried sludge heat	59,630	56.9	25,766	21.2
	Total auxiliary fuel heat	33,485	32.0	79,516	65.4
	Combustion air sensible heat	4,901	4.7	5,797	4.8
	Circulation exhaust gas sensible heat	4,428	4.2	8,369	6.8
	Total	104,786	100	121,648	100
Output	Waste steam heat output	59,156	56.5	65,812	54.1
	Slag heat output	2,153	2.1	1,055	0.9
	Exhaust gas heat output	8,066	7.7	12,348	10.2
	Circulation exhaust gas sensible heat	4,673	4.5	8,736	7.3
	Heat loss by the whole system	30,738	29.2	33,697	27.6
	Total	104,786	100	121,648	100

Material Balance

		Organic Sludge	Inorganic Sludge
Input	Sludge feed rate (kg/h)	105.6	127.9
	Dried sludge feed rate (kg/h)	(17.8)	(15.4)
	Total auxiliary fuel feed rate (kg/h)	3.0	7.2
	Total combustion air feed rate (kg/h)	169.4	193.5
	Circulation exhaust gas flow rate (kg/h)	126.4	189.6
	Total	404.4	518.2
Output	Dried sludge output rate (kg/h)	(15.8)	(28.3)
	Slag discharge rate (kg/h)	6.5	6.9
	Fly ash output rate (kg/h)	0.9	2.7
	Waste steam flow rate (kg/h)	89.8	99.6
	Exhaust gas flow rate (kg/h)	182.8	206.6
	Circulation exhaust gas flow rate (kg/h)	126.5	189.6
	Total	406.4	505.4

Dewatered Sludge Properties				
	Water Content (%)	Ignition Loss (%)	Gross Calorific Valve (kcal/kg)	Lower Calorific Valve (kcal/kg)
Organic sludge	81.8	65.4	3,863	3,587
Inorganic sludge	78.5	36.2	1,864	1,678

Dried Sludge Properties				
	Water Content (%)	Ignition Loss (%)	Gross Calorific Valve (kcal/kg)	Lower Calorific Valve (kcal/kg)
Organic sludge	2.3	58.9	3,603	3,350
Inorganic sludge	10.0	35.2	1,870	1,682

4-5 Exhaust Gas Properties

Analytical results of exhaust gas generated in the operation using organic and inorganic sludge are exemplified in Table 4-7.

As the experimental setup used does not include a gas treatment facility, the amount of SO_X generated from organic sludge marked a significantly high level, around 1,000ppm, while inorganic sludge generated a low level of SO_X, around 80ppm. SO_X is presumably generated from combustible sulfur in sludge. Inorganic sludge, as shown in Table 4-8, contains a very low level of combustible sulfur, thus generating only a small amount of SO_X.

As concerns organic sludge, by contrast, approx. 60% of its sulfur content is combustible; thus the concentration of SO_X in its exhaust gas is high. SO_X must be removed with an exhaust gas treatment facility. Exhaust gas properties are shown in Table 4-7.

The amount of NO_X generated was 176ppm (conversion at 12% O_2) for organic sludge and 99ppm for inorganic sludge. Using a cyclone melting furnace, the effect of two-stage combustion was studied. Fig. 4-4 shows the behavior of NO_X concentration (conversion at 12% O_2), studied with total air ratio set at 1.15 and primary air ratio varied. As indicated in this figure, NO_X concentration marks its lowest level, approx. 70ppm, at around the primary air ratio 0.85. With the one-staged combustion method, NO_X concentration is expected to exceed 300ppm as air ratio is set at above 1.0 for perfect combustion.

The cyclone melting furnace uses the two-staged combustion method, permitting free setting of air raio for the first stage at the desired point; thus reduction of NO_X concentration is possible.

Concentration of dust in exhaust gas is shown in Table 4-7. The concentration is 0.96g/Nm^3 for organic sludge, while inorganic sludge marks the higher level of 1.85g/Nm^3. Thus dust concentration is measured to depand upon sludge type and sludge feed rate.

Table 4-7 Exhaust Gas Properties

Item \ Sludge Type	Organic	Inorganic
SO_X (ppm)	1,048	81
NO_X (ppm)	306	189
NO_X-12% O_2 (ppm)	176	99
Dust (g/Nm^3)	0.96	1.85
Water content (%)	1.48	17.5
O_2 (%)	5.3	3.7
N_2 (%)	79.1	79.5
CO (%)	0.39	1.72
CO_2 (%)	11.5	11.0

Table 4-8 Concentrations of Combustible and Noncombustible Sulfurs

Per amount of dry solids Unit: %

	Coagulant		Total Sulfur (a)	Noncombustible Sulfur	Combustible Sulfur (b)	(a)/(b)
	Organic	Inorganic				
Sludge	o		1.99	0.76	1.23	62
Sludge		o	2.28	2.28	0	0
Dried sludge	o		2.50	0.98	1.52	61
Dried sludge	o		1.95	0.43	1.52	78
Dried sludge		o	2.18	2.18	0	0

Note) Noncombustible sulfur: Sulfur remaining in ash after sample combustion at 800°C
Combustible sulfur : (Total sulfur) - (Noncombustible sulfur)

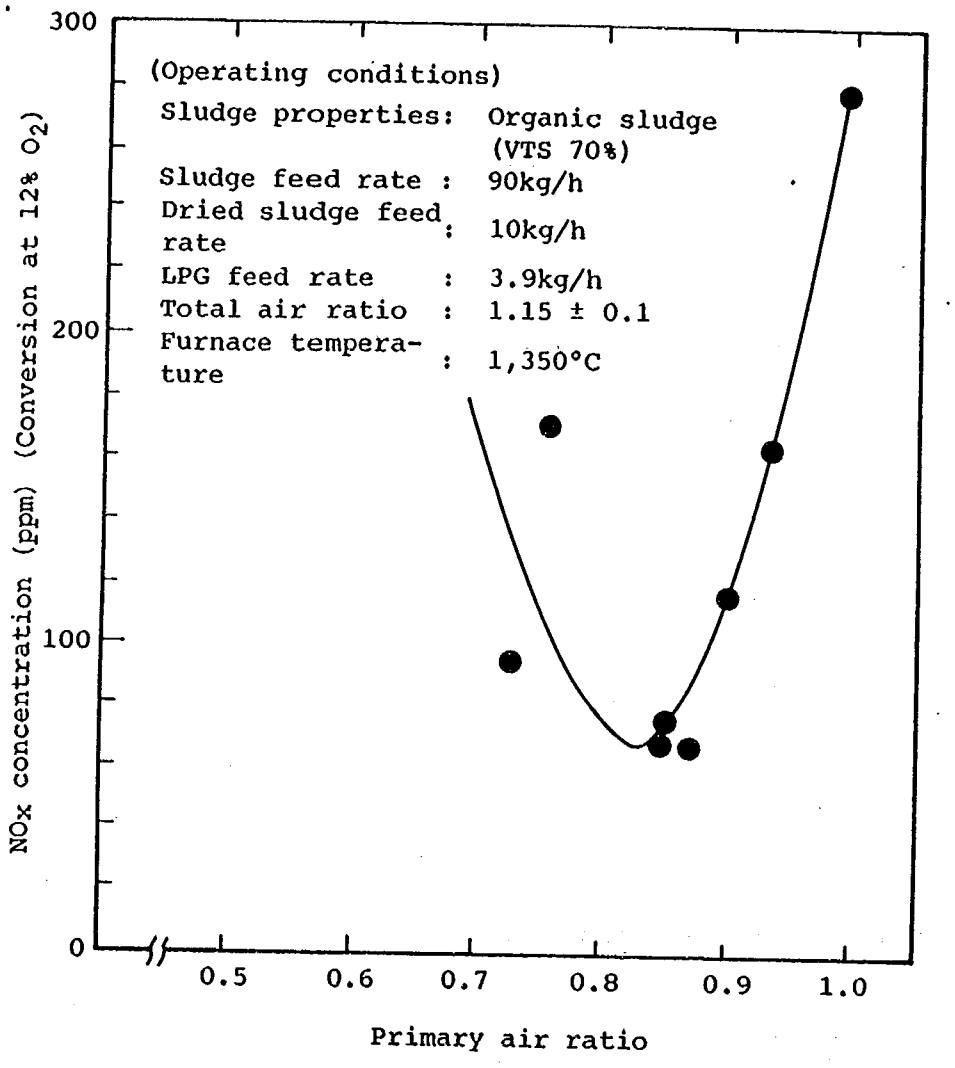


Fig. 4-4 NO_X Characteristics

4-6 Slag Volume Reduction Effect

Organic and inorganic sludge volume reduction effects are shown in Figs. 4-5 and 4-6. As those figures indicate, ratio of volume reduction to dewatered sludge is approx. 1/30 for organic sludge and approx. 1/16 for inorganic sludge, while the ratio to incineration ash is approx. 1/2.6 for organic sludge and approx. 1/3 for inorganic sludge.

Bulk density used to calculate volume reduction ratio is shown in Table 4-9.

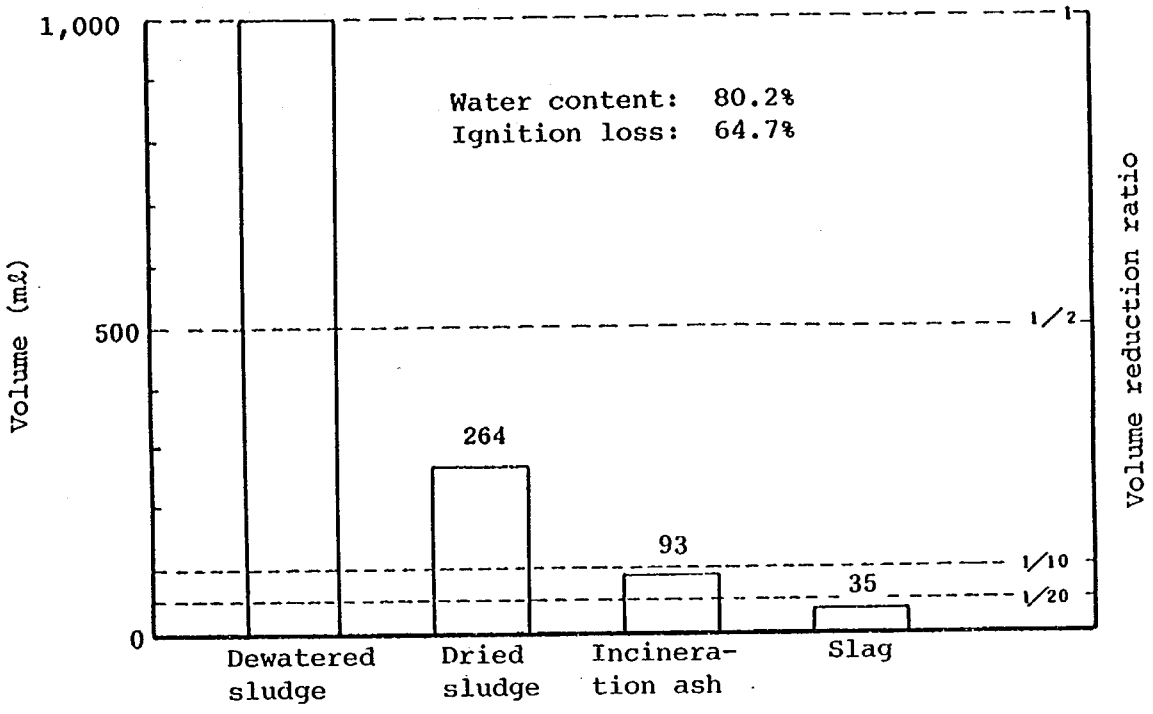


Fig. 4-5 Organic Sludge Volume Reduction Effect

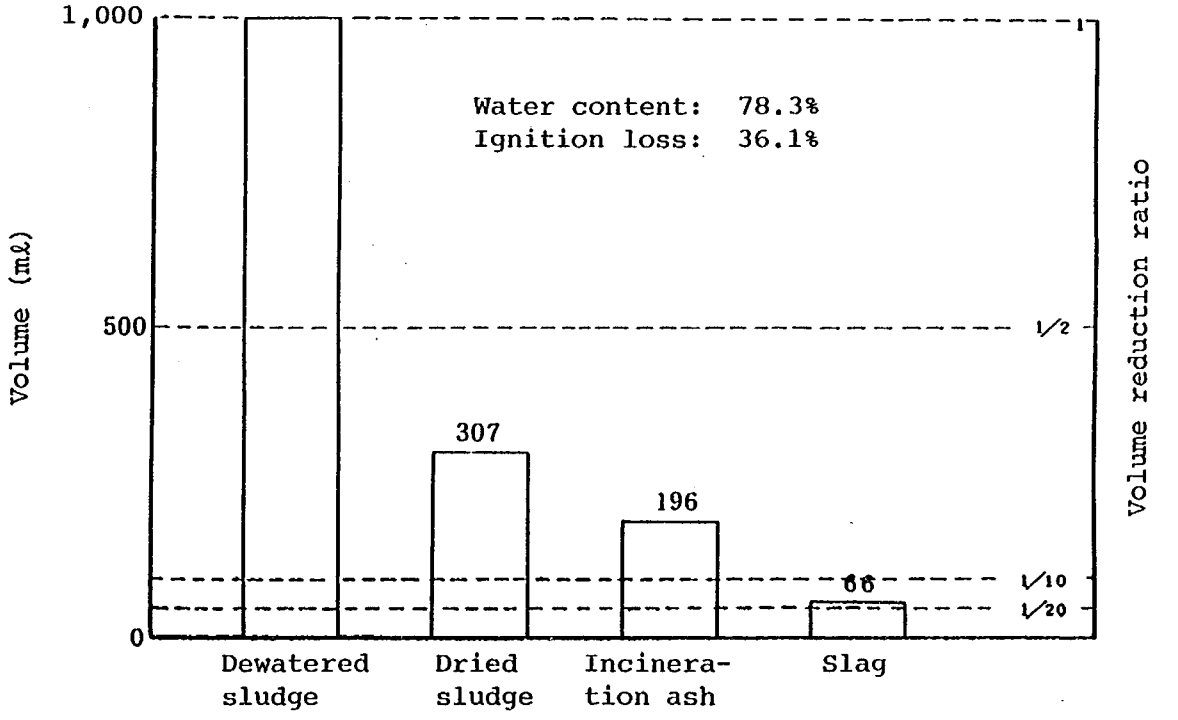


Fig. 4.6 Inorganic Sludge Volume Reduction Effect

Table 4-9 Bulk Density of Organic and Inorganic

Sludge Type	Dewatered Sludge	Dried Sludge	Incinera-tion Ash	Slag
Organic	0.8	0.6	0.6	1.6
Inorganic	0.85	0.6	0.6	1.8

5. SYSTEM EVALUATION

5-1 Evaluation of the Fluidized Bed Drying Process

The fluidized bed drying process, which employs a direct contact drying method using superheated steam, thus features a great volumetric heat transfer coefficient. Fig. 5-1 shows the relationship between dewatered sludge feed rate and overall heat transfer coefficient.

This type of dryer possesses stable treatment capacity; the water content of dried sludge obtained using it is never more than 5%. The overall heat transfer coefficient, approx. 6,000kcal/m^3·hr·°C at a sludge feed rate of 100kg/h (design value for the system), is significantly greater than that for the rotary hot gas dryer etc., i.e. 150 ∿ 350kcal/m^3·hr·°C. This value is also favorably higher than that for the indirect steam-heated drying system (paddle dryer), i.e. 65 ∿ 90kcal/m^2·hr·°C, which is used for sewage sludge drying in a number of plants.

Unfortunately, however, there is the possibility of such problems as exhaust steam pollution and reduction of VTS in dried sludge because this system performs direct drying with steam superheated to 300°C or more.

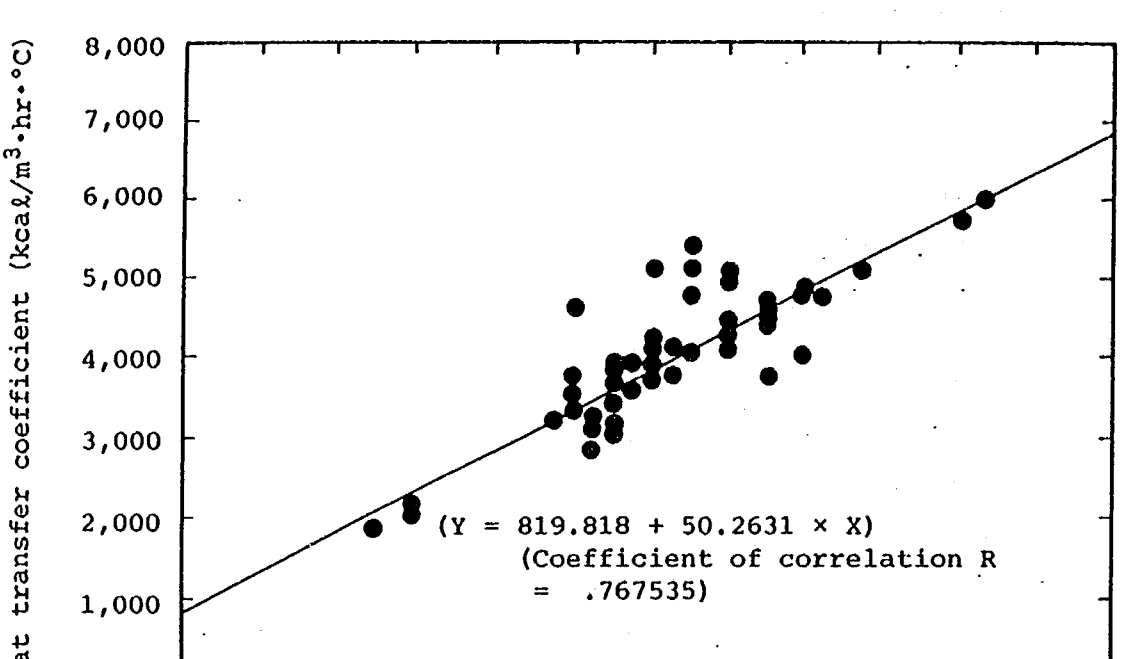
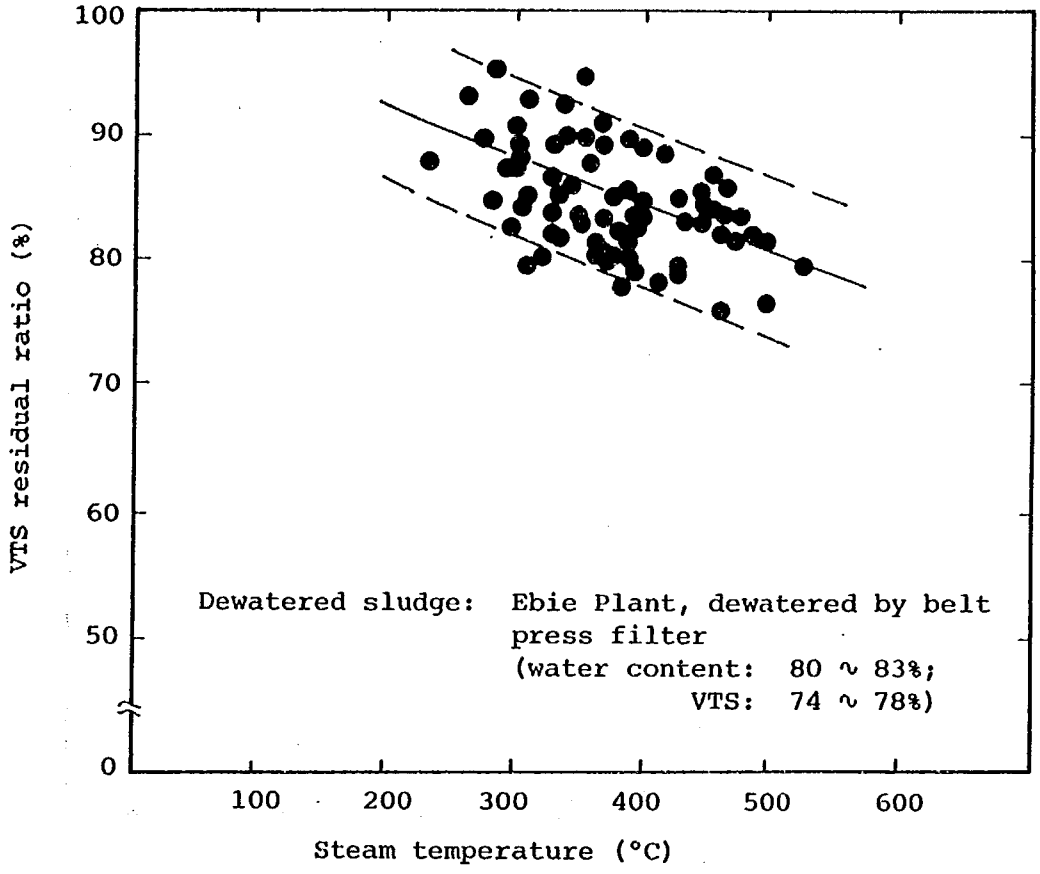


Fig. 5-1 Relationship between Dewatered Sludge Feed Rate and Heat Transfer Coefficient (drying system) Ebie Plant, Osaka (Jan. 9 ∿ 22, 1961)

Figs. 5-2 and 5-3 respectively show the relationship between steam temperature at dryer entrance and VTS residual ratio in dried sludge and between VTS reduction rate in fed sludge and COD in exhaust steam-condensed water. As shown in Fig. 5-2, VTS in dried sludge tends to decrease as entrance steam temperature increases, and as shown in Fig. 5-3, COD in exhaust steam increases as VTS decreases.

This evidence indicates that further investigation is needed to determine drying conditions necessary to minimize VTS reduction.



Note) VTS residual ratio $= \frac{\text{dried sludge VTS}}{\text{sludge VTS}} \times 100$

Fig. 5-2 Relationship between Steam Temperature at Dryer Entrance and VTS in Dried sludge

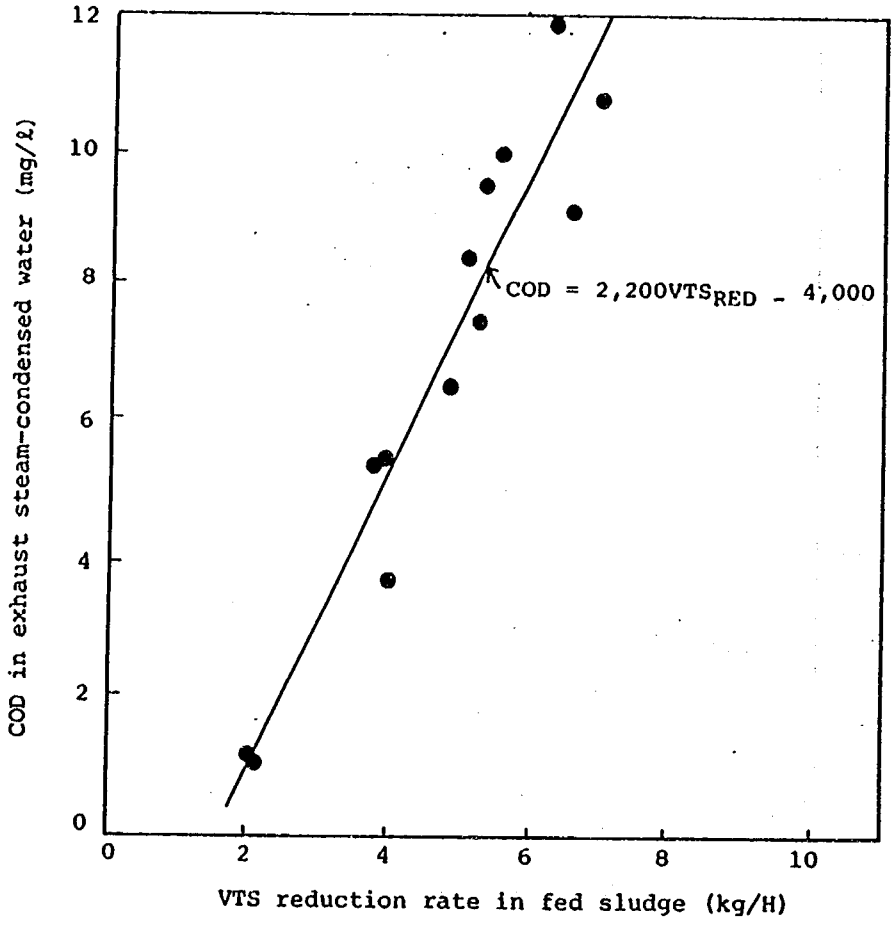


Fig. 5-3 Relationship between VTS Reduction Rate and COD in Exhaust Steam-condensed Water

5-2 Incineration/Melting Process

It has been proven that the cyclone melting furnace ensures high sludge combustibility, which makes it easy to obtain a melting state.

Table 5-1 shows the results of comparisons between this incineration/melting system and other systems.

Thermal Efficiency (I), a defining equation applied to conventional incinerators, employs the heat of vaporization of sludge water content as the effective heat. Thermal Efficiency (II) is an equation employing a recoverable heat such as exhaust steam heat, slag heat, or heat loss by furnace cooling water as the effective heat. As shown in Table 5-1, this

system is regarded as higher in thermal efficiency than other systems. For the evaluation of thermal efficiency suitable to the melting furnace system, further investigation will be needed.

Table 5-1 Comparisons in Thermal Efficiency between Present and Conventional Systems

System Type / Factor	Present System (2.4t/day, organic sludge	Present System (2.4t/day, inorganic sludge)	Multiple-hearth Furnace (40t/day)	Fluidized Bed Incinerator (50t/day)
Sludge vaporization heat (kcal/H)	5.19×10^4	6.02×10^4	7.43×10^5	8.13×10^5
Heat produced in furnace (kcal/H)	9.3×10^4	1.05×10^4	15.84×10^5	21.56×10^5
Thermal efficiency (I) (%)	55.8	57.2	46.9	37.7
Thermal efficiency (II) (%)	76.8	72.9	—	—
Source			"City and Waste", Vol. 10, No. 8, Japan Sewage Works Corporation (1979)	

Note) Thermal efficiency (I) = A/B × 100 (%)

A: Quantity of sludge vaporation heat = 597.5 × quantity of vaporized water

B: Quantity of heat produced in furnace = heat input by dried sludge + heat input by auxiliary fuel

Thermal efficiency (II) = A'/B × 100 (%)

A': Quantity of exhaust steam heat + quantity of slag heat + quantity of furnace-cooling water heat

B: Quantity of heat produced in furnace = heat input by dried sludge + heat input by auxiliary fuel

Fig. 5-4 shows the relationship between dried sludge feed rate and volumetric combustion load in the melting furnace (primary furnace). The present experiment yielded values in the range of 100 to 300×10^4kcal/m^3·H, higher than those with conventional incinerators ($10 \sim 50 \times 10^4$kcal/m^3·H). These data suggest the potential for sludge treatment with reduced furnace capacity.

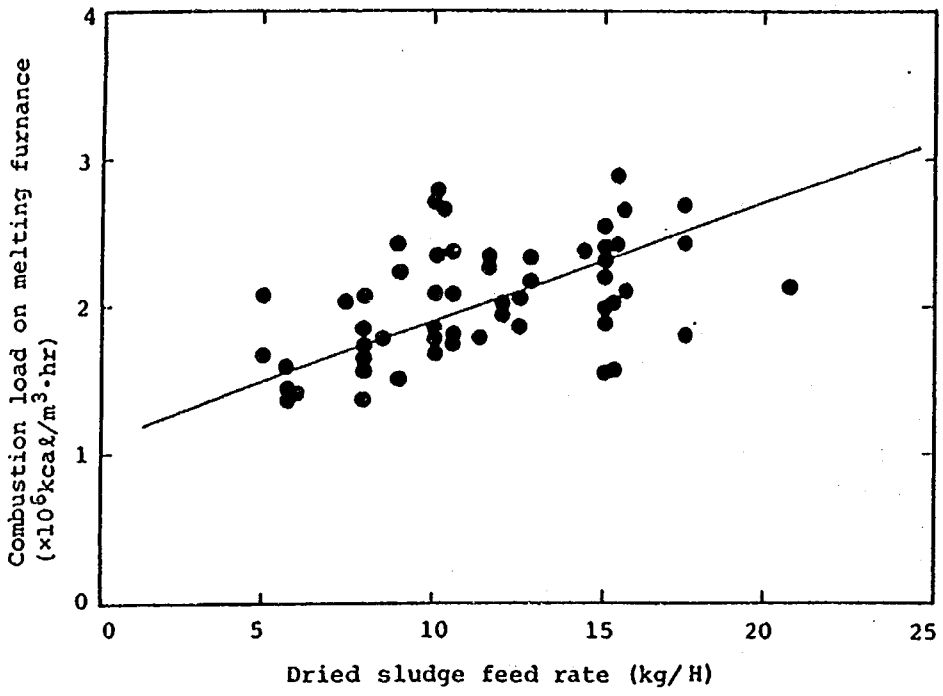


Fig. 5-4 Relationship between Dried Sludge Feed Rate and Combustion Load

5-3 Energy-saving Effect

Since the experimental system used in the present study is of relatively small scale, i.e., a sludge treatment capacity of 2.4 tons per day, a high heat loss ratio was experienced. However, the heat loss ratio decreases as system scale increases, since the surface area of the system, in general, does not increase in proportion to its treatment capacity. Based on the results of the present experiment, heat balance etc. were estimated in a simulated system of actual operating scale under the following conditions:

(1)	Treatment capacity (t/day)	: 50, 100, 200
(2)	Dewatered sludge water content (%)	: 80
(3)	Dewatered sludge VTS (%)	: 65
(4)	Dried sludge water content (%)	: 3
(5)	Dried sludge VTS (%)	: 62
(6)	Dried sludge lower calorific value (kcal/kg-DS)	: 3,440
(7)	Auxiliary fuel	: Heavy oil A

Table 5-2 shows the data on unit fuel amount required.

Table 5-2 Unit Fuel Amount and Treatment Capacity

Treatment capacity (t/day)	2.4	5.0	100	200
Unit fuel consumption (ℓ-heavy oil/t-sludge)	41.3	13.3	11.2	8.2

As shown in Table 5-2, unit fuel consumption decreases as treatment capacity increases. It is expected that unit fuel consumption at a capacity of 200t/day will be less than 10ℓ/t-sludge, as low as 1/2 to 1/4 that with the conventional multiple-hearth incinerator.

The quantity of heat required to operate the present system is the sum of the quantity required for sludge drying and of the heat loss during the incineration/melting process; it is therefore possible to reduce auxiliary-fuel consumption by lowering heat loss and/or water content.

Fig. 5-5 is an example heat balance chart, showing the relationship between sludge water content/dried sludge calorific value and auxiliary-fuel consumption per unit weight of sludge in a furnace operated at a treatment capacity of 100t/day. Lower calorific value of sludge dewatered by polymer is around 4,000kcaℓ/kg. As well, methods of reducing sludge water content are being developed. For these reasons, it is expected that incineration/melting system can be achieved without using an auxiliary fuel.

Dewatered sludge treatment capacity: 100t/day

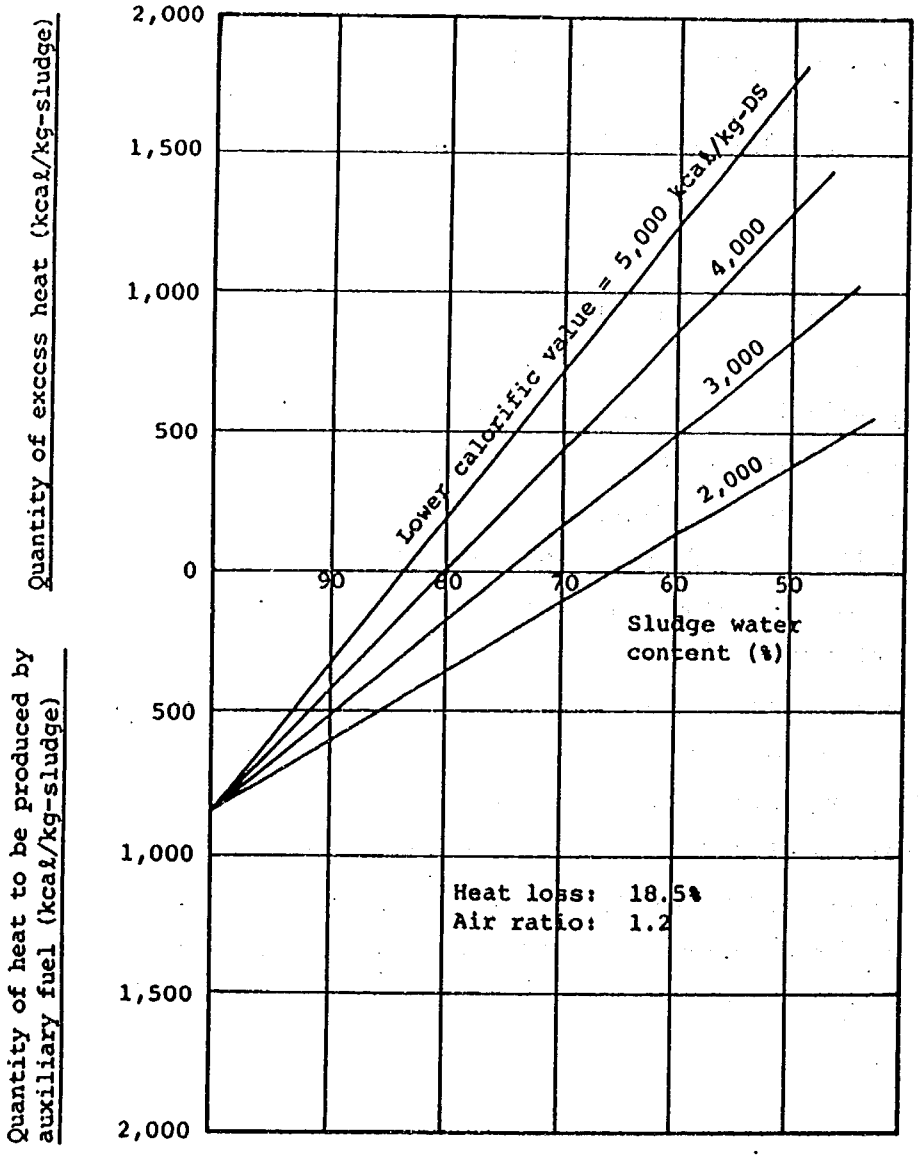


Fig. 5-5 Heat Balance Chart

6. AVAILABILITY TEST FOR SLUDGE SLAG

6-1 Chemical and Physical Properties of Sludge Slag

The slag generated in the incineration/melting process for sewage sludge is similar to iron slag, a by-product found in iron manufacture and is regarded as suitable for use as landfill material, a sub-bed material for road construction, a roadbed material, an aggregate for concrete etc. In the present study, both aggregate and concrete tests were carried out with the aim of the efficient utilization of sludge slag as an aggregate for concrete.

In general, the form of sludge slag depends on how molten slag is cooled. That is, sand-like water-cooled slag is obtained when molten slag is cooled rapidly in a water tank, while with gradually-cooled slag lumps are obtained, when the molten slag is cooled gradually in a controlled cooling chamber. If the two types of sludge slag are used as aggregates for concrete, it is preferable that the former be used in the form of a fine aggregate, the latter in coarse form.

In the present study, water-cooled and gradually-cooled slags obtained in the incineration/melting experiments in fiscal 1984 were used respectively as fine aggregate and as either fine or coarse aggregate. For the standard aggregates for comparison, river sand and crushed stone were used as fine and coarse aggregates, respectively.

The slag specimens, from dried organic sludge whose basicity (CaO/SiO_2), melting temperature, and flow temperature were 0.1 ∿ 0.2, 1,240 ∿ 1,250°C and 1,265 ∿ 1,290°C respectively, were obtained via incineration/melting at a furnace temperature of 1,300 ∿ 1,400°C. For water-cooled slag, molten slag was dropped into a water tank and cooled. For gradually-cooled slag, molten slag was transferred to a tray in a receiving tank and maintained at 900 ∿ 1,000°C for 1 ∿ 2 hours, after which it was taken from the tank and cooled in air. As stated in the paragraph "Crystallization Characteristics of Molten Slag", this gradually-cooled slag is not a crystalline but a glassy slag.

Since the particles of water-cooled slag obtained in our melting furnace were slightly too coarse for use as fine aggregate for concrete,

they were pulverized for size adjustment before testing. As for gradually-cooled slag, the lumps were crushed to two particle sizes, corresponding to fine and coarse aggregates for concrete.

Table 6-1 shows the results of a chemical analysis of sludge slag:

SiO_2 : Approx. 50%

$A\ell_2O_3$, Fe_2O_3, P_2O_3, CaO: Approx. 5 ∿ 15%

Table 6-2 shows the results of the heavy metal elution test for sludge slag, conducted in accordance with Announcement No. 13 of the Environmental Agency. There was hardly any heavy metal elution from the tested sludge slag; the requirements of the announcement for heavy metal levels for landfill were all satisfied. This shows that the heavy metals in the sludge slag were solidified after melting.

Table 6-1 Chemical composition of Slag

		Water-cooled Slag	Gradually-cooled Slag
Chemical composition	CaO (%)	5.8	5.1
	Fe_2O_3 (%)	12.6	12.5
	SiO_2 (%)	44.6	39.5
	P_2O_5 (%)	7.9	6.2
	$A\ell_2O_3$ (%)	10.6	16.6
	MgO (%)	2.5	2.4
	K_2O (%)	2.6	1.6
	Zn (ppm)	3,600	2,476
	Cu (ppm)	1,060	1,286
	T-Cr (ppm)	1,085	1,038
	Pb (ppm)	195	160
	Cd (ppm)	7.5	5.2
	T-Hg (ppm)	2.5	2.9

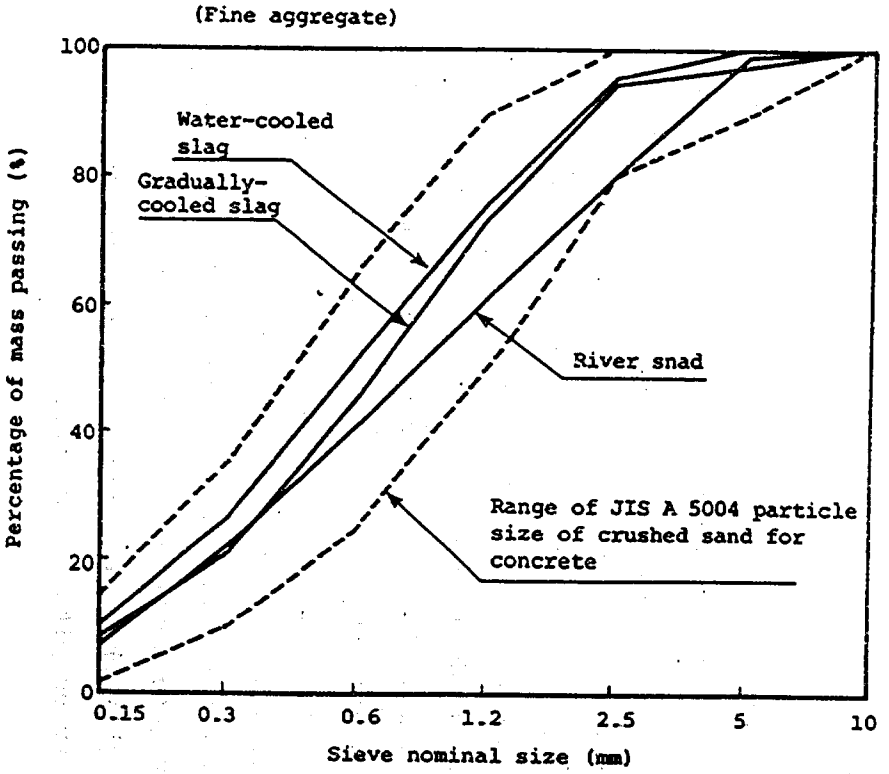
Table 6-2 Results of Heavy Metal Elution Test for Slag

		Measurement (mg/ℓ)		Regulatory Value for Landfill (mg/ℓ)	Detection Limit (mg/ℓ)
		Water-cooled Slag	Gradually-cooled Slag		
Heavey metal	T-Cr	ND	ND	—	0.05
	Cr^{6+}	ND	ND	1.5	0.05
	Pb	0.2	ND	3	0.05
	Cd	ND	ND	0.3	0.005
	Zn	0.1	0.07	—	0.005
	Cu	ND	0.14	—	0.005
	Hg	ND	ND	0.005	0.0005

6-2 Aggregate Tests

Fig. 6-1 shows the particle size distributions in the tested aggregates. All tested aggregates were in the range of standard particle size for crushed sand or crushed stone, with excellent particle size distribution.

Table 6-3 shows the results of the aggregate test. It was found that slag arregate satisfies the requirements of the standard for crushed sand or crushed stone in tests on specific gravity, the percentage of water absorption, soundness, and for the amount of material passing the standard, 74μm. The coarse aggregate of gradually-cooled slag and the fine aggregate of gradually-cooled slag differed in specific gravity. This can be explained as follows: gradually-cooled slag is low in bubbling-off quality at formation, thus containing a large number of closed bubbles. As a result, its specific gravity is low when the slag in the form of coarse aggregate, while the specific gravity is greater when it is in the form of a fine aggregate, since the bubbles largely disappear during pulverization. In addition, water absorption by slag is low when the slag is in the form of a fine aggregate because its surface is glassy, but the percentage becomes slightly higher when the slag is in the form of a coarse aggregate, since there are numerous pits and craters on its surface.



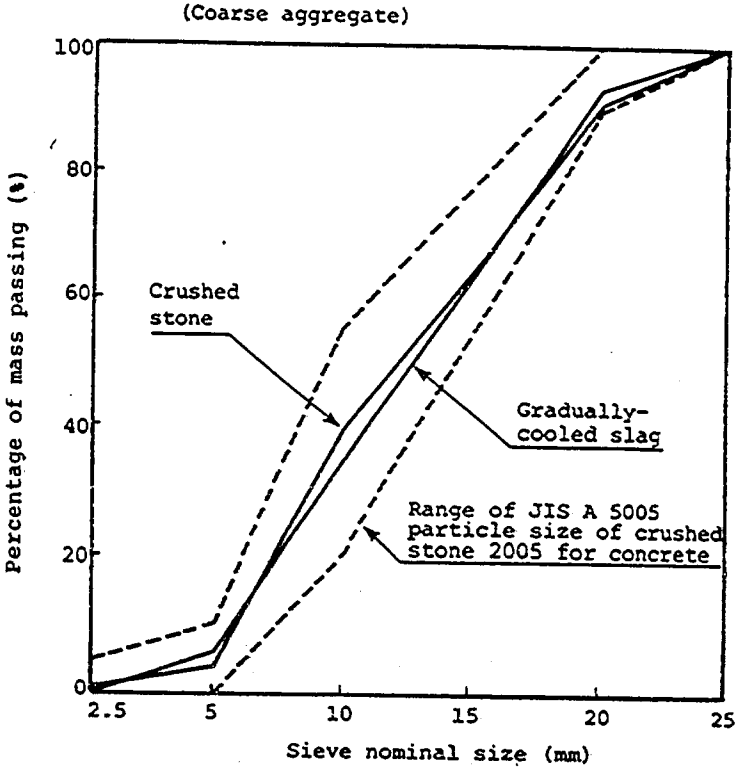


Fig. 6-1 Aggregate Particle Size Distribution

Table 6-3 Results of Aggregate Tests

Type of Aggregate \ Test Item		Specific Gravity in Saturated Surface-dry Condition	Specific Gravity in Absolute-dry Condition	Water Absorption (%)	Soundness (%)	Amount of Material Passing Standard 74μm Sieve (%)	Unit Weight (kg/ℓ)	Percentage of Absolute Volume (%)	Percentage of Absolute Volume for Particle Shape Judgement (%)	Abrasion Loss (%)	
										Classification B	Classification C
Fine aggregate	River sand	2.55	2.50	2.20	5.4	1.3	1.73	69.3	—	—	
	Gradually-cooled slag	2.71	2.70	0.34	1.3	3.9	1.63	60.4	48.9	—	
	Water-cooled slag	2.64	2.63	0.56	1.4	3.2	1.71	65.1	54.4	—	
	JIS A 5004 particle size for crushed sand for concrete	—	2.5 min.	3 max.	10 max.	7 max.	—	—	53 min.	—	
Coarse aggregate	Crushed stone	2.84	2.82	0.73	5.4	0.2	1.68	59.6	58.2	15.7	16.7
	Gradually-cooled slag	2.52	2.49	1.10	3.4	0.4	1.41	56.6	55.1	41.6	38.8
	JIS A 5005 particle size of crushed stone for concrete	2.5 min.	—	3 max.	12 max.	1 max.	—	—	55 min.	40 min.	
Test method		JIS A 1109 JIS A 1110	JIS A 1109 JIS A 1110	JIS A 1109 JIS A 1110	JIS A 1122	JIS A 1103	JIS A 1104	JIS A 1104	JIS A 5004 JIS A 5005	JIS A 1121	

Slag aggregate is low in absolute volume percentage not only because its particles are angular but also because it contains flaky particles. Gradually-cooled slag in particular contains many flaky particles; the fine aggregate of gradually-cooled slag is much lower than the standard for crushed sand in absolute volume percentage. This is attributable mainly to the method of crushing the aggregate.

The loss on abrasion in the coarse aggregate of gradually-cooled slag was more than 2 times greater than that in crushed stone, nearly equalling the standard value for crushed stone. This is attributable not only to the low strength of the aggregate but also to the presence of bubbles in the aggregate, unfavorable particle shape etc.

An alkali-aggregate reaction test was conducted in accordance with the method established by the ASTM (American Society of Testing and Materials). The results are shown in Figs. 6-2 and 6-3. In the test coducted using the chemical method, water-cooled slag proved innocuous, while gradually-cooled slag proved deleterious. In the alkali-aggregate reaction test, the degree of deleteriousness was evaluated via the mortar bar method. The two types of slag were nearly equal in expansion coefficient when their age was 6 months or less; the measurements were about 1/3 of the limit values. Based on these evidences, it can be supposed that the two types of slag are nearly free from alkali-aggregate reaction.

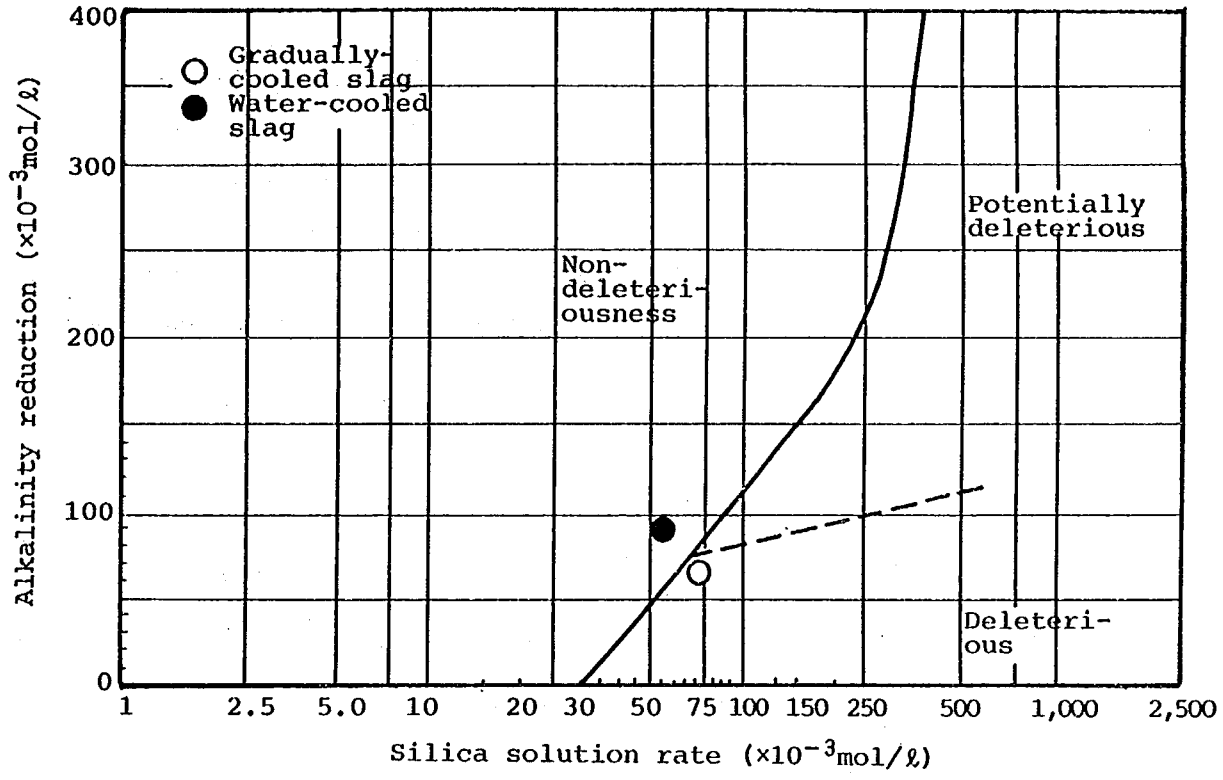


Fig. 6-2 Aggregate Deleteriousness Determined Based on Alkalinity Reduction Test in Accordance with the Chemical Method (ASTM C-289)

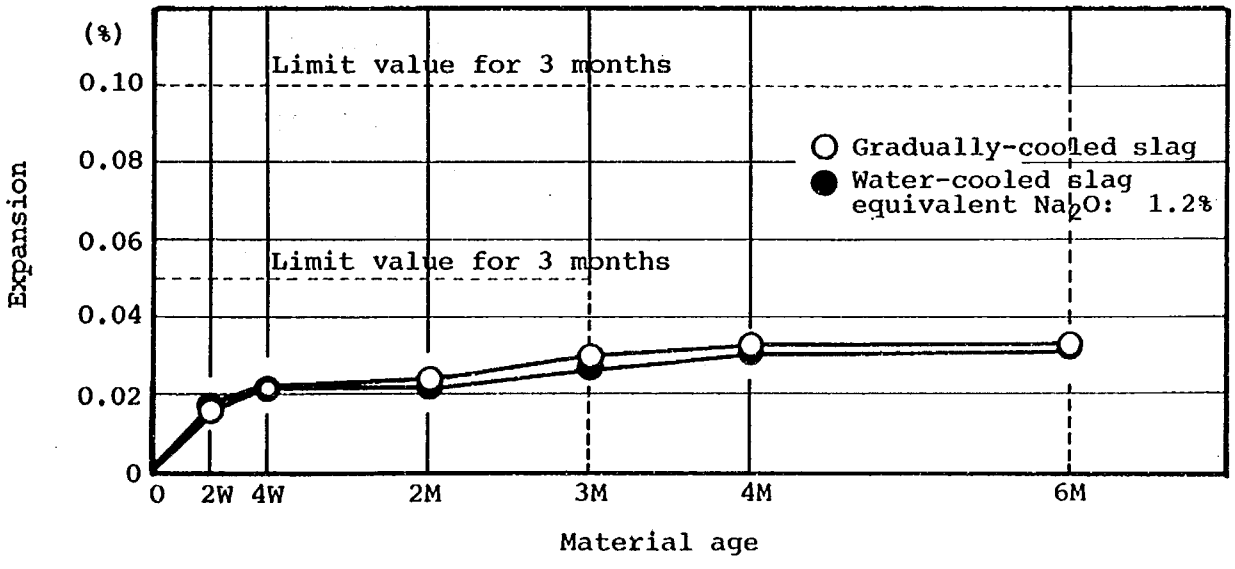


Fig. 6-3 Expansion Obtained via Moltar Bar Method (ASTM C-227)

6-3 Concrete Tests

Various concrete tests were carreid out to examine the chemical and physical properties of concrete in which sludge slag was used, either as a fine or coarse aggregate. For standard concretes for comparison, commonly used crushed stone and river sand of stable quality were used as coarse and fine aggregate, respectively. Tables 6-4, 6-5 and 6-6 each show the concrete compositions, the test items and method, and the concrete mix conditions.

Table 6-4 Concrete Materials

Materials	Types	Standards
Cement	Ordinary portland cement	JIS R 5210
Fine aggregate	Gradually-cooled slag fine aggregate Water-cooled slag fine aggregate	
	River sand	Architectural Institute of Japan The Japan Society of Civil Engineers
Coarse aggregate	Gradually-cooled slag ocarse aggregate	
	Crushed stone	JIS A 5005
Chemical admixture	Air-retaining water-reducing agent Standard type	JIS A 6204
Water	Tap water	

Table 6-5 Concrete Test

Item	Method
Slump	JIS A 1101
Air content	JIS A 1128
Bleeding	JIS A 1123
Compressive strength	JIS A 1108
Tensile strength	JIS A 1113
Bending strength	JIS A 1106
Modulus of static elasticity	JIS (Proposed)
Drying shrinkage	JIS A 1129
Freezing and thawing	ASTMC 666

Table 6-6 Concrete Mix Conditions

Coarse Aggregate	Fine Aggregate	Water-cement Ration	Air Content	Slump (cm)	Designation in Diagram
Crushed stone	River sand	49, 59, 69	4 ± 1	8, 15, 18	Crushed stone, river sand
	Gradually-cooled slag	59	4 ± 1	8, 15	Crushed stone, gradually-cooled slag
	Water-cooled slag	49, 59, 69	4 ± 1	8, 15, 18	Crushed stone, water-cooled slag
Gradually-cooled slag	River sand	49, 59, 69	4 ± 1	8, 15	Gradually-cooled slag, river sand

6-3-1 Chemical and physical properties of fresh concrete

Upon repeated trial mixing and correction based on the results of the trials, a concrete mixing proportion was chosen so as to meet the mix conditions shown in Table 6-6, and so that workable concrete was obtained. Fig. 6-4 shows the relationship between water quantity per unit volume of concrete and slump, clarified from the test results.

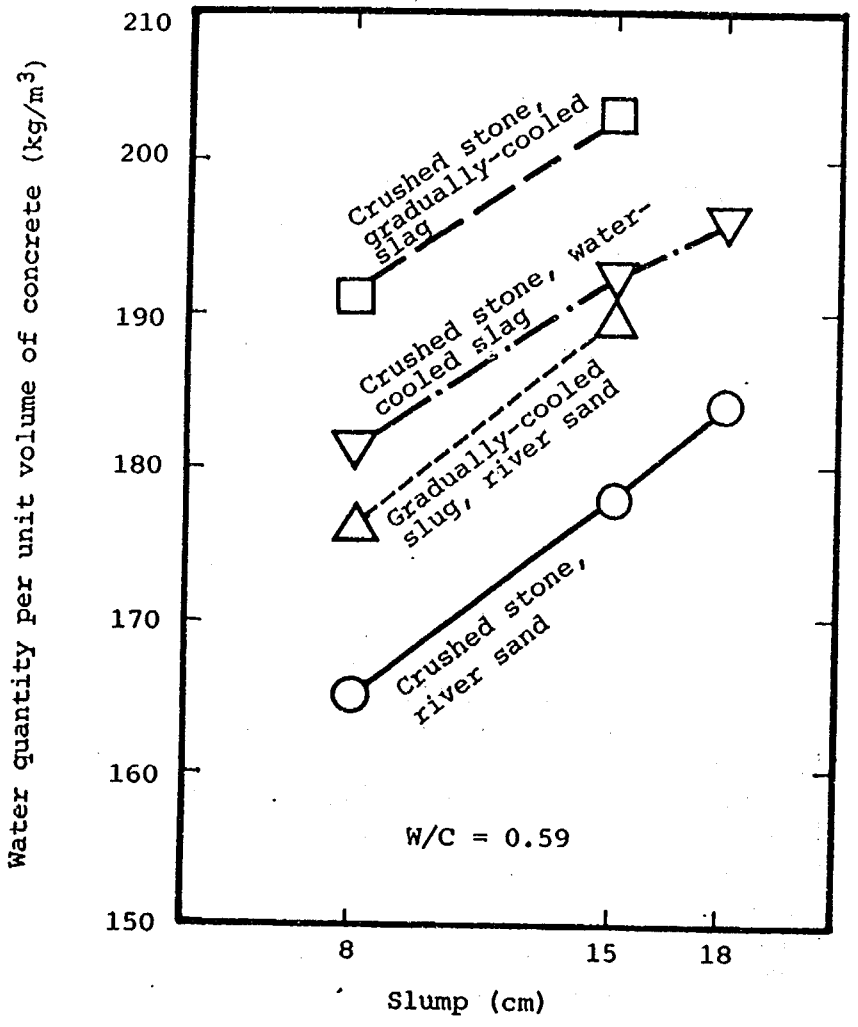


Fig. 6-4 Relationship between Water Quantity Per Unit Volume of concrete and Slump

It is preferable that water quantity per unit volume of concrete be minimized within the range in which desired workability can be obtained.

The water quantity per unit volume of concrete in the concretes using slag aggregate was 7 ∿ 16% greater than that in the standard concrete using crushed stone and river sand (hereinafter referred to as "standard concrete"). The increase rate was 7% when gradually-cooled slag was used as a coarse aggregate, while it was 8 ∿ 16% when gradually-cooled slag or water-cooled slag was used as a fine aggregate. The rate of increase in water quantity per unit volume of concrete in the fine aggregate of water-cooled slag was 8 ∿ 10%, while that in the fine aggregate of gradually-cooled slag was 14 ∿ 16%; water quantity per unit volume of concrete increased as the percentage of absolute volume decreased. It can thus be conjectured that the particle shape of the aggregate has a great effect on the increase in quantity of water per unit volume of concrete.

Fig. 6-5 shows the results of the bleeding test. Excessive bleeding should be forestalled because it has a number of harmful effects on concrete. As shown in Table 6-7, such target performances for concrete as bleeding level were specified by the Architectural Institute of Japan (JASS5). In the present test, the bleeding level in the concrete using gradually-cooled slag as a coarse aggregate tended to be slightly higher than that in the standard concrete, but the difference can be regarded as negligible. However, when either gradually-cooled slag or water-cooled slag was used as a fine aggregate, the bleeding level was approx. 2 times that in the standard concrete. Moreover, bleeding occurred at an earlier stage in the latter case, probably because slag aggregate not only is low in water retentivity, due to its glassy surface, but also contains a great amount of water per unit volume.

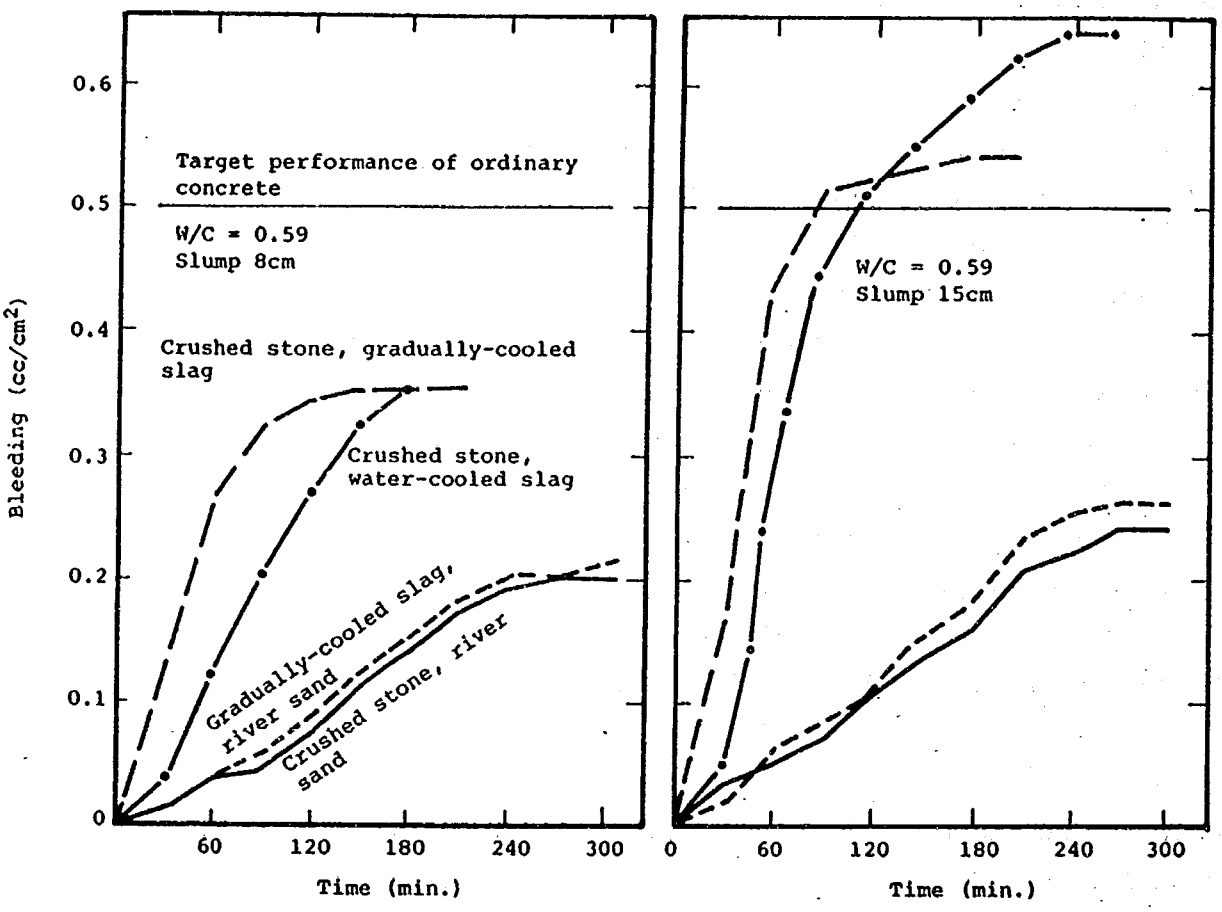


Fig. 6-5 Bleeding

Table 6-7 JASS5 Concrete Target Performance

Item / Grade	Bleeding (cc/cm^2)	Drying Shrinkage ($\times 10^{-4}$)	Durability Index (300 cycles)
High	0.3 max.	6 max.	70 min.
Ordinary	0.5 max.	8 max.	—
Low	0.7 max.	10 max.	—

6-3-2 Chemical and physical properties of hardened concrete

Fig. 6-6 shows the relationship between compressive strength and cement water ratio. The compressive strength of the concrete using slag aggregate was 5 ∿ 20% lower than that of standard concrete. When slag was used as a coarse aggregate, the rate of reduction in compressive strength was slightly higher than that observed when slag was used as a fine aggregate. In the concrete using slag aggregate, as well as in the standard concrete, compressive strength increased proportionate to the rise in cement water ratio. It can thus be inferred that there is no serious problem in the strength of slag aggregate itself, and that the reduction in compressive strength is affected by the performance related to the bond between cement paste and aggregate.

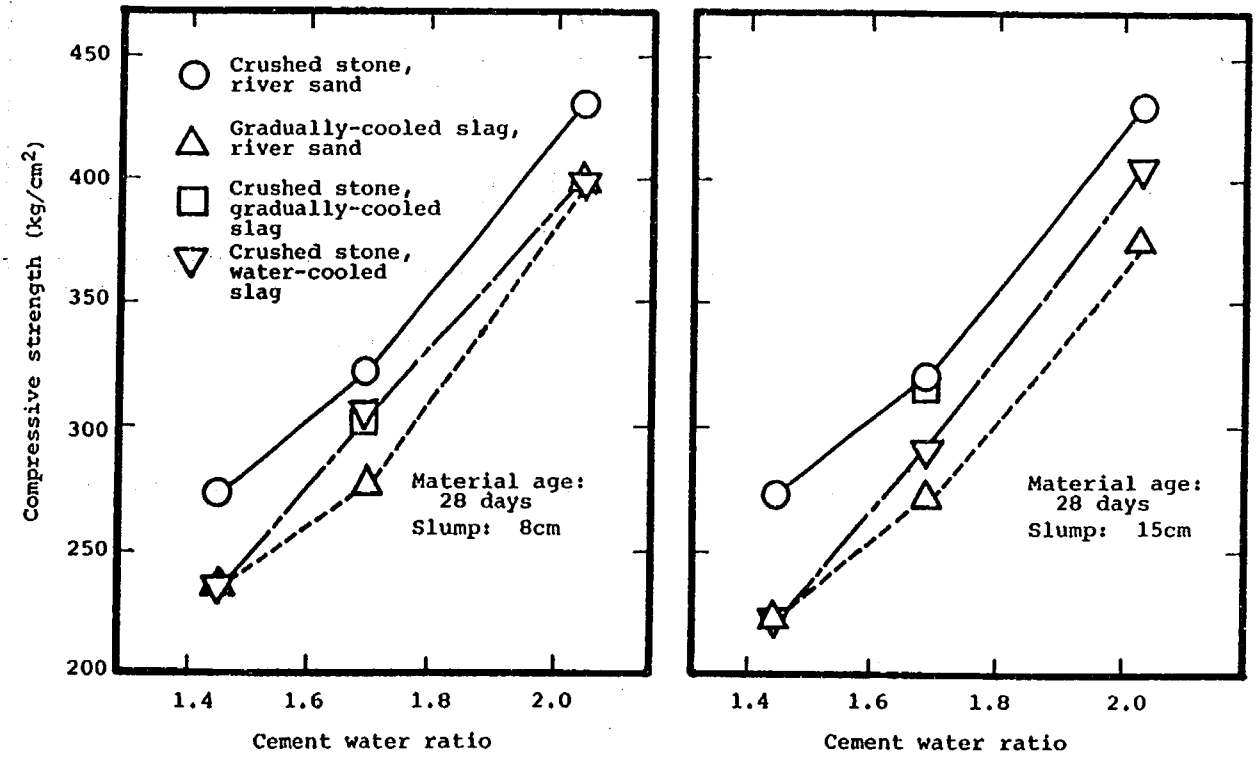


Fig. 6-6 Relationship between Compressive Strength and Cement Water Ratio

Fig. 6-7 shows the relationship between compressive strength and material age. The concrete using gradually-cooled slag as a fine aggregate was slightly higher than other concrete in rate of compressive strength increase with material aging.

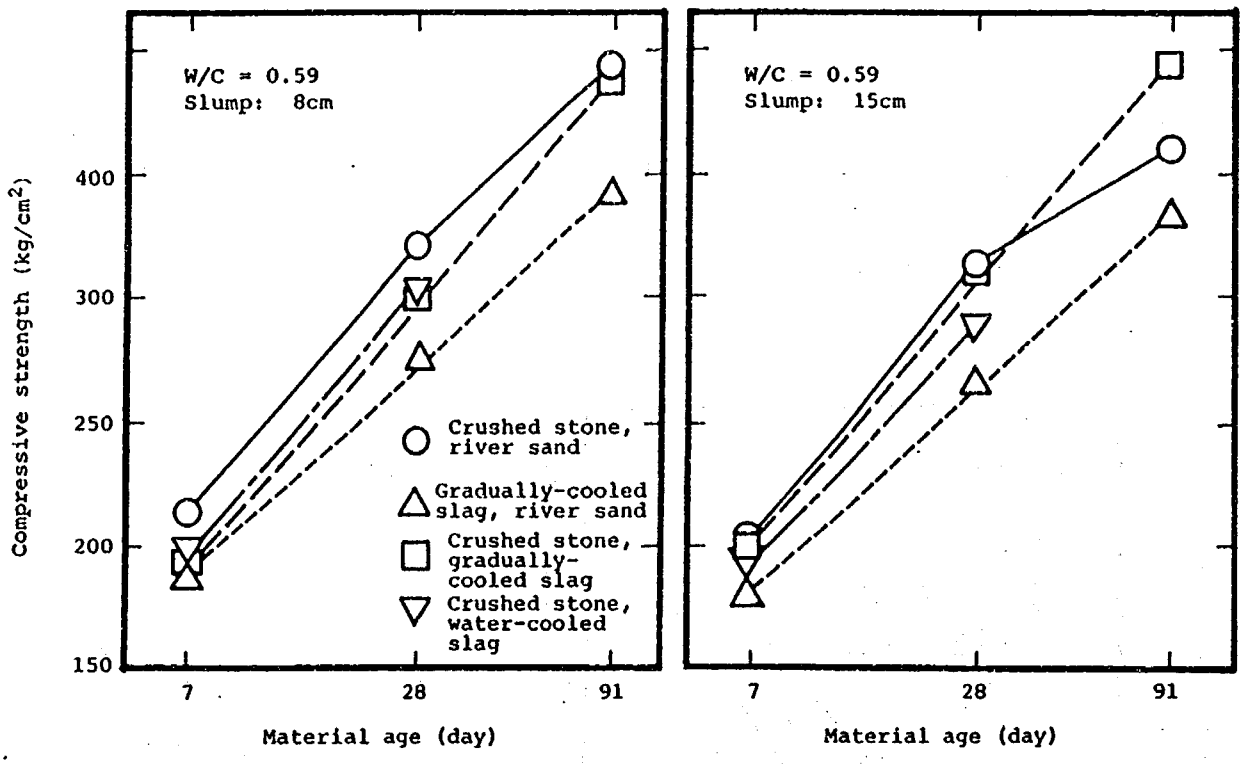


Fig. 6-7 Relationship between compressive Strength and Material Age

Fig. 6-8 shows the relationship between compressive strength and other strength factors. There was no difference between concrete using slag aggregate and standard concrete in ratio of tensile strength to compressive strength and of bending strength to compressive strength; the measurements were 1/10 ∿ 1/13 and 1/6 ∿ 1/8, respectively, corresponding to the tendency in ordinary concrete.

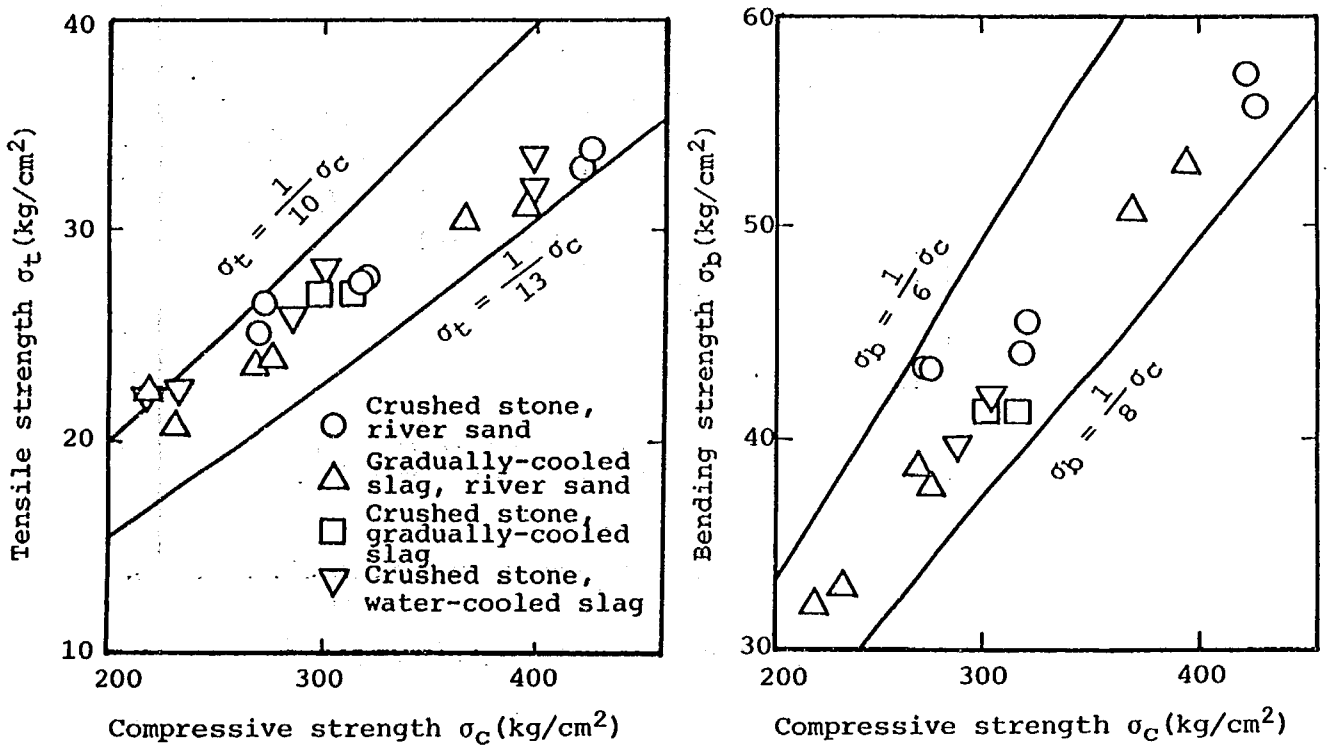


Fig. 6-8 Relationship between compressive Strength and Other Strength Factors

Fig. 6-9 shows the relationship between compressive strength and modulus of static elasticity. Based on the data shown in the figure, it can be inferred that there is no great difference in elastic modulus between concrete using slag aggregate and ordinary ocncrete.

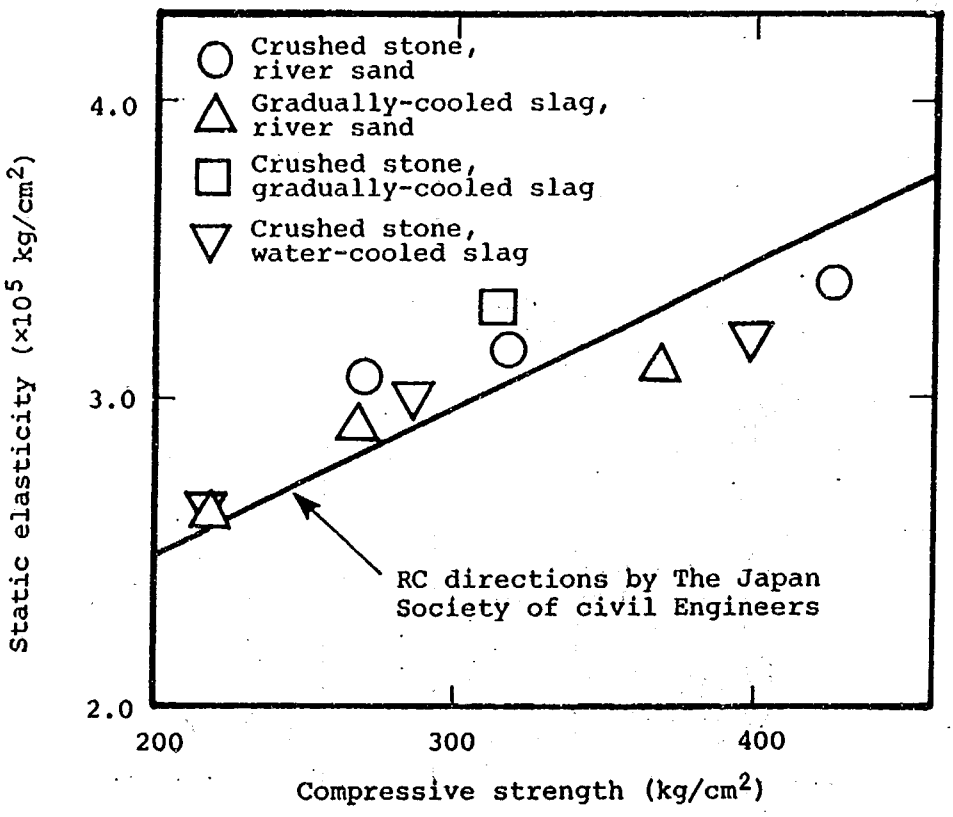


Fig. 6-9 Relationship between Compressive Strength and Static Elasticity

Fig. 6-10 shows the results of the drying shrinkage test. In general, the degree of drying shrinkage in concrete increases as the amount of water per unit volume of concrete increases. Aggregate type is also a factor causative of differences in the degree of drying shrinkage. It was anticipated that concrete using slag aggregate would be higher than standard concrete in rate of increase in drying shrinkage since the former contains a greater amount of water per unit volume. However, it was found from the test results that the concrete using slag aggregate was slightly lower than standard concrete in drying shrinkage level, contrary to expectations. This is attributed to the aggregate used.

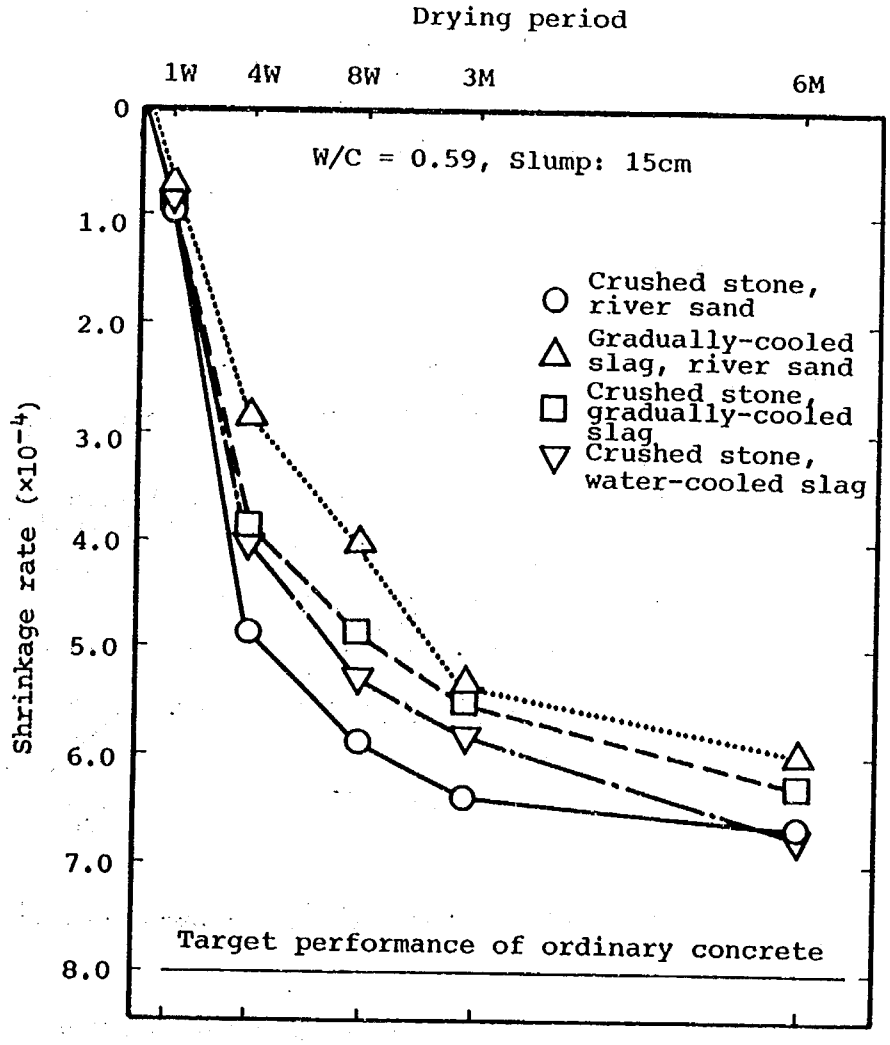


Fig. 6-10 Results of Drying Shrinkage Test

Fig. 6-11 shows the results of the freezing and thawing test. The dynamic modulus of elasticity showed a small decrease, weight reduction also being small in each test piece even after 300 cycles of freezing and thawing; it can thus be inferred that concrete is hardly degraded by freezing and thawing.

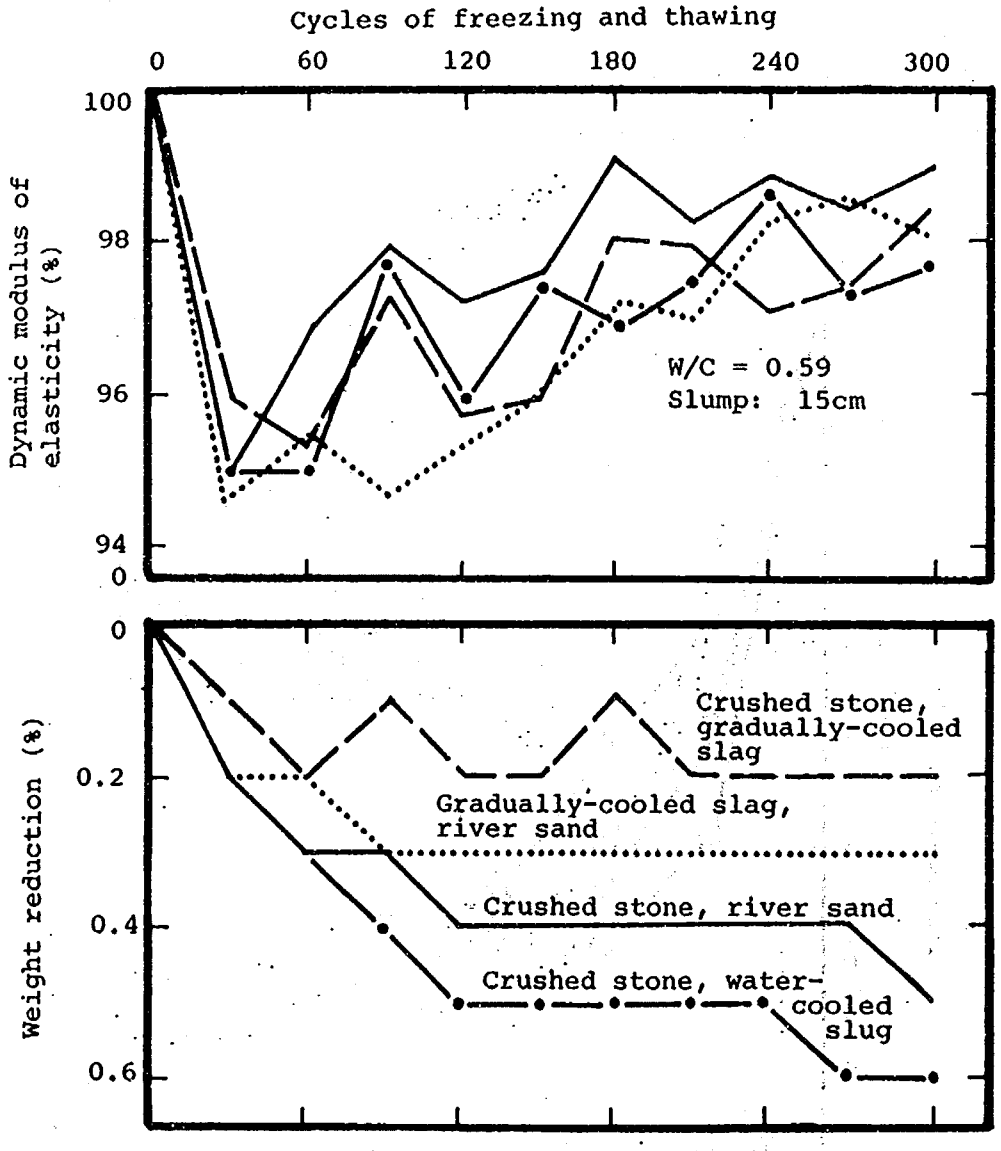


Fig. 6-11 Results of Freezing and Thawing Test

6-4 Potential for Efficient Utilization

The coarse aggregate of gradually-cooled slag has a low specific gravity and contains numerous bubbles; the amount of water per unit volume of concrete is raised by using this material as an aggregate for concrete. In addition, concrete using either gradually-cooled slag or water-cooled slag as a fine aggregate not only contains a larger amount of water per unit volume but also is high in bleeding level. It was therefore anticipated that the concrete using these types of slag aggregate would be lower in strength, water-tightness, and durability. The present study, however, revealed that the use of slag aggregate does not cause a great reduction in the strength of concrete, and that concrete using slag aggregate is of standard strength. In addition, favorable results were obtained in the drying shrinkage test and freezing and thawing test for concrete using slag aggregate; i.e., there is no problem in water-tightness or durability; it can therefore be expected that this type of concrete will work well as ordinary concrete.

When slag is used as an aggregate for concrete, the amount of water per unit volume of concrete increases because of unfavorable particle shape. If aggregate particle shape is improved by devising an efficient method of crushing slag, the amount of water per unit volume of concrete will thus be successfully reduced, and the quality of the concrete thus improved.

Since slag aggregate is darker in color than ordinary aggregate, it is necessary to consider irregular shading in concrete structures and products.

It is desirous that slag quality be stabilized, that the physical properties of concrete using slag be known clearly, and that its actual application be promoted.

7. CONCLUSIONS

The present incineration/melting system, an energy-saving type of melting system which incinerates, melts sludge with the efficient use of the heat produced by sludge proved satisfactory in the present experiment, and is thus thought of as a promising sludge treatment system.

The results of the present experiment are summarized as follows:

(1) It was proven that sludge is easily molten to slag.
Volumes of organic sludge (conditioned with polymer) and inorganic sludge (conditioned with lime) were reduced by slagging to approx. 1/30 and 1/16, respectively.
The volume reduction rate was 1/2 to 1/3 that with incineration ash obtained via the multiple-hearth furnace conventionally employed.

(2) The melting furnace used in the present experiment, with a treatment capacity of 2.4t-sludge/day, consumed heavy oil as an auxiliary fuel at the rate of 32.8ℓ/t-sludge in the case of organic sludge.
In an actual system which can treat 200 ∿ 300 ton of sludge per day, the rate of auxiliary-fuel consumption is less than 10ℓ-heavy oil/t-sludge, due to lower heat loss ratio; it is inferred that system consumes only 1/3 ∿ 1/4 of the auxiliary-fuel consumed in the conventional multiple-hearth incinerator.

(3) Both exhaust gas and NO_x production were successfully reduced by using the two-staged combustion method at a low air ratio.
Heavy metals in slag proved to be immobilized and no elution was found; it has been proven that these heavy metals are stable in slag.

(4) The slag satisfied all JIS (Japan Industrial Standards) requirements for aggregate. In addition, concrete testing evidenced the possibility of using slag as an aggregate for concrete.

(5) As the result of laboratory experiments, the minimum slag melting temperature was obtained by adjusting basicity (CaO/SiO_2) to approx. 1.
Crystallized slag was successfully obtained either by maintaining slag at a temperature approx. 50°C lower than its melting point for a specified period or by cooling it gradually.

For practical application of the present system, the following investigations should be carried out.

(1) Improvement of melting conditions for inorganic sludge

The applicability of the present melting system to organic and inorganic sludges has been proven in the present experiment. In the case of inorganic sludge, however, the optimum conditions for the overall sludge incineration/melting system must be investigated, in view of chemical composition (basicity) etc., since a higher melting temperature is required than with organic sludge.

(2) Improvement and efficient utilization of slag

In the gradual-cooling method used in the experiments, molten slag is left in a receiving tank for a specified period, after which it is taken from the tank and cooled in air. Both to stabilize and to improve slag quality, the establishment of the continuous slag ejection system under the optimum cooling conditions must be investigated further.

When the water-cooling method is used, the heat loss ratio can be reduced by increasing treatment capacity and/or improving unit structure; this approach is thought advantageous to some kinds of slag applications in the aspect of system simplification.

It is necessay to conduct a long-term availability test for slag as well as to establish the above-mentioned cooling method with the aim of the efficient utilization of slag.

(3) Establishment of comprehensive sewage sludge treatment system

It is necessary to establish a comprehensive sewage sludge treatment system by studying conventional sludge treatment systems from every point of view (revision of conventional processes, mutual utilization of surplus energy, etc.) assuming the use of the incineration/melting process.

(4) Investigation for practical application

It has been proven that the energy-saving effect of the present incineration/melting system can be improved by increasing its treatment capacity. It is desired that the technical and economical aspects of

the system be investigated on actual operating scale, with long-term proof testing on a medium scale.

To reach the goals shown above, we are planning to continue the present study with the aim of both improving energy-saving and stabilizing slag quality, as well as to conduct a proof experiment on a larger scale.

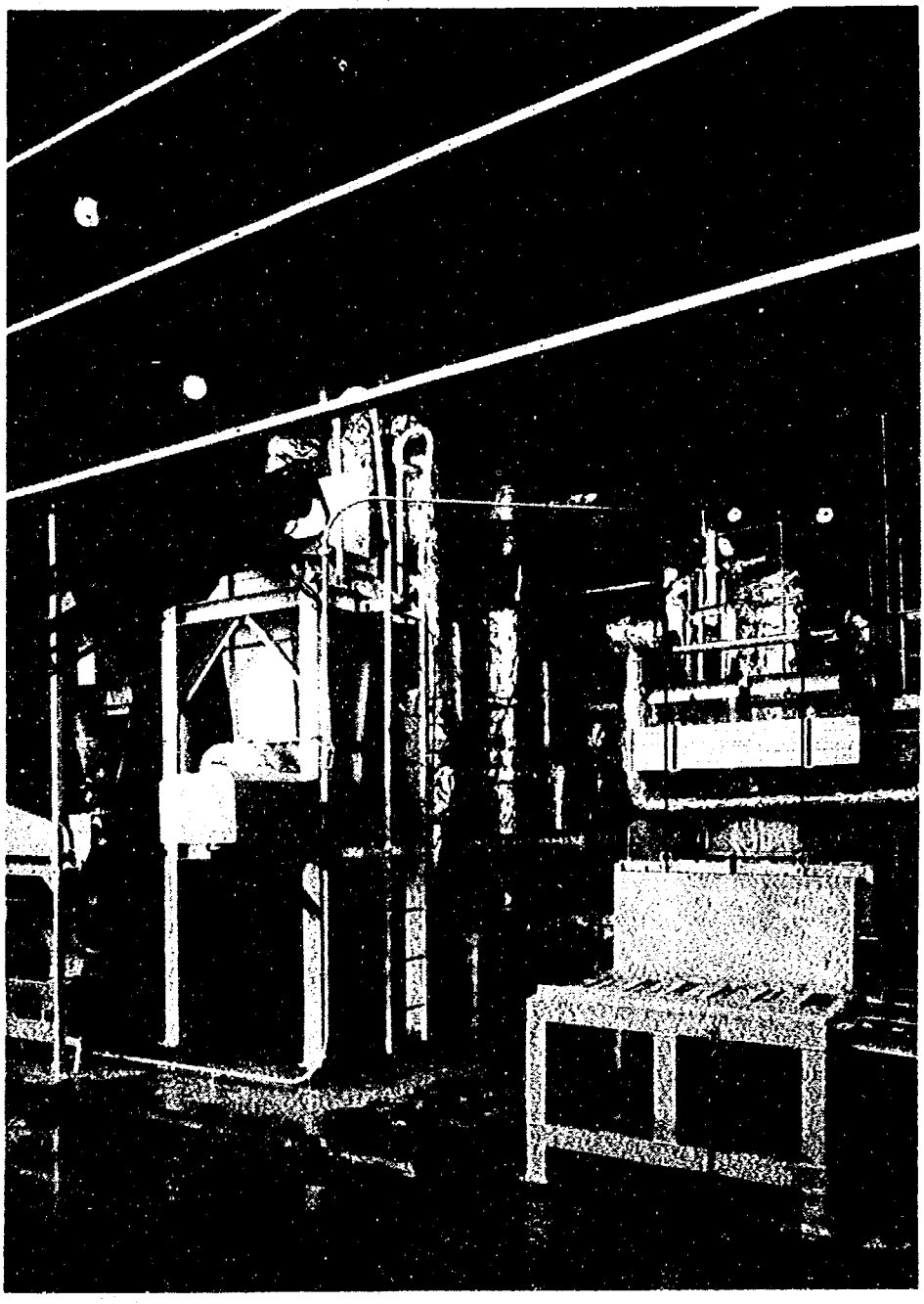


Photo 1. Incineration/Melting Pilot Plant



Photo 2. Water-cooled Slag

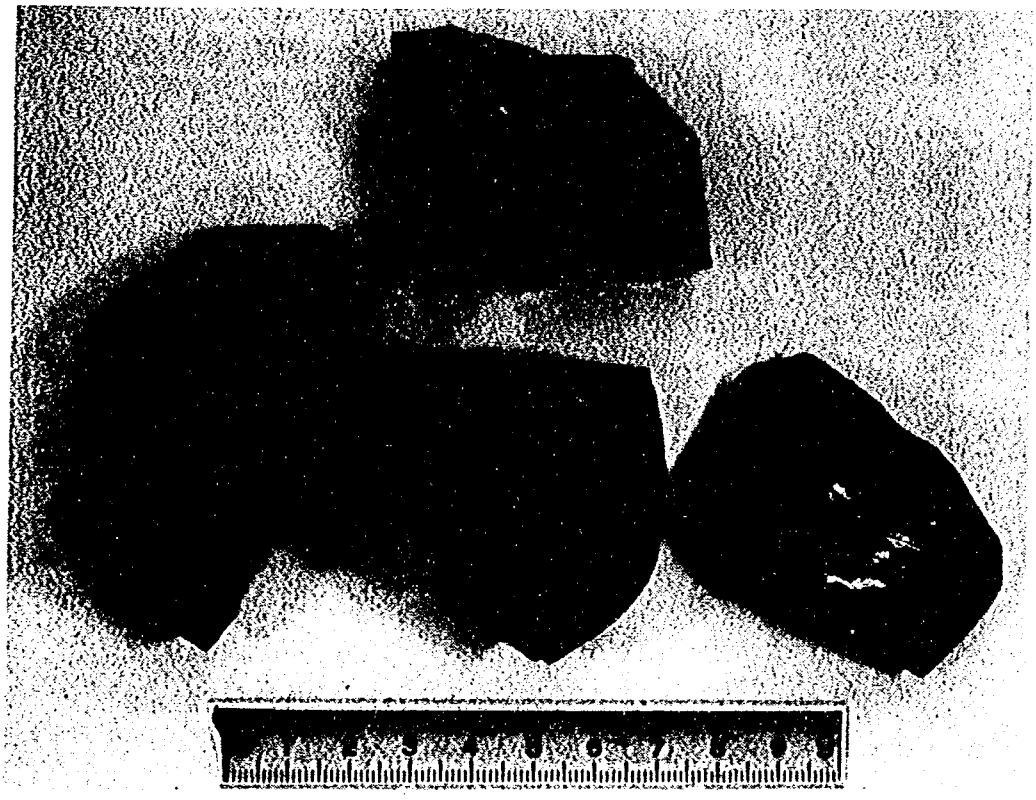


Photo 3. Gradually-cooled Slag

SEWAGE SLUDGE UTILIZATION AND DISPOSAL PRACTICES IN THE FEDERAL REPUBLIC OF GERMANY

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1. Introduction

Sludge is produced by the waste water treatment process. The general objective of processing sludge at waste water treatment plants is to render the sludge suitable for utilization or disposal at minimum cost (1). Sludge management begins with sludge generation and continues through sludge treatment, use and disposal. It is an integral consideration in the planning and design of waste water treatment plants, and can be the most complex and costly part of waste water management. Sludge contains a great amount of water as well as solids and dissolved substances that were present in the sewage and that were added by waste water treatment processes. The cleansing of sewage has demonstrably improved the quality of surface waters in many parts of the world, and dramatically so in some areas. The technology of waste water treatment has advanced considerably in the past several decades, as has the population served by treatment facilities. However, as treatment systems become more efficient and more common place, so does the volume of by-product - sewage sludge - also increase. The disposition of these ever increasing volumes of sewage sludge has become a problem of crisis proportions in many countries. The techniques used in sewage sludge treatment has been evolving since the last century. Available processing technology can produce virtually any type of end-product for utilization or disposal. Fig. 1 (2) gives a general survey on sludge treatment processes.

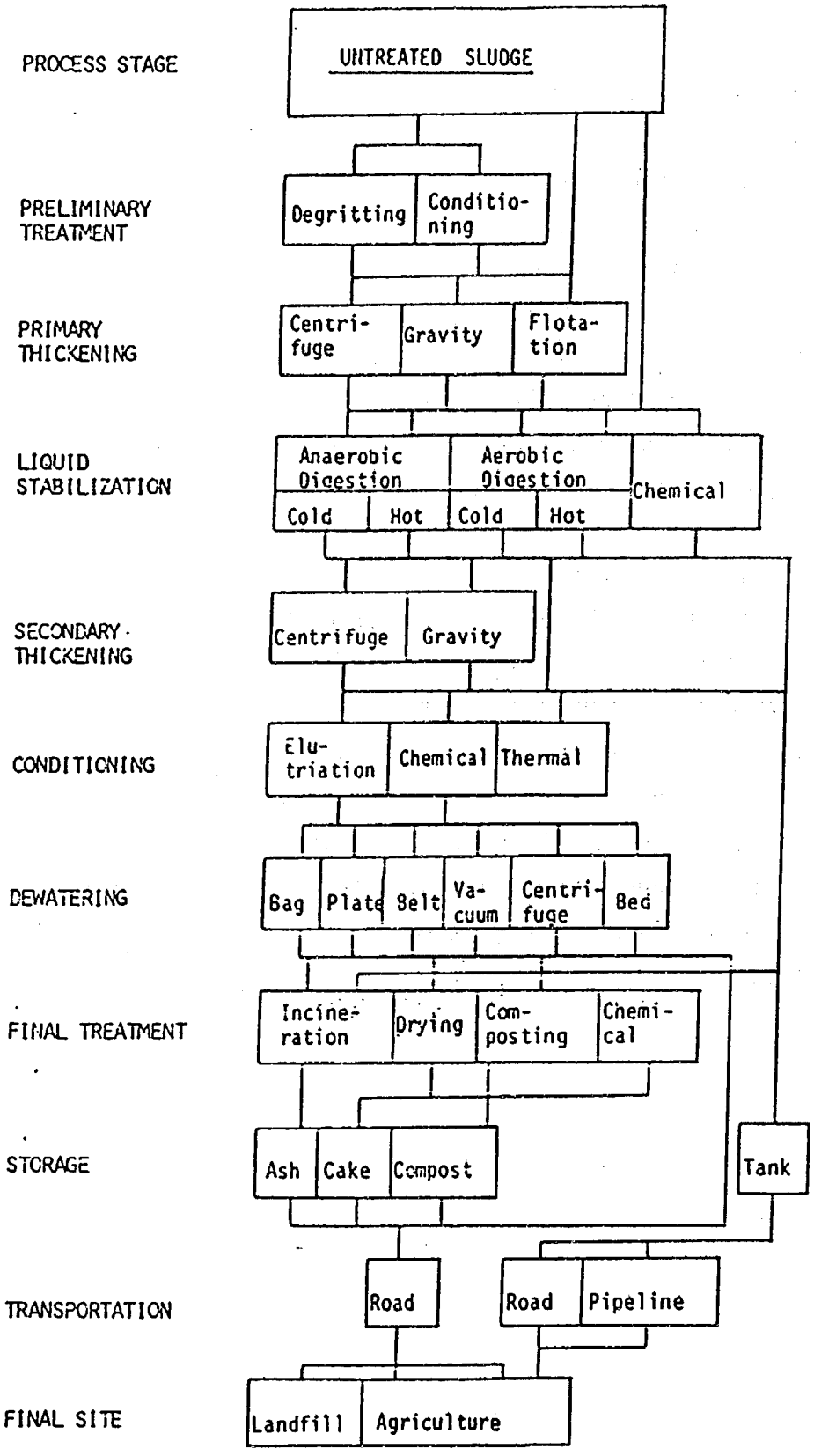


Fig. 1 Sludge Treatment Processes

Each process stage provides scope for variations in actual technique, type of equipment, and method of operation so that a wide variety of individual schemes occures in practice. Various external factors have influenced developments in sludge processing technology. For example, the steep rise in oil prices in past years has discouraged from the use of energy-intensive processes and encouraged ways of extracting as much energy as possible from the sludge itself. However, in some situations an expensive form of processing may still be justified on the ground of "easiness of disposal" of the end-product, and in other cases, a variety of treatment options may be retained on one site in order to give greater flexibility in disposal options (3). The widespread introduction of guidelines or legislation on sludge disposal to agriculture together with increased public concern about the quality of the environment, has caused a general trend towards higher levels of sludge treatment before disposal. Sludge processing greatly affects the quantity of sludge produced in waste water treatment, and can be divided in principle in three categories (4):

- Separation processes which aim at separating solid from liquid,
- Conversion processes which aim at changing the characteristics or composition of sludge in order to promote the possibilities of further sludge treatment or final disposal or utilisation,
- Disinfection processes

It should be pointed out that several sludge treatment methods can belong to all the three categories although the emphasis may be on one of them.

2. Factors Influencing Sludge Utilization and Disposal

2.1 Properties of Sewage Sludge

Sludge characterization involves description of sludge behaviour in treatment processes, utilization and disposal. Characterization methods facilitate understanding and prediction of sludge properties. The incentive for characterization could be one of the following (2)

- Because sludges exist in large quantities it is necessary to know the amounts and the expences involved in their treatment and disposal,
- because sludges are environmental nuisances, hazards, energy sources, fertilizers or even food sources it is necessary to know their potential,
- because the processes change the sludge properties it is necessary to know their treatability,
- because sludges have a history it is necessary to know how the generation process affects the quantities and the properties of raw sludge.

Parallel to the characterization of the sludges runs characterization of sludge treatment processes and the environment in which the sludge is disposed of.

The following categories are of interest:

- Sludge quantity: Sludge production data from sewage and sludge treatment methods
- Treatability parameters: Characteristics of how the sludge behaves in a specific process (e.g. dewaterability, digestability)
- Sludge quality parameters: Characteristics of how the sludge could influence the environment when disposed of by chemical and/or microbiological compounds (e.g. pathogens).

2.1.1 Sludge Quantity

An accurate estimation of sludge production rate is a prerequisite to economic planning and design of sludge handling, processing and disposal facilities. Sludge production can be given in mass or volume, in total or in

per inhabitant values. However, there are several factors which are difficult to quantify. The quantity is depending on raw water quality and treatment, water distribution system, household habits and diet, for example. Also the chosen sewerage system as well as constructions and dimensioning principles of the sewage collection system influence the quantity.

The total quantity of sludge requiring processing has increased considerably in most developed countries in the past decade and this trend is expected to continue. Several factors may contribute to sludge production increases, quite apart from population growth and higher industrial output. Upgrading of existing treatment facilities has also occured in many cases raising sludge production. An other reason for increased sludge production is the requirement to remove phosphorus from effluents discharging to rivers and lakes. A lot of works have new introduced phosphorus removal by means of chemical precipitation.

The quantity of secondary sludge generated by secondary biological treatment processes is depending on

- type of process (e.g. activated sludge system, attached growth systems such as tricklung filters)
- load on the process (kg BOD/kg MLSS, kg BOD/m^3 d)
- whether primary sedimentation is included or not

The sludge production rates of biological waste water treatment processes, used in planning are given in Table 1.

		Mean spec. Dry Solids Amount	Mean spec. Volatile Dry Solids	Dry Solids Content	Mean spec. Sludge Amount
		g OS/Inh. d	g VDS/Inh. d	%	l/Inh. d
Raw Sludge	Primary Sludge	54	38	5 - 10	0.72
	Sludge from a Trickling Filter B_R = 400gBOD_5/m^3 d	32	14 - 16	4 - 8	0.54
	Sludge from a Trickling Filter B_R = 200gBOD_5/m^3 d	24	9 - 11	4 - 8	0.40
	Excess Sludge BOD_5-SS load = 0.30 kg/kg d	40	20.4	0.5 - 2.5	2.67
	Excess Sludge BOD_5-SS load = 0.15 kg/kg d	36	16.2	0.5 - 2.5	2.40
	Excess Sludge[1] BOD_5-SS load = 0.05 kg/kg d	60 - 70	25 - 40	2 - 6	1.5 - 1.75
Digested Sl.	Primary	34	12 - 15	5 - 8	0.52
	Prim. + Trickl. F.	55 - 61	22 - 27	4 - 6	1.16
	Prim. + Act. Sl.	62 - 66	25 - 29	4 - 6	1.28

1) Primary and Secondary Sludge

Table 1: Sludge Production Rates of Biological Waste Water Treatment Processes

The characterization of tertiary sludges depend on the advanced waste water treatment process that generated it. Processes which add chemical (e.g. lime, organic polymers) to waste water result chemical sludges. One can calculate with the following approximate sludge production rates.

- Aluminium and ferrous precipitation generate 60 - 70 g DS/inh d with dosages ranging from 100 - 200 g/m^3 of aluminium sulfate or 30 - 50 g/m^3 of Fe
- Lime precipitation generates 200 - 500 g DS/inh d with a lime dosage of 300 - 700 g/m^3

Aluminium sulfate is normaly used in post-precipitation after secondary treatment but can also be used in simultaneous precipitation in the activated sludge process. Ferrous compounds are more frequently used in simultaneous precipitation. The influence of 60 - 125 g/m^3 aluminium sulfate or 15 - 25 g/m^3 of Fe in increasing sludge production has normally been estimated to 25 - 35 g DS/inh d.

The amount of the sludge production in the F.R. Germany depend on the treatment process given in Table 2 (10).

Treatment Process	Amount (x 10^6tDS/a)	Amount (%)	Portion of 1.4 (%)
1 Sludge Production			
1.1 primary	2.340	91.7	
1.2 secondary			
1.3 tertiary	0.213	8.3	
1.4 Σ	2.553	100.0	100.0
2 Stabilisation			
2.1 input amount	2.042	100.0	
2.2 aerobic stab.	0.327	16.0	
2.3 anaerobic stab.	1.716	84.0	
2.5 reduction	0.388		
2.6 output amount	1.654		64.8
3 Conditioning			
3.1 input amount	1.940	100.0	76.0
3.2 org.flocculants	1.213	62.5	
3.3 anorg.flocculants	0.446	20.9	
3.4 thermic/others	0.296	16.6	
3.5 output amount	1.955		76.6
4 Dewatering			
4.1 input amount	1.955	100.0	
4.2 belt filter		45.9	
4.3 centrifuge		13.5	
4.4 filter press		38.9	
4.5 others		1.7	
4.6 output amount	1.955		
5 Advanced Treatment			
5.1 input amount	0.286	100.0	11.2
5.2 drying	0.005	1.8	
5.3 incineration	0.230	80.4	
5.4 others	0.051	17.9	

Tab.2 Sewage Sludge - Amount and Treatment in the F.R.Germany (9)

2.1.2 Sludge Quality

Sewage sludges from different sources can exhibit wide differences in their physical, chemical and biological characteristics. This means that their behaviour in the various processing operations and their characteristics after processing can vary widely. In order to arrive at the most appropriate choice, specification and method of operation of a sludge processing facility, it is desirable to characterise the particular sludge in respect of the relevant processing parameters. Sludge quality parameters are indicators of how the sludge could influence the environment when disposed of. Besides heavy metals, there are numerous other factors influencing the characterization and evaluation of sewages sludges. Table 3 has been prepared listing methods of characterization that have either been used in the field of sludge investigation or proposed for such purpose on the basis of scientific studies (6). This list has been related to an enumeration of the common methods of sludge treatment and disposal and includes a rating that provides information of the necessity of the application of the former. This list does not include the microbiological methods needed for an evaluation of the agricultural use of sludges.

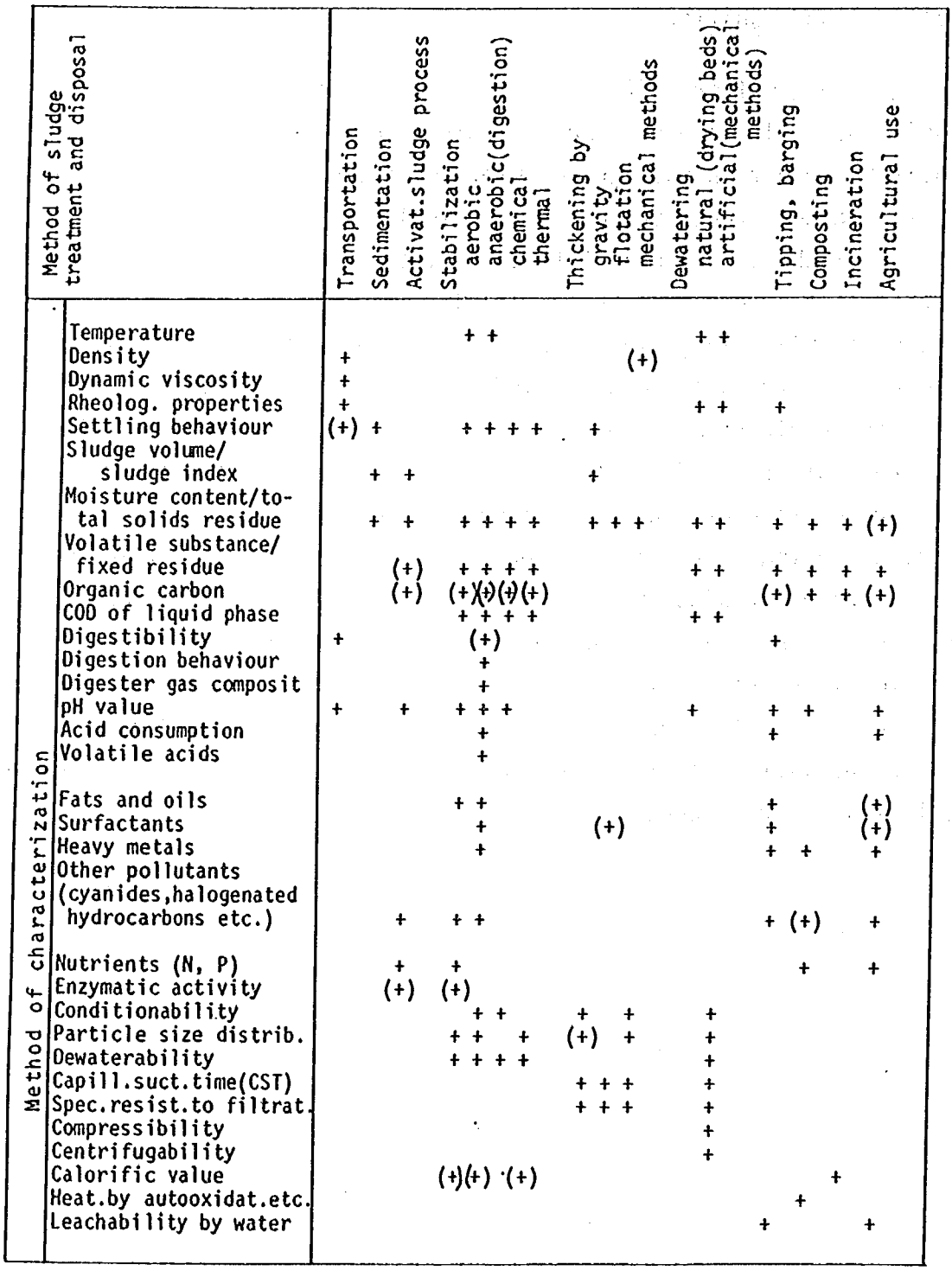


Table 3: Characterization of Sewage Sludge for Evaluation of Methods of Treatment and Disposal (6)

+: necessary (+): desirable

As an example in Table 4 the results of examinations of 344 sludge samples in the state of Baden-Württemberg is shown.

Parameter	Dimension	Average	Range
Dry Solids	%	35,3	2,6- 93,3
Volatile matter	%	47,9	10,3- 82,7
pH		7,3	5,9- 12,3
Calcium (Ca)	%	7,4	0,5- 39,9
Total Nitrogen	%	2,1	0,4- 12,3
Ammonia Nitrogen (NH_3-N)	mg/100 g	296	1 - 2276
Nitrate Nitrogen (NO_3-N)	mg/100 g	65	0 - 480
Total Phosphorus (P)	%	1,7	0,2- 4,9
Phosphorus (P)	mg/100 g	655	11 - 3722
Potassium (K)	mg/100 g	143	17 - 1149
Magnesium (Mg)	mg/100 g	39	0 - 289
Sodium (Na)	mg/100 g	397	3 - 960
Boron (B)	mg/1000 g	23,2	0,9- 358
Copper (Cu)	mg/1000 g	378	20 - 2500
Zinc (Zn)	mg/1000 g	2071	70 -15752
Manganese (Mn)	mg/1000 g	357	57 - 1245
Lead (Pb)	mg/1000 g	299	19 - 1500
Cadmium (Cd)	mg/1000 g	17,3	1 - 150
Chromium (Cr)	mg/1000 g	630	10 - 6400

Table 4: Results of examination of 344 sludge samples in the state Baden-Württemberg (15)

Municipal sewages contain considerable amounts of infectious agents like bacteria, viruses and parasites which derive from acute sick persons and carriers but also from stockyards, slaughterhouses and other sources from trade, industry and agriculture. These pathogenic agents reach the sewage treatment plants via the sewers. It was for a long time common opinion that by the treatment processes these were also relieved from the suspended pathogenic agents.

But it is known now that an unfailing elimination of pathogens can be achieved in none of the steps of sewage treatment. Investigations in Denmark and Sweden have shown that the effluents of all investigated sewage treatment plants still have various pathogens in up to 100 % of all samples. Therefore it can be stated that the receiving waters are loaded with salmonellas and other infectious agents by the not decontaminated effluents of sewage works.

Due to the sedimentation processes during the sewage purification many of the pathogens adsorbed to particulate matter are sedimented and enriched in the sludges. Therefore the raw and excess sludges contain considerable concentrations of salmonellas according to investigations in Switzerland. 93 % of the primary and 85 % of the secondary sludges were infected. In the past it was expected that by the mesophilic, anaerobic, alkaline digestion process the pathogens were destroyed. But because the digestion is a continuous process it is not possible. Also digested sludges contain more or less large quantities of infectious agents. Therefore neither the effluents nor the sludges of municipal sewage treatment plants are hygienically safe if they are not disinfected (7, 8).

Therefore it is necessary to reduce the number of pathogenic microorganisms. Which specific pathogens should be killed and the required reduction factor depends on many factors, the initial number of pathogens and the type of application being the most important ones. For purposes, e.g. where crops that are consumed by humans raw could come into contact with sludge (e.g. horticulture) also complete viral kill might be necessary. The control of microbiological contamination by specific sludge processing techniques is a simpler proposition than the control of chemical contamination. Recently, we got a rather good knowledge about the effectiveness of different techniques for removal of bacterial pathogens (e.g. Salmonellae) and of parasites (e.g. Taenia). An end-product which is acceptable from the hygienic point of view can be obtained by using the

sewage sludge disinfection methods like pasteurization, composting, aerobic-thermophilic stabilisation, aerobic-thermophilic treatment with subsequent anaerobic digestion, lime treatment, thermal drying or irradiation (10).

Table 5 (11) gives a survey on essential critical substances for the disposal of sewage sludge, the possible impairments and solutions attached to them.

Constituents of sewage sludge with a crucial bearing on disposal	Possible detrimental effects	Elimination in the treatment plant or prior to discharge to sewer system	Process used to treat sewage sludge prior to disposal, or necessary evasive action
Biodegradable organic matter	Emission of odours	Sewage Treatment Plant	Stabilization: biological aerobic (including composting) anaerobic (digestion) thermal oxidation: Incineration
High water content	Hindrances associated to landfilling	Sewage Treatment Plant	Water separation by means of: - Thickening: by flotation, or mechanically - Dewatering: mechanically or naturally (including conditioning), - Drying
Pathogens	Possible restrictions associated in agricultural use (health risks)	Sewage Treatment Plant	Disinfection by means of: - Heat Treatment (eg. Pasteurization) - Irradiation - Chemical Treatment (Lime Treatment)
Noxious substances	Risk of high concentration of noxious substances entering the food chain through soil treated with sludge	Prior to discharge to sewer system	Elimination prior to discharge of the contaminated waste-water into the public sewerage system

Table 5: Constituents of sewage sludge with a crucial bearing on disposal, possible harmful effects of each, and treatment processes used. (11)

3. Sewage Sludge Utilization and Disposal in the F.R. of Germany

The total amount of sewage sludge generated in the F.R. of Germany is about 46 x 10^6 m^3/a (liquid sludge). The population amounts to 61,4 Mio. of which approx. 90 % are served by public sewer system. It means that 2,3 x 10^6 t DS per year has to be treated, disposed or utilized. This number has been increased from approx. 0,6 x 10^6 t DS/a in 1955 over 1.8 x 10^6 t DS/a in 1975 and will continuously increase in the following years. The generated sludge is disposed of as follows by means of

Agricultural Utilization	29 %
Controlled Dumping (landfilling)	59 %
Incineration	9 %
Composting	3 %

3.1 Agricultural Utilization of Sewage Sludge

Sludge utilization on land is desirable because it recycles resources and is usually more economic than disposal. Sludge contains several plant macro-nutrients, principally N and P, and in most cases, significant of micronutrients such as boron (B), manganese (Mn), copper (Cu), molybdenum (Mo) and zinc (Zn). The exact ratio of the nutrients will not be that of a well-balanced formulated fertilizer. Nevertheless, most agronomic crops respond favourably to the nutrients in sludge.

Sludge may also be a valuable soil conditioner. In coarse-textured sandy soils, sludge can increase the water-holding capacity of the soil, and provide chemical sites for nutrient exchange and adsorption.

On the other hand sludge can contain constituents which are potentially harmful to the crops themselves (phytotoxicity), or to animals and humans who consume the crops. To avoid this problem, the quantity of sludge which may be applied per unit of land area, both on an annual and cumulative basis, is to be controlled in accordance with regulatory limits or guidelines, and good management practices.

The management of treatment and disposal is the responsibility of municipalities except in the Federal state Nordrhein-Westfalen, where there are 10 regional water authorities (e.g. Ruhrverband, Niersverband, etc.) which are waste water treatment co-operatives between municipalities.

Waste water treatment is coordinated by water management agencies in the Federal States with judicial powers conferred by the Water Management Act. There is a multitude of ways in which the authorities cooperate with farmers in the agricultural use of sludge.

The F.R. of Germany with its total area of approx. 250.000 km^2 has an area used as agricultural land of 140.000 km^2 (76.000 km^2 arable land, 64.000 km^2 grassland, pastures). The portion of the agricultural utilization of the sewage sludge generated in the F.R. of Germany is different among the Federal States, as shown in Fig. 2.

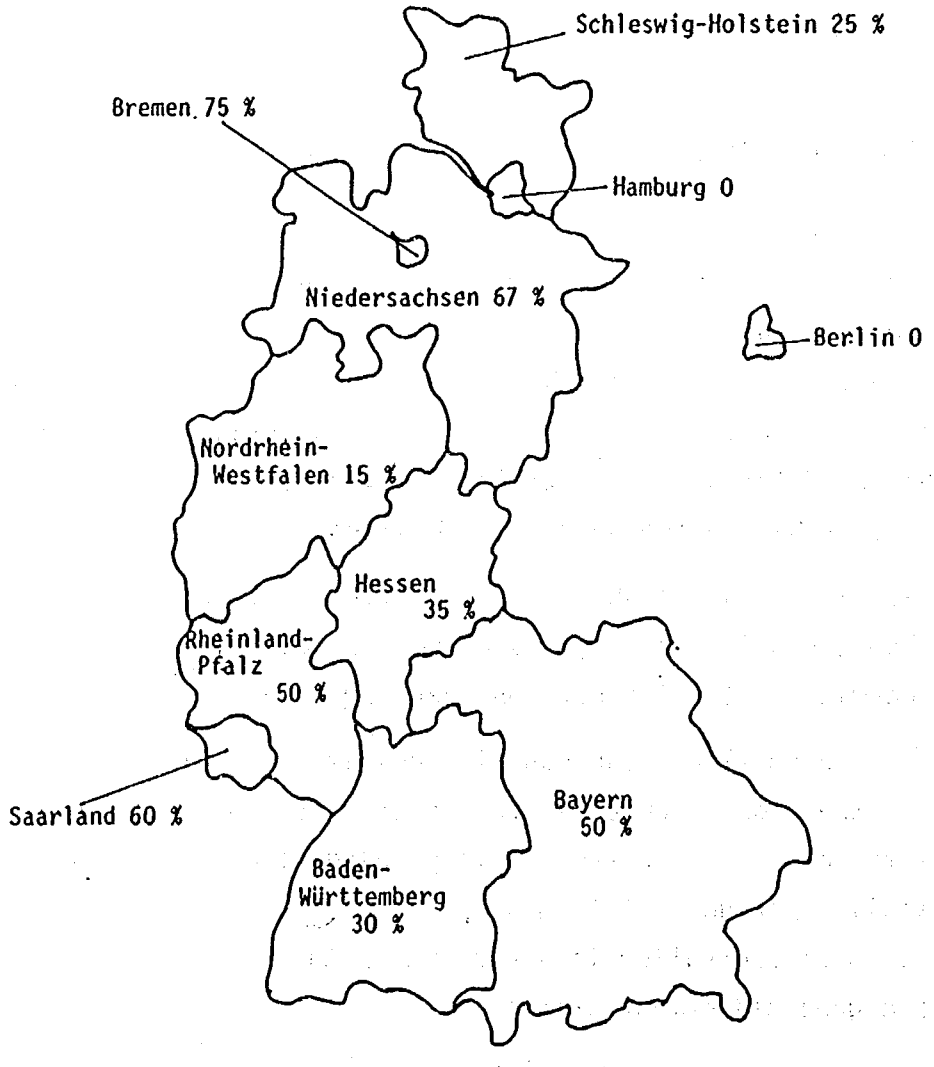


Fig. 2: Percentage of Agricultural Utilization of Sewage Sludge in F.R.G

Table 6 gives a survey on the cost of sludge disposal by means of agricultural utilization, sanitary landfill and combustion.

Sludge Disposal by means of	Spec. Cost after anaerobic digestion DM/t DS	spec. Energy-demand
Agricultural Use of Liquid Sludge		
- without disinfection	~ 100 - 200	very low
- with disinfection	~ 160 - 320	low
Landfill after Dewatering		
- by centrifuges, filter-presses, belt filter presses without deposit cost	~ 120 - 450	middle
- with deposit cost	~ 250 - 580	
Addition of Quick Lime (CaO) for increasing of dry matter from 25 % to 35 %	~ 65	middle
Combustion		
- without deposit cost	~ 400 - 1000	
- with deposit cost	~ 450 - 1050	very high

Table 6: Cost of Sewage Sludge Utilization and Disposal (16)

3.1.1 Sludge Utilization Guidelines

Guidelines for land utilization of sewage sludge have been developed in several countries. They are similar in many respects and consider sludge quality, time and rate of application, nutrient and metal addition to soil, and soil characteristics. In general, sludges must be stabilized by biological, thermal or physical-chemical treatment to reduce pathogen concentrations prior to land application. Several of the guidelines recommend that annual applications of sludge should meet the crop demand for N. Some also recommend that the soil pH should exceed 6.0 to minimize heavy metal solubility and crop uptake. Most define maximum permissible heavy metal concentrations in sludge for use on agricultural land and maximum heavy metal loadings to soil. The German Regulations in the Ordinance for the application on agricultural land ("Klärschlammverordnung") have been described on the occasion of the first Japanese-German Workshop on Waste Water and Sludge Treatment in 1982 by Möller (12). For your remembrance the limits for heavy metals in the sludge and its concentration in the soils are shown in Table 7.

Element	Sewage Sludge [mg/kg DS]	Soil [mg/kg DS]
Cadmium (Cd)	20	3
Zinc (Zn)	3000	300
Copper (Cu)	1200	100
Nickel (Ni)	200	50
Lead (Pb)	1200	100
Chromium (Cr)	1200	100
Mercury (Hg)	25	2

Table 7: Maximum permissable content of heavy metals in sewage sludge and soil according to the German "Klärschlammverordnung"

Whereas the disparity between the Member States' provisions on the agricultural applications of sewage sludge might affect the functioning of the common market within the European Community the approximation of laws should therefore be investigated.

The council of the European Communities has adopted a directive on the protection of the environment, in particular of the soil, when sewage sludge is used in agriculture (14). Whereas special arrangements should be made to ensure that man, animals, plants and the environment are fully safeguarded against the harmful effects arising from the uncontrolled use of sludge. Furthermore this directive aims at establishing certain initial Community measures in connection with soil protection. The Member States should be able to draw up more stringent provisions than those laid down in the directive.

The most important parts of the directive are the following ones:

Article 7

Member States shall prohibit the use of sludge or the supply of sludge for use on:

a) grassland or forage crops of the grassland is to be grazed or the forage crops to be harvested before a certain period has elapsed. This period, which shall be set by the Member States taking particular account of their geographical and climatic situation, shall under no circumstances be less than three weeks;

b) soil in which fruit and vegetable crops are growing, with the exception of fruit trees;

c) ground intended for the cultivation of fruit and vegetable crops which are normally in direct contact with the soil and normally eaten raw, for a period of 10 months preceding the harvest of the crops and during

the harvest itself.

Article 8

The following rules shall be observed when using sludge:

- the sludge shall be used in such a way that account is taken of the nutrient needs of the plants and that the quality of the soil and of the surface and ground water is not impaired,
- where sludge is used on soils of which the pH is below 6, Member States shall take into account the increased mobility and availability to the crop of heavy metals and shall, if necessary, reduce the limit values they have laid down in accordance with table 8.

Article 10

1. Member States shall ensure that up-to-date records are kept, which register:

(a) the quantities of sludge produced and the quantities supplied for use in agriculture;

(b) the composition and properties of the sludge in relation to the parameters dry matter, org. matter, pH, N, P, Cd, Cu, Ni, Pb, Zn, Hg, Cr

(c) the type of treatment carried out;

(d) the names and addresses of the recipients of the sludge and the place where the sludge is to be used.

2. The records shall be available to the competent authorities and shall provide a basis for the consolidated report referred to in Article 17.

3. Information on the methods of treatment and the results of the analyses shall be released upon request to the competent authorities.

Article 17

Five years after notification of this Directive, and every four years thereafter, Member States shall prepare a consolidated report on the use of sludge in agriculture setting out the quantities used, the criteria followed and any difficulties encountered; they shall forward this report

to the Commission, which shall publish the information contained therein. In the light of that report, the Commission shall if necessary submit appropriate proposals for increased protection of the soil and the environment.

Values for concentrations of heavy metals in soil to which sludge is applied, concentrations of heavy metals in sludge and the maximum annual quantities of such heavy metals which may be introduced into soil intended for agriculture are given in the tables 8, 9 and 10.

Parameters	Limit values(1)
Cadmium	1 to 3
Copper(2)	50 to 140
Nickel(2)	30 to 75
Lead	50 to 300
Zinc(2)	150 to 300
Mercury	1 to 1,5
Chromium(3)	---

(1) Member States may permit the limit values they fix to be exceeded in the case of the use of sludge on land which at the time of notification of this Directive is dedicated to the disposal of sludge but on which commercial food crops are being grown exclusively for animal consumption. Member States must inform the Commission of the number and type of sites concerned. They must also seek to ensure that there is no resulting hazard to human health or the environment.

(2) Member States may permit the limit values they fix to be exceeded in respect of these parameters on soil with a pH consistently higher than 7. The maximum authorized concentrations of these heavy metals must in no case exceed those values by more than 50 %. Member States must also seek to ensure that there is no resulting hazard to human health or the environment and in particular to ground water.

(3) It is not possible at this stage to fix limit values for chromium. The Council will fix these limit values later on the basis of proposals to be submitted by the Commission, within one year following notification of this Directive.

Table 8: Limit values for concentrations of heavy metals in soil (mg/kg of dry matter of soil with a pH of 6 to 7)

Parameters	Limit values
Cadmium	20 to 40
Copper	1000 to 1750
Nickel	300 to 400
Lead	750 to 1200
Zinc	2500 to 4000
Mercury	16 to 25
Chromium(1)	---

(1) It is not possible at this stage to fix limit values for chromium. The Council will fix these limit values later on the basis of proposals to be submitted by the Commission within one year following notification of this Directive.

Table 9: Limit values for heavy-metal concentrations in sludge for use in agriculture

(mg/kg of dry matter)

Parameters	Limit values(1)
Cadmium	0,15
Copper	12
Nickel	3
Lead	15
Zinc	30
Mercury	0,1
Chromium(2)	---

(1) Member States may permit these limit values to be exceeded in the case of the use of sludge on land which at the time of notification of this Directive is dedicated to the disposal of sludge but on which commercial food crops are being grown exclusively for animal consumption. Member States must inform the Commission of the number and type of sites concerned. They must also ensure that there is no resulting hazard to human health or the environment.

(2) It is not possible at this stage to fix limit values for chromium. The Council will fix these limit values later on the basis of proposals to be submitted by the Commission within one year following notification of this Directive.

Table 10: Limit values for amounts of heavy metals which may be added annually to agricultural land, based on a 10-year average

(kg/ha/yr)

3.2 Other Sludge Disposal Practices

The choice of a sludge disposal based on a variety of considerations including costs, regulations and public opinion. Costs may be highly variable depending on economic conditions and a number of site specific factors (e.g. transport distance, land requirements etc.). Increased regulation generally increases cost due to increased monitoring and surveillance requirements.

Composting

Composting is a sludge treatment process involving aerobic thermophilic decomposition of organic matter and it is practiced in Germany as a little component (3 %) of disposal by land utilization. Its primary value is as a soil conditioner and not as a fertilizer material because it contains only small concentrations of the major plant nutrients (N, P, K). Sometimes municipal sludge is mixed and composted with a variety of other solid wastes including municipal refuse. The product is often used in public gardens and recreation areas.

Landfills

Disposal by landfilling accounts for a much more municipal sludge as agricultural utilization. Its use in the F.R. of Germany has increased due to increasing regulation of agricultural utilization. Sanitary landfilling involves application to land and subsequent covering with a layer of soil. The sludge has to be stabilized and dewatered to a water content $\leq$ 65 %. Co-disposal of sludge with municipal solid waste is the most common landfilling method. Landfilling is simple and convenient, but it offers no opportunity for resource recovery. Figure 3 shows the possibilities for the sewage sludge disposal.

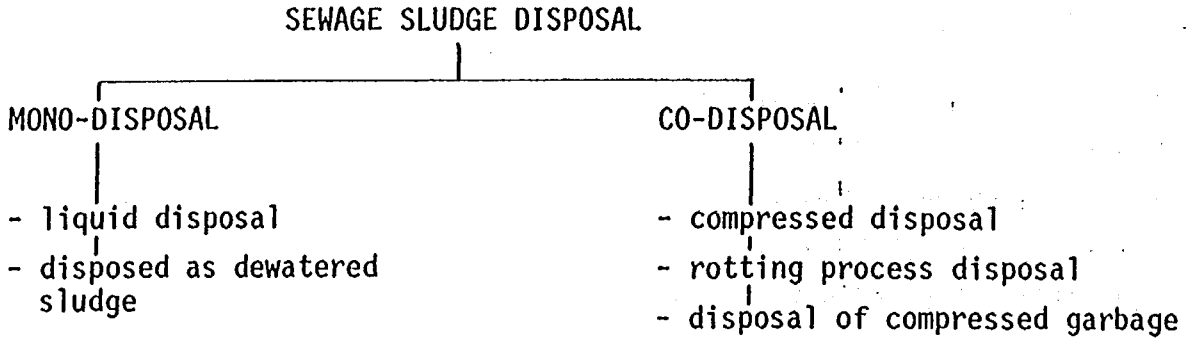


Fig. 3 Possibilities for Sewage Sludge Disposal (17)

3.3 New Developments in Sludge Disposal Possibilities

Within the framework of projects sponsored by the Federal Ministry for Research and Technology a few unconventional sludge disposal techniques are under research and development. They have not been adopted on a large scale:

- Utilization of sewage sludge by pyrolysis in a fluidized bed reactor
- Utilization of dried sewage sludge for heating drying drums of asphalt mixing facilities and placing the ash into pavement material
- Recovery of chemical raw materials from sewage sludge
- Technique for separation and recovery of phosphorus, iron salts and heavy metals from precipitation sludges of the phosphorus elimination
- Utilization of sludges as feed additives

4. Conclusions

Sewage sludge is a nutrient-rich, largely organic by-product of municipal waste water treatment processes. Sludge production in the F.R. of Germany is approx. 2,5 Mill. t DS per year and it has been estimated that it will continuously increase in the future. The increased production will result

from a upgraded treatment process to satisfy increasingly stringent effluent quality standards.

Sludge treatment and disposal is the most costly part of waste water treatment and can account for as much as 60 % the Water Pollution Control budget. There are several sludge treatment and disposal alternatives, but it is to pay attention to its environmental impact. The most popular practice has involved anaerobic digestion of sludge followed by sanitary landfilling or agricultural utilization. In some cases, esp. in large cities where land is not available or where transportation distances are too great, sludge has been incinerated.

Increases in energy cost and environmental awarness have resulted in a careful assessment of sludge handling and disposal practices. Incineration has become expensive due to the energy demand for dewatering and burning sludge. The two main options for sludge disposal in Germany are sanitary landfilling and agricultural utilization. Landfilling is resource wasteful, whereas agricultural utilization is resource conservative because it employs the nutrient and soil conditioning value of sludge. Agricultural utilization of sewage sludge involves advantages and disadvantages. The advantages are that sludge contain 3 % N, 2 % P and soil conditioning value. The disadvantages are that the sludge may contain a variety of pathogenic organisms, toxic organic compounds, soluble salts and heavy metals.

For environmental reasons, an increase of the agricultural utilization of sludge is necessary. This aim can only be reached if the use of it will become safer with special regard to soil and groundwater protection. New methods of sludge stabilisation and low cost methods of disinfection are needed. To increase the acceptability of the sewage sludge for agricultural utilization it is necessary to increase the knowledge about the presence of all the pollutants (esp. organic micropollutants) in the sludges.

There is a need to examine how the different types of sludges can be utilized in different farming systems to give maximum efficiency with minimal risks of pollution. The assessment of nitrogen losses, the leading of nutrients and its influence on groundwater, the evaluation of more efficient methods of application and of the effects of different sludges on long-term soil biological activity are of high priority.

With regard to the negative effects of agricultural sludge utilization, efforts should be concentrated on the long-term effects of contaminants introduced to the soil, the chemical characterisation of metal availability and changes in relation to soil properties (18).

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LONG-TERM CONTINUOUS APPLICATION OF SEWAGE SLUDGE IN TEST FARMS

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Abstract

The eleven treatment plants in the City of Yokohama currently generate approximately 170,000 tons of sludge in dewatered cakes per year. The increasing urbanization in the Yokohama area has made it more difficult to secure sewage disposal sites. The City of Yokohama has consequently set up intensive sludge treatment facilities at two of the treatment plants in the coastal district. It is the purpose of these facilities to reduce sludge volumes by incineration, while a certain portion of the dewatered cakes are dried for use as fertilizer in parks, golf courses, and other such locations.

The attractiveness of recycling sewage sludge as fertilizer is that it is in conformance with the cycle of nature. It was in view of this and other such considerations that the City of Yokohama embarked on a ten-year program of basic research on this subject beginning in 1976. The research consisted of tests concerning the effects of a long-term continuous usage of sewage sludge as fertilizer.

The test crops were kidney beans, small turnips, cabbage, soybeans (harvested green), garland chrysanthemums, spinach, white potatoes, and Japanese radishes. Test farms were constructed in the two treatment plants. Each test farm contained four plots: a standard plot (fertilized with chemical fertilizer and compost), a chemically fertilized plot (fertilized with chemical fertilizer only), a "sludge plot I" (fertilized with sewage sludge applied at a level of 1 ton per 10 ares and chemical fertilizer), and a "sludge plot II" (fertilized with sewage sludge applied at a level of 2 tons per 10 ares and chemical fertilizer). Each of the plots was planted twice a year (in the spring and fall). Studies were made of the fertilizing effect based on yield, and the influence of heavy metals.

The studies concluded that the fertilizing effect in the sludge plot II was on a par with that in the standard plot, and that there was no particular need for concern about harmful effects of heavy metals arising from the use of sewage sludge as fertilizer at an application level of 1 to 2 tons per 10 ares.

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1. Foreword

As of April 1986, the City of Yokohama was operating eleven sewage treatment plants. The sludge generated at these plants taken together totals approximately 174,000 tons of dewatered cakes annually. This quantity has been increasing, and is expected to continue to increase in the years ahead. Meanwhile, it is becoming increasingly difficult to secure disposal sites, because of the development taking place around the city. In order to hold the volume of sludge disposal to the minimum, virtually the entire quantity is incinerated and then used as landfill, with a small portion (approximately 3,000 tons) dried for use as fertilizer.

Of the sludge generated by sewage treatment in Japan in 1984, some 14 percent was put to effective use, 76 percent was used as fill on land or sea, and 10 percent fell in the category of "other." Ninety-three percent of the sludge in the first category was utilized on farmland, with the remaining 7 percent used for construction material and in other ways. Eighty-five percent of the sludge used on farmland was in the form of dewatered cakes, 9 percent in the form of compost, 2 percent in the form of dried sludge, and 4 percent in some other form.[1)]

Treatment and disposal of sewage sludge must take such factors as secondary pollution control, energy conservation, and resource recycling into account. The recycling of sludge as fertilizer would be particularly welcomed, in that it is in keeping with the cycle of nature.

These and other considerations motivated the City of Yokohama to conduct basic research concerning the adding of sewage sludge to farmland in the years 1971~74. This research examined such matters as the effect of using sewage sludge as fertilizer and its influence on soil structure. This effort was followed in 1976 by a ten-year program of studies concerning the effects of the long-term use of sewage sludge.

This report presents the data and findings from a 6-year period of this long-term test usage.

2. Test Method

2-1 Drier Structure

During the basic experimentation phase, dewatered cakes obtained through a vacuum-type belt filter were dried with a forced-air drier, resulting in a powdered product. This powder poses no great problems for experimental purposes, but in actual use, such sludge powder could easily be scattered or cause other difficulties. After various tests, it was decided to manufacture a pelletized product through the use of a drum-type drier described below. A diagram of the drier facilities and the structure of the drier itself are presented in Figures 1 and 2.

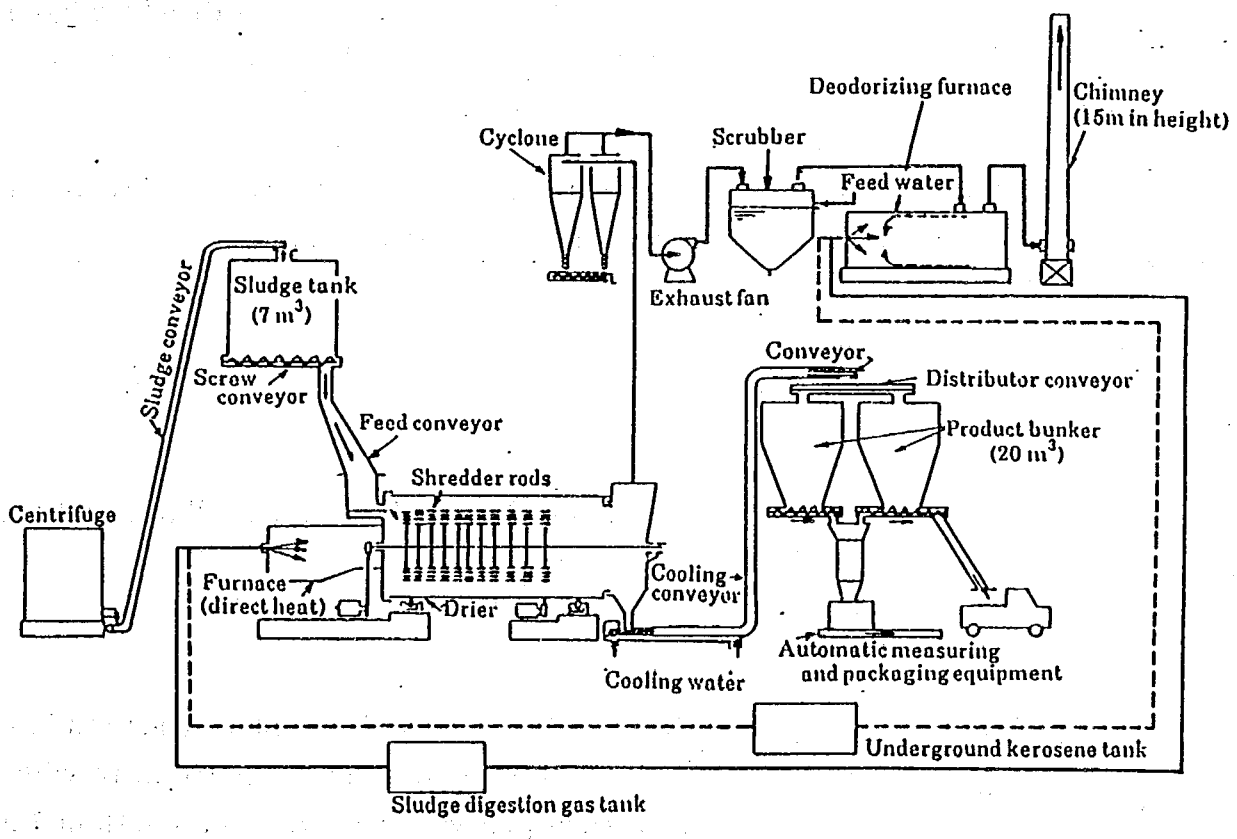


Figure 1. Drier Facility Diagram

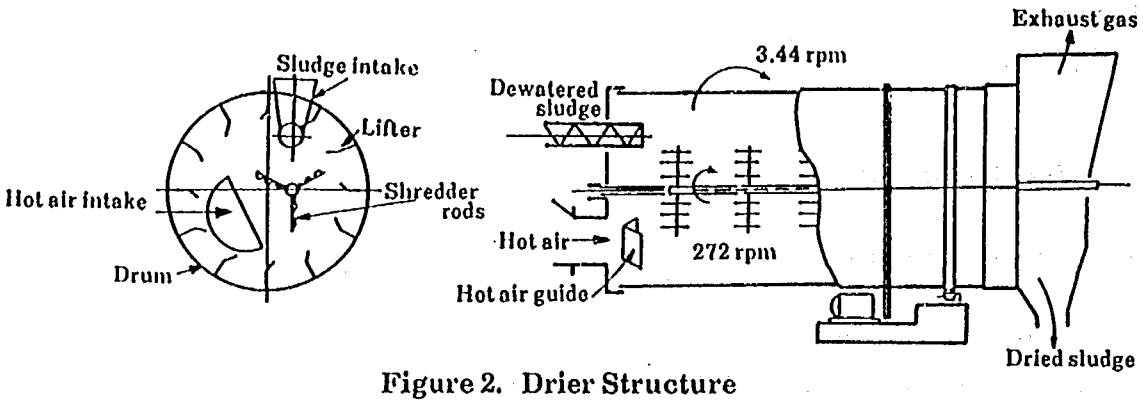


Figure 2. Drier Structure

As the sludge moves axially with the rotation of the drum, it is blown by a parallel current of air heated to approximately 900°C. The drying temperature is regulated to approximately 900°C at the intake and 120°C at the outlet. There are lifters fitted to the inner surface of the drum that turn at 3.44 rpm and lift the sludge upward. As the sludge falls back down, it is shredded by rods spinning at 272 rpm. About 40 minutes of this processing yields dried sludge in the form of pellets approximately 3 mm in diameter, with a moisture content in the range of 10~15 percent.

The drying facilities are also furnished with exhaust gas treatment equipment and automatic weighing and packaging equipment.

2-2 Creation of Test Farms

(1) Location

There are two locations: the Kohoku Sewage Treatment Plant, and the Midori Sewage Treatment Plant.

(2) Structure

Four plots were set up in each of the test farms. Each plot measured 2.80 by 2.30 meters, and was divided by a concrete wall measuring 10 cm in width and 1 m in depth. The surface soil in each plot was removed to a depth of 1 m and replaced with planting soil.

(3) Conditions

Four test plots were set up in each of the test farms: a standard plot (fertilized with compost and chemical fertilizer), a chemically fertilized plot (fertilized with chemical fertilizer only), a "sludge plot I" (fertilized with sewage sludge applied at a level of 1 ton per 10 ares and chemical fertilizer), and a "sludge plot II" (fertilized with sewage sludge applied at a level of 2 tons per ares and chemical fertilizer).

The level of chemical fertilizer application was the same in the standard plots and chemically fertilized plots. The level of chemical fertilizer application in the sludge plots was half that of the others.

The basic conditions applied in fertilizing the test plots are shown in Table 1.

Table 1 Test Plot Conditions

Number	Name	Fertilization
1	Standard plot	Chemical fertilizer and compost (1 t/10 ares)
2	Chemically fertilized plot	Chemical fertilizer
3	Sludge plot	Chemical fertilizer (applied at half the level of the standard plot and chemically fertilized plot) and sewage sludge (1 t/10 ares)
4	Sludge plot	Chemical fertilizer (applied at half the level of the standard plot and chemically fertilized plot) and sewage sludge (2 t/10 ares)

Chemical fertilization was conducted according to the Kanagawa Prefecture standards for each of the crops in question.

(4) Crops

The crops cultivated and the cultivation cycles in each of the test farms are indicated in Table 2.

Table 2 Cultivation Cycle

Year	Season	Kohoku Test Farm	Midori Test Farm
1976	Spring Fall	Soybeans, harvested green Garland chrysanthemums	- -
1977	Spring Fall	Cabbage Japanese radish	- Spinach
1978	Spring Fall	Soybeans, harvested green Garland chrysanthemums	White potatoes Small turnips
1979	Spring Fall	Cabbage Japanese radish	Kidney beans (Spinach)
1980	Spring Fall	Soybeans, harvested green Garland chrysanthemums	White potatoes Small turnips
1981	Spring Fall	Cabbage Japanese radish	Kidney beans Spinach

Note: crops in parenthesis failed due to poor weather

(5) Other

Insect control was carried out at the start of the testing and when insect pests were discovered, under the guidance of the Yokohama Greenery Center. Each of the plots was watered and weeded as necessary.

The composition of the dried sludge used in the tests is indicated in Table 3.

2-3 Testing Parameters

(1) Fertilizing effect

In order to gauge the effectiveness of sewage sludge as fertilizer, measurements were made of the vegetative quantity (e.g., total crop weight, weight of edible portion only, and the weight in terms of product value).

Table 3 Sewage Sludge Composition

	Sewage sludge					
	1976	1977 (Cabbage)	1977 (Japanese radishes, spinach)	1978	1979 1980	1981
TS (%)	93.9	93.6	97.8	95.4	96.2	96.0
pH	7.4	-	6.9	9.8	-	-
EC (μS/cm)	8,340	-	2,360	6,560	-	-
T-N (mg/kg)	27,600	33,200	27,900	24,800	22,300	22,000
P_2O_5 (mg/kg)	41,700	67,500	38,400	22,900	27,300	28,400
K_2O (mg/kg)	5,610	3,900	2,400	1,740	3,250	2,890
MgO (mg/kg)	10,100	12,300	9,320	6,270	10,100	11,000
CaO (mg/kg)	-	32,500	135,000	115,100	146,000	120,000
Cu (mg/kg)	435	409	303	321	262	233
Zn (mg/kg)	2,600	2,560	1,820	1,560	1,470	1,170
Pb (mg/kg)	186	124	78.7	58.8	74.9	61.1
Ni (mg/kg)	102	71.7	67.3	66.0	44.3	30.1
T-Cr (mg/kg)	116	190	151	93.1	42.7	55.4
Cd (mg/kg)	-	-	3.20	2.68	2.26	2.40
T-Hg (mg/kg)	-	-	-	1.22	1.26	1.40
Mn (mg/kg)	-	1,130	870	577	365	389
Fe (mg/kg)	-	33,400	17,800	45,130	28,000	17,900

(2) Influence of heavy metals

The influence of the heavy metal content of sewage sludge was investigated with respect to: i) heavy metal accumulation in the soil and the effect on crop cultivation; ii) heavy metal absorption by crops and the effect on the human body.

(3) Other

To test for soil-improving effects, measurements were made of the pH coefficient of the soil in order to determine the extent of the buffer action.

2-4 Method of Analysis

(1) Sample preparation

To obtain samples for analysis, the crops were first thoroughly washed with water to remove all soil and other matter. The edible portions were then minced, placed in a porcelain dish, and dried at 80°C, reducing the moisture content to around 5 percent. The samples were remeasured to precisely determine their moisture content.

Soil samples were taken at 5 locations to a depth of 15 cm from each plot. These were mixed and crumbled, air dried, and finally put through a 2-mm sieve. The samples were measured to precisely determine their moisture content.

(2) Analytical methods

Each subject of analysis was based on sewage analysis procedure, fertilizer analysis procedure, and soil analysis procedure.

3. Results and Discussions

3-1 Fertilizing Effect

The effectiveness of sewage sludge as fertilizer has been noted in numerous previous studies. In the City of Yokohama as well, this has been verified through various experiments[2),3)] since 1971.

Current testing also verified the fertilizing effects of sewage sludge. In short, the plots fertilized with sludge (hereafter termed "sludge plots") in the Kohoku Test Farm produced roughly the same yield of soybeans (harvested while still immature), garland chrysanthemums (Chrysanthemum coronarium), cabbage, and Japanese radishes as the standard plots. Similarly, there was no substantial difference in the yield of spinach, white potatoes, small turnips, and kidney beans between the sludge plots and standard plot in the Midori Test

Farm. It is interesting to note that in the case of the Midori Test Farm the yield of the chemically fertilized plot was markedly low in the case of kidney beans and highest in the case of spinach.

Since it contains nitrogen, phosphorus, and potassium, sewage sludge is seen to be a promising organic fertilizer. Comparisons were made between the crop yield of the chemical fertilizer plots and those of the sludge plots, in order to determine the performance of sewage sludge as an organic fertilizer. The results are indicated in Figures 3~7.

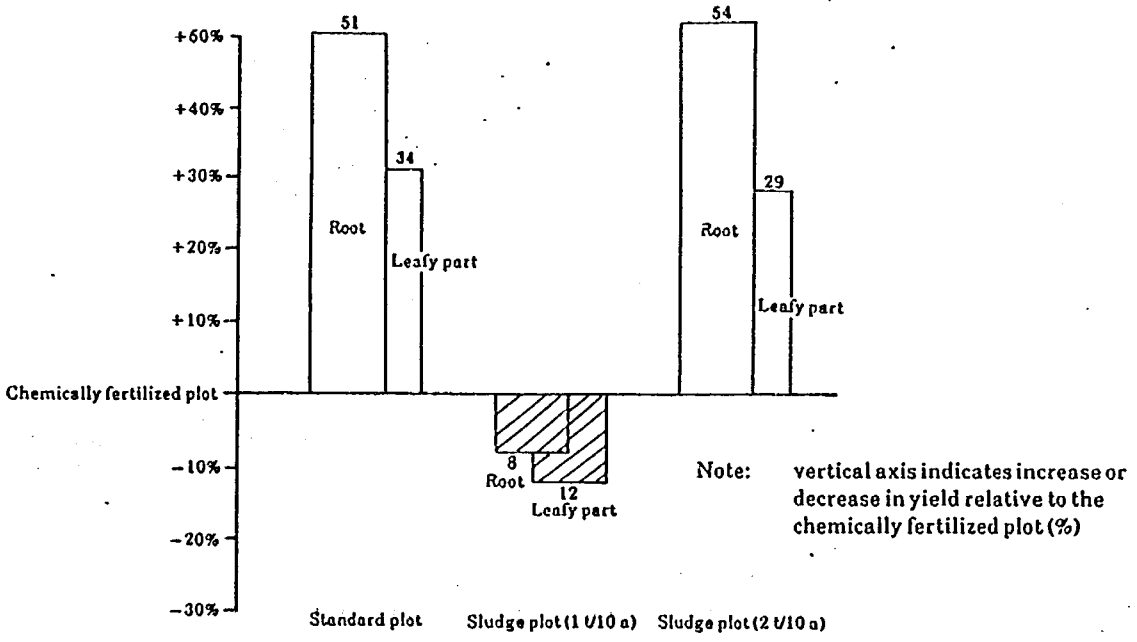


Figure 3. Yield Comparison (Small turnips)

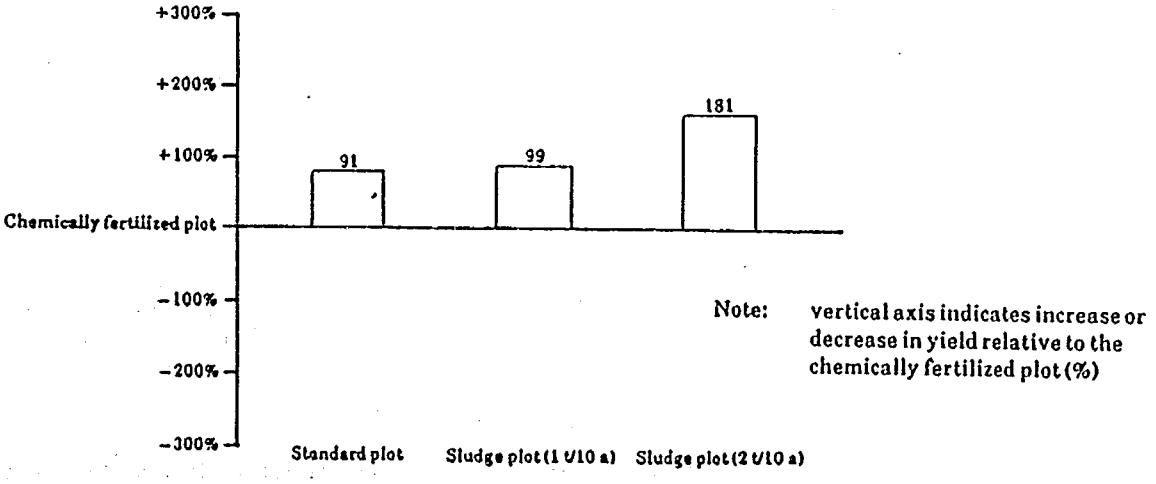


Figure 4. Yield Comparison (Kidney beans)

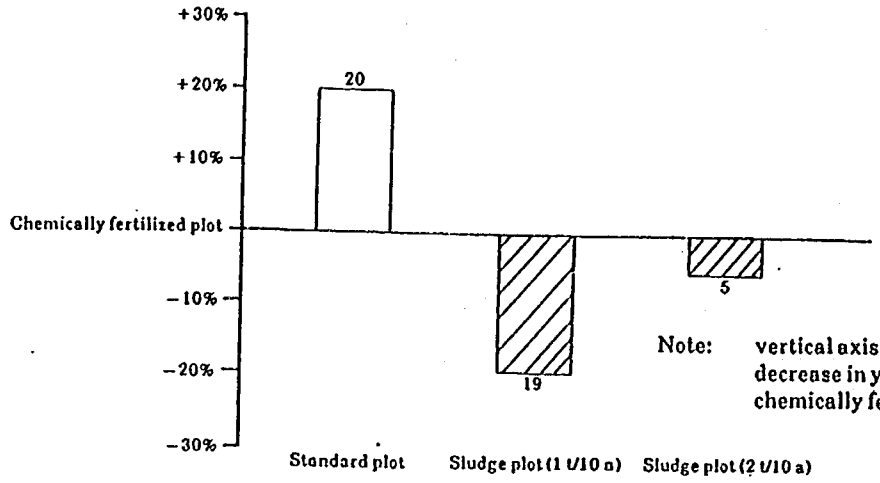


Figure 5. Yield Comparison (Spinach)

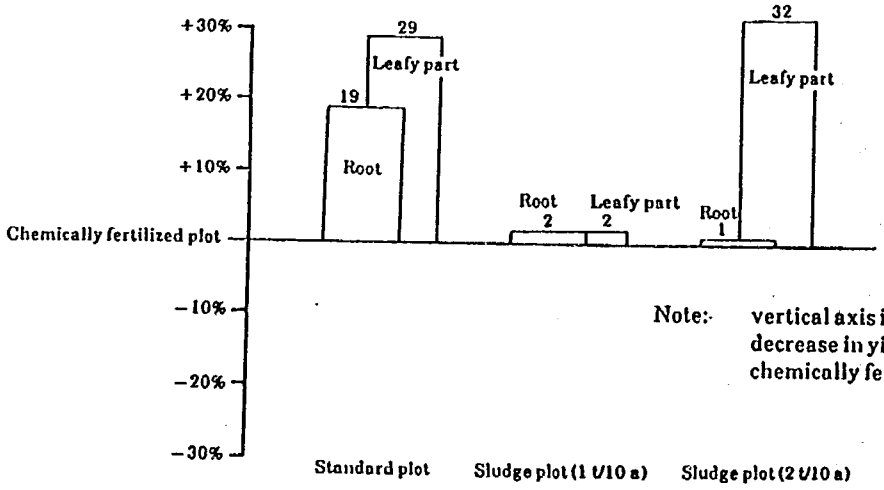


Figure 6. Yield Comparison (Japanese radishes)

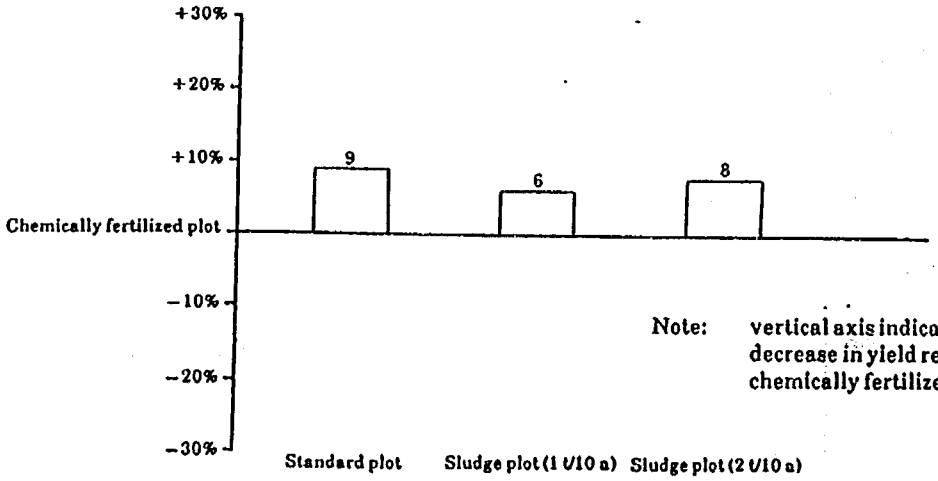


Figure 7. Yield Comparison (Soybeans, harvested green)

The standard plots, which were fertilized with compost, produced a greater yield than the chemically fertilized plots for all crops. This apparently illustrates the superior nutrient and soil-improvement characteristics of organic fertilizer.

The yield of the sludge plots varied with the type of crop. It was greater than that of the chemically fertilized plots in the case of soybeans and kidney beans, whether the level of sludge application was 1 or 2 tons per 10 ares. The yield of kidney beans was particularly great, amounting to some 2 to 3-fold to that of the chemically fertilized plots. In the case of Japanese radishes and cabbage, the yield was roughly the same as that of the chemically fertilized plots at an application level of 1 ton per 10 ares, but at 2 tons per 10 ares, it was greater than that of chemically fertilized plots, and roughly the same as that of the standard plots. In the case of small turnips, the yield was less than that of the chemically fertilized plots at an application level of 1 ton per 10 ares, and about the same as that of the standard plots at an application level of 2 tons per 10 ares. The yield of white potatoes and garland chrysanthemums was virtually the same as that of the chemically fertilized plots, regardless of application level. In the case of spinach alone, the yield of the sludge plots was significantly interior to that of the chemically fertilized plots.

The relationship between the test plots was investigated as illustrated by Figure 8, where the horizontal axis is the increase or decrease in the yield of the standard plots relative to the chemically fertilized plots, and the vertical axis is that of the sludge plots relative to the chemically fertilized plots. The results for each of the crops are indicated in Figures 9~10.

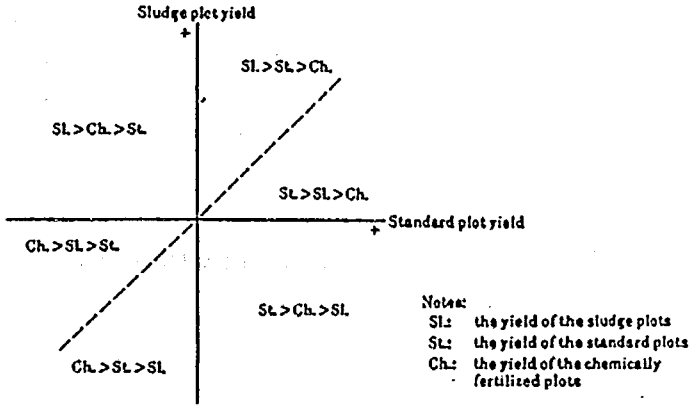


Figure 8. Pattern of Yield by Plot

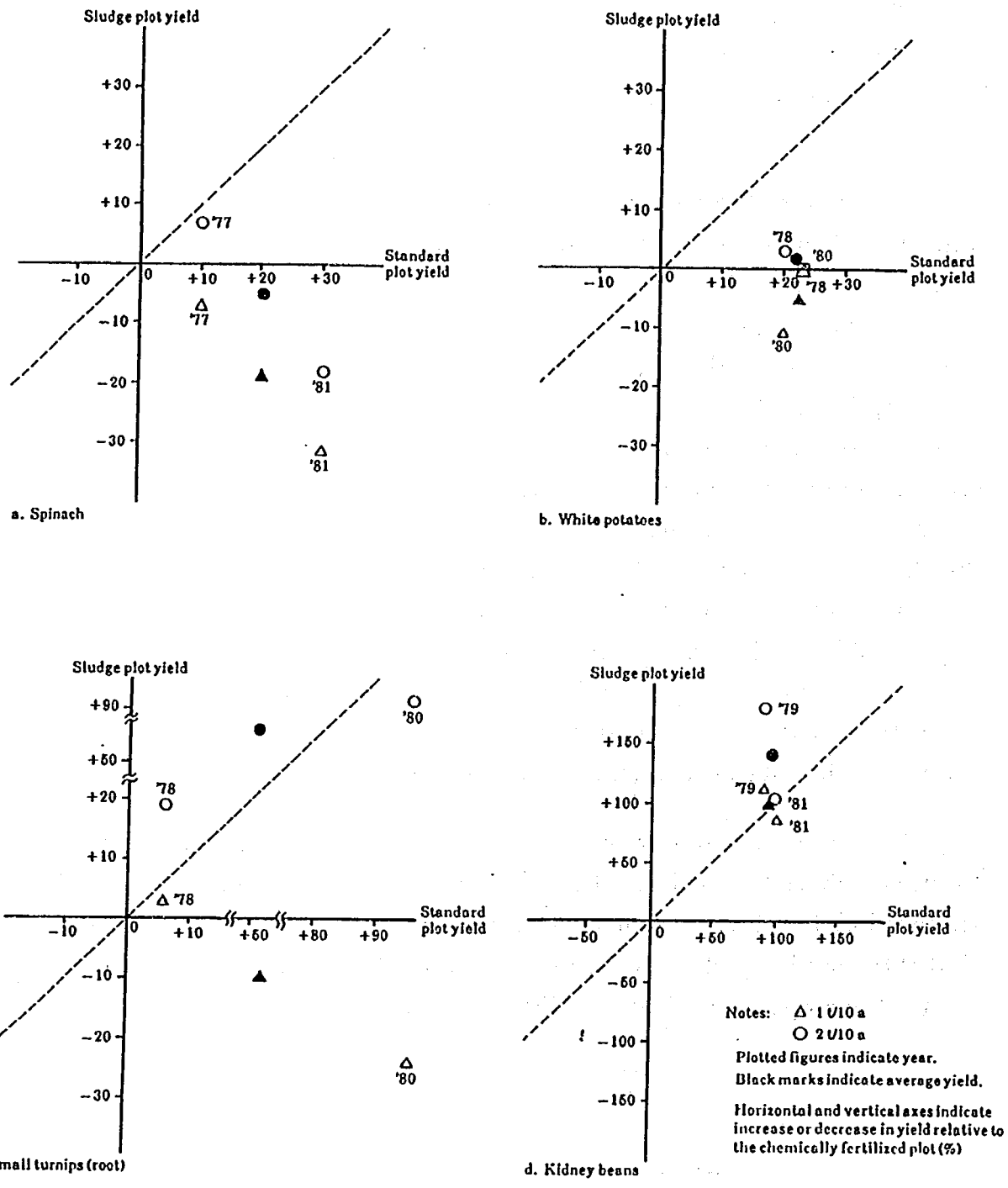


Figure 9. Pattern of Yield by Plot (I)

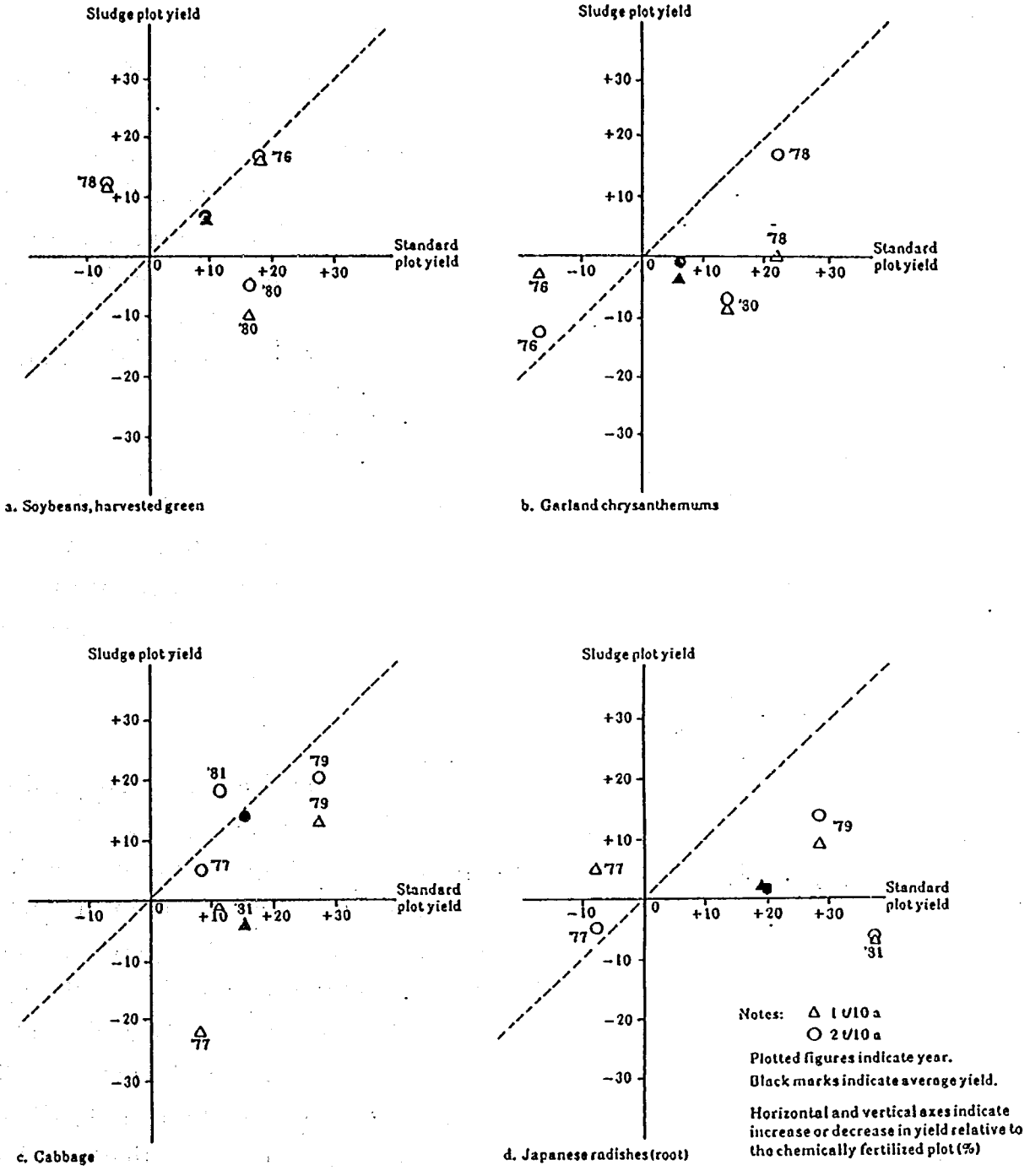


Figure 10. Pattern of Yield by Plot (II)

The following is a summary of the relationship between the test plots based on the dispersion of the points plotted for each crop (excluding those on the mean). Looking at the horizontal axis, only 6 points fell to the left of zero, and even these are close to it. The great majority of points fell to the right, and at a considerable distance from zero. This indicates the obvious superiority of the standard plots over the chemically fertilized plots as regards crop yield. Looking at the vertical axis, there are 15 points below zero, 4 points at zero, and 21 points above zero, and in some cases the points fall in 2 or more sections for the same crop. The relationship here is therefore not a uniform one, but in general, it could be said that the sludge plots produced yields equal to or better than the chemically fertilized plots.

Tables 4 and 5 present the data of the above figures by year, crop, and test plot. The sludge plots produced yields on a par with those of the chemically fertilized plots when the application level was 1 ton per 10 ares. When the concentration was 2 tons per 10 ares, the sludge plots produced yields that were greater than those of the chemically fertilized plots and equal to the standard plots in the case of kidney beans, small turnips, cabbage, and soybeans.

Next the effects of the continuous usage of sludge fertilizer on yield were investigated. Table 6 indicates the quantitative change in yield as of the final year relative to the starting year, for each type of plot and crop. By awarding 5 points for an increase in yield of 10 percent or more, 3 points for an increase or decrease of less than 10 percent, and 1 point for a decrease of 10 percent or more, the point total for the standard plots is 18, those of the chemically fertilized plots and sludge plots with an application level of 1 ton per 10 ares 16 each, and that of the sludge plots with an application level of 2 tons per 10 ares 18. This indicates that the fertilizing effect of the continuous use of sludge fertilizer is equal to that of compost or chemical fertilizers, with no particular adverse effects.

Table 4 Fertilizer Effectiveness (level of sludge application: 1 t/10 a)

	First crop	Second crop	Third crop	Average
(Plot yield) sludge≧standard≧chemical	Japanese radishes, kidney beans	-	Cabbage	Kidney beans
standard≧sludge≧chemical	Soybeans (harvested green), white potatoes	Garland chrysanthemums, Japanese radishes, white potatoes, kidney beans	Japanese radishes	Japanese radishes, soybeans
standard≧chemical≧sludge	Cabbage, small turnips, spinach	Spinach, white potatoes, small turnips	Soybeans, garland chrysanthemums	Cabbage, spinach, white potatoes, small turnips, garland chrysanthemums
chemical≧sludge≧standard	Garland chrysanthemums	Soybeans	-	-

Table 5 Fertilizer Effectiveness (level of sludge application: 2 t/10 a)

	First crop	Second crop	Third crop	Average
(Plot yield) sludge≧standard≧chemical	Cabbage, Japanese radishes, kidney beans	Garland chrysanthemums, kidney beans	-	Kidney beans, small turnips
standard≧sludge≧chemical	Soybeans, small turnips, white potatoes	Cabbage, Japanese radishes, white potatoes, small turnips	Cabbage, Japanese radishes	Japanese radishes, white potatoes, cabbage, soybeans
standard≧chemical≧sludge	Spinach	Spinach	Soybeans, garland chrysanthemums	Spinach, garland chrysanthemums
chemical≧sludge≧standard	Garland chrysanthemums	Soybeans	-	-

Table 6 Yield Trends

	Standard plot	Chemically fertilized plot	Sludge plot (1 t/10 a)	Sludge plot (2 t/10 a)
Soybeans (harvested green)	↑	↑	→	↑
Garland chrysanthemums	↓	↓	↓	→
Cabbage	→	→	↑	→
Japanese radishes	↓	↓	↓	↓
Spinach	↓	↓	↓	↓
White potatoes	→	→	→	→
Small turnips	↓	↓	↓	↓
Kidney beans	→	↓	↓	↓

Notes: ↑ Increase of 10% or over
→ Increase or decrease of less than 10%
↓ decrease of 10% or over

Crop yield is of course strongly influenced by soil characteristics, climate, method of fertilization, and other factors. It should also be noted that the characteristics of sewage sludge used as fertilizer vary by region. These considerations suggest that it would be risky to draw conclusions on the basis of a simple comparison of test data for different years and locations. Still, comparing the results of other studies with two or three of the same crops cultivated in the current test would perhaps provide a useful overview.

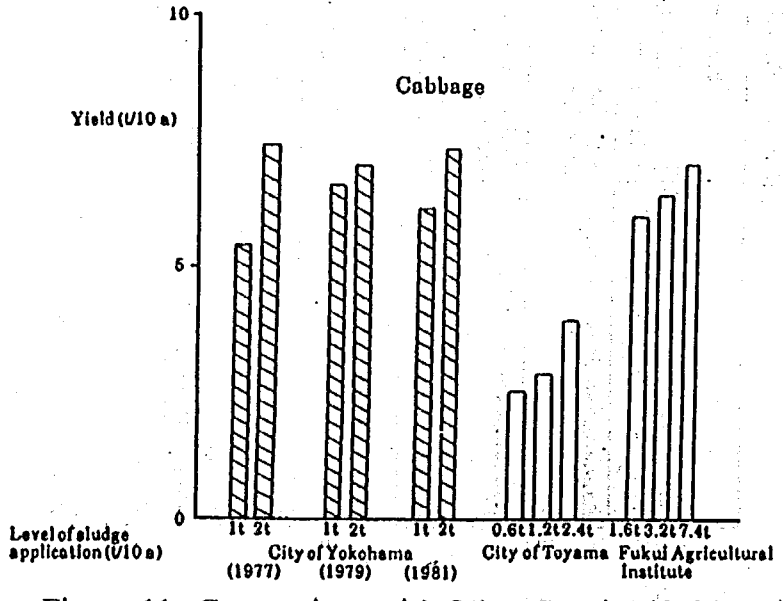


Figure 11. Comparison with Other Studies (Cabbage)

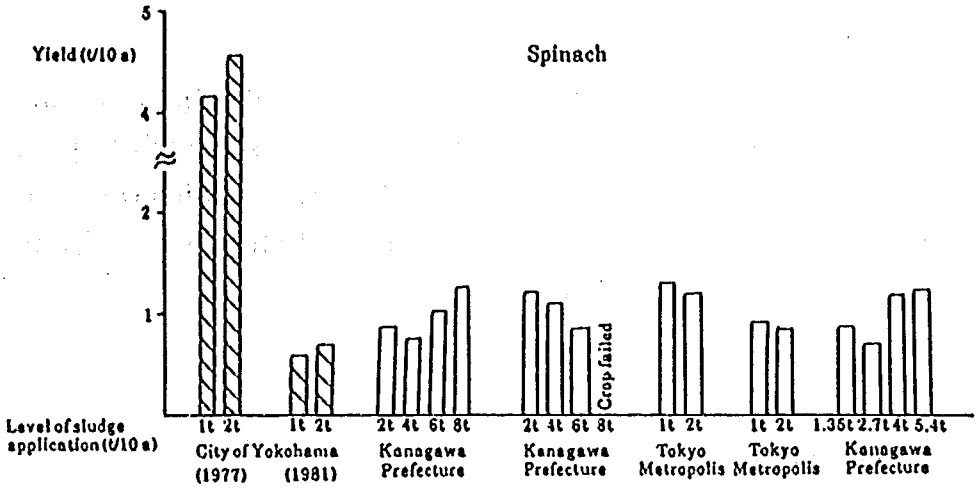


Figure 12. Comparison with Other Studies (Spinach)

a) Cabbage

Studies conducted by the Tokyo Metropolis[4] concluded that as a fertilizer, sewage sludge was as effective as chemical fertilizer, while the studies conducted by the City of Toyama[5] and the Fukui Agricultural Institute[6] concluded that it was more effective than chemical fertilizer. Studies conducted by the City of Kobe[7] concluded that as compared to chemical fertilizer, fertilization by sewage sludge alone was less effective at an application level of 3 tons per 10 ares, but more effective at 10 tons per 10 ares. The yield obtained from the plots fertilized with sludge in the City of Yokohama was approximately twice that of the City of Toyama, and about the same as that of the Fukui Agricultural Institute.

b) Spinach

Although the tests in the City of Yokohama could not verify the effectiveness of sludge as fertilizer for spinach, significant results were obtained in studies conducted by Kanagawa Prefecture[8], the Tokyo Metropolis[4], and the National Institute for Environmental Studies[9]. It should be added that there was a great yearly variation in the case of the City of Yokohama tests.

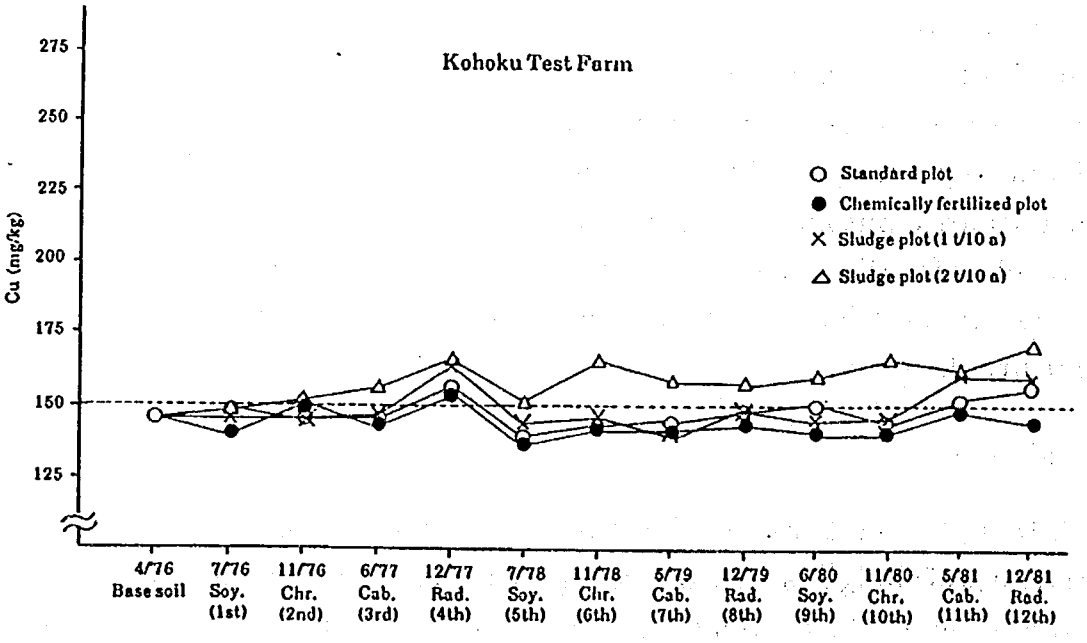
c) Other

In the case of Japanese radishes, the current test obtained the same results as the studies conducted by the Tokyo Metropolis. As for white potatoes, the results appear to be the same as those reached by the studies conducted by the City of Kobe, which imply that the effectiveness of sludge as a fertilizer is in direct proportion with the amount used.

3-2 Influence of Heavy Metals

(1) Concentration in soil

Figures 13~19 indicate examples of the secular change in the concentration of copper, zinc, lead, nickel, chromium, cadmium, and mercury in the soil after harvesting for the Kohoku or Midori test farms. The results may be summarized as follows:



Notes:

Soy.: Soybeans, harvested green Cab.: Cabbage
Chr.: Garland chrysanthemums Rad.: Japanese radishes
Figures in parentheses indicate crop order.

Figure 13. Concentration of Copper in Soil

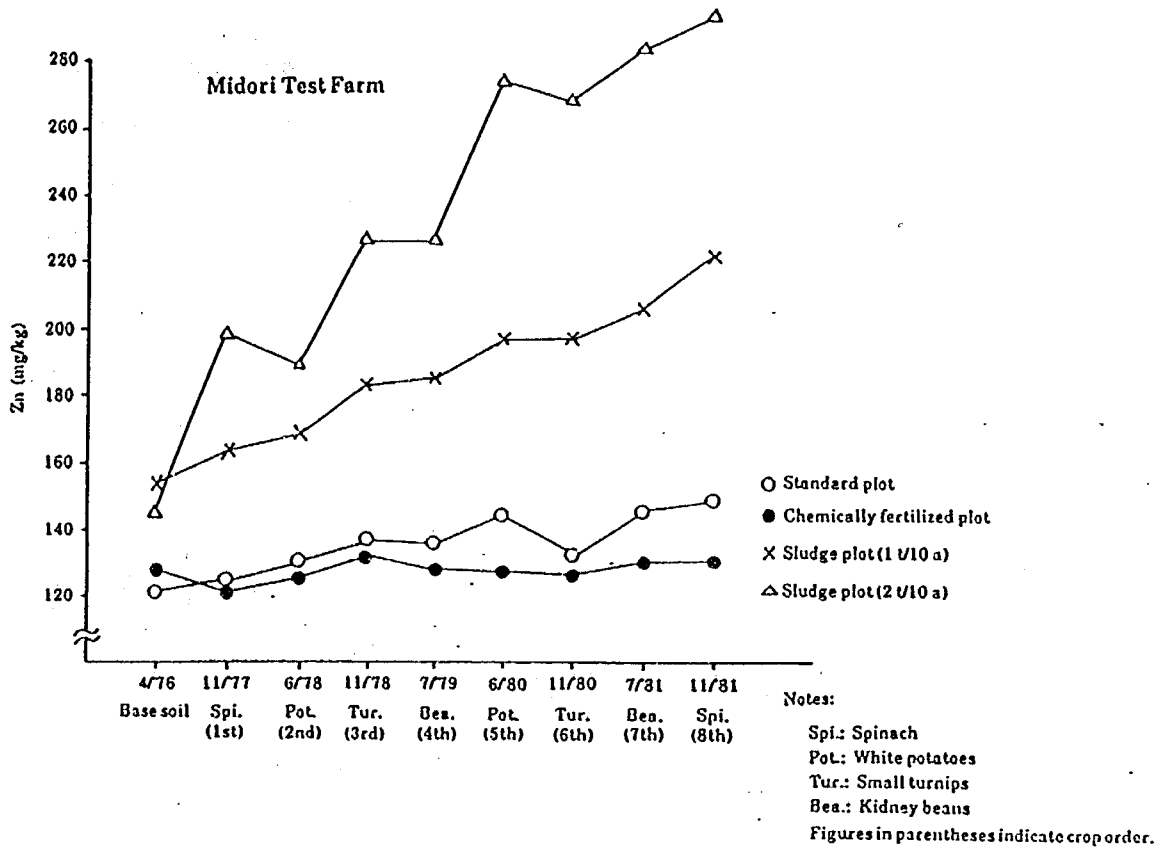


Figure 14. Concentration of Zinc in Soil

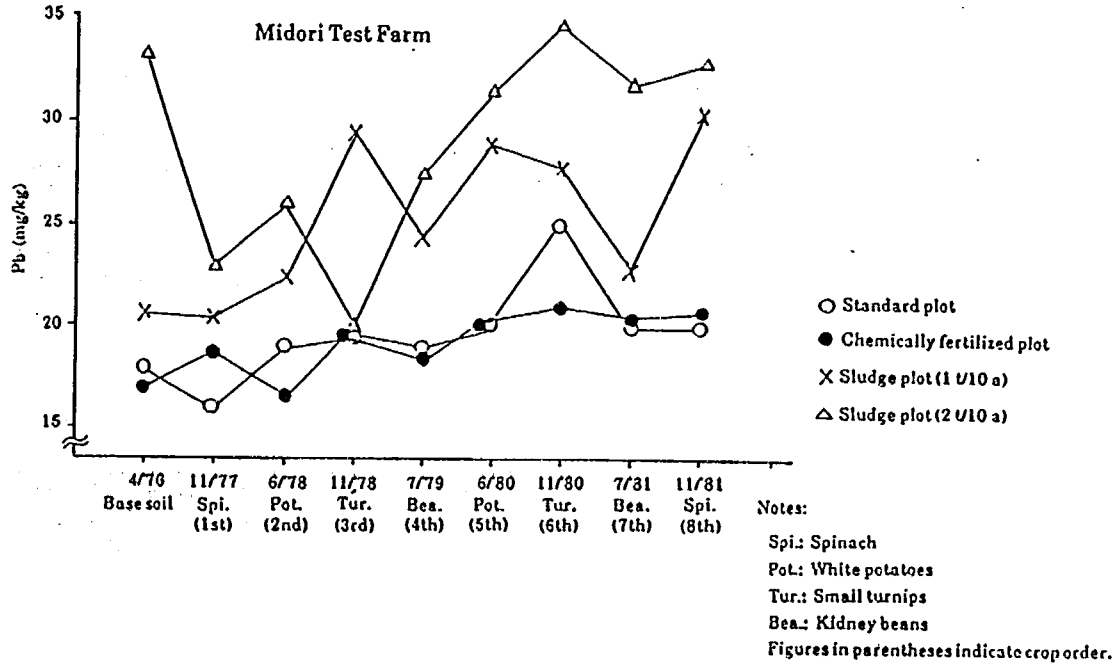
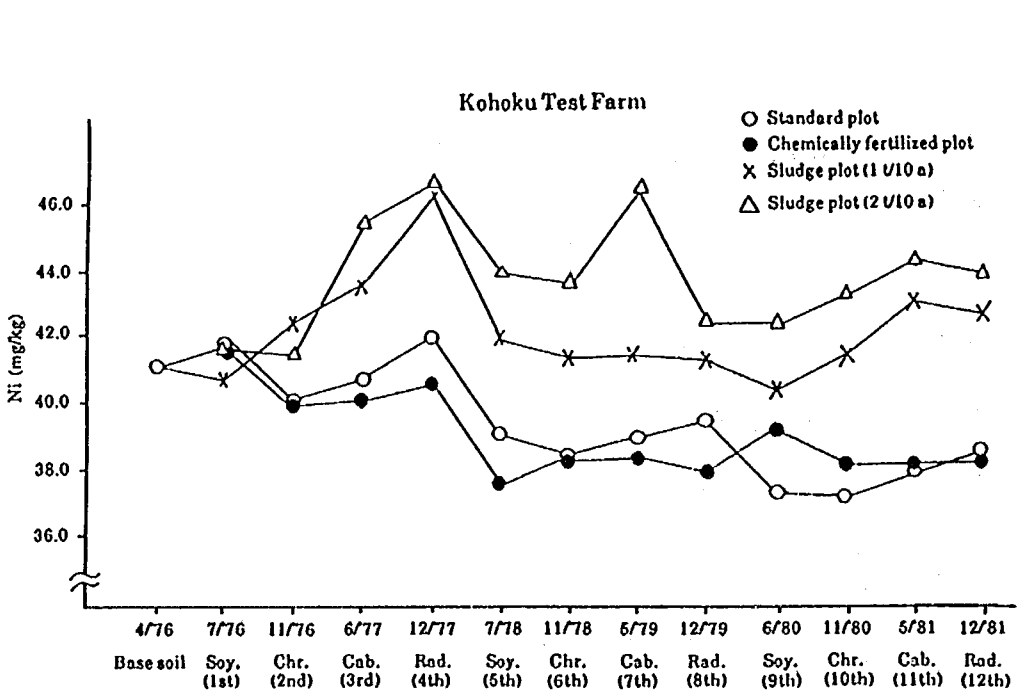


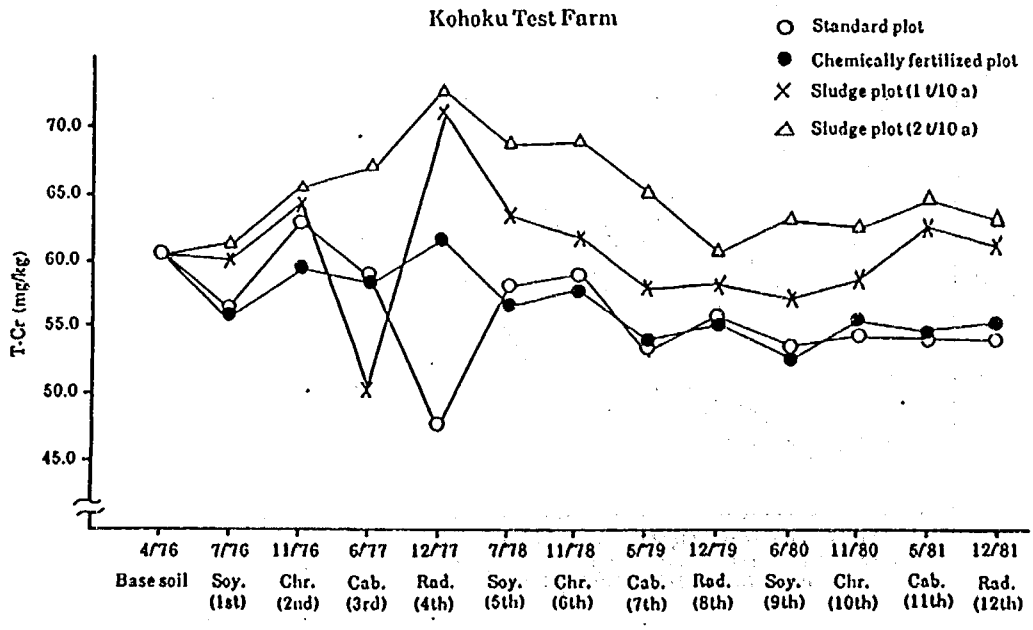
Figure 15. Concentration of Lead in Soil



Notes:

Soy.: Soybeans, harvested green Cab.: Cabbage
Chr.: Garland chrysanthemums Rad.: Japanese radishes
Figures in parentheses indicate crop order.

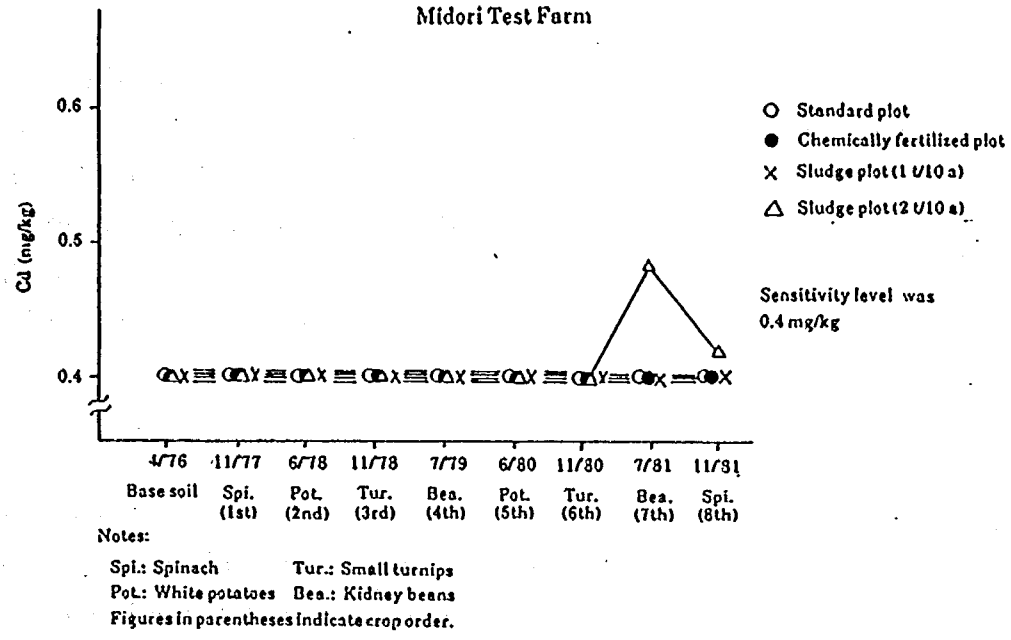
Figure 16. Concentration of Nickel in Soil



Notes:

Soy.: Soybeans, harvested green Cab.: Cabbage
Chr.: Garland chrysanthemums Rad.: Japanese radishes
Figures in parentheses indicate crop order.

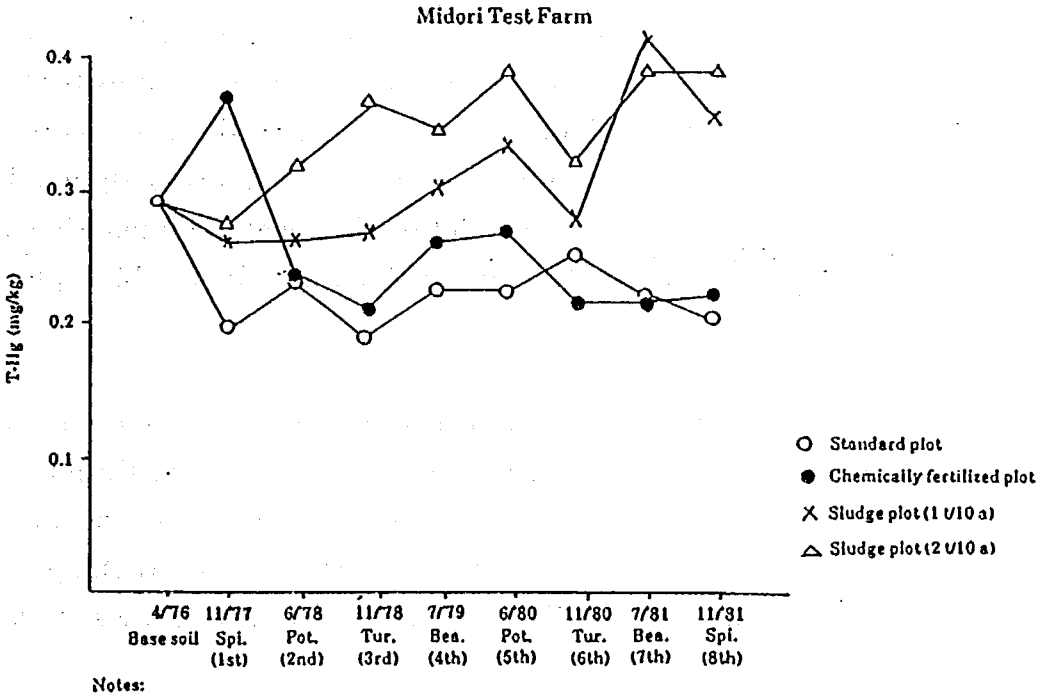
Figure 17. Concentration of Total Chromium in Soil



Notes:

Spi.: Spinach Tur.: Small turnips
Pot.: White potatoes Bea.: Kidney beans
Figures in parentheses indicate crop order.

Figure 18. Concentration of Cadmium in Soil



Notes:

Spi.: Spinach Tur.: Small turnips
Pot.: White potatoes Bea.: Kidney beans
Figures in parentheses indicate crop order.

Figure 19. Concentration of Total Mercury in Soil

Copper:

Up to and including the fifth planting, none of the plots in the Kohoku Test Farm exhibited an increase in concentration, and there was also no difference between the plots in this regard. The concentration in the plot fertilized at 2 tons per 10 ares began to rise around the sixth planting, but exhibited no substantial difference with other plots. The corresponding figures for the Midori Test Farm are not uniform, but show a similar increasing tendency in the plot fertilized at 2 tons per 10 ares.

Zinc:

In both test farms, after the final harvesting, zinc concentration in the soil of the plots fertilized at 2 tons per 10 ares was double the base concentration. Zinc concentration exhibited a rising tendency in the plots fertilized at 1 ton per 10 ares in both test farms as well.

Lead:

In the Kohoku Test Farm, lead concentration was highest in the plot fertilized at 2 tons per 10 ares, while concentration in the plot fertilized at 1 ton per 10 ares was equal to that of the standard plot and the chemically fertilized plot. In the Midori Test Farm as well, lead concentration was highest in the plot fertilized at 2 tons per 10 ares, and the plot fertilized at 1 ton per 10 ares showed a higher concentration than the standard and chemically fertilized plots.

Nickel:

In the Kohoku Test Farm, nickel concentration in the plots fertilized at 1 ton and 2 tons per 10 ares, respectively, are somewhat higher than in the case of the other plots, but there is only a slight rise from the base soil concentration. In the Midori Test Farm, a rising concentration is shown only by the plot fertilized at 2 tons per 10 ares; the plot fertilized at 1 ton per 10 ares shows the same trend as the standard and chemically fertilized plots.

Chromium:

In both test farms, the concentration is highest in the case of the plots fertilized at 2 tons per 10 ares, which exhibit an increase of approximately 10 percent over the base soil concentration.

Cadmium:

Virtually all of the figures fell below the sensitivity level, and there is no definite trend.

Mercury:

In the Kohoku Test Farm, the concentration in the plot fertilized at 2 tons per 10 ares began to rise somewhat relative to the other plots subsequent to the third harvesting, but followed a fairly constant trend overall subsequent to the fourth harvesting. The concentration in the plot fertilized at 1 ton per 10 ares exhibited the same trend as the standard and chemically fertilized plots. In the Midori Test Farm, the concentration in the sludge plots was somewhat higher than the other plots.

The above indicates that the concentration of heavy metals in the soil rises with the amount of sludge used as fertilizer. The important factor in this connection, however, is the soluble concentration of metals in the soil, which is apparently related to absorption by crops. In the case of zinc, less than 20 percent was in soluble form. The corresponding figure for copper and nickel was less than 10 percent, and zero percent in the case of chromium, lead, and mercury. In the case of cadmium, this percentage fluctuated greatly, and there was no definite trend.

(2) Concentration in crops

Of the above 7 metals, copper and zinc were detected in all crops in every test. Out of the total of 20 crops, mercury was detected in 10, lead in 6, and nickel in 4, while chromium and cadium were detected in only 2. The heavy metal concentration in the crops themselves varied with the year and the type of

crop, and did not exhibit a clearcut trend. However, the rough tendency is indicated in Table 7.

Table 7 Crops with High Heavy Metal Content

Heavy Metal	Crop
Cu	Spinach, garland chrysanthemums, soybeans (harvested green)
Zn	Spinach, garland chrysanthemums, soybeans
Pb	Japanese radishes (leafy part)
Ni	Garland chrysanthemums, kidney beans, soybeans
T-Cr	Spinach
Cd	Spinach
T-Hg	Small turnip (leafy part)

No fixed relationship was found between the heavy metal concentrations in the crops and the test plots based on the cases studied. In the Kohoku Test Farm, for example, the concentration of copper in the first crop, namely, soybeans, was 18.9 mg/kg for the standard plot, and 17.7, 17.9, and 16.5 mg/kg for the chemically fertilized plot and the sludge plots fertilized at 1 and 2 tons per 10 ares, respectively. The copper concentration in the sixth crop (garland chrysanthemums) at the same test farm was 13.7 mg/kg for the standard plot, and 14.4, 13.4, and 14.5 mg/kg for the other three respective plots. In the case of mercury, the concentration in the second crop (garland chrysanthemums) at the Kohoku Test Farm was 0.026 mg/kg for the standard plot, and 0.030, 0.034, and 0.029 mg/kg for the chemically fertilized plot and the sludge plots fertilized at 1 and 2 tons per 10 ares, respectively. The corresponding figures for the sixth crop (garland chrysanthemums) at the same test farm were 0.046, 0.054, 0.047, and 0.047 mg/kg.

A statistical study found no definite correlation between the heavy metal concentration in the soil and that in the crops.

Table 8 presents data for the concentration in crops.

Table 8 Crop Composition and Heavy Metal Content

(1) Kohoku Test Farm ①

Crop	Plot	Item													
		T-N (mg/kg)	T-P (mg/kg)	K (mg/kg)	Mg (mg/kg)	Ca (mg/kg)	Cu (mg/kg)	Zn (mg/kg)	Pb (mg/kg)	Ni (mg/kg)	T-Cr (mg/kg)	Cd (mg/kg)	T-Hg (mg/kg)	Fe (mg/kg)	Mn (mg/kg)
Soybeans, harvested green First crop spring '76	1	70,300	6,620	21,000	2,100	2,760	18.9	87.6	nd	2.63	nd	nd	nd	99.2	29.1
	2	70,300	6,200	21,000	2,060	2,660	17.7	74.7	5.12	3.18	"	"	"	92.5	27.5
	3	67,100	6,810	21,000	2,020	2,690	17.9	69.3	4.46	2.26	"	"	"	100	29.5
	4	69,700	6,560	21,000	2,020	2,480	16.5	72.2	nd	3.20	"	"	"	81.1	28.1
Garland chrysanthemums Second crop fall '76	1	54,000	6,030	80,000	2,520	12,400	14.0	112	5.45	2.48	nd	nd	0.026	294	40.6
	2	54,000	6,030	84,000	2,630	13,800	13.5	131	5.06	1.64	"	0.47	0.030	309	60.5
	3	54,400	5,740	86,000	2,750	13,300	15.7	130	nd	1.64	"	nd	0.034	298	47.6
	4	54,100	5,970	78,000	2,590	13,000	13.9	149	5.28	1.54	"	"	0.029	276	40.7
Cabbage Third crop spring '77	1	31,300	5,650	37,100	1,500	3,310	2.98	29.9	nd	nd	nd	nd	nd	44.2	14.2
	2	30,600	5,610	36,000	1,620	4,620	3.12	30.0	"	"	"	"	"	49.3	17.3
	3	29,200	5,000	38,500	1,550	3,770	2.99	28.4	"	"	"	"	"	43.7	14.7
	4	33,400	5,680	42,300	1,730	5,160	3.39	39.6	"	"	"	"	"	53.4	16.8
Japanese radishes (root) Fourth crop fall '77	1	31,100	6,060	68,600	1,680	4,740	3.23	52.9	15.3	nd	nd	nd	nd	43.4	15.0
	2	30,200	5,210	69,200	1,300	3,720	3.39	45.7	11.1	"	"	"	"	36.1	14.6
	3	25,600	5,120	62,400	1,210	3,680	2.92	37.4	9.55	"	"	"	"	37.1	9.86
	4	32,600	5,870	61,500	1,380	4,770	3.09	43.2	5.96	"	"	"	"	43.9	9.33
Japanese radishes (leafy part) Fourth crop fall '77	1	57,100	9,990	87,600	1,970	14,200	6.84	76.0	10.4	nd	nd	nd	0.027	214	44.8
	2	46,700	7,580	68,300	1,780	14,600	5.67	56.9	17.5	"	"	"	0.036	286	56.3
	3	48,200	7,700	51,300	1,750	14,400	5.17	48.6	14.2	"	"	"	nd	199	37.5
	4	49,900	7,690	72,600	1,870	23,500	5.16	64.4	5.80	"	1.87	"	0.029	430	56.9

Note: In the category 'plot,' 1 = standard plot, 2 = chemically fertilized plot, 3 = sludge plot (1 t/10 a), and 4 = sludge plot (2 t/10 a)

(1) Kohoku Test Farm ②

Crop	Plot	Item													
		T-N (mg/kg)	T-P (mg/kg)	K (mg/kg)	Mg (mg/kg)	Ca (mg/kg)	Cu (mg/kg)	Zn (mg/kg)	Pb (mg/kg)	Ni (mg/kg)	T-Cr (mg/kg)	Cd (mg/kg)	T-Hg (mg/kg)	Fe (mg/kg)	Mn (mg/kg)
Soybeans, harvested green Fifth crop spring '78	1	72,200	7,510	20,700	1,900	2,400	17.4	72.3	nd	1.71	nd	nd	nd	91.5	26.3
	2	74,000	8,130	19,900	1,990	2,650	16.4	64.5	"	2.43	"	"	"	86.3	27.8
	3	72,800	7,700	20,500	1,920	2,380	16.4	66.5	"	1.65	"	"	"	89.0	22.8
	4	77,900	7,430	34,300	2,720	3,380	24.1	90.0	"	2.86	"	"	"	128	32.8
Garland chrysanthemums Sixth crop fall '78	1	38,800	6,500	100,200	2,780	17,900	13.7	69.5	5.36	nd	nd	0.45	0.046	160	41.9
	2	39,400	5,480	90,000	2,830	18,200	14.4	62.8	nd	1.66	"	nd	0.054	251	50.9
	3	43,900	6,070	96,700	2,600	16,000	13.4	69.9	"	1.64	"	"	0.047	123	34.3
	4	44,200	6,190	81,600	2,600	18,800	14.5	65.3	"	nd	"	"	0.047	169	32.9
Cabbage Seventh crop spring '79	1	34,800	7,350	43,600	1,900	10,700	4.05	40.8	nd	nd	nd	nd	nd	64.1	20.3
	2	28,100	5,870	35,500	1,440	6,360	3.42	34.6	"	"	"	"	"	51.3	18.1
	3	32,300	6,150	40,400	1,610	9,370	4.00	38.0	5.43	"	"	"	"	56.1	17.1
	4	40,200	6,700	40,300	1,700	10,600	4.66	49.7	4.61	"	"	"	"	64.2	20.8
Japanese radishes (root) Eighth crop fall '79	1	16,200	4,470	45,800	1,140	5,420	2.96	41.6	nd	nd	nd	nd	0.038	28.6	11.3
	2	21,000	4,650	48,400	1,050	5,250	2.09	43.2	"	"	"	"	0.024	31.0	7.40
	3	19,400	3,520	46,000	1,010	6,360	1.71	33.7	"	"	"	"	0.028	33.6	5.81
	4	19,600	3,800	39,200	964	6,190	2.08	39.6	"	"	"	"	0.027	29.3	7.02
Japanese radishes (leafy part) Eighth crop fall '79	1	45,100	8,640	65,100	2,100	20,400	5.20	46.4	7.28	1.78	nd	nd	0.065	169	31.1
	2	38,000	4,600	62,600	2,300	24,900	3.75	47.9	7.32	1.79	"	"	0.052	144	31.1
	3	37,900	4,420	61,200	1,870	29,600	3.69	43.7	7.20	nd	"	"	0.064	187	34.1
	4	36,300	5,460	51,200	1,540	31,100	3.93	45.1	8.03	"	"	"	0.059	173	25.6

Note: In the category 'plot,' 1 = standard plot, 2 = chemically fertilized plot, 3 = sludge plot (1 t/10 a), and 4 = sludge plot (2 t/10 a)

(1) Kohoku Test Farm ③

Crop	Plot	Item													
		T-N (mg/kg)	T-P (mg/kg)	K (mg/kg)	Mg (mg/kg)	Ca (mg/kg)	Cu (mg/kg)	Zn (mg/kg)	Pb (mg/kg)	Ni (mg/kg)	T-Cr (mg/kg)	Cd (mg/kg)	T-Hg (mg/kg)	Fe (mg/kg)	Mn (mg/kg)
Soybeans, harvested green Ninth crop spring '80	1	69,300	7,670	23,500	2,060	2,470	17.1	66.4	nd	nd	nd	nd	nd	95.7	23.9
	2	66,700	6,640	22,300	1,970	2,530	16.3	62.6	"	"	"	"	"	85.5	27.1
	3	67,200	7,680	22,200	1,850	2,500	18.7	59.7	"	"	"	"	"	82.7	23.4
	4	68,800	7,650	22,200	2,100	2,460	19.3	62.9	"	"	"	"	"	77.2	23.9
Garland chrysanthemums Tenth crop fall '80	1	48,100	6,480	76,500	2,520	14,300	14.3	79.3	nd	nd	nd	nd	0.044	226	52.9
	2	47,400	6,710	78,700	2,780	14,900	14.3	72.1	"	"	"	0.56	0.070	283	71.1
	3	49,800	6,190	73,900	2,830	15,300	16.6	67.1	"	"	"	0.42	0.048	223	58.7
	4	50,700	6,850	66,900	2,680	15,500	16.8	61.4	"	"	"	nd	0.049	233	54.8
Cabbage Eleventh crop spring '81	1	33,700	6,310	39,500	1,780	7,120	2.16	50.5	nd	nd	nd	nd	nd	55.7	19.1
	2	29,100	5,200	32,900	1,540	6,590	2.30	38.9	"	"	"	"	"	42.7	17.3
	3	31,000	6,630	35,100	1,430	6,950	2.30	38.9	"	"	"	"	"	47.5	15.8
	4	36,500	6,750	33,800	1,430	8,800	2.61	45.1	"	"	"	"	"	56.9	18.3
Japanese radishes (root) Twelfth crop fall '81	1	24,300	6,660	49,200	1,400	6,990	2.65	42.3	nd	nd	nd	nd	nd	25.7	7.46
	2	24,300	6,020	36,800	1,310	6,760	2.46	41.3	"	"	"	"	"	28.9	7.26
	3	23,400	5,330	40,500	1,080	7,160	3.02	48.0	"	"	"	"	"	29.2	7.21
	4	23,600	6,460	38,400	1,190	7,620	2.95	41.5	"	"	"	"	"	27.2	6.29
Japanese radishes (leafy part) Twelfth crop fall '81	1	44,300	7,640	41,200	2,060	27,400	3.33	86.8	nd	nd	nd	nd	0.037	132	36.4
	2	45,200	6,280	33,600	1,400	29,600	3.84	43.8	"	"	"	"	0.034	154	39.2
	3	45,500	7,700	44,400	1,750	36,400	4.05	39.4	"	"	"	"	0.041	154	27.4
	4	44,600	7,540	31,000	1,200	32,400	3.84	46.7	"	"	"	"	0.038	126	26.9

Note: In the category 'plot,' 1 = standard plot, 2 = chemically fertilized plot, 3 = sludge plot (1 t/10 a), and 4 = sludge plot (2 t/10 a)

(2) Midori Test Farm ①

Crop	Plot	Item													
		T-N (mg/kg)	T-P (mg/kg)	K (mg/kg)	Mg (mg/kg)	Ca (mg/kg)	Cu (mg/kg)	Zn (mg/kg)	Pb (mg/kg)	Ni (mg/kg)	T-Cr (mg/kg)	Cd (mg/kg)	T-Hg (mg/kg)	Fe (mg/kg)	Mn (mg/kg)
Spinach First crop fall '77	1	55,500	7,320	122,000	13,700	12,800	17.3	170	9.66	2.42	2.22	0.93	0.073	239	107
	2	55,600	5,980	102,000	14,100	12,900	16.4	171	8.14	1.99	1.94	1.09	0.079	267	82.8
	3	54,700	6,080	107,000	13,400	13,700	17.7	229	7.54	nd	1.92	0.84	0.078	224	53.6
	4	56,800	6,850	100,000	12,400	12,900	18.9	190	6.28	"	1.92	0.68	0.064	250	35.5
White potatoes Second crop spring '78	1	11,800	2,470	23,600	942	432	7.74	16.8	nd	nd	nd	nd	nd	23.4	4.57
	2	13,500	1,860	23,300	959	517	6.23	16.3	"	"	"	"	"	30.4	3.90
	3	12,400	2,600	22,500	824	710	8.48	21.1	"	"	"	"	"	43.1	3.90
	4	13,700	2,300	18,500	763	627	6.19	15.2	"	"	"	"	"	33.9	3.32
Small turnips (root) Third crop fall '78	1	29,500	8,110	63,200	2,920	7,400	7.16	38.0	nd	nd	nd	nd	nd	42.0	14.4
	2	29,400	6,930	58,600	2,920	8,480	5.49	34.4	"	"	"	"	0.023	43.6	14.4
	3	33,400	6,820	47,100	2,260	6,880	4.70	37.2	"	"	"	"	nd	46.5	10.9
	4	32,400	7,600	53,600	2,620	9,350	6.67	49.9	"	"	"	"	nd	52.2	11.6
Small turnips (leafy part) Third crop fall '78	1	42,900	8,390	69,700	3,670	39,600	7.11	41.6	4.60	nd	nd	nd	0.265	127	51.1
	2	41,800	6,760	72,000	3,670	50,400	5.41	40.4	5.19	"	"	"	0.273	157	49.8
	3	40,200	7,050	66,700	3,310	49,200	6.28	51.6	6.61	"	"	"	0.234	131	36.3
	4	41,900	7,990	58,600	3,020	49,000	6.28	54.6	6.42	"	"	"	0.243	148	29.6
Kindney beans Fourth crop spring '79	1	30,600	4,760	30,100	2,620	5,430	7.25	46.6	nd	1.69	nd	nd	nd	67.0	14.9
	2	28,300	4,430	27,600	2,650	5,660	7.54	31.6	"	nd	"	"	"	62.2	15.2
	3	30,000	3,990	26,700	2,530	4,890	7.49	41.3	"	1.65	"	"	"	62.7	12.7
	4	29,400	4,670	25,500	2,150	4,710	7.92	34.3	"	nd	"	"	"	56.1	12.3

Note: In the category 'plot,' 1 = standard plot, 2 = chemically fertilized plot, 3 = sludge plot (1 t/10a), and 4 = sludge plot (2 t/10a)

(2) Midori Test Farm ②

Crop	Plot	Item													
		T-N (mg/kg)	T-P (mg/kg)	K (mg/kg)	Mg (mg/kg)	Ca (mg/kg)	Cu (mg/kg)	Zn (mg/kg)	Pb (mg/kg)	Ni (mg/kg)	T-Cr (mg/kg)	Cd (mg/kg)	T-Hg (mg/kg)	Fe (mg/kg)	Mn (mg/kg)
White potatoes Fifth crop spring '80	1	13,400	3,150	28,500	826	402	6.19	15.8	nd	nd	nd	nd	nd	27.8	4.19
	2	13,700	2,360	25,500	939	362	6.35	15.2	"	"	"	"	"	21.6	3.54
	3	15,700	3,360	24,300	849	601	8.27	19.9	"	"	"	"	"	22.3	4.58
	4	11,700	3,290	21,100	682	557	8.52	18.3	"	"	"	"	"	26.9	4.26
Small turnips (root) Sixth crop fall '80	1	32,800	7,070	63,600	3,050	7,440	5.09	38.6	nd	nd	nd	nd	0.028	44.8	12.3
	2	37,900	7,160	53,900	3,220	7,440	4.48	34.6	"	"	"	"	nd	46.7	13.4
	3	32,100	6,200	43,100	2,660	7,630	4.55	37.6	"	"	"	"	"	40.9	9.53
	4	32,000	6,640	43,500	2,070	7,150	4.58	37.7	"	"	"	"	"	41.6	9.06
Small turnips (leafy part) Sixth crop fall '80	1	51,300	7,680	64,200	3,980	36,400	6.77	47.6	nd	nd	nd	nd	0.299	232	43.3
	2	50,900	6,330	59,100	3,710	39,100	6.45	43.2	"	"	"	"	0.253	282	58.3
	3	54,100	5,180	43,200	2,580	40,700	5.73	43.7	"	"	"	"	0.252	271	28.9
	4	53,300	6,670	39,900	2,930	45,700	6.98	50.5	"	"	"	"	0.228	263	29.0
Kindney beans Seventh crop spring '81	1	34,400	6,040	32,800	2,580	6,440	6.96	39.9	nd	1.54	nd	nd	nd	113	24.5
	2	30,900	4,760	24,300	2,370	5,820	6.02	34.6	"	nd	"	"	"	87.6	23.6
	3	38,000	6,040	32,300	2,550	7,100	9.34	42.5	"	1.91	"	"	"	128	19.8
	4	37,900	7,080	29,600	2,570	5,880	9.89	53.1	"	2.10	"	"	"	124	18.3
Spinach Eighth crop fall '81	1	50,900	1,120	110,000	10,600	14,500	11.3	108	nd	nd	nd	0.53	0.0534	203	33.6
	2	49,200	6,510	98,600	11,600	17,800	10.6	114	"	"	"	0.54	0.0513	199	51.9
	3	49,600	7,140	68,800	8,830	23,600	11.1	111	"	"	"	nd	0.0438	174	19.9
	4	53,300	8,640	67,000	9,660	25,900	13.4	114	"	"	"	0.48	0.0511	205	14.9

Note: In the category 'plot,' 1 = standard plot, 2 = chemically fertilized plot, 3 = sludge plot (1 t/10a), and 4 = sludge plot (2 t/10a)

(3) Effects on the human body

Heavy metals are also found naturally in the human body, and indeed such metals as copper and zinc are essential elements. It is estimated, for example, that the body of a healthy adult contains 80~120 mg of copper and 1.4~2.3 g of zinc[10]. A deficiency of these metals can cause various physiological disorders. Generally speaking, an ordinary diet provides a sufficient amount of these metals. An excessive intake, however, could also cause significant disorders. Some well-known examples are the "itai-itai" disease linked to excessive cadmium intake, the Minamata disease arising from mercury poisoning, as well as lead poisoning, and various disorders arising from hexavalent chromium.

Bearing in mind the fact that the types and quantity of food consumed vary with individual lifestyles and preferences, the basic guideline for the daily diet of a Japanese adult is presented in Table 9[11]. This information can be used to examine the intake of heavy metals in a diet of the crops cultivated in the sludge plots at the Kohoku and Midori Test Farms.

Table 9 Suggested Daily Diet

(Prepared by Aya Kagawa)

Group	Four food groups and suggested intake (in grams)	Examples	Nutritional value
Group 1	Milk and milk products 250 Egg 50 } 300	Milk: 1 and 1/3 glasses Cheese: 1 slice Eggs: one	Completes a balanced diet: high-quality protein fats calcium vitamin B2
Group 2	Meat, fish 100 Beans and soybean products 80 } 180	Meat: 50g Fish: 50g Fermented soybeans: 5g 'Tofu' bean curd: 1/4 'Miso' bean paste: 1/2 tablespoon	For muscles and blood: high-quality protein fats vitamin A vitamin B1 and B2 calcium
Group 3	Green vegetables 100 Light-colored vegetables 200 Potatoes 100 Fruit 200 } 600	Spinach: 3 stalks Cabbage: 1 leaf Potatoes: 1 Cucumbers: 1 Apple: 1	To keep fit: vitamin A vitamin C minerals fiber
Group 4	Cereals 180 Sugar 20 Oils and fats 20 (including sweets and nuts, etc.) } 220	Rice: 1 cup Bread: 1 slice Sugar: 2 tablespoons Oil: 2 tablespoons	For strength and body heat: sugars protein fats

Source: *Zoku shyokuniku no chishihi* (More About Meat)[11]

The examination assumes a daily intake of 80 grams of beans in the form soybeans or kidney beans, 100 g of green vegetables in the form of spinach or chrysanthemums, 200 g of light-colored vegetables in the form of either cabbage, small turnips, or Japanese radishes, and 100 g of white potatoes. The figures used for the metal concentration within the plants are the maximum values measured during the course of the testing period. With these preconditions, the following formula was used to calculate metal intake.

$$\text{Metal intake} = \text{concentration in crops (maximum value)} \times \text{crop quantity ingested} \times \left(1 - \frac{\text{moisture (\%)}}{100}\right)$$

The figures for copper are presented in Table 10.

The same calculations were performed for the other metals as well. The maximum quantities of intake for each one are shown in Table 11.

Table 12 presents the average and maximum permissible levels of daily metal intake per person, according to the standards of the United Nations Food

Table 10 Heavy Metal Intake from Crops (Copper)

Food group	Crop	Maximum heavy metal concentration	Food intake	$1-\frac{\text{moisture}(\%)}{100}$	Intake	Maximum intake
Beans	Soybeans, harvested green	24.10 mg/kg	0.08 kg	0.304	0.586	0.586
	Kidney beans	9.89 mg/kg		0.065	0.051	
Green vegetables	Spinach	18.90 mg/kg	0.10 kg	0.063	0.119	0.119
	Garland chrysanthemums	16.8 mg/kg		0.048	0.081	
Light-colored vegetables	Cabbage	4.66 mg/kg	0.20 kg	0.058	0.054	0.077
	Small turnips	7.15 mg/kg		0.054	0.077	
	Japanese radishes	3.39 mg/kg		0.064	0.043	
Potatoes	White potatoes	8.52 mg/kg	0.10 kg	0.174	0.148	0.148
Total						0.930

Table 11 Heavy Metal Intake from Sludge Plot Crops

Metal	Intake from crops (mg/person/day) [A]	Maximum permissible level (mg/person/day) [B]	Intake as percentage of maximum permissible level (%) [A/B×100]
Copper	0.930	25	3.7
Zinc	4.676	50	9.4
Lead	0.382	0.357	107
Nickel	0.094	-	-
Total Chromium	0.014	44	0.03
Cadmium	0.007	0.3	2.3
Total Mercury	0.001	0.0357	2.8

Table 12 Average and Maximum Permissible Levels of Heavy Metal Intake (mg/person/day)

Metal	Average intake	Maximum permissible intake
Copper	2-4	25 (1)
Zinc	14	50 (1)
Lead	0.15	0.357 (1)
Nickel	0.3-0.6	-
Total Chromium	0.8	44 (2)
Cadmium	0.0385	0.3 (3)
Total Mercury	0.02	0.0357 (1)

(1) Proposed by UN Food and Agriculture Organization/World Health Organization
(2) USSR atmospheric standards
(3) Ministry of Health and Welfare

and Agriculture Organization, World Health Organization, and Ministry of Health and Welfare of Japan.

Since the above foods would not constitute the entire diet, the figures in Table 11 by no means indicate the total daily metal intake. Still, the figures for copper, zinc, chromium, cadmium, and mercury are less than 10 percent of the daily maximum permissible levels, and would therefore pose no health problems. Likewise, the figure for nickel would also pose no problems, since it amounts to approximately 20 percent of the average intake. The figure for lead, however, is close to the maximum permissible intake level. Nevertheless, such factors as the use of the maximum concentration values, despite the fact that no lead was detected in any of the crops in 1980 and 1981, suggest that this does not necessarily represent a critical problem. At any rate, this is a matter for future study.

3-3 Other

In order to test for one of the soil-improving effects of organic fertilizers, measurements of the pH coefficient were made as an index of the buffer action.

For the cultivation of most crops, the optimal range of the pH coefficient is between a weak acidity and neutrality. For this reason, lime is generally applied to farmland to adjust the pH coefficient.

In both test farms, the pH coefficient was adjusted by applying calcium carbonate.

In the Kohoku Test Farm, calcium carbonate was applied to both the standard plot and chemically fertilized plot on seven occasions, for a total application level of 1,410 kg per 10 ares. Calcium carbonate was applied to both of the sludge plots on five occasions up to the seventh planting, for a total application level of 800 kg per 10 ares. In the standard plot and chemically fertilized plots, the pH coefficient exhibited no great fluctuation, and remained in a condition of weak acidity. In the sludge plots, however, the pH coefficient rose significantly with the application of calcium carbonate up to the seventh

planting, particularly in the case of the sludge plot II. With the discontinuation of the application of calcium carbonate from the eighth planting on, however, the pH coefficients of both sludge plots remained virtually constant, on the order of 7.

In the Midori Test Farm, the standard plot and chemically fertilized plot exhibited the same trend as in the Kohoku Test Farm. Calcium carbonate was applied to the sludge plots in this test farm only at the first planting, at a level of 40 kg per 10 ares. The pH coefficient of the sludge plot I remained virtually constant, in the range of 6~6.5, while that of the sludge plot II rose to 7.

The above indicates that the pH coefficients of the sludge plots did not grow more acidic even when lime was not applied, but instead remained virtually constant or rose slightly. This result is due to the action of the lime mixed into the sludge as coagulant.

In sum, although the test data did not yield any conclusions about the buffer action of sludge itself as an organic fertilizer, it was confirmed that the lime used as coagulant in the sludge acted to adjust the pH coefficient.

4. Conclusion

This test concerning the long-term, continuous application of sewage sludge to test farms yielded conclusions regarding the soil-improving effects of sludge fertilizer, its fertilizing effect, the accumulation of heavy metals in the soil arising from the use of sludge, and the migration of heavy metals to the crops.

(1) Soil-improving effects

It was determined that the lime mixed in with the sludge acts to prevent the acidification of the soil and raises the pH coefficient.

(2) Fertilizing effect

An application of sludge at a level of 2 tons per 10 ares offers a greater fertilizing effect than an application level of 1 ton per 10 ares. At this level of application, sludge has proven to be an effective fertilizer for kidney beans, small turnips, cabbage, and soybeans harvested green. To judge from the results of six years of testing, the impact of a continuous application of sewage sludge upon the fertilizing effect maybe regarded as on a par with that in the case of compost (it should be noted, however, that chemical fertilizer was also applied to the sludge plots, and therefore the effectiveness verified by the test is not necessarily attributable to sludge alone). There is also no need for applying lime to adjust the pH coefficient, since the lime mixed in with the sludge performs this function.

(3) Behavior of heavy metals

Although there was a verified accumulation of heavy metals in the soil of sludge plot II, no positive correlation was found between the concentration of heavy metals in the soil and that in the crops. Fertilization with sludge at a level of 1 to 2 tons per 10 ares is not thought to exert any influence on the migration of heavy metals to crops.

With the exception of lead, the intake of heavy metals from a daily diet, in an average quantity, of crops from the sludge plots would be less than 10 percent of the maximum permissible daily levels, and the use of sludge as fertilizer is therefore thought to pose no health hazards. Even in the case of lead, there is thought to be no serious problem, but in any case, this is a topic that requires further study.

As for the prospects of the practical use of sludge fertilizer, the following issues demand investigation in the years ahead:

1) Statistical analysis of fertilizing effect;
2) Fertilizing effect of sludge when used alone;

3) Clear apprehension of soil-improving effects (including influence on soil microbes); and
4) Behavior of heavy metals and effect on the human body in the event of an even longer continuous application of sludge.

5. Afterword

This report has presented the conclusions of a six-year program of testing to examine the prospects for the continuous application of sewage sludge to fermland as fertilizer. As has been mentioned above, the testing found that sewage sludge can be looked to as an effective fertilizer, and one that poses basically no health problems owing to the influence of heavy metals.

It must be remembered, however, that since the testing program is still continuing, the present report must be termed an interim report. It is hoped that a final report will be released in the future, presenting new information acquired during the course of subsequent testing.

In conclusion, special thanks must be extended to Mr. Shigeyuki Suzuki, head of the Environmental Management Office, Pollution Prevention Bureau (former special councilor for industrial wastewater control, Sewage Works Bureau), for his valuable cooperation in the preparation of this report.

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SESSION 6

UPGRADING OF WASTEWATER TREATMENT

"Application of Biotechnology to Wastewater Treatment"
S. Kyosai 639
"Micro Bio Kinetic Bases for the High Process-stability and Purification Efficiencies of Two-stage Biological Plants with Preceding A-Stage"
B. Boehnke 659
"Role of Smaller Metazoa in the Waste Water Treatment"
Y. Inamori 667
"Application of Anaerobic Fixed Bed Process to Domestic Wastewater Treatment, followed by Membrane Separation"
Y. Tanaka, N. Okuno 703
"Odor Emissions and Their Prevention in Municipal Waste Water Treatment Plants"
U. Rudolph 739
"Experiments for Practical Use of Soil Deodorizing Equipments used together with Compost and Zeolite"
E. Sato 759

CHAPTER 6

WASTEWATER TREATMENT

APPLICATION OF BIOTECHNOLOGY TO WASTEWATER TREATMENT

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ABSTRACT

A five year research project named "Development of New Wastewater Treatment Systems Employing Biotechnology" was started by the Ministry of Construction in 1985. This project, commonly known as "Biofocus W.T.", intends to develop new wastewater treatment systems with following features achieved through the use of biotechnology.

a. Less energy requirement and lower O & M costs
b. Less land required for the wastewater treatment facilities
c. Upgrading the effluent quality
d. Recovery of valuable resources

The topics of research and development for the attainment of the above-mentioned objectives are as follows.

a. Immobilization technology for enzymes and microorganisms
b. Bioreactors with immobilized microorganisms for wastewater and sludge treartment
c. Biosensors for treatment process control and environmental measurement
d. Separation, identification, preservation and cultivation of microorganisms useful for wastewater and sludge treatment
e. New bacteria through recombinant DNA
f. Combined technology for physical/chemical processes having biological processes
g. New wastewater and sludge treatment systems

The development plans and the development targets of the project are summarized in this paper.

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APPLICATION OF BIOTECHNOLOGY TO WASTEWATER TREATMENT

1. Preface

A five-year research project named "Development of New Wastewater Treatment Systems Employing Biotechnology" was started by the Ministry of Construction of the Japanese Government in 1985. This project is commonly called "Biofocus W.T.", and has as its purpose the development of new wastewater treatment systems having the following features making use of biotechnology.

a. Less energy requirement and lower O & M costs
b. Less land required for wastewater treatment facilities
c. Upgrading the effluent quality
d. Recovery of valuable resources

This paper summarizes the research and development plans and the targets of the project.

2. Research and Development Plans of Biofocus W.T.

2-1 Research and Development Subjects and Timetable

Table 1 shows the research and development subjects and the timetable of Biofocus W.T.. As shown in the timetable, biotechnology, which can be defined, in a wide sense, as "technology to utilize the life maintenance functions of biological organisms for human life", will be applied to improve wastewater and sludge treatment in order to attain the above mentioned objectives of this project.

2-2 Research Organization of Biofocus W.T.

Two research institutes of the Ministry of Construction are in charge of this project. The Public Works Research Institute is responsible for publicly-owned wastewater treatment plants (POWTP), and the Building Research Institute is responsible for on-site wastewater treatment plants. This paper is mainly concerned with the POWTP field.

Table 1 Research Subjects for the Development of New Wastewater Treatment Systems Employing Biotechnology

Research Subjects (Research Period in F.Y.)	Outline of Research
1. Development of immobilization technology for enzymes and microorganisms a. Study on immobilization methods (1985-1989) b. Study on bioreactors (1985-1989) c. Study on biosensors (1985-1989)	Following are to be developed. Immobilization methods for enzyme and microorganisms for increasing microorganisms concentration in order to make compact bioreactors and to decrease operation costs of WT plants. Efficient WT bioreactors in which general microorganisms found in activated sludge or useful microorganisms separated or developed under the subject 2 or 3 are immobilized. Efficient ST bioreactors to digest sludge anaerobically. Biosensors having advantages such as reacting with specific substances in a short time under normal temperature and pressure and high selectivility for particular substances.
2. Development of enzyme and microorganisms bank a. Study on separation and classification of microorganisms (1985-1989) b. Study on evaluation of enzymes (1987-1989) c. Study on preservation and mass cultivation of microorganisms (1985-1989)	Following are to be studied. Separation and classification of microorganisms useful for WT, ST and recovering valuable resorces from wastewater and sludge. Biochemical characteristics and applicability to WT of enzymes extracted from microorganisms related to WT. Preservation and mass cultivation of enzymes and microorganisms useful for WT and ST. Systematizing of information on enzymes and microorganisms useful for WT in cooperating with other research institutes.
3. Development of novel bacteria by genetic alteration a. Study on needs and feasibility of genetic control (1986-1989) b. Development of novel microorganisms by recombinant DNA (1987-1989) c. Development of new microorganisms by cell fusion (1988-1989)	Following are to be studies and developed. Development, preservation and mass cultivation of novel microorganisms useful for WT and ST by recombinant DNA, cell fusion and so on. Optimum cultivating conditions of novel microorganisms which tend not to survive and grow in mixed cultures employed in WT. Influence to ecosystem, safety, effect and economy of new microorganisms.
4. Development of combined technology for physical/chemical processes with biological processes a. Study on evaluation of physical/chemical treatment processes (1986-1989) b. Study on total treatment system(1986-1989)	Following are to be developed. Solids-liquid separation processes put prior to WT processes employed immobilized microorganisms which have a difficulty in treating solid substances. Solid-liquid separation processes for effluent from WT processes.
5. Application of new technologies to WT system a. Application to POWTP (1987-1989) b. Application to septic tank (1987-1989)	Following are to be practised. Construction and operation of a piot plant of WT system employed new technologies which are practical and have a higher performance than conventional WT systems.

WT: wastewater treatment
ST: sludge treatment
POWTP: publicly owned wastewater treatment plant

Biofocus W.T. has been carried out through the Committee of Development of New Wastewater Treatment Systems Employing Biotechnology, a contract research with the Japan Sewage Works Agency, and also joint researches with private companies supplementing that performed by its own researches. The Committee to supervise this project is organized as shown in Figure 1. Some 44 joint research topics and about 9 subjects have been carried out since 1985 and 2 subjects were added this year as can be seen in Table 2.

3. Development Targets of Biofocus W.T in the field of POWTP

The Committee prepared development targets of this project as one of its outcomes in 1985. Folowings are outlines of the targets abuot POWTP.

3-1 Sub-Commitee for Bioreactor Development

The Sub-Commitee for Bioreactor Development has prepared development targets for immobilization technology for enzymes and microorganisms, bioreactors for wastewater treatment, bioreactors for sludge treatment, and also for bisensors. This Sub-Committee has proposed 4 new wastewater tratment systems that also employ biotechnology.

3-1-1 Immobilization Technology for Enzymes and Microorganisms

The development targets for the immobilization technology which is to be utilized for increasing amounts of useful microorganisms and upgrading the efficiency of bioreactors for wastewater treatment and sludge treatment, are as follows.

Objective enzyme and microorganisms are microorganisms in activated sludge and of sludge digestion at present. When useful enzymes and/or microorganisms are obtained through the study of enzymes and microorganisms, such enzyme and/or microorganisms will be added as the objects of study. Immobilization methods to be developed are divided into those methods without attachment mediums and those methods with attachment mediums, as follows.

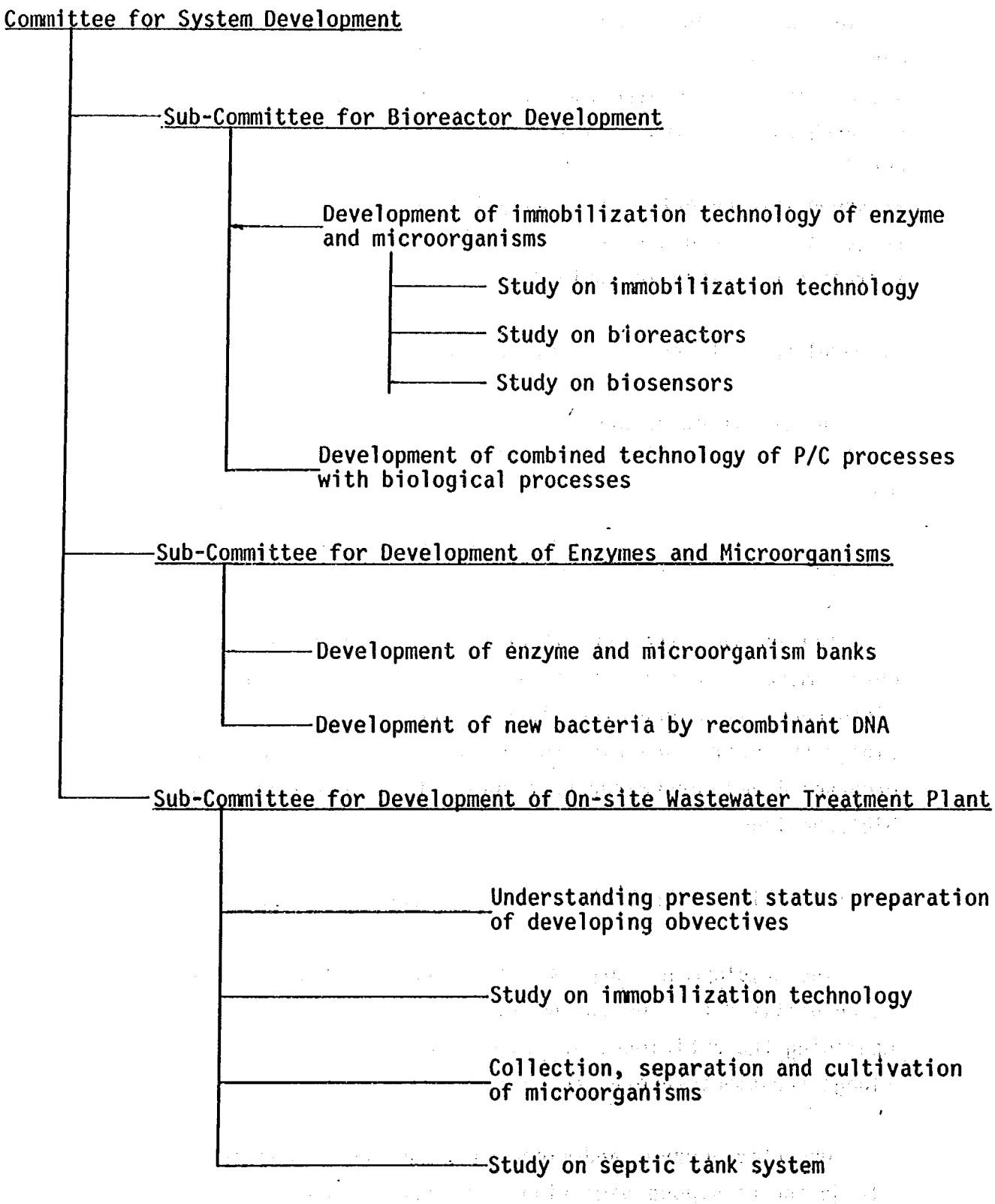


Figure 1 Committee of Development of New Wastewater Treatment System Employing Biotechnology

Table 2 Subjects of Joint Research (for POWPT)

Initiated year		Subjects of Joint Research	Number of Companies
1985	1.	Development and evaluation of attachment medium for microorganisms (binding methods)	11
	2.	Development and evaluation of attachment medium for microorganisms (entrapping methods)	6
	3.	Development of biosensors	2
	4.	Separation and cultivation of microorganisms useful for wastewater treatment	3
	5.	Development of bioreactors for removing organic substances *	15
	6.	Development of bioreactors for removing phosphorus and nitrogen	2
	7.	Development of bioreactors for sludge treatment	3
	8.	Development of technology to remove nitrogen using immobilized microorganisms **	1
	9.	Development of technology to remove highly concentrated organic substances **	1
1986	10.	Development of solid-liquid separation processes combined with bioreactors	5
	11.	Removal of heavy metals in sludge by bacteria leaching	2

* This subject is divided into four categories according to aerobic or anaerobic and a fluidized bed or a fixed bed.

** With the Japan Sewage Works Agency. Others are with the Public Works Research Institute of the Ministry of Construction.

a. Methods without attachment mediums
Self-immobilization methods
Membrane methods

b. Methods with attachment mediums
Binding methods
Entrapping mehtods

Immobilization technology will be evaluated with respect to the items of the density & stability of immobilized microorganisms, their durability, and the cost and disposability of immolilization materials.

3-1-2 Bioreactors for Wastewater Treatment

Table 3 shows the development targets of bioreactors for wastewater treatment. Bioreactors for organic removal and bioreactors for advanced wastewater treatment are to be developed in line with these targets .

1) Bioreactors for Organic Removal

a. Energy-Saving Type Bioreactors

Energy saving type bioreactors are intended to be used for medium-size and/or small-size wastewater treatment plants, of which construction will increase in number in Japan, because those plants require more energy for per unit volume treatment when compared with large size plants. A primary target of development of energy saving type bioreactors is that electricity consumed in wastewater treatment is between 0.1 - 0.2 kWh per cubic meters. The candidate processes are anaerobic treatment and aerobic treatment with immobilized microorganisms.

b. Land-Saving Type Bioreactors

There are two types of land-saving bioreactors. Type 1 is for areas where the available land for a wastewater traetment plant is quite limited such as in mountanous regions. Type 2 is for on-site reuse of effluent in buildings. The land requirment is reduced by half in type 1 and one tenth in type 2. The candidate processes are aerobic

Table 3 Developing Targets of Bioreactors for Wastewater Treatment (draft)

		Bioreactors for Organic Removal			Bioreactors for Advanced Wastewater Treatment		
		Energy-Saving Type	Land-Saving Type		for Nitrogen Removal		for Phosphorus and Nitrogen Removal
			Type 1	Type 2	Type 1	Type 2	
Water Quality in Effluent	BOD_5 (mg/ℓ)	5-10	5-10	less than 5	△	○	5
	TN (mg/ℓ)	△	△	○	10 less than 1 in NH_4-N	less than 2	10
	TP(mg/ℓ)	△	△	○	△	○	0.5
Land Requirement of Facilities		△	1/2	1/10	1/2	1/10	◎
Energy Consumption in Wastewater Treatment (kwh/m^3)		0.1-0.2	0.2-0.4	0.4-0.8	△	○	△
Excess sludge Generation		◎	○	◎	△	△	△
Operation and Maintenance		○	○	○	○	○	○
Resource Recovery		△	△	◎	△	△	△
Present Process for Comparison		conventional activated sludge process ($5,000-10,000m^3$/day)			single-stage nitrification denitrification process (SSNDP)		SSNDP with coagulant addition
Candinate Processes to be Developed		anaerobic process and aerobic process with immobilized microorganisms	aerobic process with immobilized microorganisms	liquid-solid separation by membrane	nitrification process with immobilized nitrifier	nitrogen removal process with immobilized nitrifier and denitrifier (NRPIND)	NRPIND with efficient liquid-solid separation process

◎ about 50% improvement compared with present process
○ about 25% improvement compared with present process
△ equal to present process

treatment with immobilized microorganisms for type 1 and liquid solids separation by membrane instead of sedimentation for type 2.

2) Bioreactors for Advanced Wastewater Treatment

a. Bioreactors for Nitrogen Removal

Selectively immobilized nitrifiers and denitrifiers are to be utilized effectively in this development. There are two types of bioreactors to be developed for nitrogen removal. Type 1 is applied to existing treatment plants with minimum alternation of the treatment facilities. Energy and land savings are the primary objectives in the development of a type 2 reactor which is to be applied to newly-designed plants.

b. Bioreactors for Phosphorus and Nitrogen Removal

The biological nitrification-denitrification method with coagulant addition to an aeration tank and biological phosphorus and nitrogen removal method have been applied to full-scale plants as methods to simultaneously remove phosphorus and nitrogen. In addition to the necessity of a long detention time for nitrification in both methods, the prior method has problems such as excess sludge increase and decrease in the nitrification rate due to chemical addition, while the latter method has problems such as disposal of sludge containing more phosphorus and the instability of phosphorus removal. The bioreactors for phosphorus and nitrogen removal to be developed aim at solving these above-mentioned problems.

3-1-3 Bioreactors for Sludge Treatment

A primary objective of the application of biotechnology to sludge treatment in Biofocus W.T. is to recover valuable resources including digestion gas from sludge. As shown in Table 4 indicating the development targets of bioreactors for sludge treatemtn at present, the energy-saving type anaerobic sludge digestion and the energy-creating type anaerobic sludge digestion will be developed.

Table 4 Development Targets of Bioreactors for Sludge Treatment (draft)

	Present Mesophilic Anaerobic Sludge Digestion for Comparison	Bioreactors to be Developed	
		Energy-Saving Type	Energy-Creating Type
Decomposition Rate (% of VSS)	mixed sludge 40-50% primary sludge 50-60%	△	60-75% 75-85%
Digestion Period	20 days in the first stage tank	less than 10 days	△
Energy Consumption		○	○
Digestion Gas Volume Generated ($CH_4 \cdot Nm^3/VS$ Kg)	mixed sludge 0.3 primary sludge 0.35	○	0.45 0.50
Candinate Process to be developed		immobilization of microorganisms relating digestion	introduction of liquefying bacteria

○...about 25% improvement compared with present process

△...equal to a present process

1) Energy-Saving Type Bioreactors for Sludge Digestion

Microorganisms relating to anaerobic sludge digestion are immobilized to reduce the digestion period and result in conserving the energy used in heating the digestion tanks. Reducing the digestion period of the first stage tank in mesophilic digestion to less than 10 days from 20 days, which is the design value at present in Japan, is a primary objective of this development.

2) Energy-Creating Type Bioreactors for Sludge Digestion

The decomposition rate of solids in sludge is increased by effectively introducing liquefying bacteria to digestion process. As a result, the digestion rate and the recovered digestion gas will increase and the sludge volume will decrease. Objective digestion rates are 60-75 % for mixed sludge and 75-85 % for primary sludge. These rates are 40-50 %, and 50-60 % in mesophilic digestion at present for mixed sludge and primary sludge, respectively.

3-1-4 Biosensors

Application of sensors has been attracting attention in various fields. As the fields of application of chemical sensors has spread widely, chemical sensors have been vigorously developed. One type of chemical sensors is the biosensor which mainly measures biological substances. Biosensors utilize biological materials such as enzymes which have high selectivity for particular substances. The biosensors to be developed will be used in wastewater tratment plants and include organic acid sensors, ammonia sensors, and BOD sensors as shown in Table 5.

1) Organic Acid Sensors

Organic acid sensors are intended to measure the organic acid content in anaerobic wastewater treatment and in anaerobic sludge digestion. Therefore, the ease of operation and durability are important factors for the evaluation of the sensor.

Table 5 Development Targets of Biosensors (draft)

	Organic Acid Sensor for Process Control	Ammonia Sensor for Monitoring	BOD Sensor for Laboratory
Place for use	indoor	indoor	indoor
Size	○	○	○
Accuracy	○	◎	○
Response	○	○	◎
Frequency of Measurement	continuous or semi-continuous	continuous or semi-continuous	manual
Life of Sensor	one month	one month	one month
Durability	◎	◎	○
Ease of O & M	◎	◎	○
Output Signal	necessary	necessary	not necessary

◎ very important factor for development

○ important factor for development

2) Ammonia Sensors

Ammonia sensors are intended to measure ammonia concentration in effluent and nitrification process. Therefore, ease of operation and durability are also important factors to evaluate the sensor.

3) BOD sensors

Quick response is an important factor in the development of these sensors.

3-1-5 New Wastewater Treatment Systems

Biofocus W.T. is scheduled to develop unit technologies and unit processes employing biotechnologies, according to the subjects shown in Table 1. It is also necessary to consider new wastewater treatment systems based on unit development from the early stages of development. From such a standpoint, the sub-committee for bioreactors proposed the following 4 new wastewater treatment systems last year.

a. Wastewater treatment system with little sludge treatment
b. Self-energy sustaining wastewater treatment system
c. Wastewater treatment system utilizing light
d. Wastewater treatment system recovering valuable resources

The self-energy sustaining wastewater treatment system is shown in Figure 2. The total electricity consumed by wastewater treatment in Japan amounted to about three billion kWh per year in fiscal 1983. This value will increase along with the increase in the sewered population and so self-energy sustaining system was proposed to solve this problem.

Most suspended solids are removed from raw wastewater before flowing to a wastewater treatment bioreactor, by introducing a high efficiency solids-liquid separation unit to the system instead of a primary sedimentation tank. Reduction of the influent loadings and the application of an energy-saving wastewater treatment bioreactor contribute to reduce energy consumption in wastewater treatment. On

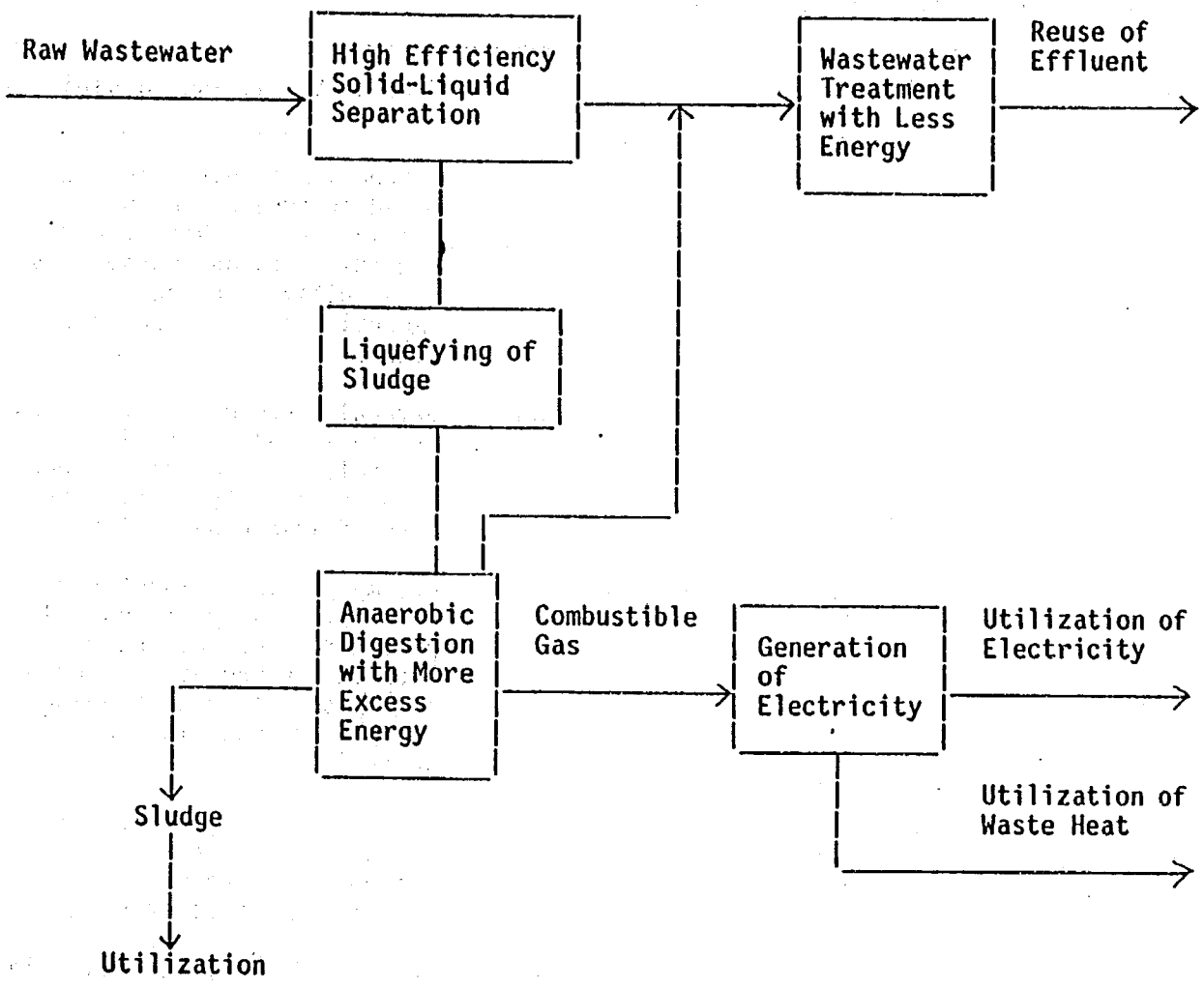


Figure 2 Self-Energy Sustaining Wastewater Treatment System

Table 6 Objective Microorganisms (draft)

Microorganism	Selected Reason	Present Status and Possibility
Nitrifier	For upgrading nitrification	Metabolism of nitrifier in mixed culture has been made fairly clear. Nitrification process is to be improved by mass cultivation and immobilization of separated nitrifier.
Denitrifier	For upgrading denitrification	Several kinds of denitrifiers are separated and their metabolism are clear. There is a possibility of finding more efficient denitrifiers.
Phosphorus Accumulating Bacteria	For upgrading phosphorus removal	0.1 mg·p/ℓ in effluent has been obtained by biological phosphorus removal processes such as A/O process. There is a possibility of finding phosphorus accumulating bacteria.
Resistant Bacteria of Heavy Metals	For heavy metal free sludge and recovering heavy metals from sludge	Metabolism of Hg resistant bacteria is clear. Metabolism of Zn resistant bacteria is to be studied.
Solids Liquefying Bacteria	For increasing decomposition rate of solids in sludge in anaerobic digestion	Bacteria liquefying solids in sludge are unknown. Solids liquefying bacteria are to be separated after identifying object solids in sludge.
Photosynthetic Bacteria	For improving sludge treatment and reusing the products	Metabolism of photosynthetic bacteria are clear. Application conditions of the bacteria to sludge treatment are to be studied.
Bacteria Decomposing Natural Refractory Organic Substances	For upgrading effluent quality by decomposing natural refractory organic substances which increase COD in effluent	Bacteria decomposing lignin and tannin are practically unknown. There is a possibility to found the bacteria from humus soil.
Bacteria Decomposing Organochlorine Compounds	For upgrading effluent quality by decomposing refractory organochlorine compounds	Plasmid relating to decomposition of organochlorine compounds is known. The bacteria are to be separated from activated sludge cultivated in aromatic halide.

the other hand, sludge treatment (anaerobic digestion) produces much more excess energy by applying an energy-creating type bioreactor for sludge treatment and by increasing the primary sludge in the feed sludge to the sludge reactor. The generation potential of digestion gas for this sludge is higher than that of excess sludge. A key point of the system development is the development of a high efficiency solids-liquid separation unit as well as the bioreactors.

3-2 Sub-Committee for Development of Enzyme and Microorganism Bank

The following eight kinds of microorganisms were selected as object microorganisms for study and/or development.

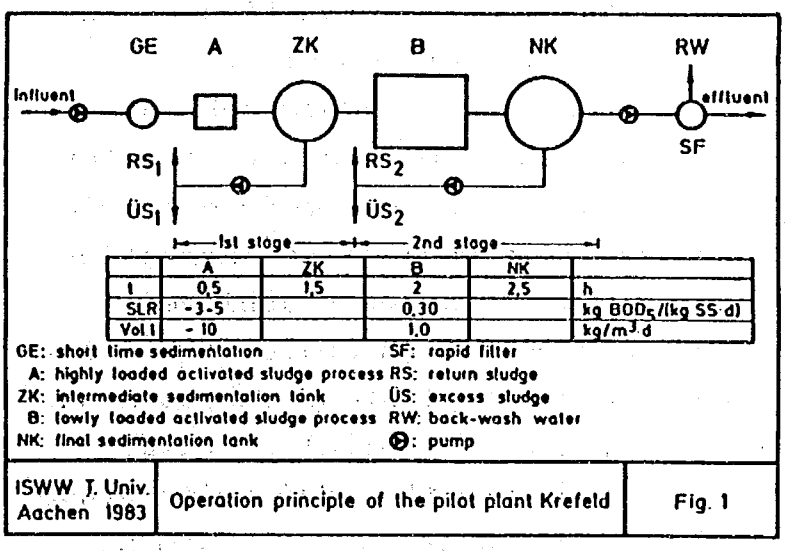
a. Nitrifier
b. Denitrifier
c. Phosphorus accumulating bacteria
d. Resistant bacteria to heavy metals
e. Solids-liquefying bacteria
f. Photometabolic bacteria
g. Bacteria decomposing natural refractory organic substances
h. Bacteria decomposing organochlorine compounds

Table 6 summarizes the selected reasons and the present status and possibilities of the eight kinds of microorganisms.

MICRO BIO KINETIC BASES FOR THE HIGH PROCESS-STABILITY AND PURIFICATION EFFICIENCIES OF TWO-STAGE BIOLOGICAL PLANTS WITH PRECEDING A-STAGE

Prof. Dr.-Ing. B. Boehnke
RWTH Aachen, Institut fuer Siedlungswasserwirtschaft

The two-stage biological activated sludge treatment plants, which have preceding adsorption stages, show a much higher process-stability and much improved purification efficiencies, with much reduced specific space requirements.



	A	ZK	B	NK	
t	0,5	1,5	2	2,5	h
SLR	~3-5		0,30		kg BOD_5/(kg SS·d)
Vol l	~ 10		1,0		kg/m³·d

GE: short time sedimentation
A: highly loaded activated sludge process
ZK: intermediate sedimentation tank
B: lowly loaded activated sludge process
NK: final sedimentation tank
SF: rapid filter
RS: return sludge
ÜS: excess sludge
RW: back-wash water
⊕: pump

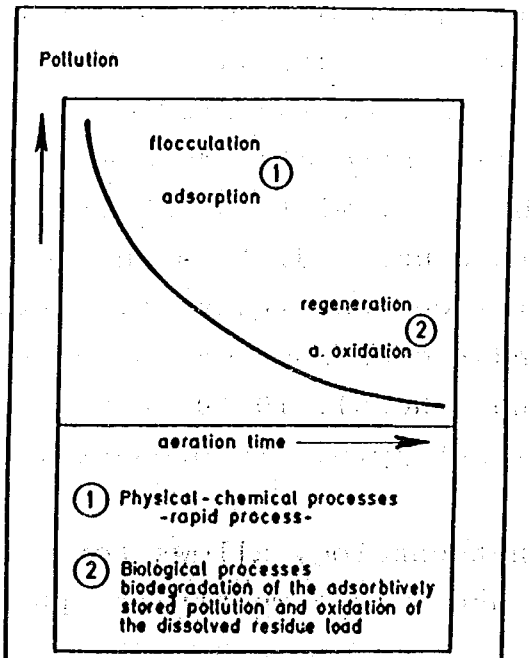
ISWW T. Univ. Aachen 1983 | Operation principle of the pilot plant Krefeld | Fig. 1

This has been reported on in a number of publications (1, 2,3). Hereafter; further results of the AB-technology, with special emphasis on the micro-biokinetic bases, will be presented. To facilitate recall, the first illustration shows the operating schema. There is a high-rate loading at the first biological stage, whilst there is a low-rate loading at the second stage. The sludge circuits of both stages remain separate. There already are 19 large scale AB-plants in existence, where 2.1 million inhabitants/population equivalent (p.e.) are connected. A further 8 large scale plants, where 2.46 million inhabitants/p.e. will be connected, are presently under construction (Saloniki, Rotterdam, Salzburg, Innsbruck and others). 10 further plants, servicing 4.85 million inhabitants/p.e. are presently at planning stage.

AB-technology allows for a development to be effected in stages. If a mechanical purification stage with an aerated

grit removal tank, effects a treatment efficiency of about 30 % - 33 %, this efficiency can be increased to 55 % - 65 % by recycling the sludge from pre-sedimentation to the grit removal tank. The only pre-condition for this is the circulation of the sludge, as well as a slight increase in the detention period in the grit removal tank. In this case the grit removal tank additionally takes on the function of the first stage, loaded at maximum rate. Where considered necessary, the second activated stage can be added subsequently in a second development stage. Thus, on changing the former pre-sedimentation at the treatment plant Krefeld (ηold = 28 %) to an adsorption stage and intermediate sedimentation, a purification efficiency of η = 63 % has been achieved, whilst saving 25 % of the pre-sedimentation space. A similar result was achieved when the treatment plant Rheinhausen was extended to AB-technology. Here, the mechanical circular treatment tanks were re-structured into the adsorption stage with an inside A-stage, and outside intermediate sedimentation.

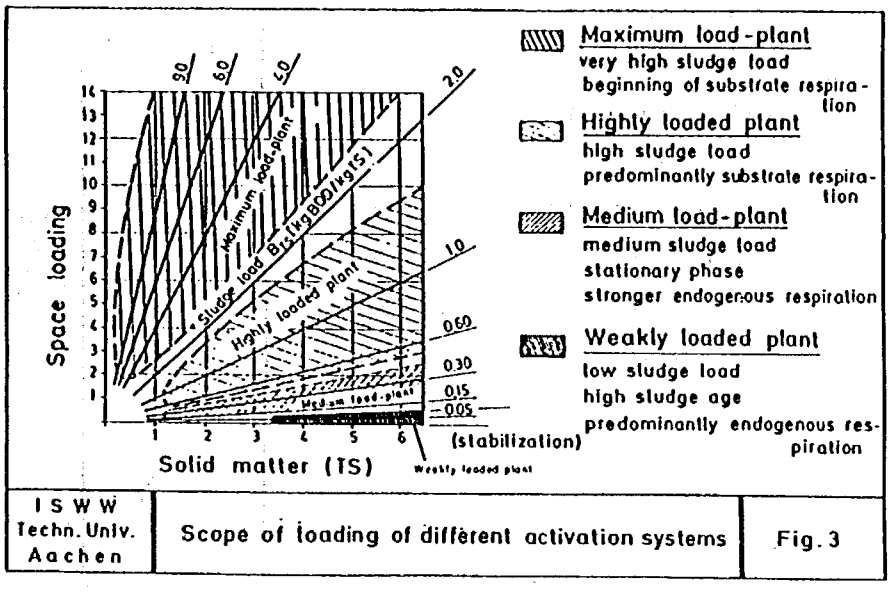
We know from process technology that multiple-stage processes produce more even results. This principle not only applies to chemical and physical processes, but also to biological and biochemical processes. When looking at the schema of the purification process at an activated sludge treatment plant, as published by Jung in 1953/54, the rapidly occurring physico-chemical, and, partly, bio-chemical processes should happen in a first stage, and the biological fine-purification in a second stage (4,5).



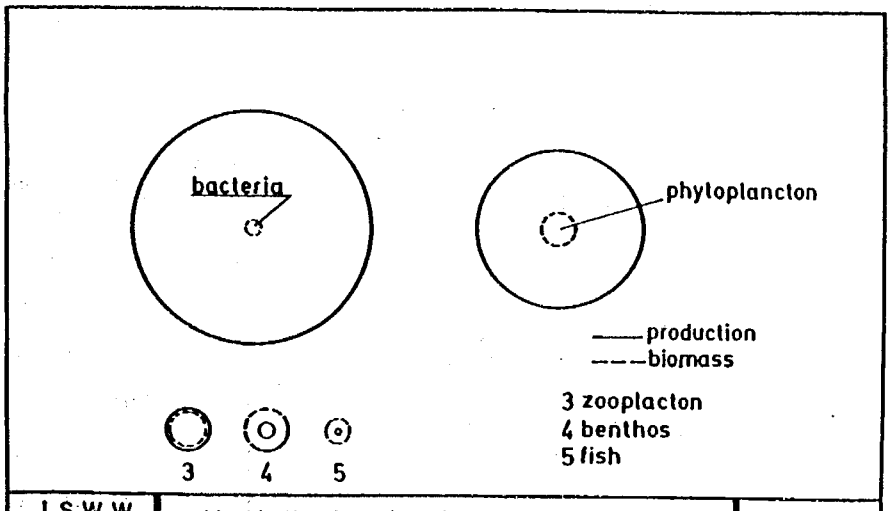
(taken from: H. Jung in Kommunalwirtschaft 1953/54)

ISWW Techn. Univ. Aachen	Course of the clarification process of an activated sludge plant	Fig 2

Since the processes for municipal/industrial waste waters occur very rapidly, i.e. in about 15 mins. to 30 mins., very high rate space as well as sludge loading result at the first activated stage.



I S W W Techn. Univ. Aachen	Scope of loading of different activation systems	Fig. 3



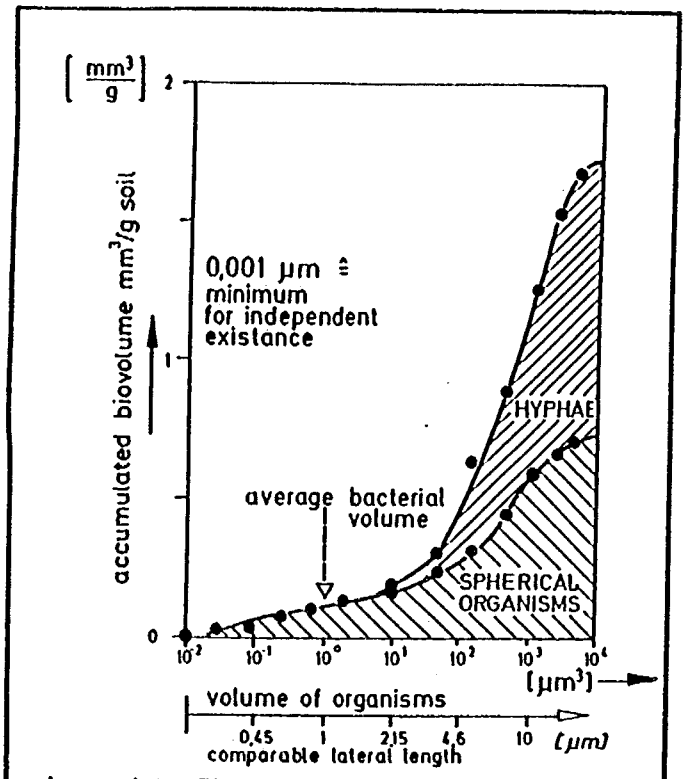
I S W W Techn. Univ. Aachen	Metabolic Intensity of Organisms (acc. to O.Klee)	Fig. 4

The sludge age is in the region of 0.4 - 0.5 days, and does not allow for the survival of superior micro-organisms (Eucaryotes).

Where there is such a change in the composition of species of the "bacteria population" those bacteria predominate, which have an average lateral length of 1 µm, achieve a low bio-volume, and which have about 1/1000 of the weight of Eucaryotes (e.g. protozoa), as also shown by Jenkinson and Ladd (Fig. 5).

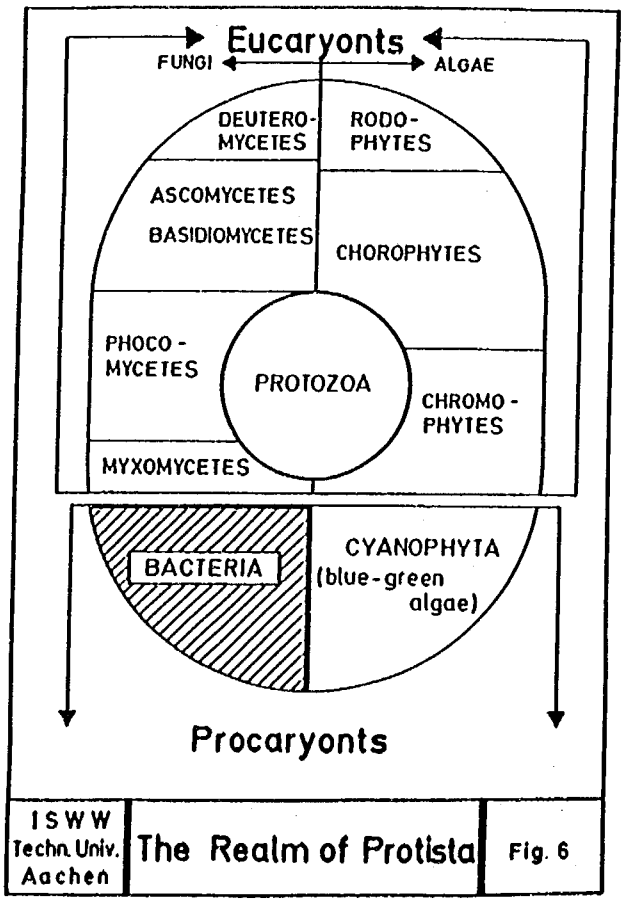
The metabolic intensity is proportionally reverse to the size of the organisms. Klee (Fig. 4) shows by way of an example the metabolic intensities of varied organisms proportionally to their bio-mass (6). It is, therefore, expe-

dient, to use the bacteria in a purposeful way, because of their nutritional greed and high breeding rate.
Taking the microbes as a whole, only bacteria can be used thus in the first activated stage (Fig. 6).
Experiments carried out at the Institute have shown unequivocally that pure bacteria cultures occur whenever there are high sludge loads.
Fig. 7 shows such a biocenosis, yielding extraordinarily high specific elimination efficiencies (7). The extent to which the elimination efficiency of the working activated sludge augments, where there is an increasing sludge load, is shown in Fig. 8.
In order to purify the respective pollution of 1 EW (EW = total number of inhabitants and population equivalents) (= 60 g BOD_5) at a rate of 98 %, at a stabilizing plant, 1200 g of dry solid matter (TS) are needed. To eliminate about 55 % in a maximum loaded stage, in contrast, on B_{TS} = 5 only 12 g/EW

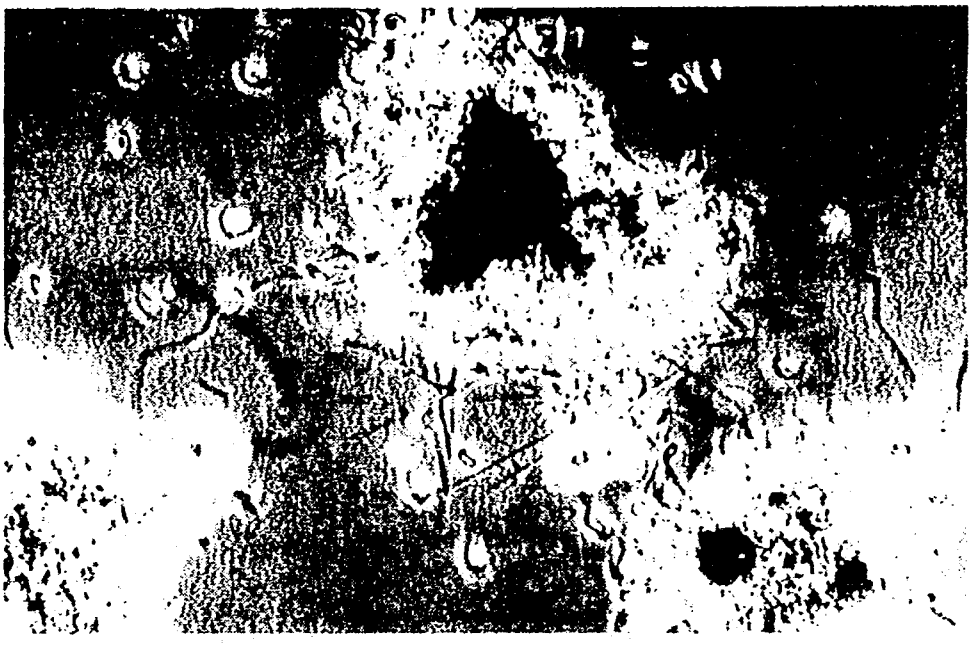


Accumulatet Biovolume In The Soil
Illustration acc. to Jenkinson and Ladd 1981

ISWW Techn. Univ. Aachen	Size And Volume Of Organisms In The Soil	Fig. 5



ISWW Techn. Univ. Aachen	The Realm of Protista	Fig. 6



and on B_{TS} = 10 kg BOD_5 / kg TS only 6 g/EW are required. This equals an increase of the sludge degradation efficiency by factor 57 or 83 resp. This high efficiency is accompanied by an extensive insensibility to loading and pH-shocks, forming of a heavy sludge with lower sludge index and the retaining, or the improvement respectively, of an easy, biological degradation of the load still remaining.

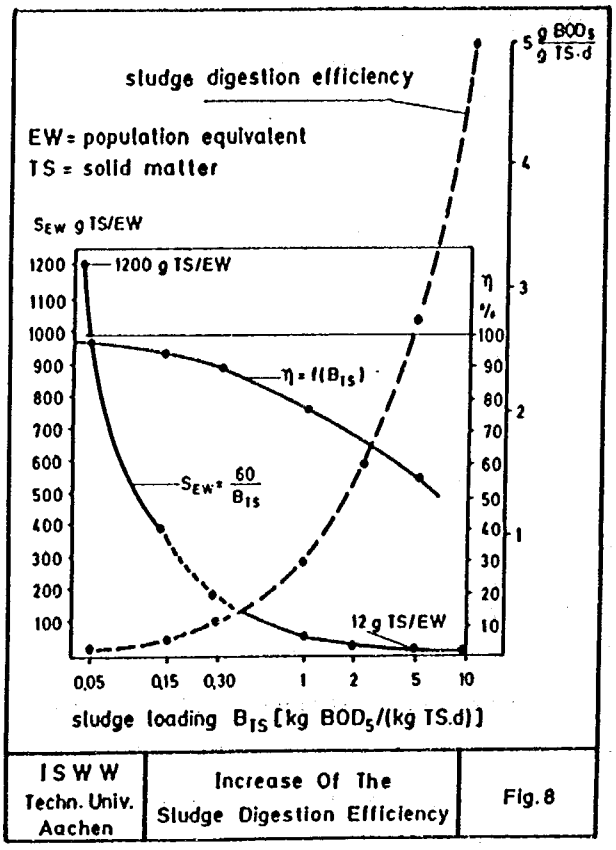
The high processstability and the fast recovery from noxious influences on the biocenosis at A-stage, can be explained to a large extent by the following properties existing in this biocenosis (8,9):

1. Existence of an effective survival mechanism, which is due to the short generation time and high degree of adaptability.

2. Presence of a broadly distributed bio-chemical potential, coupled to highly developed "job-sharing".

3. Extensive variability of the pure as well as the mixed populations to changing and extreme environmental influences when shock-loads occur.

4. High selection and mutation capacity during long-term



loads, which lead to the breeding of adapted and mutated bacteria cultures.

5. Presence of a highly active capacity to resist enforced changes in environmental influences, as well as a high stabilizing ability, characterized by the quick return to the original condition, once the influent shock load ceases.

In order to adapt rapidly to constantly changing loads, especially to overloads lasting for considerable periods of time (chemo-therapeutic substances, heavy metals, acid/basic pH values, toxic substances etc.), a high degree of variability is required.

To stand a slightly better chance in the hard fight for survival, bacteria have found a variety of possibilities to achieve new, resistant properties, as e.g.:

increase of the mutation rate:	- through mutagenic influences ba factor 10^3 - 10^4
conjugation:	- transfer of plasmids
recombination:	- new combination of the genotype in the meiosis
mobilization:	- transfer of not autotransferable plasmids
transformation:	- genetransfer through free DNA
transduction:	- transfer of DNA through bacteriophages

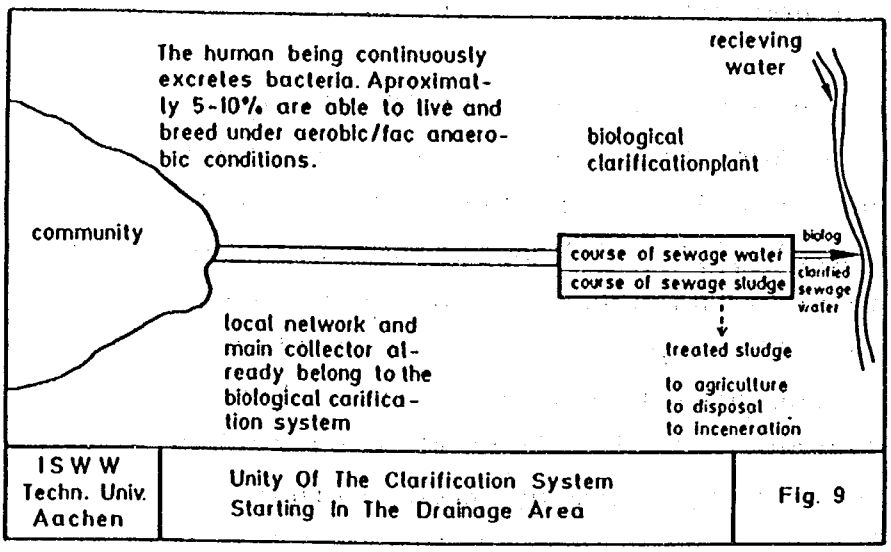
In my publications I have already discussed the influences of mutation on the creation of new resistant properties at length. However, the high adaptability and great stability in case of forced changes in the environment cannot be explained with this solely (10).

In my conversations with bacteriologists and hygenic experts I became aware of the fact that much could be learned from the experiences in the medical field - especially from bacteriologists of hospitals. It is obvious that there is a constant hard battle in the clinical field being fought between the experts of the medical and pharmaceutical sciences an the attacked group of microbes to suppress infections and deseases. It is clear that there has been no success in developing a remedy with definately suppress infections, instead there is a continuous creation of new resistant types. The hospital phenomenon is an exellent example for the defense capacity and adaptability of the bacteria to difficult external living conditions. The experiences gained in the medical field concerning this topic will therefore point out some useful directions in terms of what biological clarification technology ought to do or refrain from doing to support the bacteria in their fight for survival when confronted with noxious and toxic substances. At this point I would like to remind you of Wiesmann's explanations, who points out that in over 90 % of the resistant bacteria found in hospitals the resistance was brought about by transfer of plasmides (11). Jawetz et al., too, point out this fact (13).

A further accumulation of resistant properties is also being brought about by the fact that the prescribed pharmaceuticals have a strongly mutagenous effect and, according to Hopwood (14), increase the spontaneous rate of mutation by factor 1.000 to 10.000.

Permit me, therefore, to qoute a few passages from the reports and books of Strasburger, Mudrack/Kunst, Kaudewitz, Bogen, Praewe et al., Klee, Sajonski/Smollich, Nultsch, Jawetz et al.,

Schlegel, Wiesmann, Dawes, Sutherland, Sengbusch, Potel, Demain, Solomon, Hopwood, Weide/Aurich, Harbers and Kaplan (12).



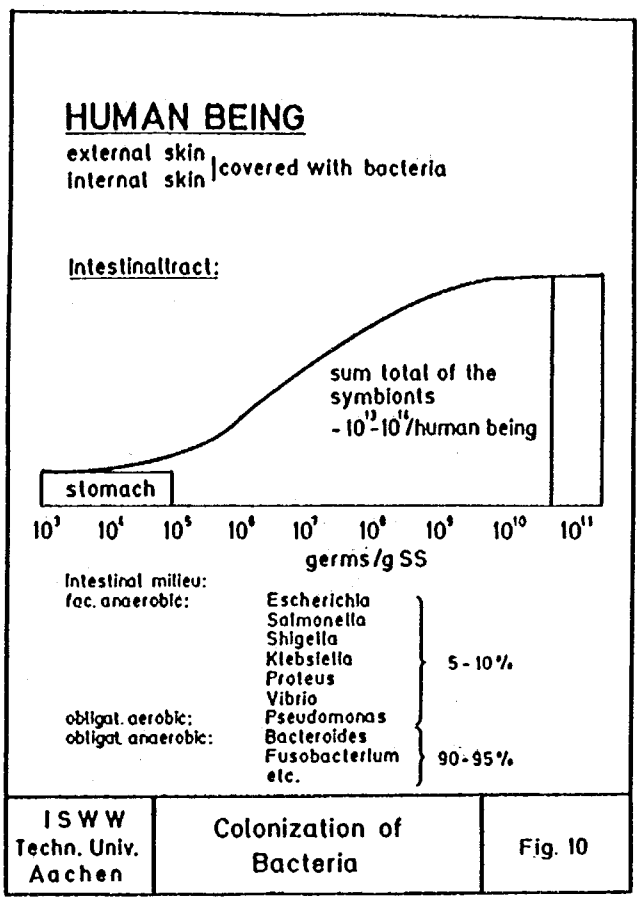
Jawetz et al. point out that the intestinal bacteria are a large group of gramnegative rods. This group includes e.g. Escherich's bacillus and related forms, Salmonella, Proteus Vibrio Shigella, Klebsiella, Pseudomonas, Bacterioides and Fusobacterium (13).

According to Potel (15), up to 10 % of the intestinal bacteria are able to survive outside the intestine, and the aerobians populate the canalization. The local canalization is, therefore, part of the purification system. Microscopic surveys show that the raw sewerage itself is very highly populated, as are the flushed pipe interiors. These are pure bacteria populations as in the maximum loaded first activated stage.

Since the forming of resistant cultures in the hospitals on the one hand, and the densely populated intestine (intestinal flora), on the other, point to a very close bond/dependency resp., between human beings and the bacteria world, one should further examine this relationship.

According to Potel the intestine is populated by bacteria. In the stomach the bacterial count is lowest at 10^3 - 10^5 bacteria/g TS, and highest in the large intestine at 10^{11} bacteria/g TS (15). When evaluating the number of bacteria

within the human body, about 10^{13} - 10^{14} bacteria live in symbiosis with the human being. Since the living being, i.e. the human being itself, has about 10^{14} - 10^{15} cells, the number of bacteria is virtually equal. In my opinion, such a relationship already points to the fact that each participant profits from the symbiosis. What are the profits, though? According to Potel, it seems to be doubtless that the intestinal flora has meaning for local, unspecific defence mechanisms. Furthermore, the assistance during digestion must be mentioned.



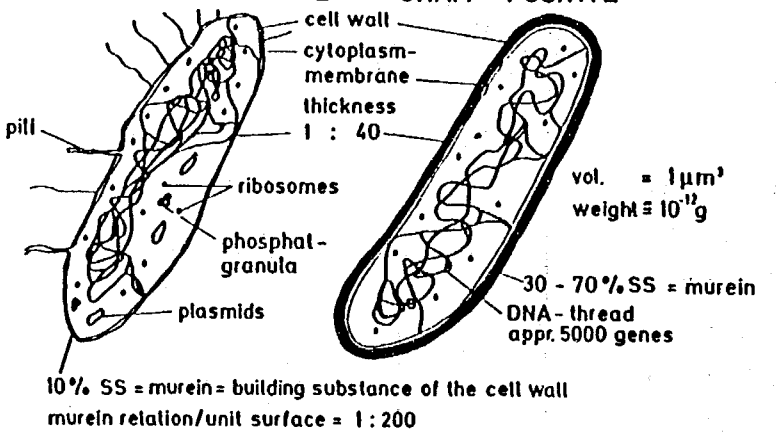
If this is so, it must be concluded that the warm blooded being, as well as the bacteria, have looked for the optimum partners in order to improve survival. Is this so?

According to Jawetz et al. there are only, or a vast majority of, gram-negative bacteria present in the intestinal flora. The question to be asked is: Why these in particular?

In comparison with gram-positive bacteria, gram-negative bacteria have a very thin cell-wall (only 1/40) and per unit of area only 1/200 of the murein-layer. The gram-positive bacteria have been stably developed by nature and carry their shield around with them. Why does the warm blooded being, the human, select "thin-skinned" and rather sensitive bacteria as symbionts? In my opinion this is done because during their development, such sensitive cells must have found or created a defence mechanism, which reacts sensiti-

vely and grants them a good chance for survival.

These seem to be plasmides, which are carriers of resistant properties, which can be transferred to non-plasmide carrying bacteria. According to Jawetz et al., hardly any plasmide transfers have as yet been observed in gram-positive bacteria. Latest findings show, however, that gram-positive bacteria, also, possess plasmides.

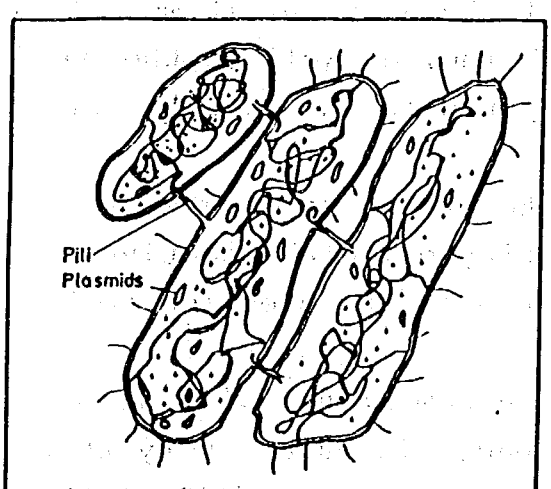


ISWW Techn. Univ. Aachen	Structure of Bacteria	Fig. 11

Plasmides are DNA rings which are preferred as carriers of resistant properties e.g. against heavy metals, penicillin, chemotherapeutics, toxic hydrocarbons etc., and whilst they are not vital for the host, they protect him when noxious substances occur, if corresponding resistant properties are coded in the genes.

The transfer of such plasmides to non-plasmide carrying bacteria is excellently organized by nature.

By direct contact, the exchange of information and transfer to the other bacterium is effected via the fertility gene (= F-Factor). It is remarkable that not



Transfer of Plasmids = Conjugation
DNA - Transfer via Pili
R - Plasmids: Resistancegenes
(chemo - therapeutics, heavy metals, sulphonamides, antibacterial substances, etc.)
En bloc - transfer

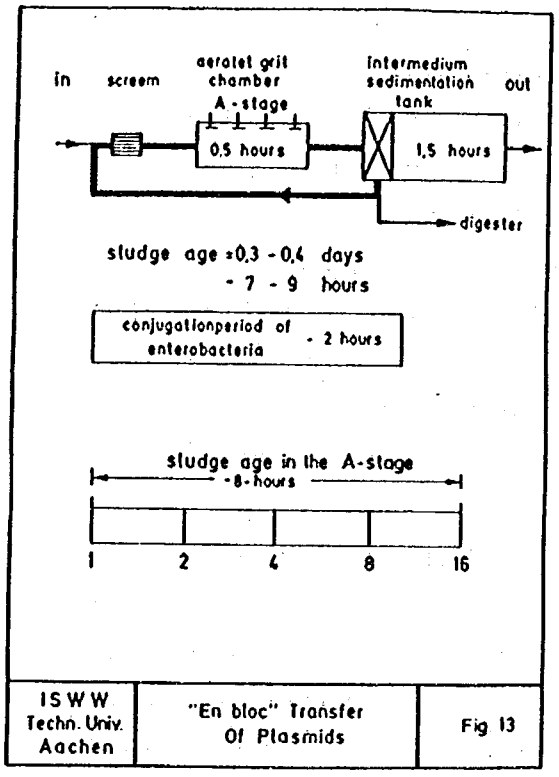
ISWW Techn. Univ. Aachen	Transfer - Schema	Fig. 12

all plasmides carry F-Genes. These can only be transferred to other cells, if they are mobilized by F-Gene carrying plasmides (auto-transferable). According to Schlegel (16), the conjugation process is a highly developed one in the entero-bacteria, and also takes place, if the partners are in a liquid medium, followed by a period of rest (e.g. peristalsis). Other bacteria need days for the transfer and have to intermingle intensely. The entero-bacteria can further effect the transfer in 90 mins. - 120 mins. If this amount of time is available - by receiving plasmides - which were heretofore unprotected, can rapidly develop a latently existing protection against a number of noxious substances.

How is this transfer effected? Some flagella of the plasmide carrying bacteria are formed as tubes, which bore through the cell wall of the other bacterium. The plasmide ring parts in one place and wanders into the other bacterium as a DNA-culture. After this transfer there is an immediate protection against those substances, against which corresponding R-genes are present. This transfer mechanism corresponds to a bi-sexual behaviour, which is also called a pseudo-sexual behaviour of the bacteria (Fig. 11 and 12).

If looking at the sewerage system and the first maximum rate activated stage in the light of the above, it can be ascertained that: The raw sewage in the influent shows on average 4 cm^3/l active bacteria sludge. A-stage and intermediate purification show a total retention time of 2 h altogether. The sludge age, however, is about 0,4 to 0,5 days, and one can, therefore, count on the sludge age being at least 10 h. Thus, 5 conjugations are possible on average. Since, in the opinion of the bacteriologists, more than 10 % of bacteria are plasmide carriers, there is an "en-bloc" transfer of plasmides at A-stage, i.e. all bacteria are being provided with existing and constantly incoming plasmides (= resistant properties). Moreover, the dangerous loads have a mutagenous effect and increase the rate of mutation quite considerably.

The resistant properties which develop daily in dependency on the influent load, together with the transfer of these properties to the whole of the bacteria population, are certain to represent one of the basic reasons for a process-stable working bacteria population.



Because of the high breeding-rate and the frequency of splitting, the new combination of the genotype in the meiosis (recombination), the transfer of DNA by way of bacteriophages (transduction), and the transfer of genes by way of free DNA (transformation) work in favour of an improvement of the genotype and an increase in the chances for survival.

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ROLE OF SMALLER METAZOA IN THE WASTE WATER TREATMENT

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Abstract

Smaller metazoa appearing in biological treatment process are divided into two types. One is filter feeder such as Rotatoria and another is detritus feeder such as Oilgochaeta. To make clear the role of smaller metazoa in water purification, *Philodina roseola* and *Aeolosoma hemprichi* were used as Rotatoria and Oligochaeta individually in this experiment. Smaller metazoa were cultivated under static and dark conditions in LE (lettuce and egg yolk extract) medium containing several bacteria. The run of experiment consisted of only bacteria (B), bacteria + *A. hemprichi* (BA), bacteria + *P. roseola* (BP), and bacteria + *A. hemprichi* + *P. roseola* (BAP). Purification efficiency was estimated by TOC, viable count and suspended solid in cultivation medium. The results obtained from this experiment were B, BA, BP, BAP in case of viable count (B: 1.4×10^8/ml, BAP: 5.4×10^5/ml in suspension) and BAP, BP, BA, B in case of TOC removal (B 49%, BAP 62% in filtrate). From these results, it was made clear that coexistence of filter feeder and detritus feeder was very important to promote the water purification efficiency.

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Role of Smaller Metazoa in the Waste Water Treatment

1. Preface

Biological treatment process is a typical ecosystem composed of bacteria, fungi and smaller animals such as protozoa and smaller metazoa. There should be an indigenous picture of competition, predation, mutualism and so forth, among the microorganisms which contribute to the removal of organic matters from sewage. Particularly, specific smaller metazoa have been detected in the sludge when the process proceeds satisfactorily. Therefore, some studies on the role of smaller metazoa in the biological wastewater treatment to warrant a good quality of effluent and a reduction of sludge production are essential. However, the role of smaller metazoa in the purification process has been known only little compared to that protozoa in the biological treatment process.

The present study, therefore, was performed to aim at elucidating the role of smaller metazoa by examining their growth and purification characteristics on the purification in biological treatment process. In this study, a strain of Rotatoria such as *Philodina roseola* and a Oligochaeta such as *Aeolosoma hemprichi* were used. Because their frequency of appearance and dominance is very remarkably and methods for their isolation and mass culture is well established.

This paper deals with the results of the studies on the investigation of smaller metazoa in full scale biological treatment process and on the growth and purification characteristics of *P. roseola* and *A. hemprichi* under monoxenic and mixed cultivation and on their roles on the reduction of biomass.

2. Experimental Methods

2-1 Investigation of Smaller Metazoa in the Biological Treatment Plants

In order to elucidate the role of smaller metazoa in biological treatment process, it is necessary to obtain important animals from the view points of the high frequency of appearance and dominance. Therefore, qualitative and quantitative investigations of smaller metazoa are required in biological treatment process at first. In this study, the important smaller animals were selected from 15 small-scale biological treatment plant, which has been expected to be used further widely to prevent water pollution in Japan, with regard to their frequency of appearance and dominance. Samples were collected from about 5 different parts per one tank. After these samples were mixed, 1 ml preparation put on a frame-attached lined slide glass and the smaller animals were observed using a light microscope for their count and identification

2-2 Experiments of smaller metazoa cultivation isolated from biological treatment plants.

2-2-1 Smaller Metazoa supplied to Experiment

A strain of Archioligochaeta (*Aeolosoma hemprichi*, see Photos 1 and 2) and a strain of Rotatoria (*Philodina roseola*, see Phots 3 and 4) had been isolated from biological treatment plants, and were used in this study.

A. hemprichi belongs to the family Aeolosomatidae of Oligochaeta is characterized by the following properties: its body size of about 1 mm,

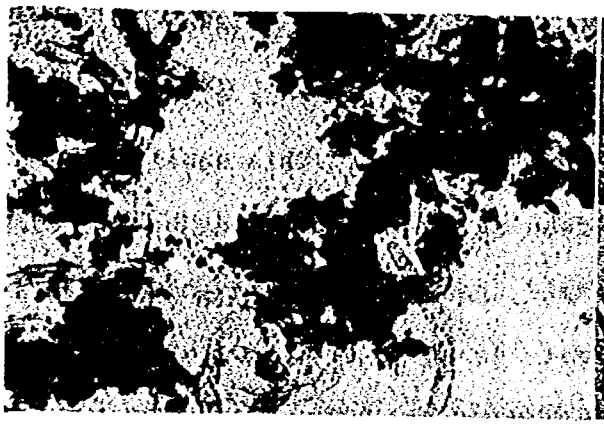


Photo 1 *A. hemprichi* dominated in bio-reactor

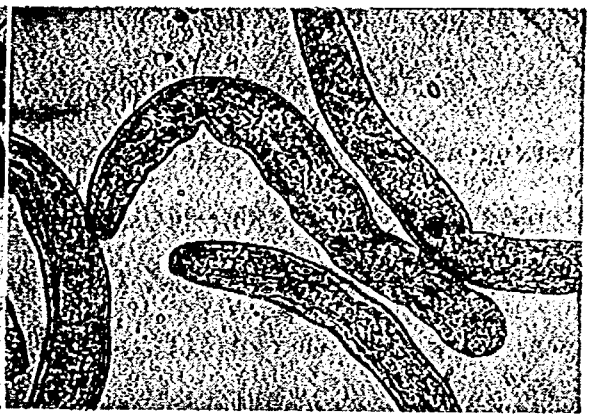


Photo 2 *A. hemprichi* isolated from biological treatment process



Photo 3 *P. roseola* dominated in bio-reactor

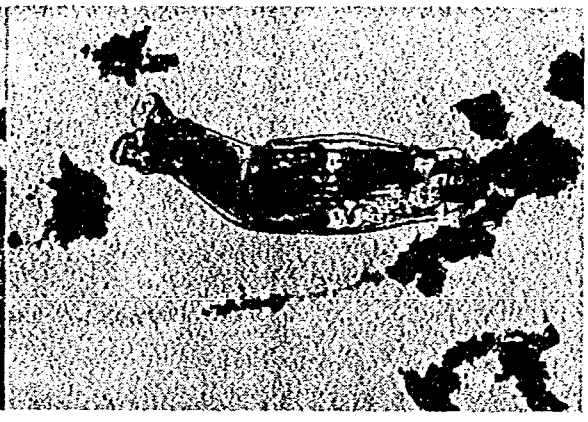


Photo 4 *P. roseola* isolated from biological treatment process

the presence of red-colored oil droplets under the skin, two pairs of seta bundles on its back and belly parts of each segment and the asexual reproduction method for its growth by means of transverse division. This species is classified as a stenophagous Oligochaeta which feed on a restricted species of detritus bacteria. The specific growth rate is around 0.4 day^{-1} when it was fed on the favorable bacteria. *A. hemprichi* is known as detritus feeder which fed on the coagulated organic matters by its mouth located on the head side (Fig. 1).

The characteristics of *P. roseola* are as follows: its body size is 200-400 μm, each of the back and tip sides of its foot has a pair of

toes, it moves like a leech by swimming or attaching to a substance with the toes and it feeds on a suspended from food by a strong ciliary movement of the trochus-attached corona on the head. This species is an euryphagous Rotatoria who can grow by feeding on various kinds of bacteria as food. Its multiplication is performed, generally, by parthenogenosis laying eggs, and the specific growth rate is about 0.4 day^{-1} when it was fed on the favorable bacteria. *P. roseola* is named as filter feeder, because it feeds on the dispersed bacteria and small flagellata by filtering them with ciliary movement (Fig. 2).

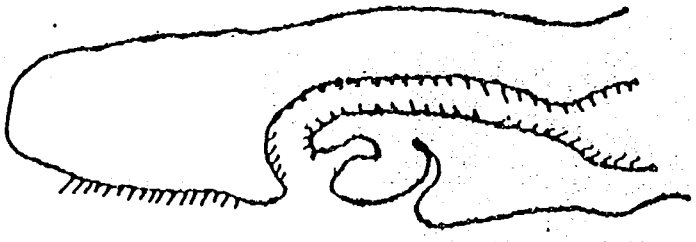


Fig. 1 Structure of mouth of *A. hemprichi*

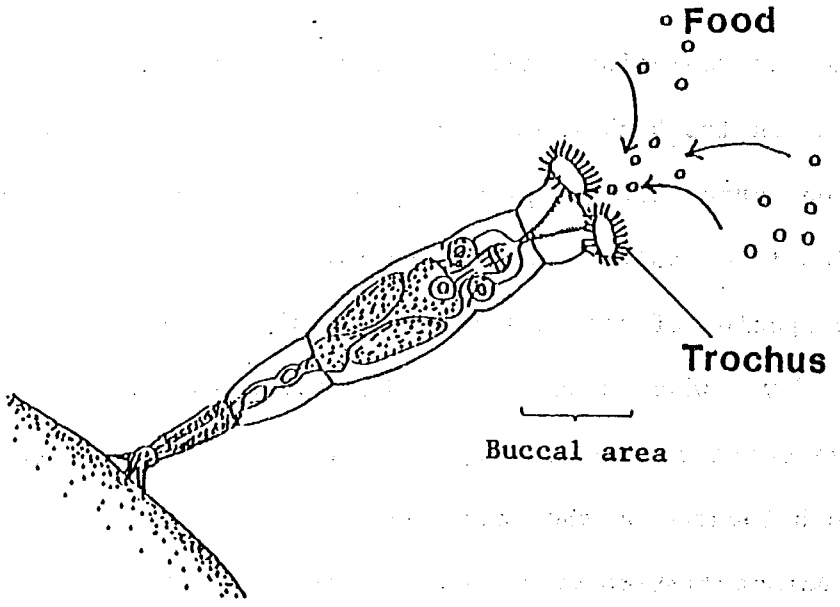


Fig. 2 Structure of buccal area of *P. roseola*

2-2-2 Culture Methods for Smaller Metazoa

1) Media

Selection of culture medium is quite important for the growth of smaller metazoa, because they require specific composition of medium. LE medium composed of lettuce and egg yolk extracts is the best nutrient for this purpose. The preparation method of LE medium is as follows. Lettuce (dried at 105°C for 5hr) and yolk of boiled egg is extracted in boiling water for 30 min individually. After they are filtered using absorbent cotton, the same volumes of the two extracts are mixed, pH of the mixture is adjusted to 6.7-7.0, and then the medium is sterilized using an autoclave.

2) Culture conditions

A 300 ml triangle flask containing 98 ml of the LE medium was used as the culture vessel (Fig. 3). Bacteiral suspension and/or culture broth of smaller metazoa (2 ml in total) was inoculated to the medium, and static cultivation conducted at 20°C for 10 days in the dark, and the changes in biochemical properties were measured daily throughout the culture period. The culture systems tested in this study were called as follows individually. B system (consisted of bacteria alone), BA system (mixture of bacteria and *A. hemprichi*), BP system (mixture of bacteria and *P. roseola*), and BPA system (mixture of bacteria, *P. roseola* and *A. hemprichi*). In this report, the BA and BP systems are designated as monoxemic culture, the BPA system which contains both species of smaller metazoa and the B system without smaller metazoa are also designated as mixed culture and control respectively.

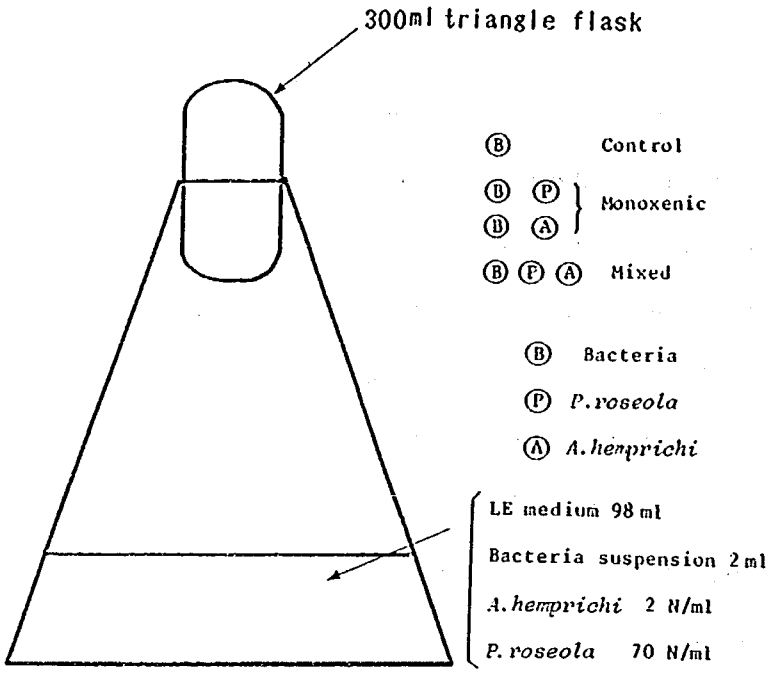


Fig. 3 Cultivating conditions

The initial inoculum numbers of *A.hemprichi* and *P.roseola* in these culture systems were fixed to 2 and 70 numbers/ml, respectively. Because the change of the relationship between the numbers of smaller metazoa and the viable counts, water qualities were able to clear under these inoculum conditions untill the maximum numbers of 100 numbers/ml (*A.hemprichi*) and 2,000 numbers/ml. (*P.roseola*) were obtained after about 10 days of culturing.

Since the optimum environments for the growth of the two species of smaller metazoa have been reported in detail as summarized in Table 1, cultivation conditions were set using these data in this study.

3) Evaluation methods of growth characteristics

Their growth rates are expressed as doubling time (td) or specific

Table 1 Optimum growth conditions
P. roseola and *A. hemprichi*

Items	*P. roseola*	*A. hemprichi*
Temp. (°C)	30	33
pH	6.0∿7.7	6.0∿7.7
Phosphate Buffer conc. (M)	1/50 >	1/375 >
Shear forces	Strong	Moderate
μmax(day^{-1})	0.39	0.42
Food habit	Wide	Narrow

growth rate (μ) in the logarithmic growth phase based on the fact that neither the space nor nutrient can be a limiting factor for growth during this phase.

As described already, the experimental culture period was set for 10 days, just before their growths reached to the stationary phase. Their growth rates in the monoxenic and mixed culture were evaluated by calculating the μ values during logarithmic growth phases according to the following expression: $\mu = 2.303 \log (N-No)/t-to$ where μ was the specific growth rate (day^{-1}), N was the population number at t day and No was the number at 0 day.

2-2-3 Measurements and Methods

1) Numbers of smaller metazoa

To obtain homogeneous suspension of smaller metazoa cultured liquid was stirred, and after that, a portion of the suspension (0.05 ml for *P. roseola* and 1 ml for *A. hemprichi*) was put on a lined slide glass and the numbers were measured using a light microscope of 40 magnification. The population was expressed as the average numbers/ml of medium

after 5 microscopic observations.

2) Viable counts of bacteria

The homogeneously mixed sample of 1 ml was transferred to a test tube containing 9 ml of sterilized pure water, cooled in on ice bath and treated with ultrasonic wave for 30 seconds. The sonic-treated sample was diluted stepwisely at a constant rate of 1/10, 1 ml of the dilution was put on a Petri dish and then a 10 ml portion of CGY (Casitone-Glycerol-Yeast extract)-agar medium was poured on the dish. After the agar medium had hardened, the plate was incubated at 20°C for 10 days and the colonies formed were measured as the viable counts of bacteria. Each of the data was expressed as the average of 3 plate cultures. The CGY agar medium was used in this study because the highest viable counts were obtained.

3) Biomass

Biomass was evaluated by measuring concentration of suspended solids in the culture broth by the following method. The sample was filtered through using a Millipore filter (Pore size: 0.45 μm) whose weight had been measured. After that, the filter was dried at 105°C for 2hr and the weight of the dried filter measured. Biomass was expressed as the mass weight before and after the filtration.

4) Floc size

Floc size was expressed as average floc surface area (μm^2) by measuring 30 times of its length and breadth using an ocular micrometer. A lined slide glass holding 0.1 ml of culture broth was used with no cover glass in the case of measurement.

5) Turbidity

Culture broth was stood still for 1 hr, and the turbidity of the supernatant was measured using a turbidity meter.

6) TOC

A suspension of homogeneously mixed culture broth and a supernatant of the culture after 1 hr of standing were used as samples, and they were applied to a TOC-analyzer (Model TOC-10B, Shimazu Seisakusho Ltd., Japan)

3. Results and Discussion

3.1 Smaller Metazoa at the Biological Treatment Plants

Many samples were collected from gray water biological treatment plants, and the samples were observed and counted about smaller metazoa. As shown in Table 2, various smaller metazoa were observed such as *Alona, Paracyclops, Cyclops, Asellus* of Arthropoda, *Lymnaea* of Gastropoda, *Aeolosoma, Nais, Pristina* of Oligochaeta and *Philodina, Cephalodella, Lecane* of Rotatoria.

Among them, Oligochaeta such as *A. hemprichi* and Rotatoria such as *P. roseola* were observed specifically at the highest frequency. Table 3 shows the rank of important species of Rotatoria and Oligochaeta. As can be seen in the table, *P. roseola* of Rotatoria and *A. hemprichi* of Oligochaeta were found to be the most important species in terms of their frequency and dominancy. In consequence, these two species were used in this study as the test strains of smaller metazoa.

Table 2 Important smaller metazoa appearing in biological treatment plant

Crustacea	*Paracyclops, Alona. Cyclops, Moina, Asellus*
Gastropoda	*Lymnaea, Physa*
Turbellaria	*Stenostomum*
Oligochaeta	*Aeolosoma, Pristina, Nais, Dero, Chaetogaster*
Rotatoria	*Philodina, Rotaria, Lecana, Cephalodella, Lepadella*

Table 3 Important Oligochaeta and Rotatoria appearing in biological treatment plant

Oligochaeta	Rotatoria
Aeolosoma	*Phiolodina*
Pristina	*Rotaria*
Nais	*Lecane*
Dero	*Cephalodella*

3.2 Growth Characteristics of Smaller Metazoa under Monoxenic and Mixed culture

Growth curves of *P. roseola* and *A. hemprichi* under monoxenic and mixed culture conditions are shown in Fig. 4. *P. roseola* showed almost the same growth patterns independent of the two culture systems. Its specific growth rate was 0.48 day^{-1} and the maximum numbers were about 2,000 numbers/ml in both systems. *A. hemprichi*, on the other hand, showed different growth patterns depending on the systems. Its specific growth rate in the monoxenic culture was 0.42 day^{-1}, but the rate was reduced to 0.34 day^{-1} when it was cultured under the mixed culture conditions. Also, the maximum numbers reached to 138 numbers/ml in the monoxenic culture, but reduced to 28 numbers/ml in the mixed culture.

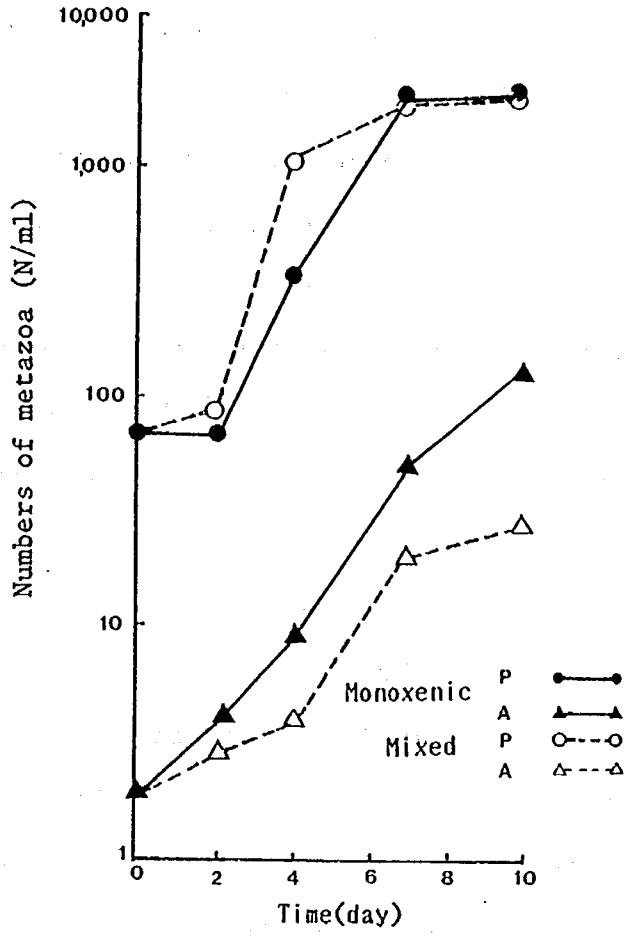


Fig. 4 Comparison of growth curves of *P. roseola* (P) and *A. hemprichi* (A) between monoxenic and mixed culture

Consequently, it was found that the growth characteristics were different between *P. roseola* and *A. hemprichi* in monoxenic and mixed culture whose feeding mechanism were not same each other. Among some possible reasons for this fact, difference in their food habits was found to be the most reasonable explanation. Namely, when *P. roseola* or *A. hemprichi* was cultured in the presence of one of the 7 different bacteria (see Table 4) as food, their growth rates changed greatly depending on the supplied bacteria (Table 5). *P. roseola* fed and grew on all the supplied bacteria, but *A. hemprichi* was able to feed only on a few specific bacteria. Decreases in the growth rate and maximum numbers of

A. hemprichi in the mixed culture seems to be explained by the food habit. Namely, *P. roseola* can feed on both inedible and edible bacteria for *A. hemprichi* by active abilities of swimming and creeping movement. If the food habit of *A. hemprichi* is wide, the growth rate and maximum numbers must be more large. Since a constant amount of substrate for bacterial growth was used in this study, the cell yield of bacteria in the medium would be constant and, therefore, the yield of smaller metazoa which feed on these bacteria would also be constnat independent of the monoxenic and mixed culture. In this mixed culture, however, *P. roseola* and *A. hemprichi* showed higher total numbers compared to those in the monoxenic culture, in spite of the same amount of the growth substrate. This will be explained as a result of increased feeding activity by their coexistence. This explanation can be supported by the fact, as will be described below, that the viable counts and biomass in 10 days cultured medium were remarkably low in the mixed culture compared to the monoxenic culture.

3.3 Role of Smaller Metazoa on Water Purification

Degree of suspension and residual organic matter in supernatant after biological treatment can be used as indicators on the capacity of water purification. Degree of suspension is measured by the turbidity and viable cells in culture supernatant, and residual organic matter in the supernatant is determined as the TOC value. The water purification characteristics using these indicaters were evaluated after 10 days of culturing. Because it was the period which the number of *P. roseola* and *A. hemprichi* reached to the maximum and water quality in the culture systems showed remarkable differences.

Table 4 Characteristics of Bacteria

Strain	Gram Stain	Color of Colony	Cell	
			Shape	Mobility
A	Positive	Yellow	Coccus	Presence
B	Positive	Yellow	Coccus	Absence
C	Negative	Red	Rod	Absence
D	Negative	Orange	Rod	Absence
E	Negative	Orange	Rod or Coccus	Absence
F	Negative	Creamy	Rod or Coccus	Absence
G	Negative	Red	Rod	Absence

Table 5 Comparison of μ between *A. hemprichi* and *P. roseola* supplied 7 kinds of bacteria as food

Kind of bacteria	*A. hemprichi*	*P. roseola*
A	0	0.37
B	0	0.32
C	0.22	0.24
D	0.10	0.17
E	0	0.33
F	0.23	0.33
G	0.32	0.16

(day $^{-1}$)

The turbidities of supernatant are shown in Fig. 5. These values in the B, BA, BP and BPA systems were 19.8, 10.7, 0.5 and 0.5, respectively. In the B system (bacteria alone), supernatant was remarkably turbid and the degree of turbidity was not so improved even when the detritus feeder *A. hemprichi* was grown in the system. On the other hand, in the *P. roseola* growing systems, transparency of supernatant was much improved as the result of filter feeding of the turbid suspended solids.

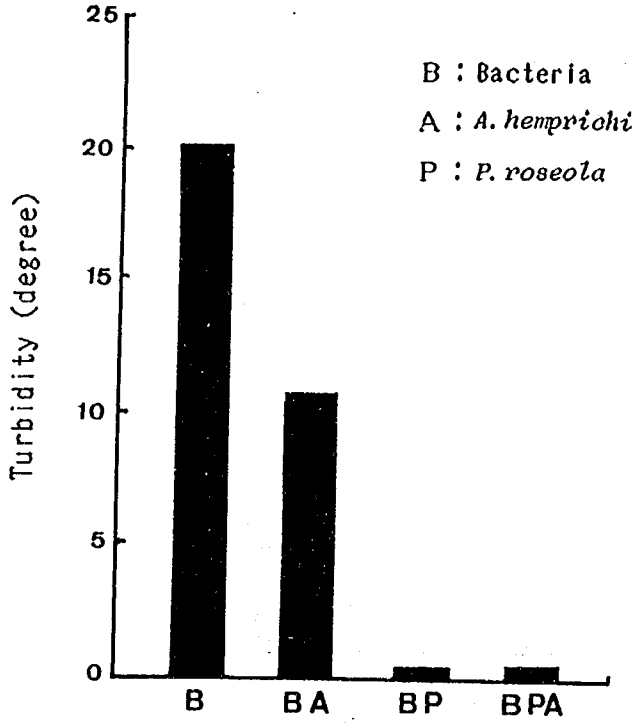


Fig. 5 Comparison of Supernatant Turbidity after 10 days cultivation between monoxenic and mixed culture

The final result after 10 days of culturing in the viable counts in supernatant are shown in Fig. 6. As can be seen in this figures, the degree of turbidity was directly proportional to the viable counts of supernatant. In other words, the turbidity becomes low as the result of the dispersed bacteria were fed by *P. roseola* which possesses characteristic filter feeding habit. Thus, the turbidity in the BP and BPA systems is reduced because of the growth of *P. roseola*.

TOC removal in the supernatant is shown in Fig. 7. Residual TOC in the supernatant was high in the B system because of the existence of large amount of dispersed bacteria, but was decreased in the presence of smaller metazoa by the flocculation and predatory effects. Especially, *P. roseola* improved transparence and reduced the residual TOC, because it was able to feed on the dispersed bacteria and cause the

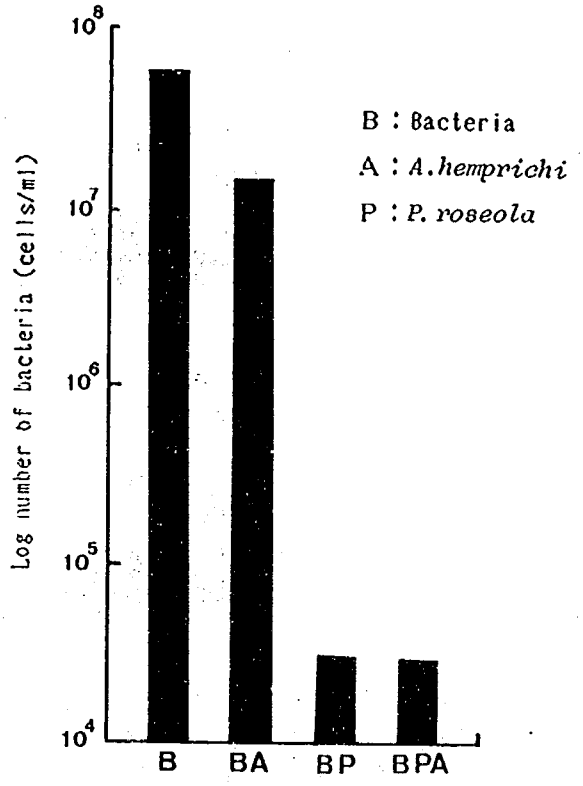


Fig. 6 Comparison of viable count in Supernatant after 10 days cultivation between monoxenic and mixed culture

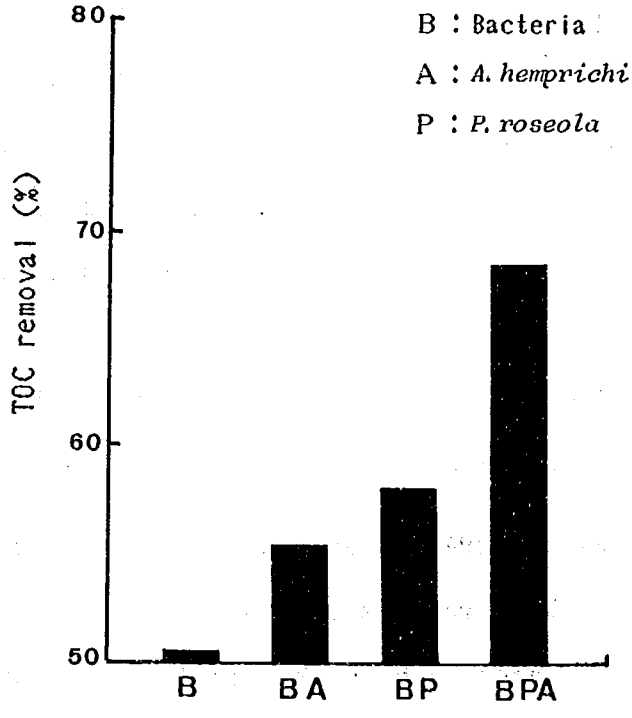


Fig. 7 Comparison of TOC removal after 10 days cultivation between monoxenic and mixed culture

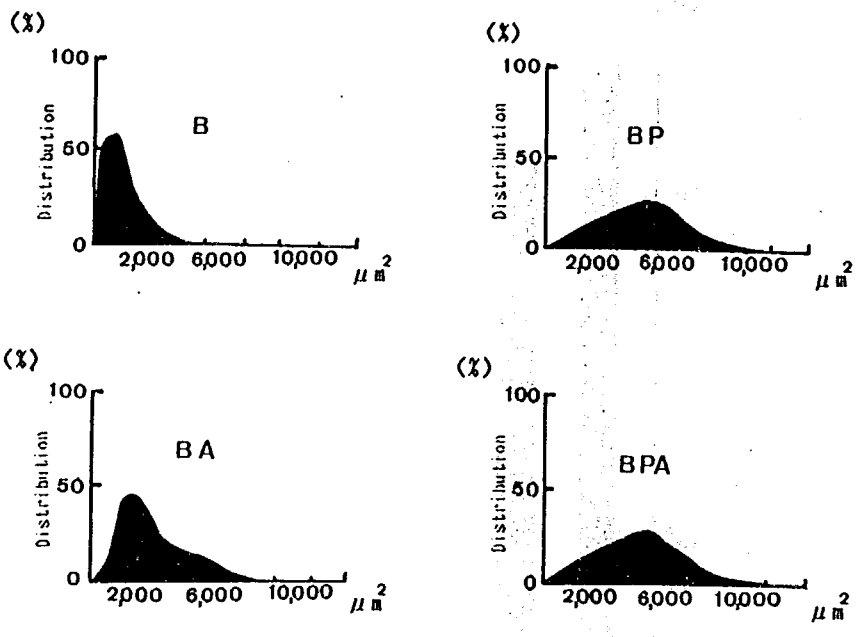


Fig. 8 Comparison of distribution of flock size after 10 days cultivation between monoxenic and mixed culture (B: Bacteria, A: *A. hemprichi*, P: *P. roseola*)

flocculation. In the mixed culture of *A. hemprichi* and *P. roseola*, efficiency of TOC removal was found to be improved further due to the increase in the bacterial and predatory activities. In consequence, the TOC removal increased in order of the B, BA, BP, BPA system. These results indicate that coexistence of *A. hemprichi* and *P. roseola* whose feeding mechanisms are different from each other, is quite important for the improvement of the efficiency of water purification.

Results on the distribution of floc area after cultivation for 10 days in the 4 systems are shown in Fig. 8. The dominant floc areas (μm^2) in the B, BP, BA and BPA systems were 1,000, 6,000 2,000 and 6,000, respectively. These results and Photo. 5, 6 indicate that the existence of smaller metazoa such as *P. roseola* is essential for the improvement of bacterial flocculation ability.

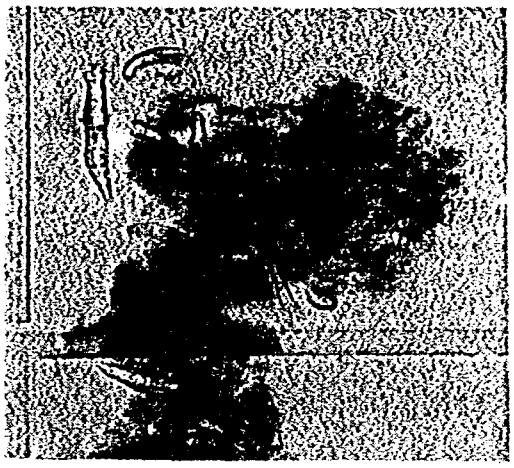


Photo 5 *P. roseola* existed in flock

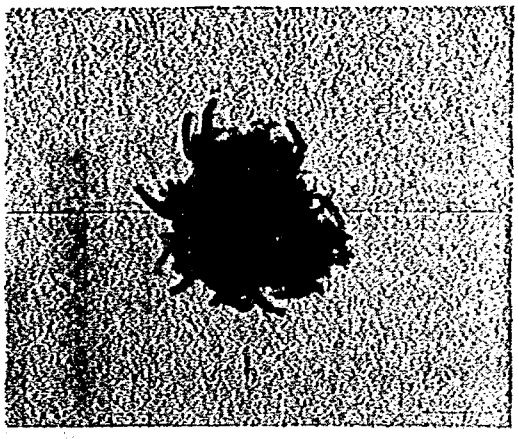


Photo 6 Role of *P. roseola* on flocculation

3.4 Role of Smaller Metazoa on the Reduction of Biamass

Biomass can be evaluated by measuring concentration of total suspended solid in culture suspension which was a medium containing high amount of organic matters such as the LE medium. The viable counts were about 10^8 cells/ml in the 4 culture systems at the starting period of the culturing. Remarkable differences of the counts were observed between the B system (bacteria alone) and other systems (containing smaller metazoa) after 4 days of culturing, especially at 10 days culturing. And, as shown in Fig. 9, these differences (rates of reduction of viable counts) became large in order of the BA, BP and BPA systems. Profile of the reduction pattern of biomass coincided with that of viable counts as the result of the feeding activity of smaller metazoa on bacteria.

Biomass in the 4 culture systems after 10 days of culturing is shown in Fig. 10. By comparing the both figure data between viable counts of Fig. 9 and biomass of Fig. 10. , a high correlation can be seen between the biomass and viable counts in the culture suspension. These

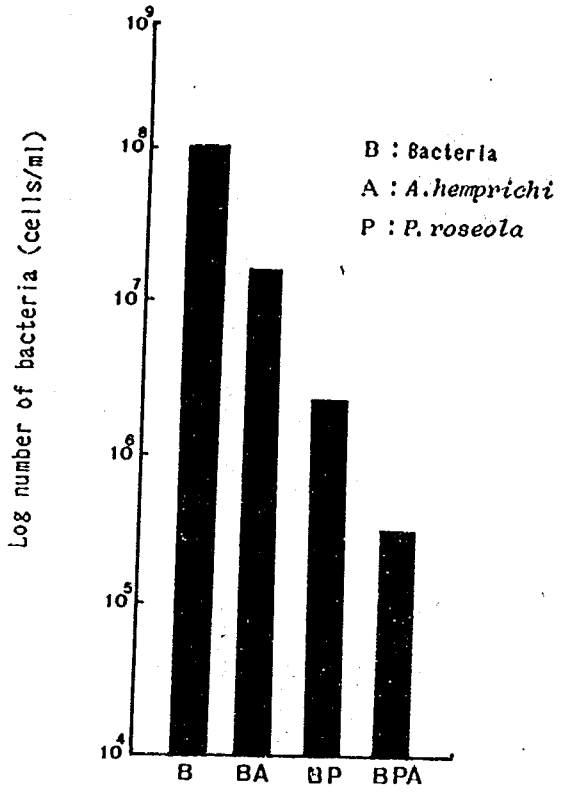


Fig. 9 Comparison of viable counts in Suspension after 10 days cultivation between monoxenic and mixed culture

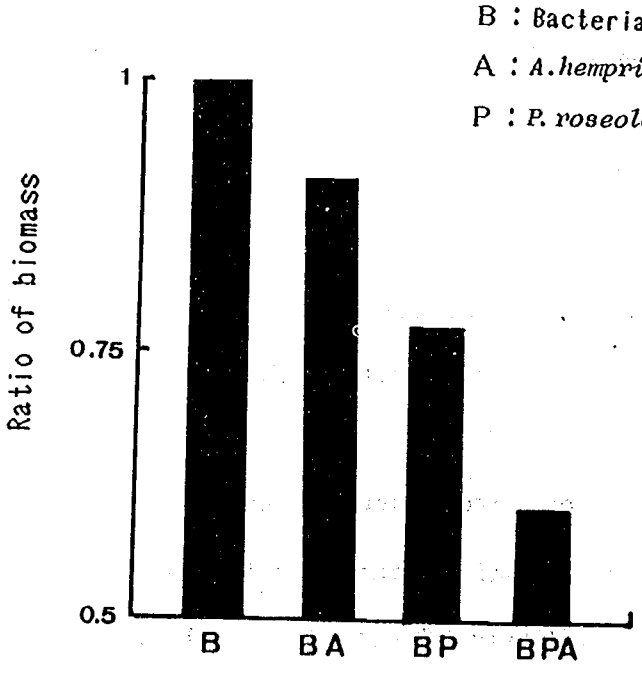


Fig. 10 Comparison of biomass after 10 days cultivation between monoxenic and mixed culture

results indicate that the reduction of biomass occurs when bacteria are fed by their predator and the reduction effect by *P. roseola* is higher than that by *A.hemprichi* in the monoxenic culture and the reduction occurs most effectively in their mixed culture. The remarkably low levels of viable counts and biomass in the mixed culture will be explained as a result of increased predatory activity induced by the coexistence of smaller metazoa having different feeding mechanisms and food habits.

In conclusion, the role of *P. roseola* on the reduction of biomass is larger than that of *A.hemprichi*, but the effect is improved further by the coexistence of both smaller animals.

4. Summary

The data obtained in this study are summarised and depicted in Fig. 11 with regard to the roles of a typical detritus feeder (*A. hemprichi*) and a filter feeder (*P. roseola*) which are the importnat smaller metazoa in the biological treatments process.

(1) In the B system, the turbidity is high and the efficiency of organic matter removal is low because of existence of the large amount of dispersed bacteria. (2) In the BA system, the amount of biomass decreases because the detritus feeder (*A. hemprichi*) feeds on the detritus, whereas the turbidity is still high and the efficiency of organic matter removal is not so high because of the presence of dispersed bacteria. (3) In the BP system, the biomass and turbidity decrease and the efficiency of organic matter removal increases, since the filter feeder (*P.roseola*) feeds on the dispersed bacteria.

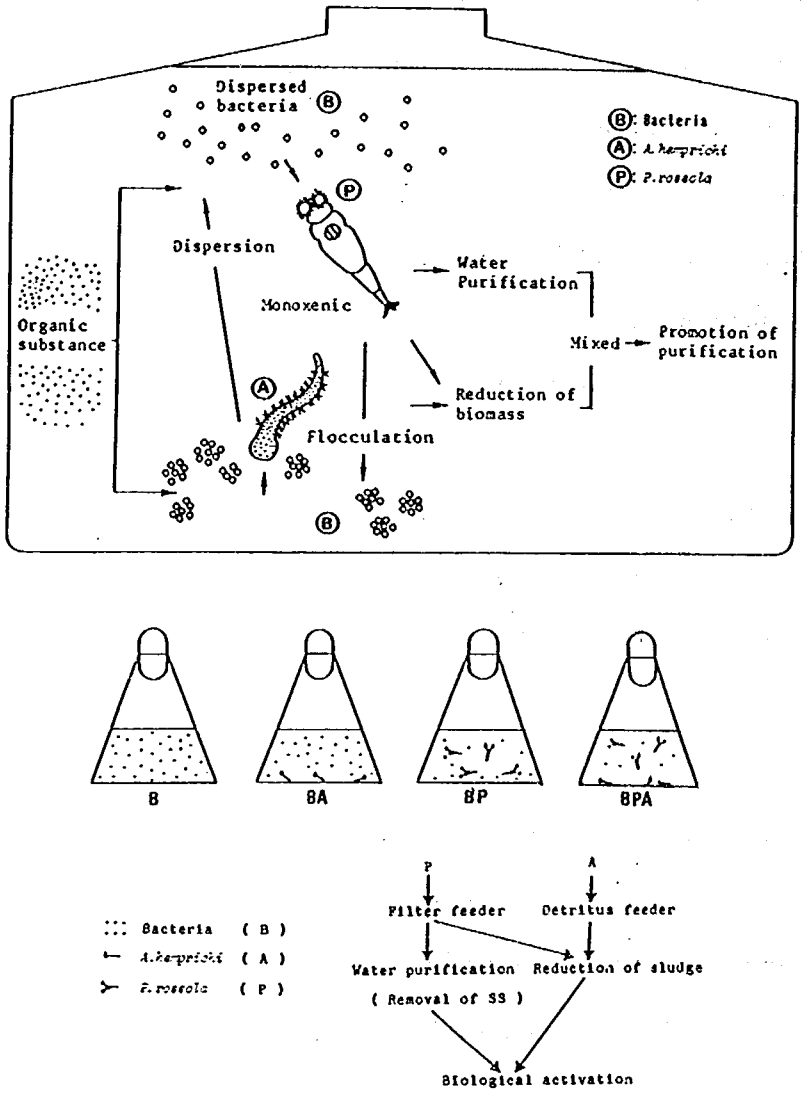


Fig. 11 Role of smaller metazoa on biological wastewater treatment

(4) In the BPA system, the biomass decreases most remarkably along with the turbidity reduction and improved efficiency of organic matter removal, because the coexistence of *A. hemprichi* and *P. roseola* whose feeding mechanisms and food habits differ from each other, promotes their predatory activities on the dispersed and flocculated bacteria.

It can be concluded that the detritus and filter feeder smaller metazoa in biological treatment are contributing greatly to water purification and reduction of biomass.

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APPLICATION OF ANAEROBIC FIXED BED PROCESS TO DOMESTIC WASTEWATER TREATMENT FOLLOWED BY MEMBRANE SEPARATION

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1. INTRODUCTION

Almost all municipal sewage is now being treated by the activated sludge process in Tokyo. This process requires a large amount of energy for aeration and produces a vast volume of excess sludge. In process of the municipal sewage treatment, one of the biggest problems is sludge treatment and disposal, especially in a densely populated city such as Tokyo. It is, therefore, important to develop a new treatment process which saves energy costs and produces a small amount of excess sludge.

In recent years anaerobic treatment has become accepted as an effective means for treating wastewater with the following advantages over aerobic treatment systems (Ramalho, 1983):

(1) Since aeration equipment is not utilized, capital costs and power consumption for aeration are saved.
(2) The biomass yield coefficient Y for the anaerobic process is much lower than that for aerobic systems. This means that less biomass is produced per unit of removed substrate and consequently there may be considerable savings in the processes for handling and disposal of excess sludge.
(3) The production of methane in anaerobic processes is advantageous owing to its fuel value.

Anaerobic treatment, however, has limitations and drawbacks. Some of the disadvantages of anaerobic over aerobic treatment are as follow (Ramalho, 1983):

(1) Longer retention time is required. Consequently, the capital investment in the vessel is higher for anaerobic treatment.
(2) Bad odors associated with anaerobic processes, mainly due to the production of sulfide and mercaptans, may constitute a serious limitation, particularly in urban areas.
(3) Higher temperatures are required for anaerobic processes to proceed at reasonable rates. Usually the optimum temperature for anaerobic processes is around 35°C, which means that preheating

the feed or heating the anaerobic reactor may be required.

(4) Sedimentation of anaerobic biomass in the secondary clarifier is more difficult than settling of biomass in the activated sludge process.

If these defects can be overcome, anaerobic treatment should become suitable for municipal sewage treatment.

Since the early 1960s, ultrafiltration has been recognized as efficient separation technique in the food, drug, and chemical process industries as well as for wastewater treatment (Bhattacharyya et al., 1974; Goldsmith et al., 1974; Pearson et al., 1976; Bhattacharyya et al., 1977; Bhattachryya et al., 1978; Bhattacharyya et al., 1979; Klemeston & Scharbow, 1979; Christensen & Plaumann, 1981; Belfort, 1984; Strathmann, 1984). Ultrafiltration has also been used in fermentation industries for preventing microorganisms and enzymes being washed out of the reactor. For example, to increase the productivity of acetone-butanol fermentation, Pierrot et al. (1986) used a hollow-fiber ultrafilter for separating and recycling cells in a continuous fermentation of *Clostridium acetobutylicum*. This type of reactor, called a membrane reactor, has also been applied to wastewater treatment. In the early 1970 s , aerobic biological treatment system consisting of a suspended growth biological reactor followed by a membrane ultrafiltration process was developed and patented. Recently, this system has been used for wastewater treatment and reuse in large buildings in Japan (Tsuchiya, 1985; Hikasa, 1985). The ultrafiltration process provides a positive means of separating solids and liquid, being independent of most of the factors which cause solids loss from conventional activated sludge systems resulting in aerobic system failures. The concept has been developed for anaerobic treatment applications, and patented by Dorr-Oliver. Fig. 1 is a schematic of the anaerobic membrane process, called the MARS process (Membrane Anaerobic Reactor System). The defects of anaerobic treatment may be overcome by using this process. The advantages of the MARS process are summarized as follows:

(1) Higher treatment rate may be expected by increasing the concentration of microorganisms in the anaerobic reactor.

(2) The washing out of methanogenic bacteria which have a slow growth rate will be prevented regardless of the hydraulic retention time (HRT).

(3) Since solid-liquid separation is complete, the effluent is clear enough for reuse.

(4) Dissolved organic matter with high molecular weights is also eliminated from the effluent.

(5) Disinfection of the effluent is not necessary, because pathogenic microorganisms are eliminated through the membrane.

Though several basic research studies have been made on the MARS process (Grethlein, 1978; Sutton & Evans, 1983), to date, there are no studies on the potential, reliability, and operational costs of the MARS process used in a pilot plant over an extended period.

The department of Sewage Works, Tokyo Metropolitan Government, have conducted research using a pilot plant to clarify the performance of the MARS process. Though this research is still continuing we have reached certain conclusions. We herein provide an outline of our findings.

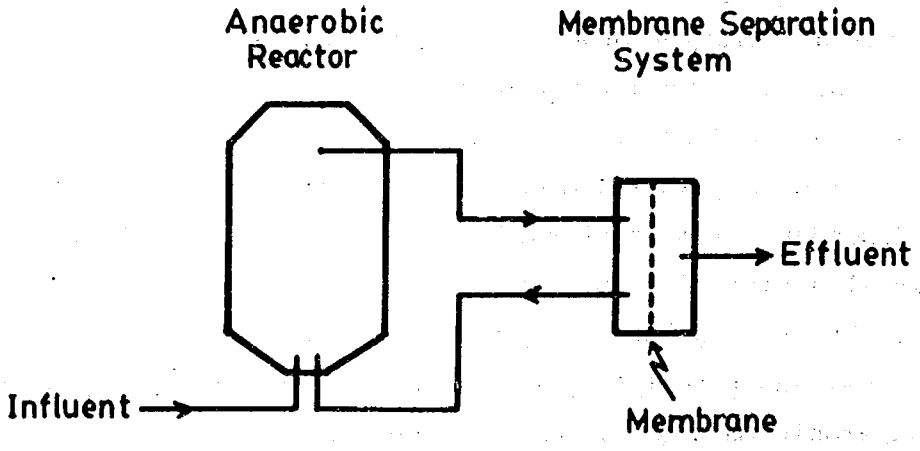


Fig. 1. Membrane anaerobic reactor system.

2. PILOT PLANT EXPERIMENT

2.1 DESCRIPTION OF THE PILOT PLANT

Figs. 2 and 3 are an overview and schematic diagram of the pilot plant, respectively. An upflow anaerobic filter was used as anaerobic reactor instead of a suspended growth biological reactor which was used in the original MARS process, to reduce the load of suspended solids on the ultrafilter (UF) membrane. The anaerobic filter used had a nominal volume of 3.3 m^3 and height of 290 cm, with a packed bed volume of 2.2 m^3. Fine gravel was filled at the bottom of the anaerobic filter up to 50 cm in height, and coarse sand particles 1.3 mm in diameter were filled above the gravel layer up to 220 cm in height. The filter bed was stirred by a spiral screw at regular intervals to prevent plugging of the filter bed. Settled municipal sewage was fed to this filter by a feed pump (P1). The anaerobic filter effluent was fed to a tubular membrane system by a pressure pump (P2). A schematic of the construction of the tubular membrane system is given in Fig. 4. The system had 60 modules. An exterior view of the UF module is given in Fig. 5. Polyacrylonitrile tubular ultrafilter (2.4 cm in diameter and 300 cm in length with a molecular weight cutoff of 13,000) was equipped for each module. The total area of the UF membrane was 13.56 m^2. The anaerobic reactor effluent was circulated in the modules by circulating pump (P3) to prevent gel formation and fouling of the membrane surface. The concentrated solution in the modules was continuously returned to the anaerobic filter through return pump (P4). At regular intervals, the inner surface of the UF membrane was mechanically cleaned by passage of over-sized sponge balls through the membrane tubes. During mechanical cleaning, the anaerobic reactor effluent which was previously stored in the cleaning tank was circulated in the UF module by the cleaning pump (P5). After cleaning, the water in cleaning tank was fed to the anaerobic reactor by the cleaning water return pump (P6), and then new anaerobic reactor effluent was stored again. All processes were controlled automatically by a programable controller.

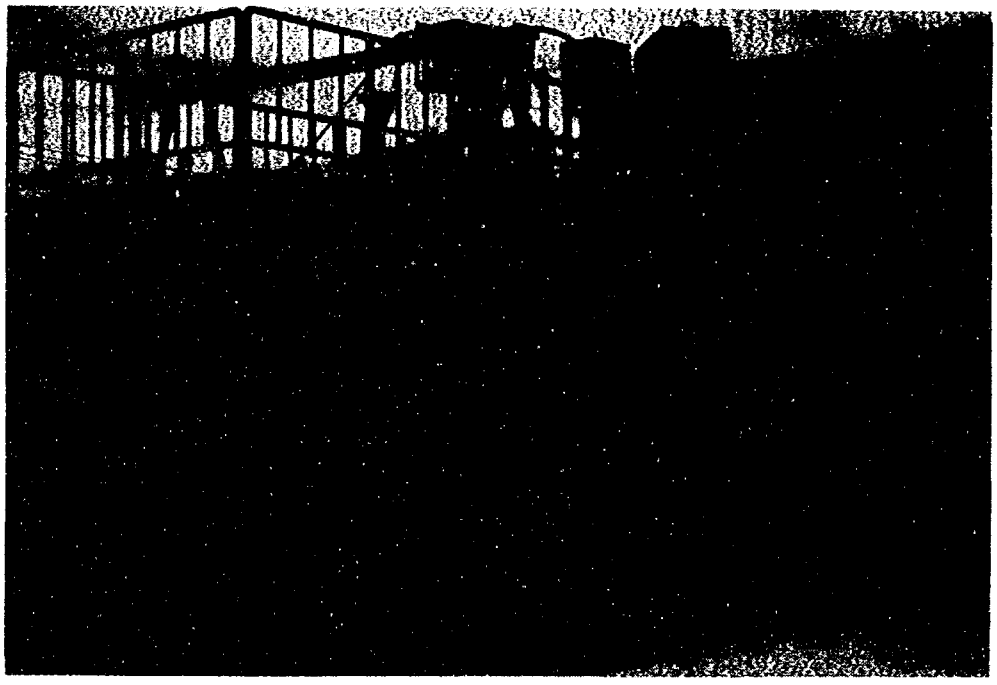


Fig. 2. Overview of the pilot plant.

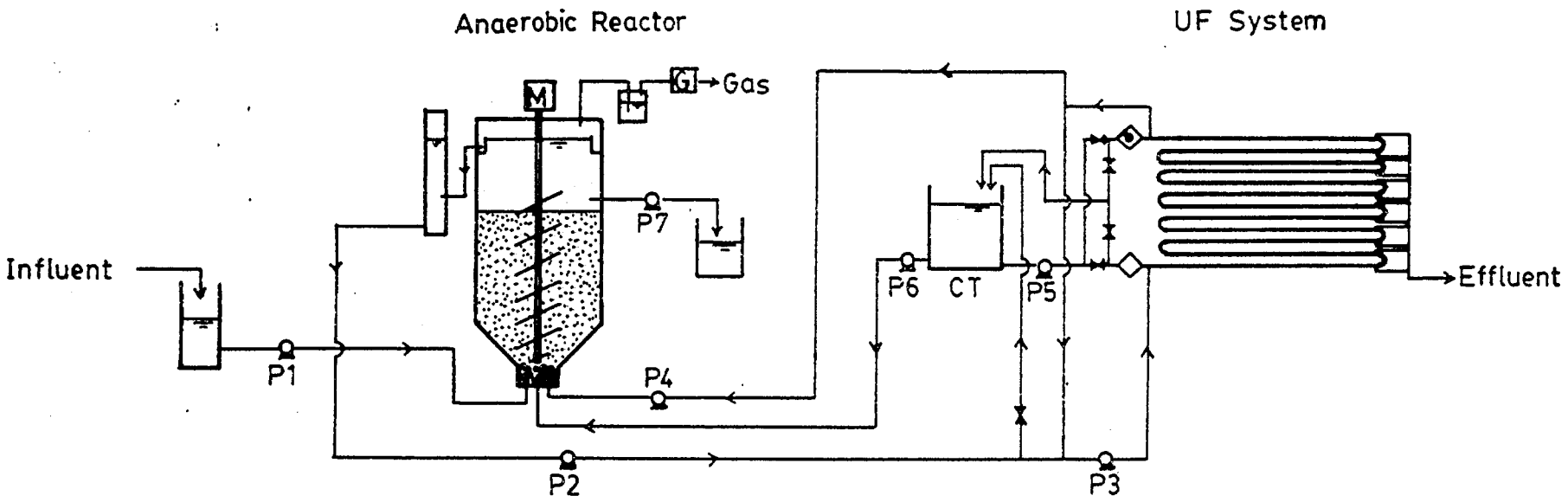


Fig. 3. Schematic diagram of the pilot plant.

P1: Feed pump
P2: Pressure pump
P3: Circulating pump
P4: Return pump
P5: Cleaning pump
P6: Cleaning water return pump
P7: Sludge discharging pump
CT: Cleaning tank
M: Stirring motor

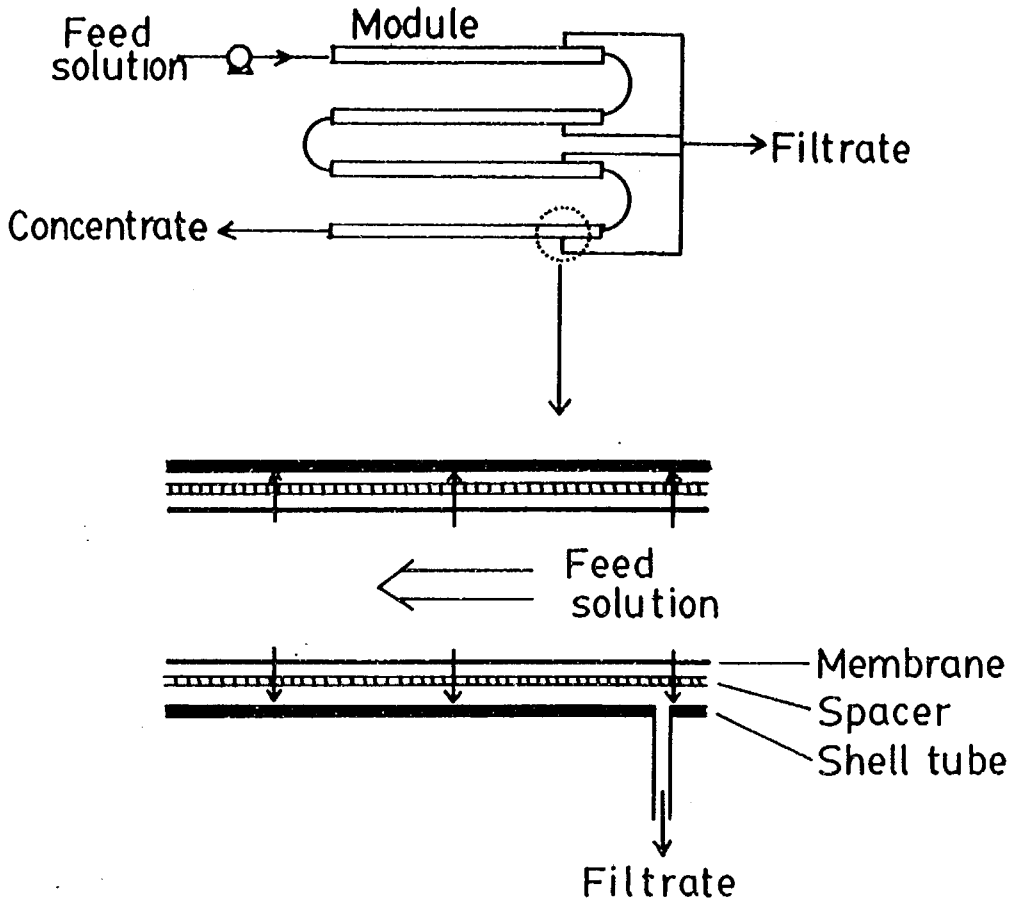


Fig. 4. Construction of the tubular membrane system.

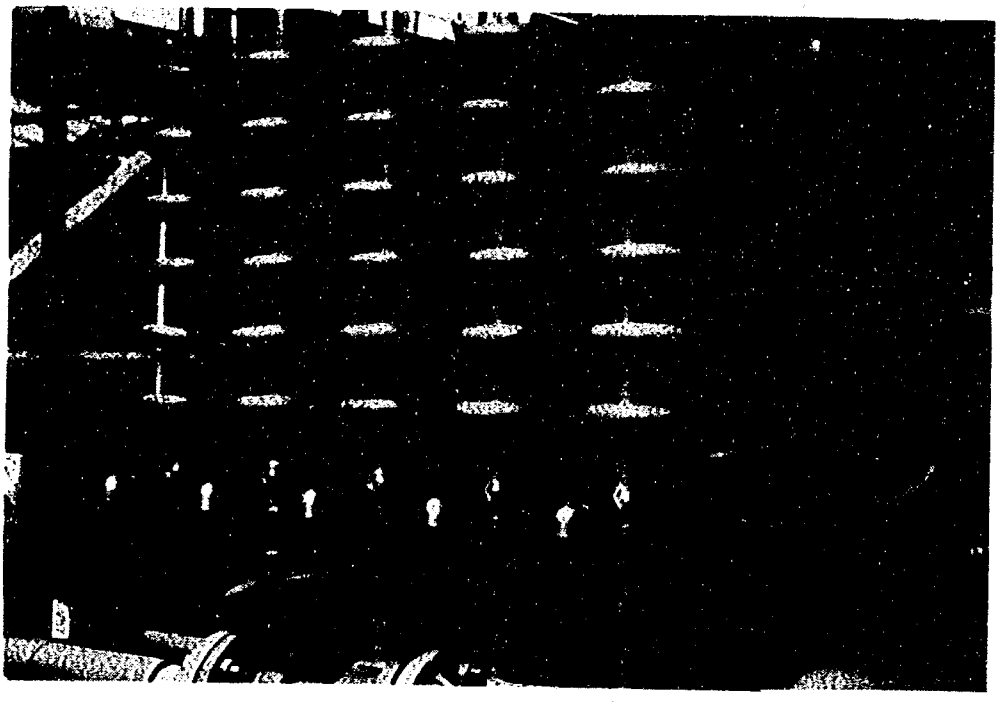


Fig. 5. Exterior view of the UF module.

2.2 OPERATION OF THE PILOT PLANT

Only the anaerobic reactor was operated from August 6 1985 to October 15 1985 to establish sufficient biological population in the anaerobic reactor, and then the reactor and the membrane separation system were combined and operated simultaneously.

Fig. 6 shows the operational condition of the anaerobic reactor. The water temperature changed in the range from 8 to 22°C. The minimum temperature was observed in February. The hydraulic retention time (HRT) setted for about 10 hours during the initial month, but then it lengthened to about 20 hours because the flux of the UF membrane declined markedly. The average reactor volumetric loadings of BOD and SS were 0.18 $kg/m^3.day$ and 0.08 $kg/m^3.day$, respectively. The filtered bed was stirred with the spiral screw for 30 minutes every day till March 3, and then 5 minutes per day.

Sludge in the anaerobic reactor was not discharged throughout whole experimental period. As shown in Fig. 7, the concentration of MLSS at about 20 cm above filter bed increased markedly after January, reached maximum (29,000 mg/ℓ) in March, and then declined. This indicates that the rate of sludge autolysis exceeded the rate of production and accumulation of sludge after March, probably because of a rise in water temperature.

Fig. 8 shows the water quality of a diurnal composite sample. Influent SS, COD-Mn and BOD concentrations were about 50 to 75 mg/ℓ, 50 to 80 mg/ℓ, and 100 to 150 mg/ℓ, respectively. The values of SS, COD and BOD in the anaerobic reactor effluent increased till March and then declined. The values were considerably higher than those of influent except in the initial stage. Though there was no detectable SS in the UF filtrate immediately after filtration, small amounts of SS (average concentration fo 2.2 mg/ℓ) were detected in the diurnal composite sample. The formation of colloidal sulfur from sulfide after filtration must be one of the causes of this phenomenon. The concentration of COD in the UF filtrate was stable throughout the

experimental period, and that of BOD increased gradually till March and then declined. The average concentrations of COD and BOD in the UF filtrate were 28 and 49 mg/ℓ, respectively. As shown in Fig. 9, the removal rate of SS was higher than 95% throughout the experimental period. Those of COD and BOD were about 50 to 70%.

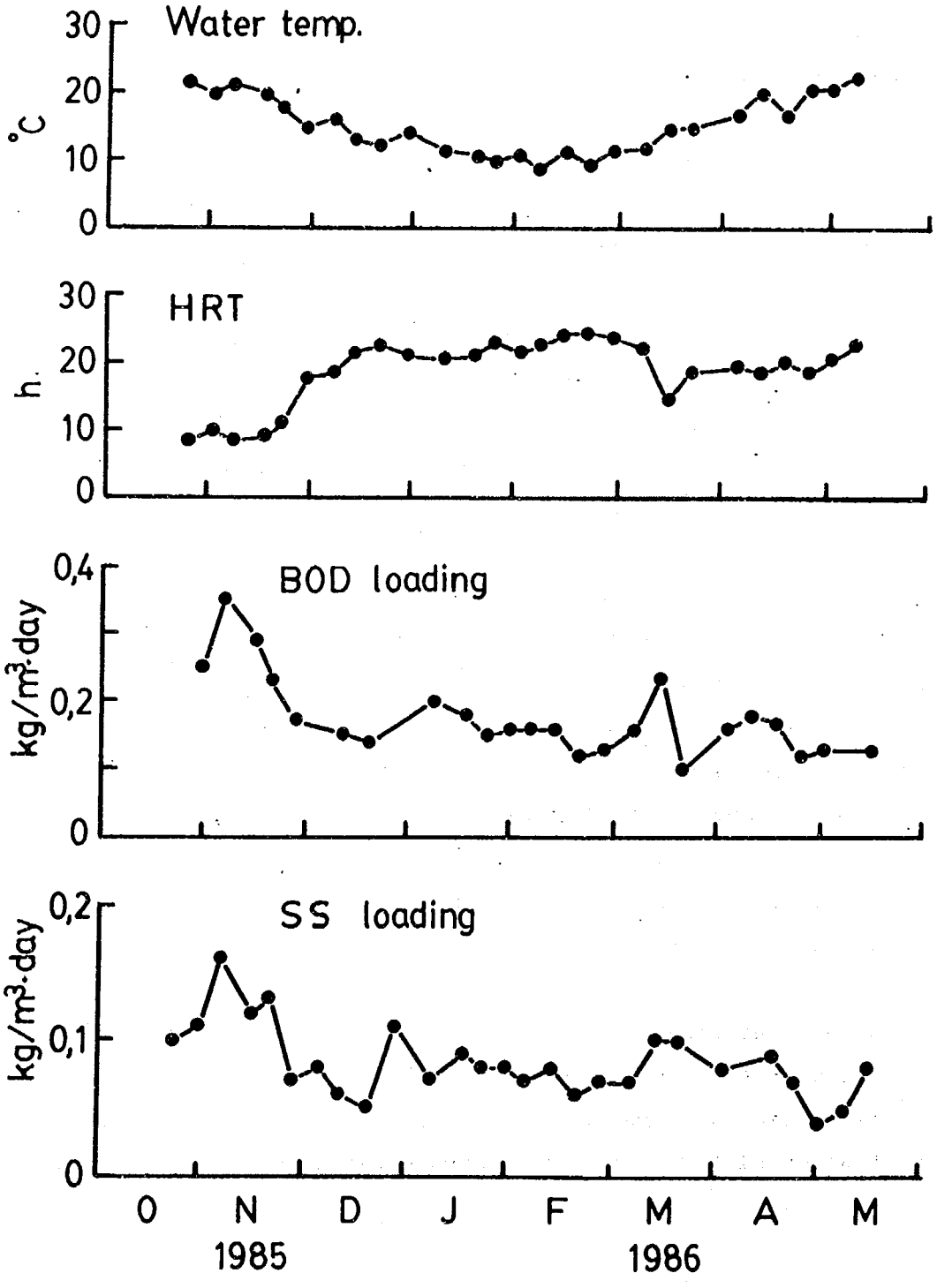


Fig. 6. Operating conditions of the anaerobic reactor.

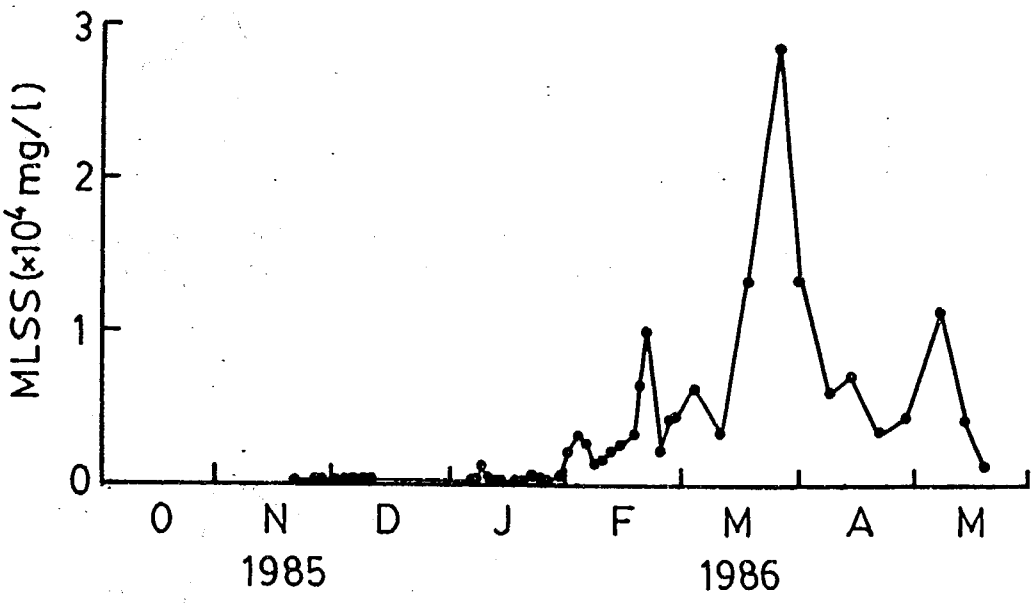


Fig. 7. Change of MLSS concentration in water layer of anaerobic reactor during the experimental period.

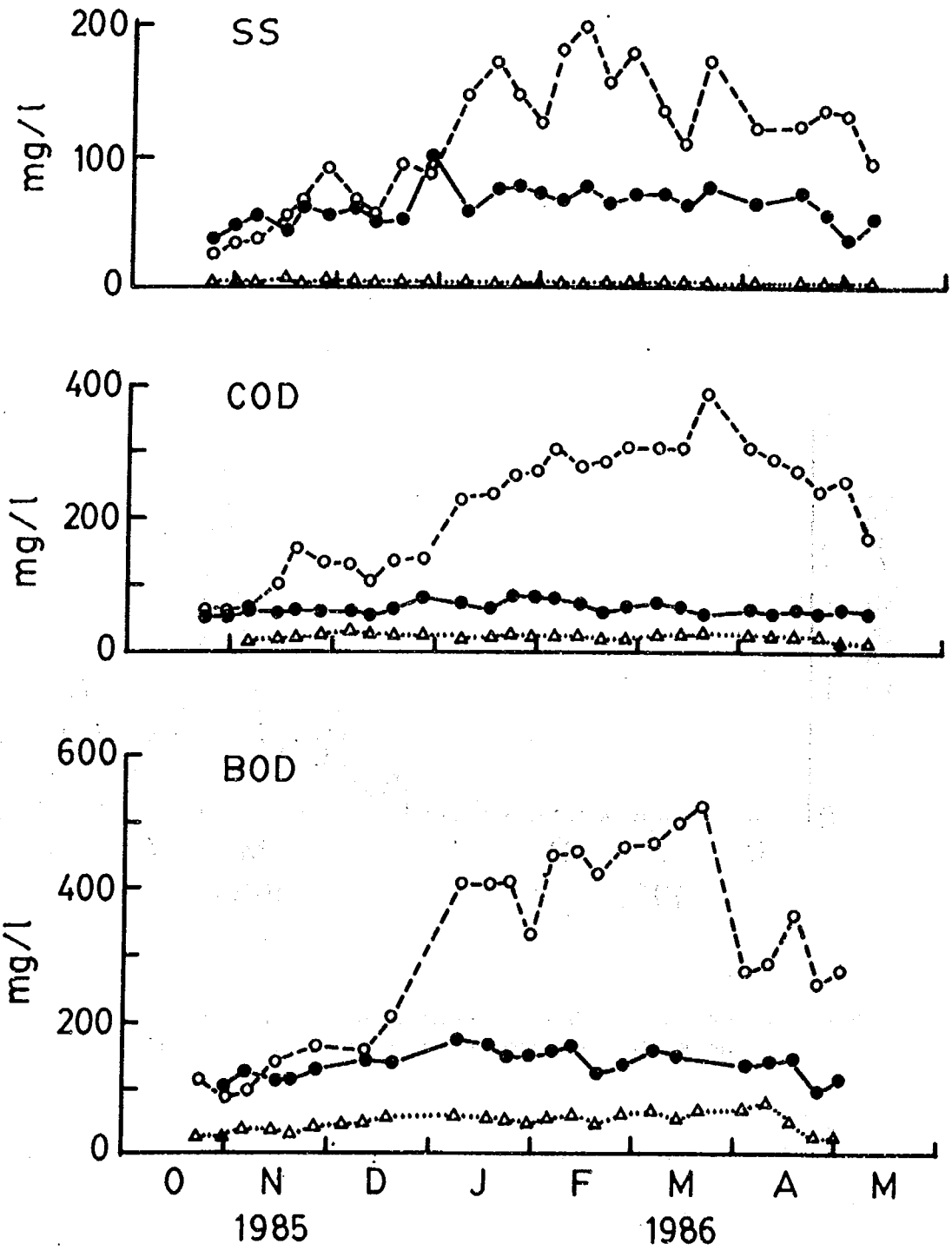


Fig. 8. Changes in SS, COD, and BOD during the experimental period.

- ● Influent
- o Anaerobic reactor effluent
- Δ Ultrafilter filtrate

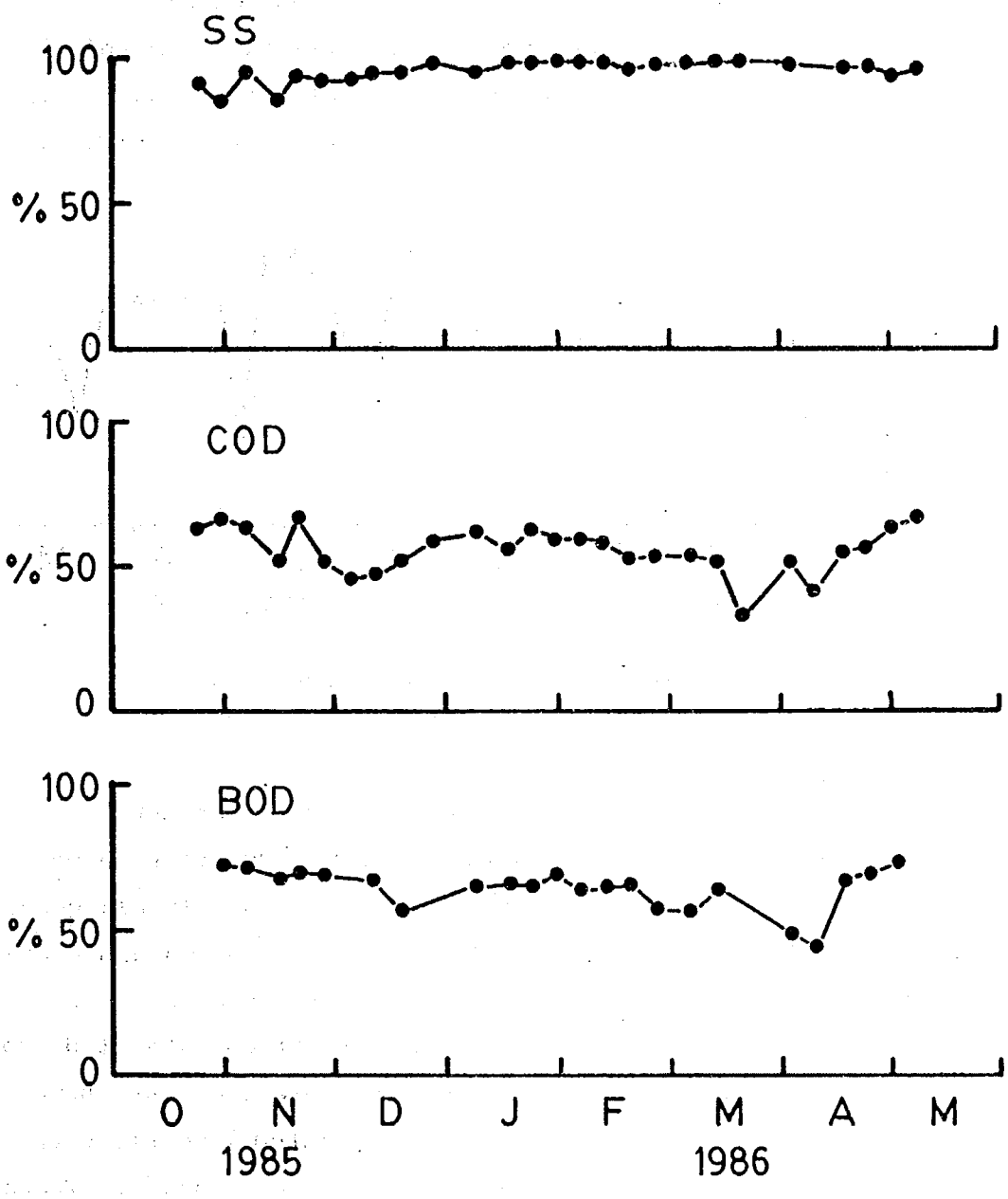


Fig. 9. Removal rates of SS, COD, and BOD.

Effluent COD and BOD were affected considerably by sulfide which was contained in the effluent at about 10 to 20 mg/ℓ (Fig. 10). The sulfide disappeared after 5 minutes aeration, and following this COD and BOD dropped by about 5 and 20 mg/ℓ, respectively. Therefore, the average values of COD and BOD in the UF filtrate after the removal of sulfide were estimated to be about 23 and 29 mg/ℓ, respectively.

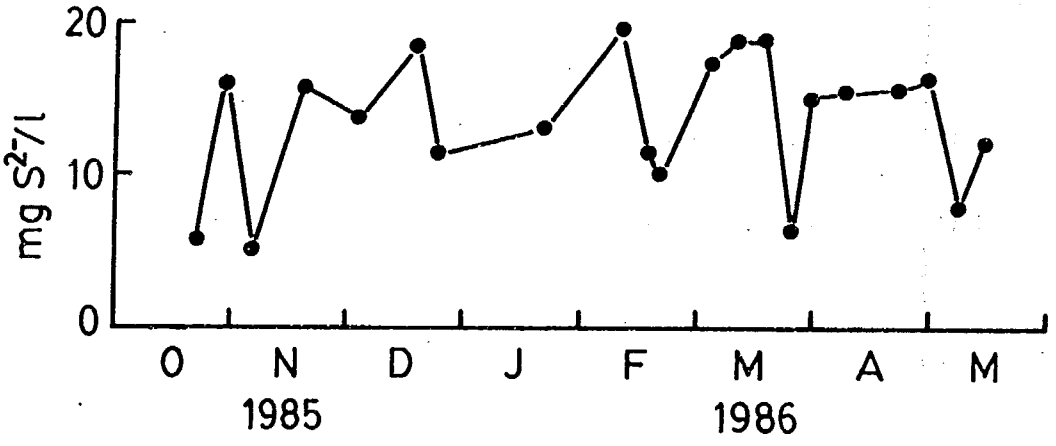


Fig. 10. Change in sulfide concentration in UF filtrate during the experimental period.

Fig. 11 shows the concentration and production of methane. Methane concentrations in both generated gas and UF filtrate were constant till March and then increased markedly. Nearly all generated methane was discharged from the pilot plant as dissolved gas contained in the UF filtrate. The recovery of methane from the UF filtrate is necessary for using methane as fuel. The generation rate of dissolved methane was also stable till March and then increased. As shown in Fig. 12, the rate of methane production correlated linearly with the water temperature after the HRT rose to about 20 hours. Therefore, an increase in the water temperature should be the cause of the increase in the generation rate after March.

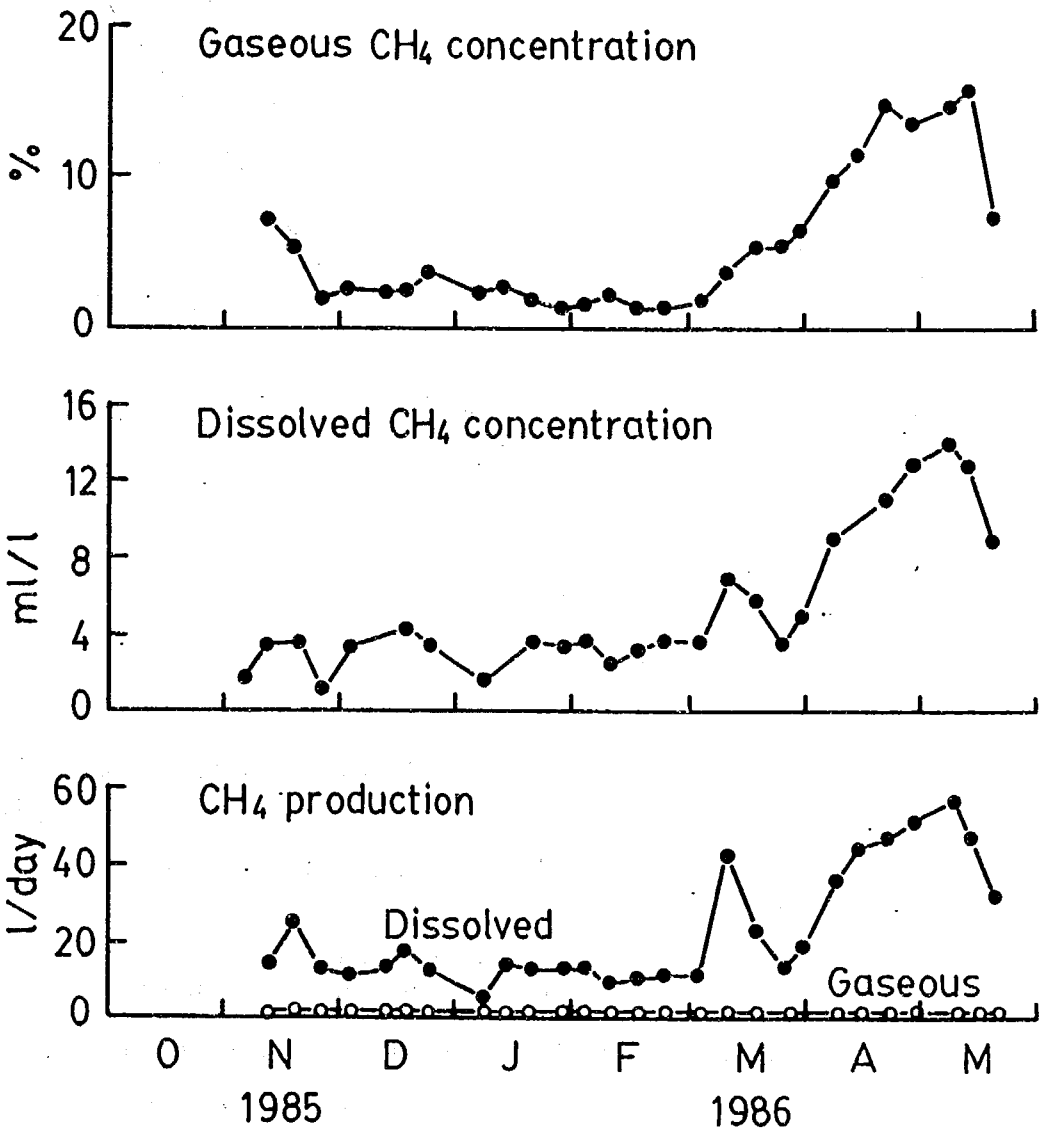


Fig. 11. Concentration and generation rate of methane.

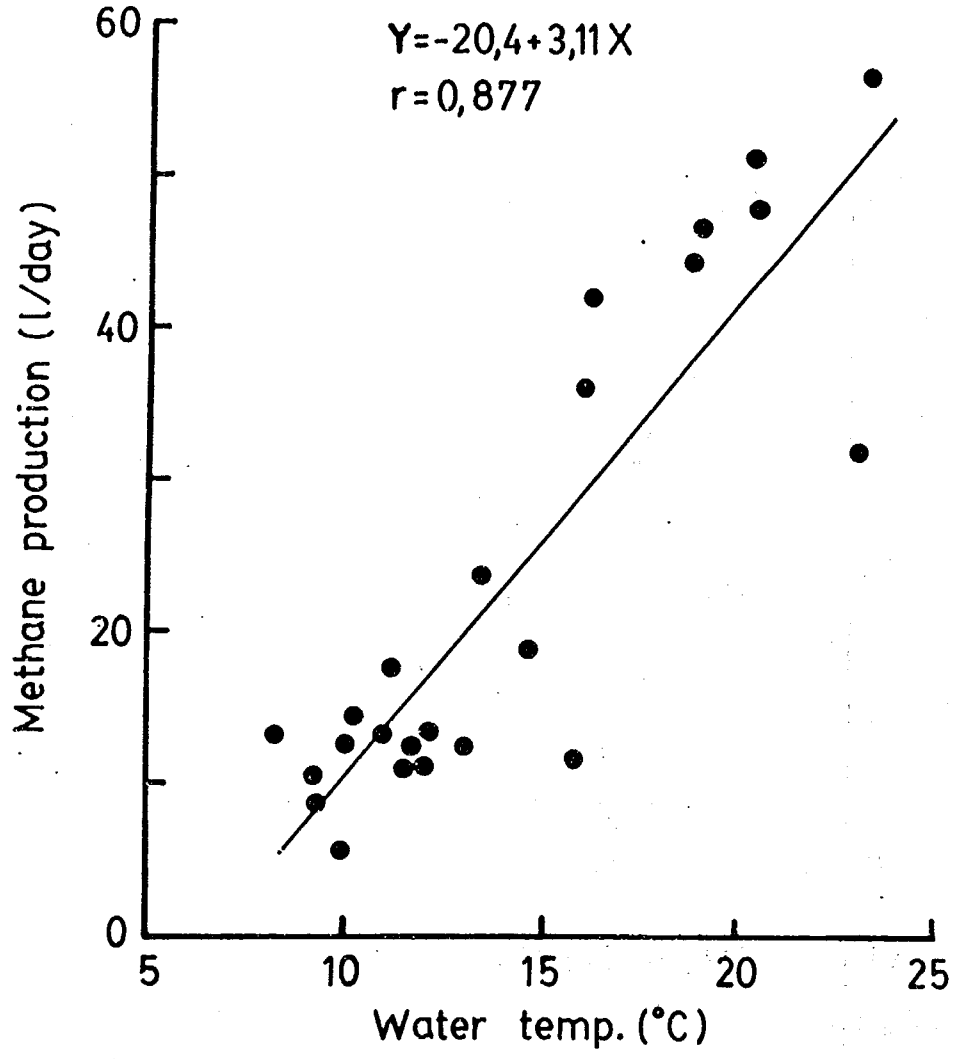


Fig. 12. Relationship between temperature and generation rate of methane.

Fig. 13 shows the operating conditions of the UF system. The operating pressure for water permeation through the membrane setted at about 0.5 kg/cm^2 at first, but rose gradually to about 2 kg/cm^2 because the membrane flux declined with time. The circulating velocity past the membrane surface was about 1 m/s at first and rose to 2 m/s after about a month. The flux (ℓ/hr.m^2.atm at 25°C) declined rapidly during first 2 months and then became constant. Mechanical cleaning by the sponge balls was done at 2-hour intervals from October 16 to December 18, 4-hour intervals from December 19 to March 3, and then

8-hour intervals. At each cleaning time, the sponge balls passed through the UF module four times.

Chemical cleaning of the UF membrane was done for 1 hour at December 29 1985 using a mixed solution of EDTA (0.2%) and sodium peroxyborate (0.4%). No recognizable effect, however, was observed with the cleaning. Low water temperature may be one of the causes of this lack of effect.

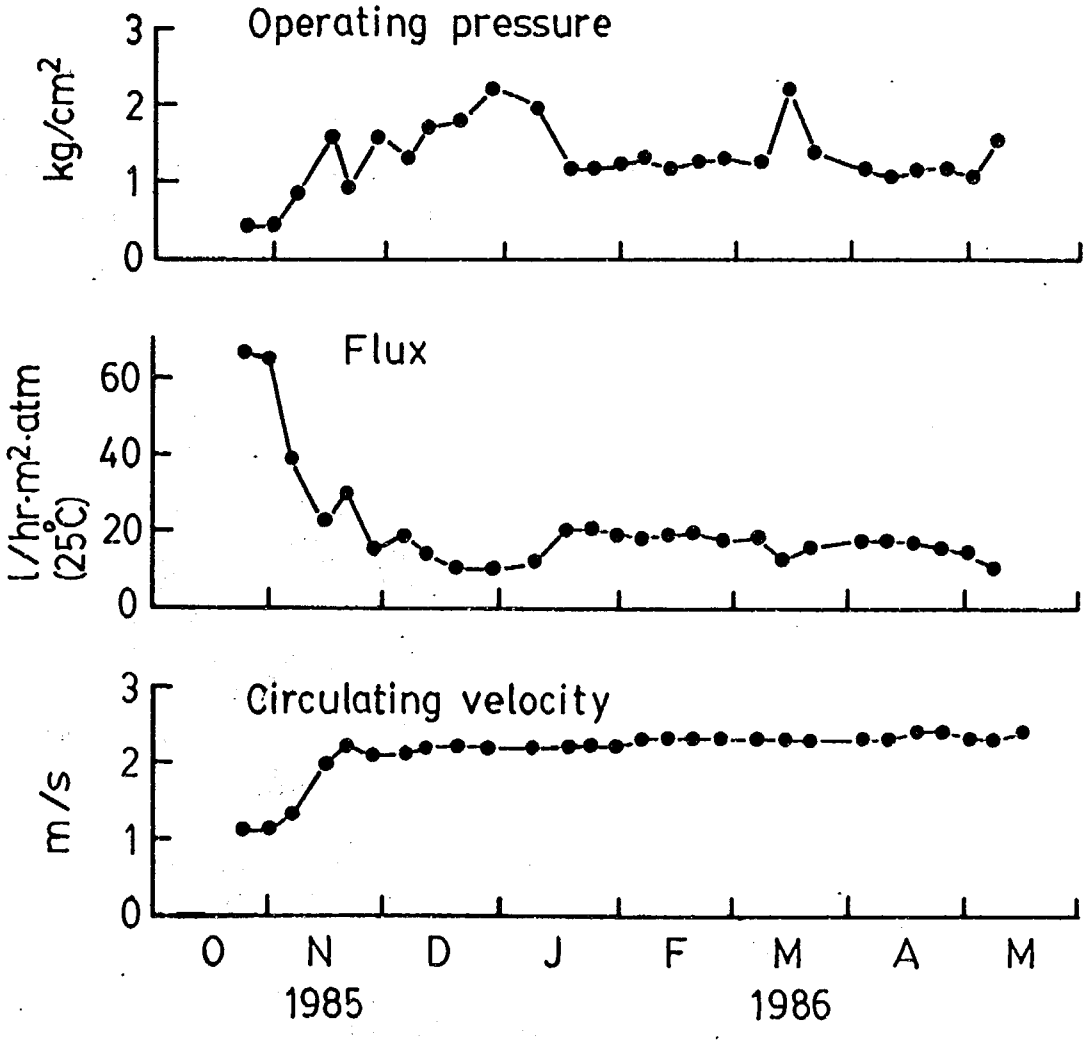


Fig. 13. Operating conditions of membrane separation system.

2.3 CHEMICAL ANALYSES OF ORGANIC MATTER

For clarifying the change in concentration and composition of dissolved organic matter by the anaerobic treatment and the UF filtration, chemical analyses were made for several typical organic matters.

Analyses of the organic acids with ICE (Ionchromatography Exclusion) revealed that mainly acetic acid was formed during anaerobic treatment. Though formic acid, propionic acid, and butyric acid were also detected, their concentrations were considerably lower than that of acetic acid. Acetic acid concentration of both anaerobic reactor effluent and UF filtrate increased gradually till April and then dropped rapidly (Fig. 14). According to McCarty (1964, 1981), about 70% of the methane generated from organic matter is produced via the methyl group of acetic acid. Therefore, the drop in acetic acid concentration after April seems to be caused by an increase of methanogenic activity (Fig. 11).

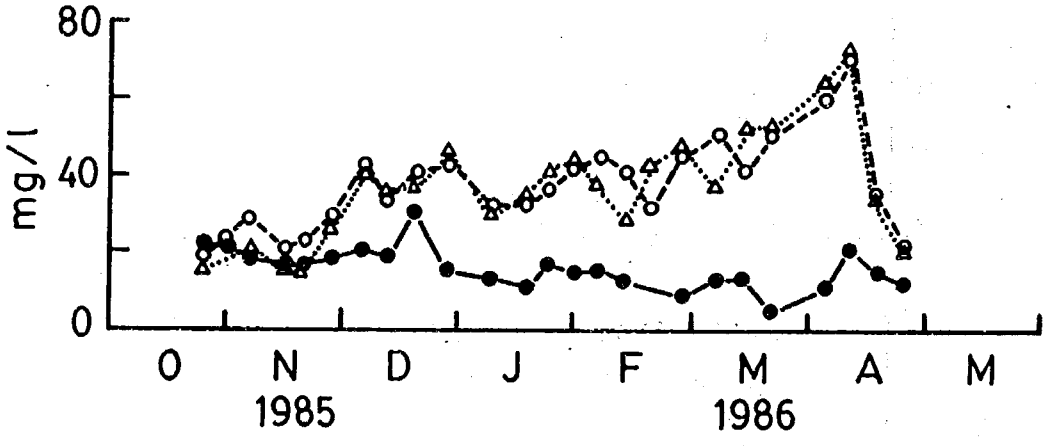


Fig. 14. Change in acetic acid concentration during the experimental period.

- ● Influent
- o Anaerobic reactor effluent
- Δ Ultrafilter filtrate

Acetic acid may permeate the UF membrane freely, because no marked difference in the concentration was observed between the anaerobic reactor effluent and UF filtrate. The average concentration of acetic acid in the UF filtrate was 34 mg/ℓ. Schulze-Forster & Gad (1938) have reported that 1 mg of acetic acid corresponds to 0.7 mg of BOD. Therefore, it can be estimated that 34 mg/ℓ acetic acid corresponds to 24 mg/ℓ BOD. As stated previously, the average BOD value of the UF filtrate not including the effects of sulfide was about 29 mg/ℓ. Therefore, it is assumed that acetic acid is the main component of carbonaceous BOD in the UF filtrate. The good correlation between acetic acid and BOD for effluent (Fig. 15) also supports this.

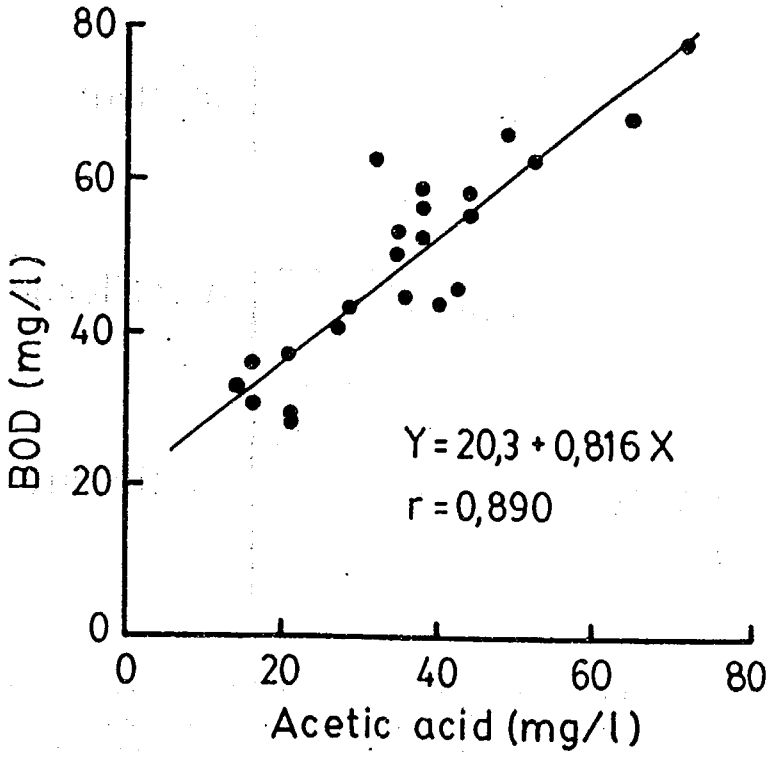


Fig. 15. Relationship between concentration of acetic acid and BOD in UF filtrate.

Dissolved protein and carbohydrate were determined by the micro-Biuret method and phenol-sulphuric acid method, respectively. As shown in Fig. 16, the concentrations of both protein and carbohydrate in the anaerobic reactor effluent were about double those in the influent. However, there was no detectable protein in the UF filtrate. The carbohydrate concentration also dropped markedly after UF filtration. It is concluded that nearly all the protein and a fair amount of carbohydrate in the anaerobic reactor effluent were rejected by the UF membrane and accumulated in the anaerobic reactor.

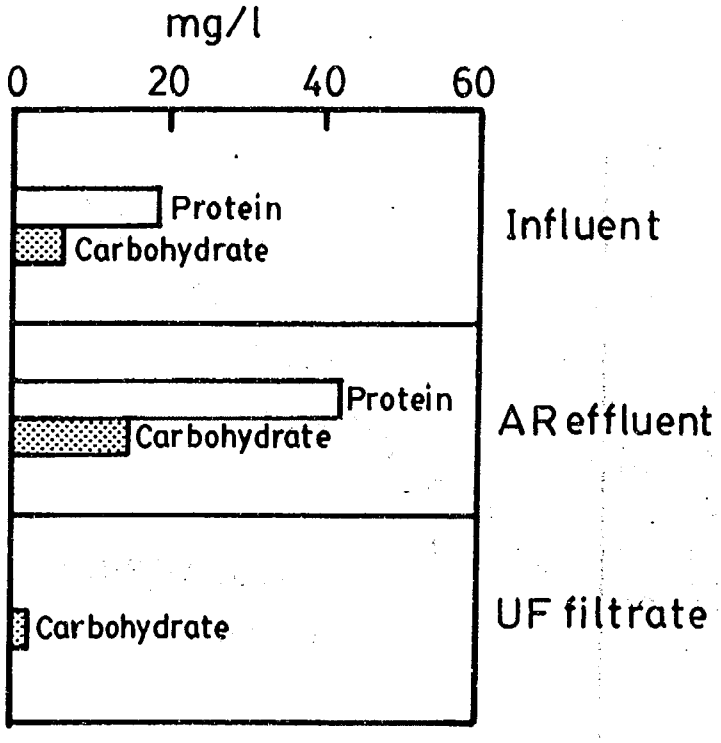


Fig. 16. Concentrations of dissolved protein and carbohydrate in influent, anaerobic reactor (AR) effluent, and UF filtrate.

Characterization of dissolved organic matter was done by gel chromatography in February 1986. The sample for gel chromatography was filtered with a glass-fiber filter (Whatmann GF/C), and concentrated to one-twentieth its original volume using a rotary evaporator, and then centrifuged at 10,000 rpm for 15 minutes. The supernatant of 8 ml was applied to the gel column (2.4 cm in diameter and 65 cm long) of Sephadex G15 or G75 using distilled water as the elution medium. Successive 5-ml fractions were collected by a fraction collector and the total organic carbon (TOC) and absorbance of 220 nm and 260 nm were determined for each fraction. Fig. 17 shows the elution patterns of influent, anaerobic reactor effluent, and UF filtrate. Three distinct peaks, called peaks I, II, and III in the order of decreasing molecular weight, were observed for the gel chromatograms of both the influent and anaerobic reactor effluent eluted with G15 gel. Peak I was markedly higher for anaerobic reactor effluent than that of influent. This peak disappeared after UF filtration, though other two peaks remained. It is concluded that organic constituents of the higher molecular weight peak (peak I) were rejected by the UF membrane and accumulated in the anaerobic reactor. Gel chromatograms by G75 gel were even better evidence of this behavior of the high molecular weight constituents. It is estimated from the gel chromatograms of G75 that the molecular weight of the high molecular weight constituents is more than 20,000.

Fig. 18 shows the relative concentrations of protein and carbohydrate for each peak. It is apparent that nearly all protein and carbohydrate were contained in peak I, which was eliminated by UF filtration. This result is compatible with above-mentioned fact that nearly all dissolved protein and carbohydrate in the anaerobic reactor effluent were rejected by the UF membrane (Fig. 16).

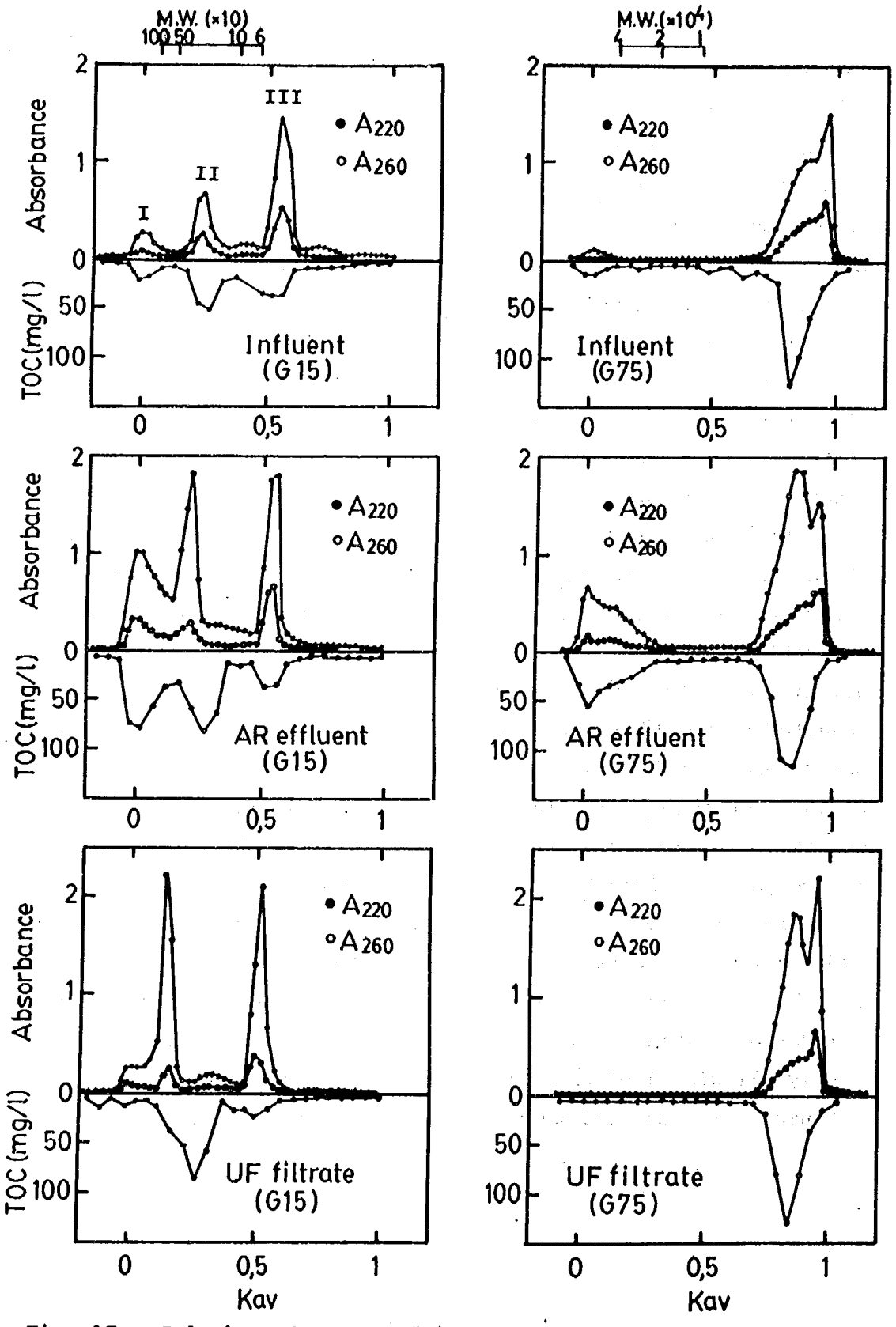


Fig. 17. Gel chromatograms of influent, anaerobic reactor (AR) effluent, and UF filtrate analyzed on Feb. 3 1986.

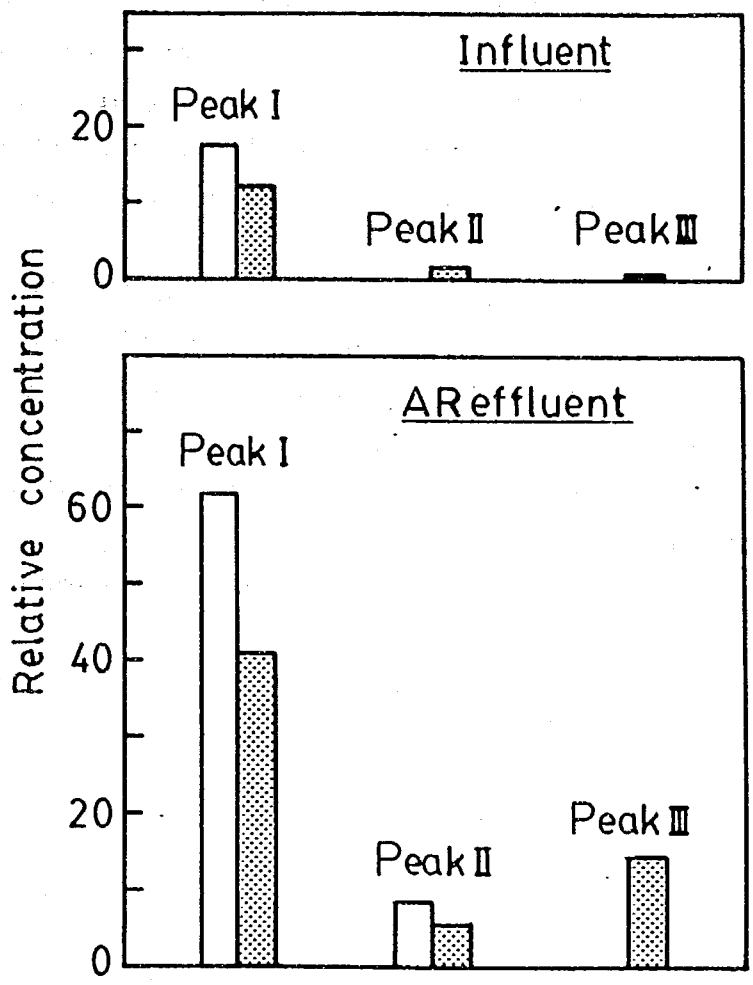


Fig. 18. Relative concentration of protein and carbohydrate for each peak of gel chromatogram on Sephadex G15.

Peak I : High molecular weight peak
Peak II : Medium molecular weight peak
Peak III: Low molecular weight peak

Protein
Carbohydrate

2.4 REMOVAL EFFICIENCY OF COLIFORM BACTERIA AND COLIPHAGE

Coliform bacteria and *E.coli* bacteriophages (coliphage) were counted as indices of pathogenic bacteria and viruses, respectively. As shown in Table 1, counts of both coliform bacteria and coliphage for anaerobic reactor effluent were considerably higher than those for influent. The counts however dropped markedly after UF filtration. The average removal rates were 99.85% for coliform bacteria and 99.89% for bacteriophage.

These results prove that the ultra-filtration is an effective mean for removing both bacteria and viruses. Disinfection by the ultra-filtration should have following advantages over a chemical disinfection:

(1) Since microbes removal efficiency is independent on water quality, stable treatment can be made.
(2) It has no bad influence on the aquatic environment.

Table 1. Counts and removal rates of coliform bacteria and coliphage. (AR: anaerobic reactor)

Coliform Bacteria

Date	Influent	AR effluent	UF filtrate	Removal rate
Jan. 22 (1986)	8.8×10^4/ml	1.6×10^5/ml	1.5×10^1/ml	99.98%
Jan. 28 (1986)	9.0×10^4	3.1×10^5	3.7×10^2	99.59
Feb. 4 (1986)	1.6×10^5	4.1×10^5	1.5×10^1	99.99

Coliphage

Date	Influent	AR effluent	UF filtrate	Removal rate
Jan. 22 (1986)	3.3×10^2/ml	2.0×10^3/ml	4.0×10^{-1}/ml	99.88%
Feb. 18 (1986)	3.5×10^2	2.5×10^3	6.0×10^{-1}	99.83
Feb. 25 (1986)	3.9×10^2	2.0×10^3	2.0×10^{-1}	99.95

3. AEROBIC POLISHING OF UF FILTRATE

Aerobic polishing of the UF filtrate must be done to dissolve oxygen and remove sulfide. Investigations were made concerning the performance of a trickling filter as an aerobic polisher. The polisher column was 15 cm in diameter and 200 cm high. Lapilli about 0.5 to 1 cm in diameter were filled in the column as a packed media. The UF filtrate was distributed over the packed media with a hydraulic loading of 20 m^3/m^2.day. Chemical analyses of influent and effluent were made with a diurnal composite sample (sulfide concentration was determined using a spot sample). Fig. 19 shows the water qualities of the influent and effluent of the polisher. Both sulfide and acetic acid were removed efficiently and, therefore, the COD and BOD dropped markedly. The odor of sulfide was no longer apparent and dissolved oxygen was detected after polishing. It is concluded that the aerobic polishing was effective for improving the water quality of the UF filtrate. The concentration of SS, however, rose slightly after the polishing, because slime on packed media fell off continuously. Therefore, it should be more appropriate for UF filtration to be done after the aerobic polishing. A bench scale plant in which aerobic polishing is done prior to UF filtration is now being operated. High removal efficiencies of SS, COD and BOD were obtained in this plant. Further precise investigations will be made concerning this treatment process in the future.

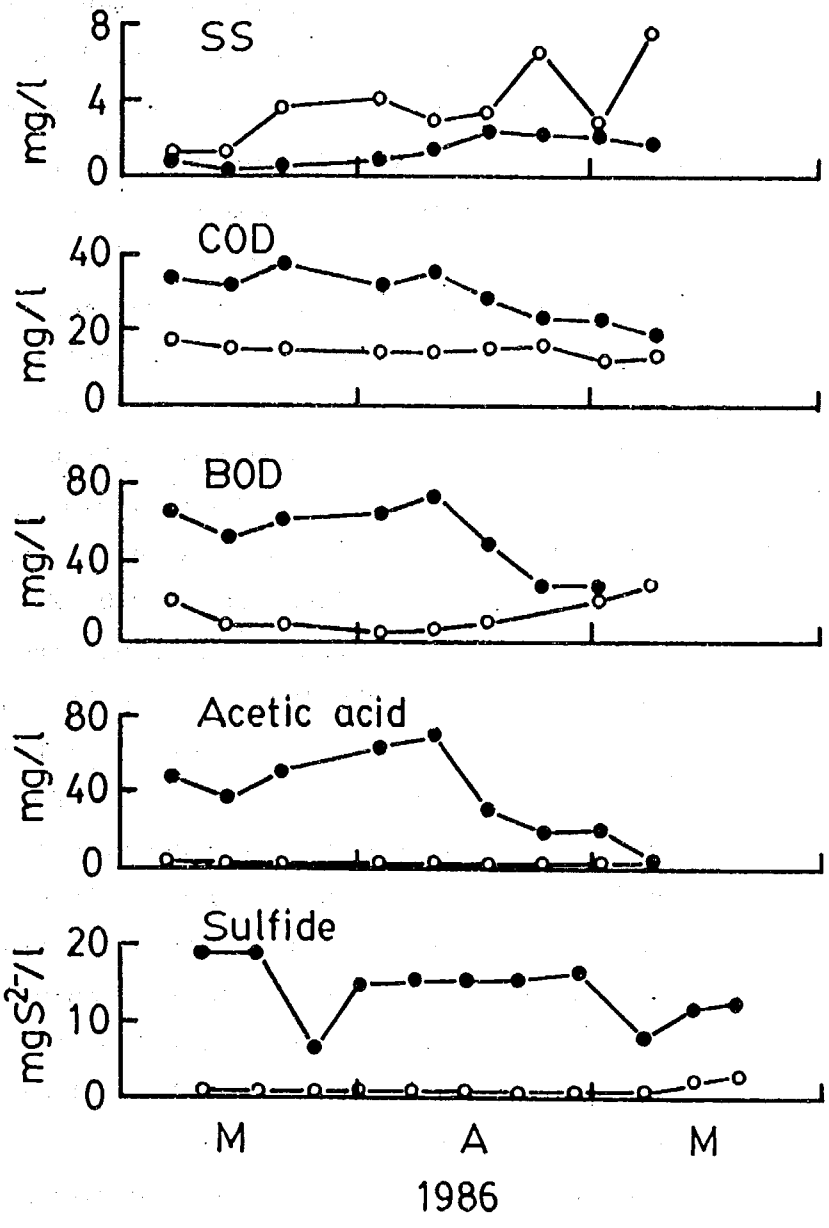


Fig. 19. Treatment performance of the aerobic polisher.

● Before treatment
o After treatment

4. ECONOMIC CONSIDERATIONS

Though no aeration energy is required for the MARS process, it requires pumping energy for circulating the bulk fluid in the module. The electrical energy consumption of the circulating pump was determined to be about 1 KWh for the pilot plant. The pumping power per cubic capacity of the filtrate is estimated to be about 3 KWh/m^3 at a filtration rate of 0.3 m^3/h and circulation rate of 20 m^3/h. Rogers (1984) has pointed out that the pumping power cost per unit volume of filtrate depends on the circulation-to-filtration ratio. The pumping power for circulation will drop to about 0.15 KWh/m^3 if the filtration rate rise to 6.6 m^3/h (circulation-to-filtration rate is about 3). This energy requirement is equal to that for aeration in the activated sludge process. To maintain the above filtration rate, the membrane flux must be higher than about 250 ℓ/hr.m^2.atm. However, it is difficult to maintain such a high flux with present technology. The development of a high-flux membrane is required for reducing the pumping power cost. The circulation-to-filtration ratio can also be dropped by reducing the circulating rate. However, a high flow rate is indispensable for maintaiing membrane permeability. Starthmann (1984) said that the thickness of the gel layer, which is formed on the membrane surface and directly effects the transmembrane flux, is determined by the concentration of the feed solution, temperature, and flow velocity of the feed solution parallel to the membrane surface. He further pointed out that the feed flow velocity is of prime importance for successful ultrafiltration.

Rogers (1984) discussed the operating and maintenance costs of ultrafiltration process as follows:

(1) Fixed power consumption

The pumping power cost is dependent on the recycle-to-feed ratio.

(2) Labor

Since ultrafiltration plants are usually well automated, they require little operating labor.

(3) Maintenance Materials

The ultrafiltration process relied heavily on pumps. The repair or replacement of pump components constitutes a good percentage of maintenance material costs. A second large cost item is cleaning chemicals. Some feeds permit prolonged plant operation without membrane cleaning. However, under severe conditions, cleaning may be required frequently.

(4) Membrane replacement

The UF membranes are subjected to a flux decay, commonly attributed to compaction. For severe applications, the membrane replacement cost can be determined on the basis of the service warranty resulting from the pilot plant tests.

An estimation of the total cost of the MARS process, including anaerobic treatment, ultrafiltration, and sludge treatment and disposal, is the subject of a further study.

5. CONCLUSIONS

(1) The average concentrations of SS, COD and BOD of the UF filtrate over 8-months operation (from October 1985 to May 1986) were 2.2, 28, and 49 mg/l, respectively. The removal rates were about 95% for SS, 50% for COD, and 70% for BOD.

(2) Sulfide was contained in the UF filtrate at about 10 to 20 mg S^{2-}/l. The sulfide in the UF filtrate was responsible for an odor and increased the values of COD and BOD significantly.

(3) Most part of the BOD for the UF filtrate was derived from acetic acid and sulfide. These were removed efficiently by short-retention-time aerobic polishing, and the values of COD and BOD also dropped significantly after this aerobic polishing.

(4) Almost none of the dissolved protein and carbohydrate in anaerobic reactor effluent could permeate the UF membrane and nearly all accumulated in the anaerobic reactor.

(5) Nearly all the methane generated from the anaerobic reactor was discharged with the UF filtrate as dissolved gas. A linear relationship was observed between the water temperature and generation rate of the dissolved methane.

(6) The flux of the UF membrane declined from about 60 l/hr.m^2.atm (25°C) to 20 l/hr.m^2.atm (25°C) during the first two months, and fluctuated in the range of 10 to 20 l/hr.m^2.atm (25°C) afterward.

(7) The removal rates of coliform bacteria and coliphage were 99.85% and 99.89%, respectively. The removal of those microorganisms was done by mainly by UF filtration.

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ODOR EMISSIONS AND THEIR PRVENTION IN MUNICIPAL WASTEWATER TREATMENT PLANTS

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0. Summary

"Odor" is not a clear physical parameter, but a sensual experience of a human being. Therefore the measurement of odor and the detection of emission sources can be very difficult. As far as municipal sewage treatment plants are concerned, odors usually are caused by a mixture of many odorants substances; at the time being, sufficiant results can mainly be provided by combined measurements, e.g. gas-chromatography and olfactometry.

It is possible to find out, whether odor immissions result from primary and/or secondary odors. It is also possible, to find site-specific parameters for a general odor-emission-formula. This is necessary for quantifying different sources of odor and for deciding on means of counteraction.

First of all, a correct plant design and proper operation are to be assured of. Only, if they are found to be not sufficient, further techniques should be applied. Special stripping-shafts are more cost-effective than big containments in many cases. Whenever containment is necessary, one must think of the problems with operation and maintenance of plants under closed covers (explosion, corrosion etc.).

As far as air-desodoration has to be realized, plant operators have the choice between different techniques. Cost-efficient is the recycling of waste-air for tank aeration. Very common are biofilters and chemical scrubbers. A new combination between biofilter and biowasher with activated-sludge-effect is discribed.

1. Odor Problems in Sewage Treatment

1.1 Historical Review and Temporary Situation

Since ever, mankind has been sensitive concerning odors. It was the year 0051 p.Chr.n., when SENECA reported the air of Rome to be "stinky and smoky" (ATV, 1885). The famous "cloaka maxima" had already been constructed 300 years a.Chr.n., because people wanted to get rid of bad smelling wastewater. Environmental and epidemical effects were still unknown in these times.

With the construction of public sewage systems in the European cities during the 19th century the rivers began to smell as a result of heavy pollution. So the first sewage treatment plants had to be built to get rid of odor problems and health diseases.

In our days, we can afford to be much more sensitive on environmental disturbances than in recent times. Growing numbers of sewage treatment plants with big sized tanks (due to high purification standards) and complete sludge treatment, initially designed to protect the environment, have become sources of odor problems themselves.

1.2 Economic Disvalue of Odor Nuisance in the FRG

Germanys law and administered justice gives any person good chances, who wants to accuse the operators of municipals sewage treatment plants, whenever odors occur. Practical judgement says that any neighbour of a sewage treatment plant is not expected to accept odor immissions for more than 36 days per year.

Thus the construction of sewage treatment plants will only be authorized, if there is sufficiant distance between the treatment plant and the next housing (e.g. 300 m for medium sized municipal plants) and/or, if special equipment is supplied to minimize odor emissions. If existing plants tend to disturb the neighbourhood, the plant operator will either be forced to take (sometimes very expensive) means to prevent from odor or to pay compensation-fees to the inhabitants of the neighbour houses, which can be as much as 20 % of the money payed for the rent of the house.

The economic importance of odor problems for the operators of municipal sewage treatment plants in Germany can thus be estimated to be very high:

- in one case (120000 p.e. STP) the compensation fee was as much as 0.8 Mio DM/a, paid to 11 house owners.

- in another case (1.8 Mio p.e. STP) the cost for odor prevention was as much as 80 % of the cost for wastewater treatment and more than 0.5 DM/m³.

Due to this situation, research in the field of Odor Emissions from Wastewater" is of great importance.

2. Characteristics of Odor

2.1 Physiological and Psychological Background

"Odor" is defined as a sensation resulting from stimulation of the olfactory organism. Different from noise, which can be defined and analized clearly using physical parameters, an "odor" is the experience of perceiving a smell, reported by human beings. Thus we have to know about physiological and psychological circumstances.

As natural science until now has not fully detected the mechanism of our nose, nobody can clearly define, which kind of chemical substance is an odorant and which not. There are some phenomenons, which should be known by sanitary engineers:

- Our nose gets "tired" (fatigatio olfactoria). After receiving a smell for a certain period, one will not be able to detect this odor any more (due to exhaustion of olfactory-receptor-cells in the nose).

  Therefore the operator working on the treatment plant is no good witness concerning odor problems.

- There is no direct corelation between the concentration of odorous substances and the impression of odor.

  Hydrogen-Sulfide, for example, in lower concentrations smells like rotten egg, in higher concentrations begins to smell like "pleasant sweetness", but in very high concentration it can not be smelled any more (in these concentrations H_2S begins to be toxic!).

  An engineer looking for sources of odor in this case might have to follow a different odor quality to trace the responsible location.

- In nearly any case the operator of a treatment plant will have to deal with not only one, but with many compounds causing odor. It has to be known that a mixture of several odorous agents might cause odors, which are different from the expected "average smell" and of different intensity than the one corresponding to the accumulated concentration of the mixtures compounds.

- In some cases we find that special chemical agent added to wastewater thus can neutralize others and prevent from odor.

Psychological circumstances are of high importance concerning odors from treatment plants. People believe to know "where the smell comes from", and a plant operator should know that the direct view of a treatment plant will also make the neighbourhood more sensitive against smell.

A wastewater irrigation site in norther Germany with more than 10 Mio m^2 is well accepted by its neighbours in spite of significant odor immissions. This is not the opinion of neighbors near a small municipal plant, in spite of the fact that only minor odor-immissions are observed. Whereas in the first case most of the neighbours work in the factory, where the wastewater comes from, in the second case the first three of the complaining neighbours have already been to court and aimed a compensation fee.

The management of odor control ("public opinions marketing") is of high importance.

2.2 The Analysis of Odorants and the Measurement of Odors

Table 1 gives an overview of odorous substances, which can be detected and quantified by adequate chemical analysis. Common instruments used nowadays are atomic-absorption, gas-chromatography and ionisation detectors. Sometimes the combination of all methods has to be proceeded to gain the results needed. Probably the most efficient way is the combination of analytical instruments with so called olfactometers.

Whereas odorant substances can be analyzed by instruments, as mentioned above, the measurement of odors can only be done with an olfactometer, which allows mixing of fresh air with a sample to be analysed. The olfactometer is capable of producing different dilution-ratios and the observer simply breathes through the instrument. At a certain dilution-ratio, the observer will smell any odor and thus mark the threshold level - that is the point where the odor concentration is just detectable.

For sewage treatment plants olfactometering alone can either be used as a rough method (if working only with one or a few observers), or it will be very expensive (if working with a sufficient number of observers, as are necessary to find a representative average). However, olfactometry can easily be used to detect odorant substances by marking peaks in a gas-chromatogram, which the observer finds to be odorous. Once these peaks are marked, measuring can continue with instruments only; even automatic monitoring can be developed.

Because odor measurement is expensive and not always reliable, the sewage treatment plant operators decisions very often are based on rough estimations and personal experiences.

3. Sources of Odor

3.1 Primary and Secondary Odorants

Odors arise anywhere, where odorous substances evaporate from water into the atmosphere. One has to differ between primary odorants (substances, already carried into the sewage treatment plant with the wastewater inflow) and secondary odorants (substances, formed during sewage treatment by biological/chemical reactions) (RUDOLPH, 1981).

The typical situation, where secondary odorants are produced, is insufficient aeration of fresh wastewater. In this case, anaerobic conditions lead to the production of H_2S (under acid conditions) or to the production of NH_4 (under alcaline conditions).

Working on a research project supported by the German Minister of Research and Technology, the author has developed a general formula on odor-emmissions-modelling, which has to be filled with site-specific parameters:

TABLE 1: Odorous substances (osmogenes), (PARTHUM et al., 1978)

	Formula	Characteristic Odor	Odor Threshold (ppm)	Detection Threshold (ppm)	Recognition Threshold (ppm)	Adsorption of Vapors by Activated Carbon (%)	Boiling Point (°C @ 760 mm Hg)	Molecular Weight
Acetaldehyde	$CH_3 \cdot CHO$	Pungent fruity	0.004	—	0.21	7	20.2	44.05
Allyl Mercaptan	$CH_2 \cdot CH \cdot CH_2 \cdot SH$	Strong garlic, coffee	0.00005	0.016	—		67	74.15
Ammonia	NH_3	Sharp pungent	0.037	—	46.8	1-2	−33	17.03
Amyl Mercaptan	$CH_3 \cdot (CH_2)_3 \cdot CH_2 \cdot SH$	Unpleasant, putrid	0.0003	—	—		126	104.22
Benzyl Mercaptan	$C_6H_5 \cdot CH_2 \cdot SH$	Unpleasant, strong	0.00019	—	—		194	124.21
Butylamine	$C_2H_5 \cdot CH_2 \cdot CH_2 \cdot NH_2$	Sour, ammonia-like	—	—	0.24		78	73.14
Cadaverine	$H_2N \cdot (CH_2)_5 \cdot NH_2$	Putrid, decaying flesh	—	—	—		178	102.18
Chlorine	Cl_2	Pungent, suffocating	0.01	0.01	0.314		−34	70.91
Chlorophenol	$Cl\ C_6H_5O$	Medicinal, phenolic	0.00018	—	—		214	128.55
Crotyl Mercaptan	$CH_3 \cdot CH : CH \cdot CH_2 \cdot SH$	Skunk-like	0.000029	0.0077	—		98	90.19
Dibutylamine	$(C_4H_9)_2\ NH$	Fishy	0.016	—	—		159	129.25
Diisopropylamine	$(C_3H_7)_2\ NH$	Fishy	0.0035	—	0.085		84	101.19
Dimethylamine	$(CH_3)_2\ NH$	Putrid, fishy	0.047	—	0.047		7	45.08
Dimethyl Sulfide	$(CH_3)_2\ S$	Decayed vegetables	0.001	—	0.001		37	62.13
Dipheny Sulfide	$(C_6H_5)_2\ S$	Unpleasant	0.000048	—	0.0021		296	186.28
Ethylamine	$C_2H_5 \cdot NH_2$	Ammoniacal	0.83	—	0.83		17	45.08
Ethyl Mercaptan	$C_2H_5 \cdot SH$	Decayed cabbage	0.00019	0.0026	0.001	23	23	62.1
Hydrogen Sulfide	H_2S	Rotten eggs	0.00047	—	0.0047	3	−62	34.1
Indole	$C_2H_6\ NH$	Fecal, nauseating	—	—	—	25	254	117.15
Methylamine	$CH_3\ NH_2$	Putrid, fishy	0.021	—	0.021		−7	31.05
Methyl Mercaptan	$CH_3\ SH$	Decayed cabbage	0.0011	—	0.0021	20	8	48.1
Ozone	O_3	Irritating above 2 ppm	0.001	0.5	—		111	48
Propyl Mercaptan	$CH_3 \cdot CH_2 \cdot CH_2 \cdot SH$	Unpleasant	0.000075	0.024	—	25	67	76.16
Putrescine	$NH_2\ (CH_2)_4\ NH_2$	Putrid, nauseating			—	25	158	88.15
Pyridine	$C_5H_5\ N$	Disagreeable, irritating	0.0037	—	—	25	116	79.10
Skatole	$C_9H_9\ N$	Fecal, nauseating	0.0012	0.223	0.47	25	266	131.2
Sulfur Dioxide	SO_2	Pungent, irritating	0.009	—	—	10	−10	64.07
Tert-Butyl Mercaptan	$(CH_3)_3C \cdot SH$	Skunk, unpleasant	0.00008	—	—		65	90.19
Thiocresol	$CH_3 \cdot C_6H_4 \cdot SH$	Skunk, rancid	0.0001	0.019	—		195	124.21
Thiophenol	$C_6H_5\ SH$	Putrid, garlic-like	0.000062	0.014	0.28		170	110.18
Triethylamine	$(C_2H_5)_3\ N$	Ammoniacal, fishy	0.08	—	—		89	101.19

$$OU \cdot t = (pO + sO) * f_e \times t$$

OU = Number of odor units emitted
(An odor unit is the amount, by volume, of odorant gas necessary to contaminate one m^3 of odor-free-air to the detection treshold level).

t = Observation time

OU x t = Loading of the odor emission

pO = Concentration of primary odors in wastewater
(e. g.: wastewater from a chicken-slaughtery carried 37.000 odor-units)

sO = Concentration of secondary odors
(e.g.: 12.000 odor units are produced within one hour, when the wastewater is treated in the activated sludge tank of a plant under certain conditions)

f_e = Function, describing the evaporation of odorant substances from wastewater into the atmosphere
("stripping-function", depending on site conditions, such as temperature, pH-value, humidity, pressure, dilution-balance-figures, effective surface area between air and water ...).

By measuring actual emissions on different locations of a sewage treatment plant and by recalculating the emission formula, several parameters can be quantified; others can be neglected. By collecting typical results, the author hopes to find typical figures

- for different kinds of wastewater,
- for the correlation of plant-operation-parameters and odor emissions (e.g. BOD, oxygen content and supply, volume- and space loading, suspended solids concentration etc.),
- for parameters depending on the type of equipment (e.g. we find different stripping intensities for different aeration systems).

3.3 Emission, Transmission, Immision and Disturbance

Odor disturbances may arise, if immission levels are definetely higher than the threshold level. This can only happen, if the emission of odor from the treatment plant is high enough to reach the observer without beeing diluted too much during its way of transmission through the atmosphere.

Thus, the prevention of odors not necessarily means the prevention of odor emissions. We can also imagine cases, where it is easier to influence transmission (for example by planting high and dense trees between a treatment plant and the neighbourhood, hoping to have more intensive air turbulences and higher rates of dilution).

3.4 Sources of Odor

Table 2 gives an overview of potential sources of odor in sewage treatment plants, which have to be checked, analyzed and selected for treatment whenever odor problems have to bee solved.

It may be mentioned that one always has to differ between primary and secondary odorants. Intensifying tank-aeration, for example, will reduce emissions, if secondary odorants have been the reason (formed under aaerobic conditions). If primary odorants have been the reason, odors will increase due to more intensive stripping.

4. Prevention of Odor

4.1 Criteria for Plant-Design

A basic rule should be, to prefer long distances between treatment plant and housing areas, if possible. The whole plant must be designed for easy maintenance, otherwise it will not be kept clean enough to prevent from odors. Some general advices can be given as follows:

- Keep wastewater and sludge well aerated
  (this can be wrong for certain industrial wastewaters!)
- Maintain short detention times in sumps, primary clarifiers, thickeners etc.
- Solid wastes from sand-trap, screening kit etc. must be kept in closed containers and removed quickly.
- Sludge must be removed quickly from claryfires etc. Esp. raw sludge should be transported off quickly to prevent uncontrolled digestion.
- Aerated sludge tanks need good hydraulic conditions; oxygen has to be supplied into corners etc.
- If very high loading is necessary (e.g. first step), intensive oxygenation is necessary and stripping effects can not be avoided. Tanks should be constructed for easy containment, which might be necessary.
- Space loading and hydraulic loading of trickling filters must be well balanced, to be sure of oxygen supply and proper washout of biomass as far as necessary.
- Anaerobic sludge digesters should be refilled continuoesly or in such a way that reactor gas cannot emerge into the atmosphere.
- Aerobic sludge stabilisation is more difficult to maintain, except, when space loading is very low.
- Sludge thickeners tend to odor emissions. Thickened sludge and scum should be removed continously and quickly.
- Dewatering of sludge is not possible without odor emmissions, especially, when chemicals have to be used (e.g. when lime is used, the pH-value increases and ammonia is stripped).
- The storage of sludge usually causes no severe problems, whenever sludge is well stabilized and not too hot.

TABLE 2: SOURCES OF ODOR IN SEWAGE TREATMENT

Legend:

- ○ communal wastewater
- high organic loading
- high anorganic loading
- only in case of disturbance or poor maintenance
- ( ) under certain circumstances only

		ODORS OCCUR, BECAUSE					
		Formation of Odorant substances			Stripping of Odorant Substances		
		biological		chemical	physical		
		as by-product of reactions ▲anaerobic ■aerobic	as result of changing pH-value	reactions when mixing wastewater of different qualities	by enlargement of active surface ΔA	as result of rising temperature ΔT	as result of lowered pressure Δp
TRANSPORT	in any canal	○ ▲	⊗				
	in pressure pipes	○ ▲					
	end of pressure pipes						○
	Archimedian screw pump				⊗		
	centrifugal pumps						(⊗)
	sewage shafts and sumps	▲			⊗		
	turbulent zones				⊗		
PURIFICATION	screening kit						
	stormwater tank	○					
	sand trap	▲					
	aerated sand trap				⊗		
	grease trap	▲					
	primary clarifyer	○ ▲					
	equalization tank, detention tank						
	neutralization						
	scrubbers etc.						
	activated sludge tank	■ ○ (▲)			⊗		
	tank outlet				⊗		
	final clarifyer	▲					
	trickling filter	■ (▲)			⊗	⊗	⊗
	sewage ponds, oxidation ponds	■ ○ (▲)	( )		( )	( )	⊗
	septic tank	○ ▲	○	(○)			
	tertiary treatment		⊗				
SLUDGE TREATMENT	sludge thickener	○ ▲			○ ▲		
	sludge storage tank	○ ▲					
	anaerobic sludge stabilization				( )		
	sludge incineration					⊗	
	thermal and chemical sludge conditioning		⊗	⊗	⊗	⊗	
	sludge filter press & vacuumfilter						⊗
	sludge pond	○					
	land irrigation (sludge)	○			⊗		
	sludge composting	○				( )	
	landfill site (sludge deposit)	○					

4.2 Means of Operation

In existing sewage treatment plants, the most difficult task usually is to locate the sources of odor. As soon as complaints are made, the operator should register telephone calls etc. and list any important operation parameter of the plant, the meteorological situation etc. Thus he can find out, whether events of odor disturbances are corelated to overloading, mis - operation of certain tanks, special activities (as sludge transport), poor meteorological circumstances, or anything else.

Once a plant is really kept clean and operated probably, there are still some means to decrease odor emmissions. For activated sludge tanks, to give one example, it is possible to

- lower the speed of rotating surface aerators, respectively to dip the aerators more deeply into the water, so that production of aerosols will be minimized.
- give a better regulation of pressurized-air-aerators to reduce stripping effects to the level necessary for oxygen supply.
- run aerators at higher speeds, whenever secondary odors are responsible for disturbance.
- run the plant at abnormal operation figures, if reasonable and necessary under special circumstances (for example: lower sludge content, submerged wastewater inflow, low pH-values etc.).

It needs quite a bit of experience, to find and proceed proper means of operation against odor nuisance. Anyhow, in more than 50 % of all cases with severe odor problems, means of operation prooved to be efficient, as far as the authors experience is concerned.

4.3 Containment

More easy to design and in some ways a safer solution is the containment of odor sources, once they are detected. However, one must be aware of the fact that operation of plants under a cover may become very difficult. Protection against explosions has to be kept with all kind of machinery, which may be quite expensive. Another point is corrosion, which is said to be twice as intensive as for uncovered plants.

There are different kinds of containement:

a) semi-covering
for tanks etc.; it may help against aerosols and direct wind drifting, such as shown in **figure 1. Figure** 2 shows, how the odor immission can be reduced by swimming plastic balls minimizing the effective surface area between water and air.

b) full covering
A textile construction as containment of a settlement tank is shown with **figure 3.** The "rotating tent" moves together with the sludge scraper.

c) gas-tight covering
and covers with negative pressure inside, as shown in **figure 4** (sludge-thickener). No odors can emerge into the atmosphere. However, odorant substances might evaporate as soon as the liquid leaves the covered area. (This does not raise any problem for pre-thickeners before anaerobic digesting,because odorants are degraded with the process of sludge stabilization.)

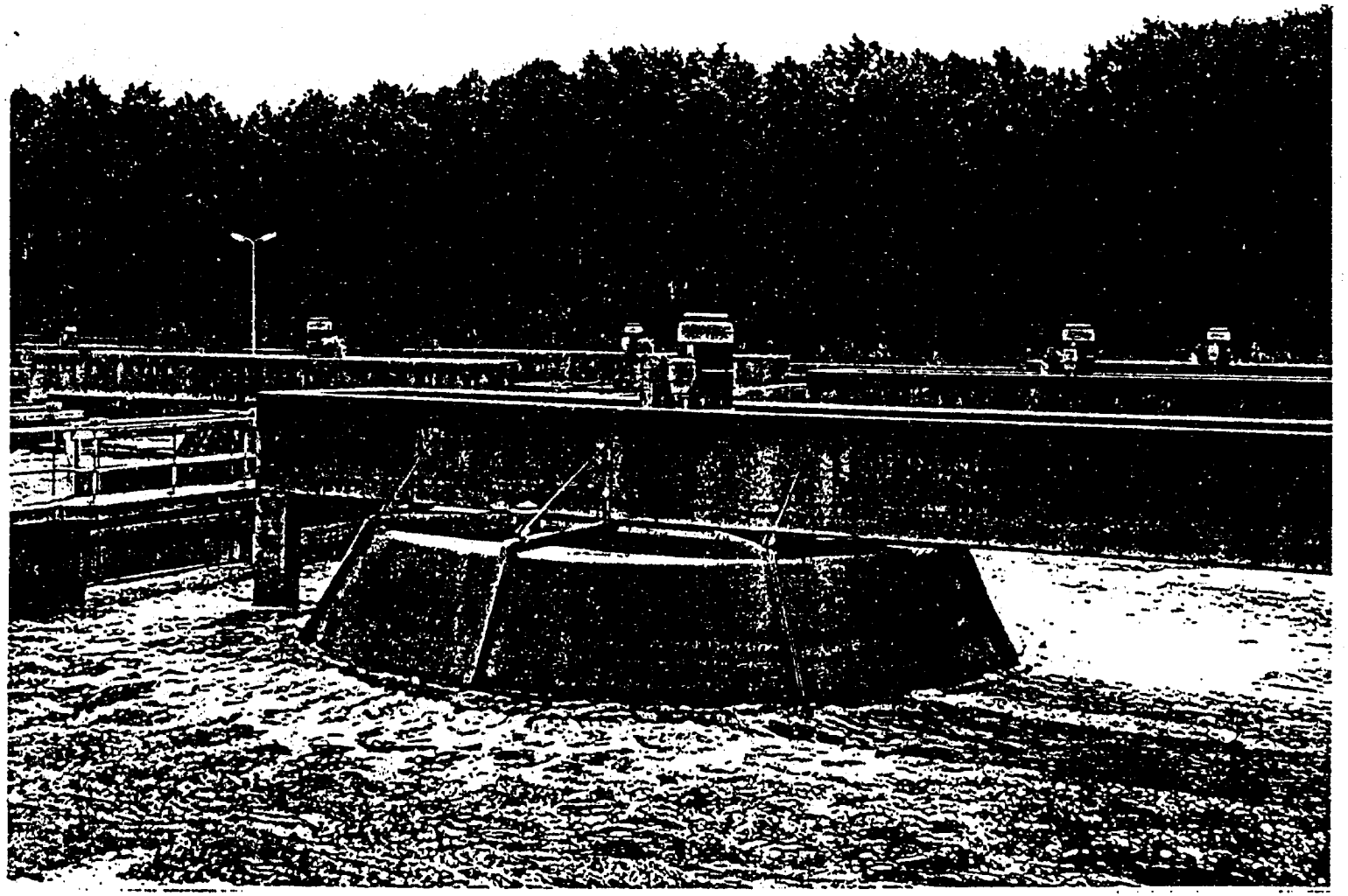


FIGURE 1: Semi-Covering of Tanks

Shown here: Plastic Cover for Surface Aerator

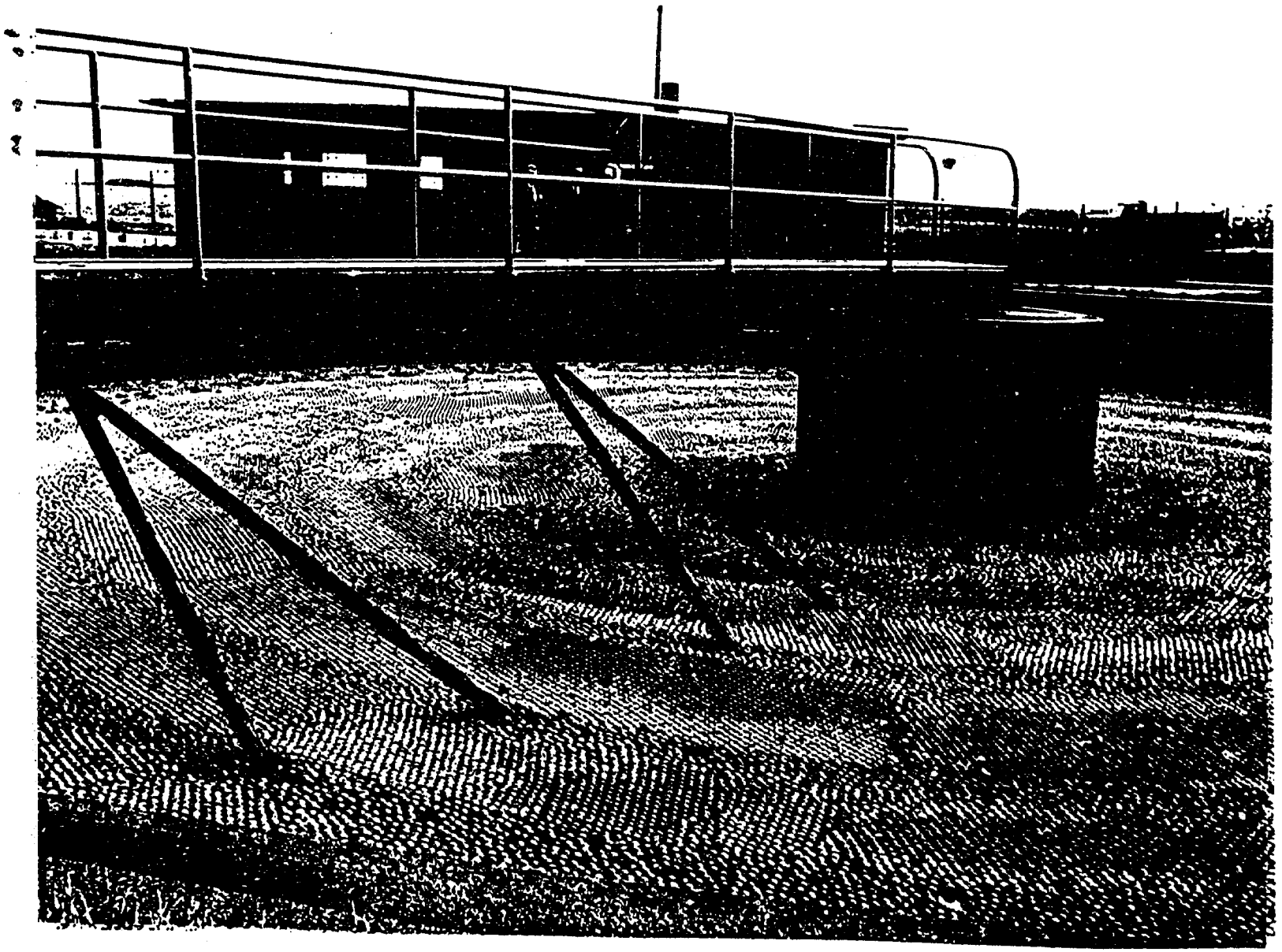


FIGURE 2: Swimming Plastic Balls to Reduce Gas Emission From Wastewater Tank

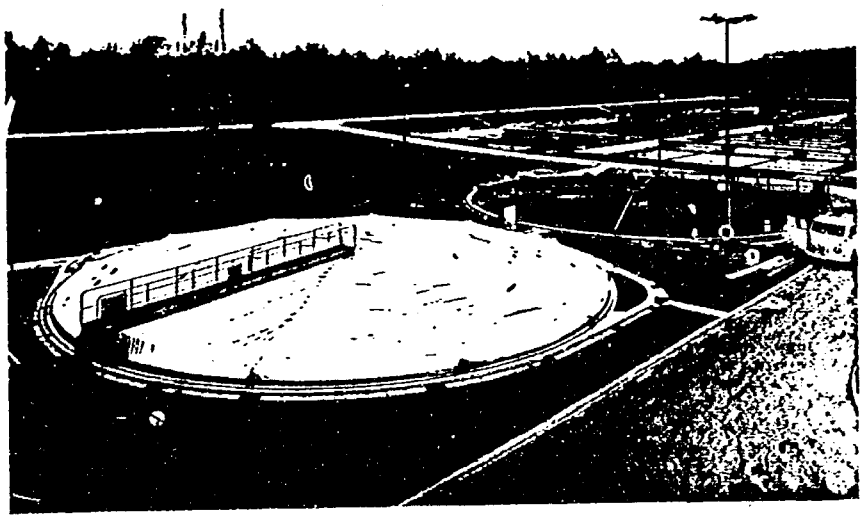


FIGURE 3: Rotating Textile Cover as Containment of a settlement tank

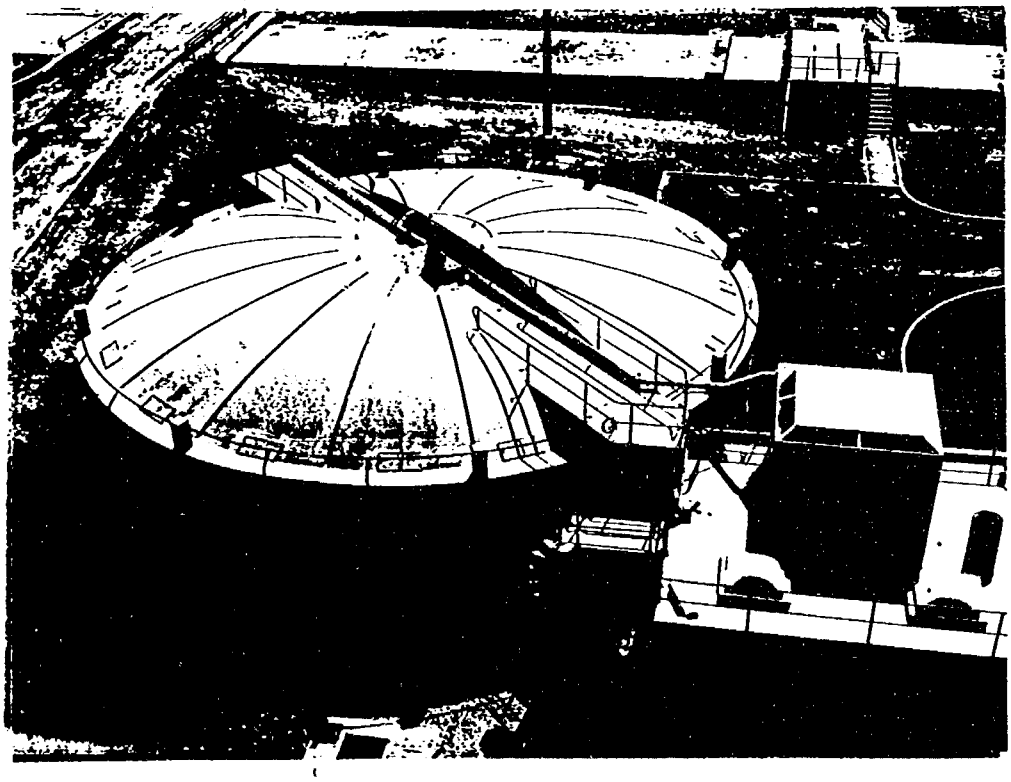


FIGURE 4: Gas-tight Cover of a Sludge Thickener (glass fiber material)

We also have to differ between contain ments designed as high buildings (dewatering house, for example) and flat tops.

As the cost for containment construction mainly depends on its size, it seems recommendable to choose small buildings to be covered. A very reasonable sollution can be a special stripping-shaft with intensive aeration. Odorant substances can be controlled here and captured easily for further treatment. Good experiences have been made with such a stripping shaft; detention time three minutes; 1,5 m³ of fresh-air per m³ of wastewater inflow.

4.4 Desodoration of Air

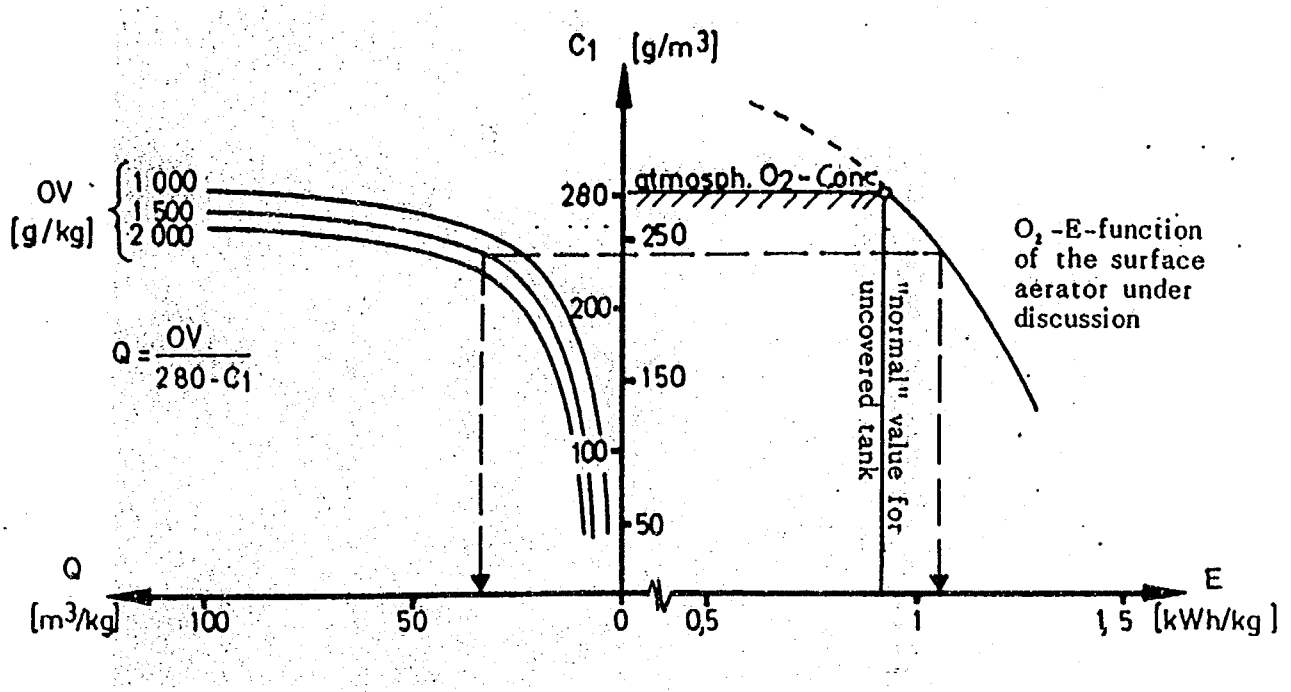
Air-desodoration should be suggested as the last step and to be realized only, if all means of operation and pure containment prooved to be unefficient.

A statistical review of what is used in the FRG for sewage treatment plants showed that in more than 70 % of all cases chemical scrubbers and biofilters are used (BÖHNKE, 1983). Other methods are thermal oxydation, activated carbon filters and (comparably cheap) the reclamation of odorous air for the aeration of activated sludge tanks.

It is experienced that high concentrations of odorants (e.g. waste-air from sludged dewatering) can be dissolved and partly eliminated during the activated sludge process (NEUMANN, 1976).

Whenever a surface-aerated tank is covered, the oxygen supply will suffer, if there is not enough fresh air provided under the cover. Thus one has to optimate between energy consumption for aeration and the cost for desodoration of waste air, which, by volume is equal to the fresh air supply under the cover. **Figure 5** gives some hints for calculation.

A new development is the combination of biowashing and biofilter, as shown with **figure 6.** The flat biofilter is permanently served with water-saturated air to be deodorized, which has just passed a biowasher. The washer is permanently served with water from the bottom drainage of the biofilter, so that some kind of "activated sludge effect" is realized. The system could also be combined with a chemical scrubber, if necessary (pre scrubbing only under certain restrictions!).



Example: Chosen oxygen concentration in covered tank:
C_1 = 240 g O_2/m³ air

Energy consumption of surface aerator:
E = ca. 1,1 kWh/kg BOD-elimination

Waste-air-volume, if OV = 1500 g O_2 - consumption per kg BOD-elimination:
Q = ca. 38 m³/kg BOD_5 - elimination

FIGURE 5: Energy Consumption and Waste-Air-Volume for Covered Activated Sludge Tanks as Function of the Air Content of Oxygen under the Cover

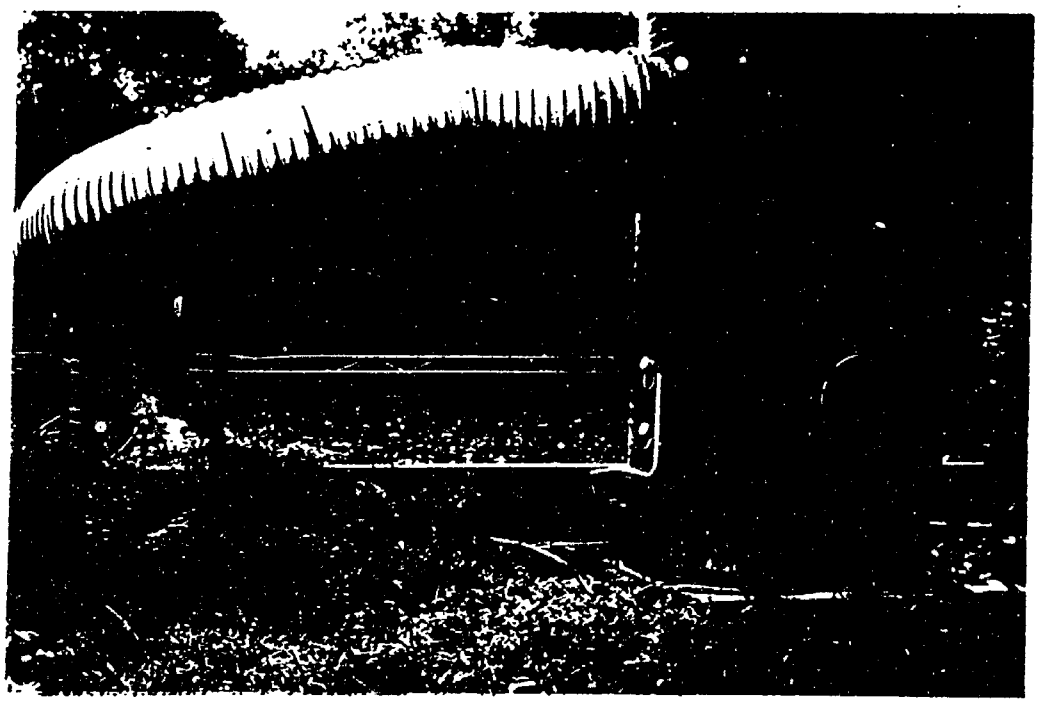


Figure 6: Biofilter (background) for air-desodorisation combined with biological washer (front, right side)

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EXPERIMENTS FOR PRACTICAL USE OF SOIL DEODORIZING EQUIPMENTS USED TOGETHER WITH COMPOST AND ZEOLITE

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SUBSTANCE

This is a report on the recent experiment carried out to study the applicability of soil deodorization to complex odors which are discharged from the city's composting plant. The main characteristic of the odors is high concentration of ammonia. The experiment was performed with full-scale models.

In the experiment, two kinds of soil beds, one composed only of Kuroboku soil which was most commonly used soil for soil deodorization in Japan and the other composed of a mixture of Kuroboku soil, sludge compost, and zeolite (soil mixture), were used.

The results of the experiment showed that the soil mixture was more capable of deodorizing air and more applicable to complex odors than unmixed Kuroboku soil and that the permeation resistance of the bed of the soil mixture was small enough to make it economical.

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EXPERIMENTS FOR PRACTICAL USE OF SOIL DEODORIZING EQUIPMENTS USED TOGETHER WITH COMPOST AND ZEOLITE

1 INTRODUCTION

In Japan these days, to protect and keep harmony with the environment has become an important consideration when constructing sewage treatment plants. Therefore, efforts have been made not only to develop the means to get rid of odors but also to improve the structure of facilities by having them built indoors. At the same time, a considerable amount of money has been spent on the construction and operation of deodorizing facilities and equipment.

Two most common deodorizing methods to date are physical control process and chemical treatment process. With research and development made on processes and controlling techniques, these two methods have now won a reputation as highly dependable technology.

However, these methods require a great deal of fuel and materials; therefore, biological deodorizing method which uses activated sludge, compost, and/or soil has now been attracting attention for its energy and material conserving feature. Among biological deodorizing methods, soil deodorization system requires less operation and lower cost than others, making itself available for sewage treatment facilities as well in recent days.

This report introduces the outline of deodorizing measures now being implemented by this city for sewage treatment facilities and the results of the experiment carried out to study possibilities of adopting soil deodorization system for the city's composting plant where solution of odor problem is urgently called for.

In this experiment, two kinds of soil beds, one composed only of soil and the other composed of soil, sludge compost, and zeolite, were used. This report focuses on the satisfactory results shown by the latter bed.

2 REGULATIONS ON OFFENSIVE ODORS

In Japan, offensive odors are controlled by the "Offensive Odor Control Law". This law stipulates that the governor in each prefecture designates areas in which malodors are to be regulated and sets the standards for odors to be restricted. The governor also exercises what is known in Japan as "administrative guidance" and recommends or orders odor emitting plants located in the controlled areas to take appropriate measures to secure a comfortable living environment for people in the neighboring areas. In the case of sewer system, the law applies to waste water treatment facilities and pumping stations excluding piping system.

Table 1-a below shows the standards for odors allowed at the ground surface level on the boundary of a sewage treatment plant and the substances regulated by the "Offensive Odor Control Law".

Table 1-a. Standards set by the Offensive Odor Control Law

Substances Regulated	Standards (Concentration at Boundary) (ppm)			Characteristics of odors
	Variation	Industrial Areas	Other Areas	
Ammonia	1 ~ 5	2 ~ 5	1 ~ 2	Pungent odor
Methyl mercaptan	0.002 ~ 0.01	0.004 ~ 0.01	0.002 ~ 0.004	Similar to the odor of rotten onions
Hydrogen sulfide	0.02 ~ 0.2	0.06 ~ 0.2	0.02 ~ 0.06	Similar to the odor of rotten eggs
Methyl sulfide	0.01 ~ 0.2	0.05 ~ 0.2	0.01 ~ 0.05	Similar to the odor of rotten cabbages
Trimethyl amine	0.005 ~ 0.07	0.02 ~ 0.07	0.005 ~ 0.02	Similar to the odor of rotten fish
Dimethyl disulfide	0.009 ~ 0.1	0.03 ~ 0.1	0.009 ~ 0.03	Similar to the odor of rotten vegetable
Acetaldehyde	0.05 ~ 0.5	0.1 ~ 0.5	0.05 ~ 0.1	Grassy-smelling odor
Styrene	0.4 ~ 2.0 Corresponding to2.5~3.5in odor strength	0.8 ~ 2.0 Corresponding to3.5~3.5in odor strength*	0.4 ~ 0.8 Corresponding to2.5~3.0in odor strength*	Similar to the odor of plastic

*See Table 1-b

Table 1-b. Six grades of odor strength

Strength	Odor Characteristics
0	No odor
1	Odor that can be barely sensed(the detectable threshold odor)
2	Weak odor that be recognized according to its kind (the recognizable threshold odor)
3	Easily detectable odor
4	Strong odor
5	Very strong odor

The "Offensive Odor Control Law" picks up eight substances which are considered to be major sources of offensive odors and stipulates for the use of an analyzer to mechanically measure and regulate concentration of these eight substances. However, offensive odors stem from many other sources and in most cases are made up of many different odors each of which is of low concentration in itself. This often brings up contradiction between the mechanically measured figures and complaints received from residents, making us search for better ways of measuring odors. As a results, "Triangle Air Bag Method" which uses the human sense of smell has been developed and adopted by several local governments.

Under this method, the original odor is diluted with odorless air and the ratio of the volume of odorless air injected to remove the original odor to the extent that the human sense does not sense it to the volume of the original odor is taken as "odor concentration", the values of which are the criteria for odor restriction. The criteria differ from one prefecture to another and Table 2 shows the odor concentration allowed by Tokyo Metropolitan Government.

Table 2. Odor control ordinance in Tokyo

Classifications	Applicable Areas	Standards for Discharging Point	Standards for Boundary Lines
1st Class	1. First type of area is exclusively used for residents. 2. Second type of area is exclusively used for residents. 3. Normal residential area. 4. Nonfixing area(excluding 2nd and 3rd classes).	Odor concentration 300	Odor concentration 10
2nd Class	1. Communal commercial area. 2. Commercial area. 3. Semi-industrial area. 4. The land and waters adjoining 1–3area.	Odor concentration 500	Odor concentration 15
3rd Class	1. Industrial areas. 2. The area exclusively used for industry. 3. The land and waters adjoining 1–2area.	Odor concentration 1,000	Odor concentration 20

3 DEODORIZING MEASURES FOR SEWAGE TREATMENT FACILITIES

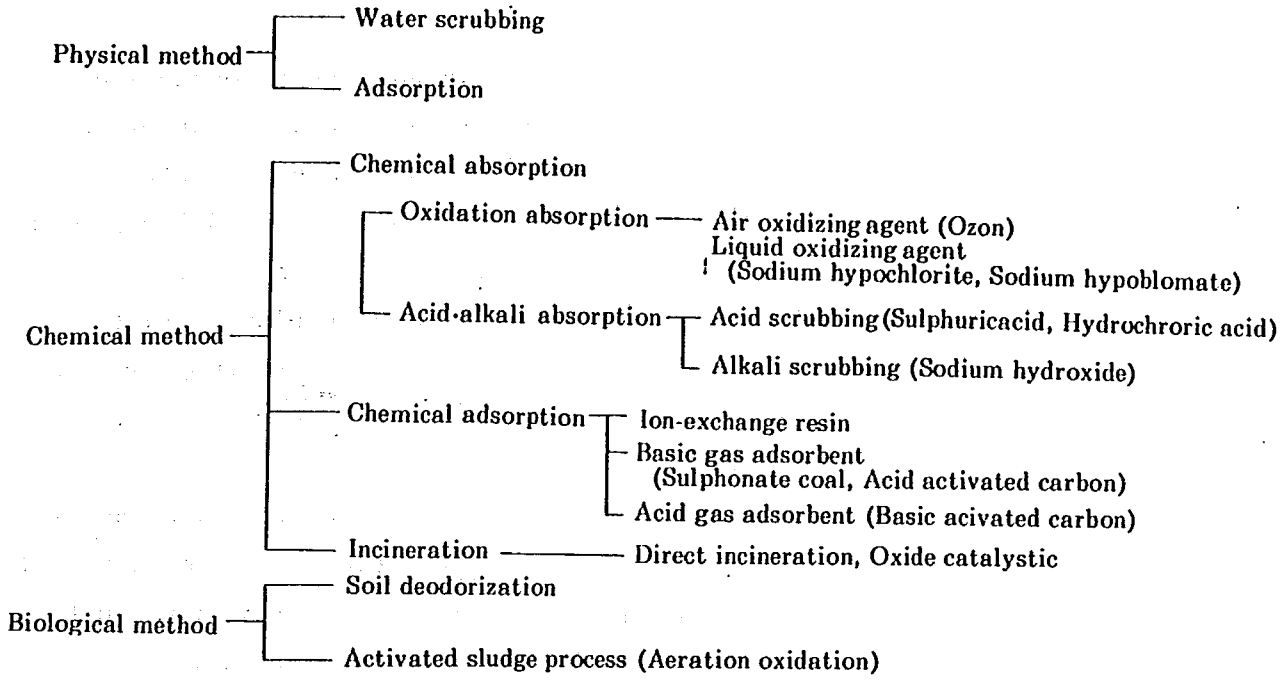
3.1 DEODORIZING MEASURES FOR SEWAGE TREATMENT FACILITIES IN JAPAN

The numbers of sewage treatment plants and pumping stations in Japan have been on the rapid increase in recent years as a result of positive moves to expand sewer system service. Today, these facilities are often constructed in or adjacent to densely-populated areas making it imperative for local governments to take proper measures to keep harmony with their surroundings. Therefore, rooftops of the facility buildings are often made into parks or recreation grounds. This necessitates the installation of deodorizing system at sewage treatment facilities and most of the newly built facilities are equipped with the system.

Commonly adopted deodorizing methods are scrubbing, adsorption, and ozonization. These three methods are often used in combination with one another in order to produce better results. Direct incineration is frequently applied to odors contained in waste gas discharged from incinerators or high concentration odors.

Table 3 below shows deodorizing methods applied to sewage treatment facilities.

Table 3. Deodorizing methods classified by principle



Physical control and chemical treatment shown above have proved to be quite effective and dependable in controlling offensive odors; however, construction and operational costs of the facilities and equipment needed for these methods are very high and affected more by the volume of evil-smelling air to be treated than the concentration of odors. Therefore, in view of economics,

(1) efforts are being made to prevent odor leakage from the facility and to collect the least amount of odors needed for treatment within the range it does not affect the working environment,
(2) the volume of evil-smelling air to be treated is reduced by reusing waste gas emanating from the aeration tank as aeration air,
(3) deodorizing methods are carefully selected in accordance with the odor concentration.

In addition, odor treatment with activated sludge using the odorous air around the grit chamber or sludge treatment facilities as aeration air is also in practice.

3.2 DEODORIZING SYSTEM FOR SEWAGE TREATMENT FACILITIES IN FUKUOKA CITY

3.2.1 GENERAL

Fukuoka City is the eighth largest city in Japan with a population of 1.17 million (as of March, 1986). As of March, 1986, 8,677 ha out of the total area of 14,595 ha where sewer-system service has been projected receive the benefit of the service. In terms of population, 760,000 people or 65% of the total population are now served by the sewer system.

At present, the city operates seven sewage treatment plants which, in total, are capable of treating approximately 507,000m^3 of sewage per day (at maximum daily capacities). All waste water is given secondary treatment by the conventional activated process.

The treatment facilities in Fukuoka City are located near residential districts or parks in urban areas because of difficulties in acquiring land and a need for efficient operation of sewer systems. Recently, more and more areas adjacent to the treatment facilities have been developed for housing and in some cases houses are built less than 100m away from the facilities.

In order not to spoil the surrounding atmosphere, the city has built all the waste water treatment equipment as well as primary setting tanks and aeration tanks indoors at major treatment plants and made the rooftops of the buildings into parks for citizens to enjoy sports and recreational activities. Other facilities which emit high concentration odors are also built indoors. (See Fig. 1)

Waste water treatment equipment, sludge scrubbing tank, and thickener tank are all equipped with covers made of fiber reinforced plastic or reinforced concrete so that only the air enveloped within the equipment or the tank can be extracted for deodorization by a duct and blower. (See Fig. 2)

In the case of the grit chamber and dewatering equipment room which need frequent inspections, ducts are installed at several points from which odors emanate. This facilitates deodorizing process by collecting high concentration odors before sending them to the deodorizing equipment.

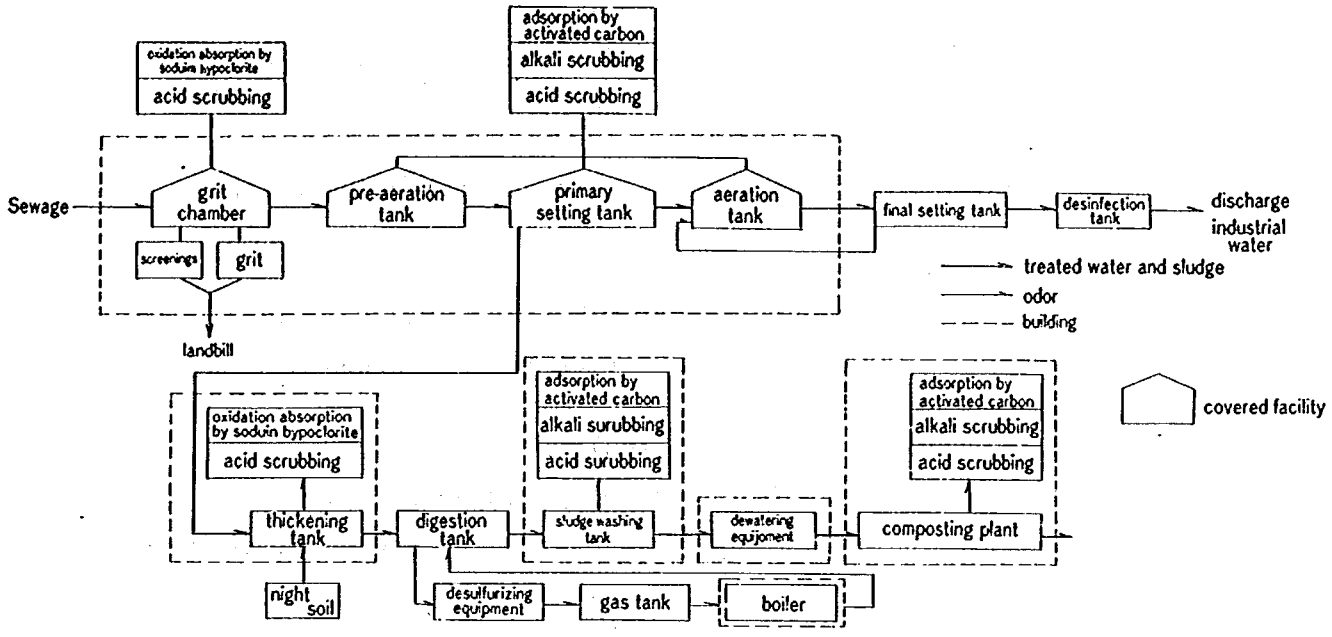


Figure 1. Flow Diagram of waste water treatment and waste gas from the Tobu Sewage Treatment Plant

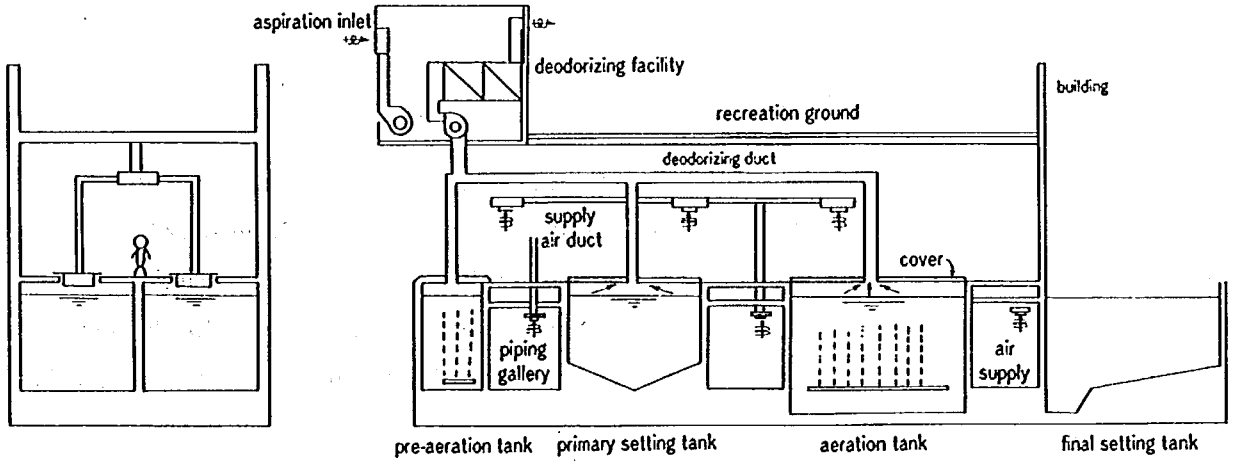


Figure 2. Deodorizing Facility for wastewater treatment plant

3.2.2 DEODORIZING METHOD

Before selecting a deodorizing method, investigation on odors is carried out to find out a most suitable and effective method.

Deodorizing methods currently in use are the following three:

(1) acid scrubbing (hydrochloric solution)
+ alkali scrubbing (sodium hydroxide solution)
+ adsorption by activated carbon

(2) acid scrubbing (hydrochloric solution)
+ alkali scrubbing (sodium hydroxide solution)
+ oxidation absorption by sodium hypoclorite

(3) acid scrubbing (hydrochloric solution)
+ alkali scrubbing (sodium hydroxide solution)
+ ozonization

Depending on the plant size, odors are treated collectively by a single set of deodorizing equipment, or separately by more than two sets of equipment each of which functions for a different facility (grit chamber, primary setting tank, etc.)

3.2.3 EFFECTS OF DEODORIZATION

Fukuoka City aims at meeting the requirements prescribed by the "Offensive Odor Control Law" at the discharge port of the deodorizing equipment in order to avoid annoying people with evil odors as much as possible. It also carries out routine inspection of odors by the human sense of smell as well as regular analysis of odors by use of an analyzer.

Table 4. Efforts of deodorization at the Tobu watstewater treatment plant facility (1985 fiscal year)

(concentration:ppb)

Sample	Ammonia	Methyl Mercaptan	Hydrogon Sulfide	Methyl Sulfide	Torimethyl -amine	Dimethyl Disulfide
Grid chamber;Inlet	150	144	1,000	8.9	<1.0	<1.0
After scrubbing	<100	95	750	6.5	<1.0	<1.0
After adsorption	<100	<1	1.3	2.2	<1.0	<1.0
Total removal (%)	>66.7	99.3	99.9	75.3	–	–
Primary setting tank and aeration tank;Inlet	1,000	75	2,400	15	<1	<1
After scrubbing	560	33	23	8.9	<1	<1
After adsorption	140	1.7	<1.0	6.7	<1	<1
Total removal (%)	86	97.7	≒100	55.3	–	–
Thickening tank;Inlet	180	410	10,000	56	<1	<1
After acid scrubbing	<100	120	2,600	ND	<1	<1
After oxidation absorption	<100	<1	<1	ND	<1	<1
Total removal (%)	>55.5	≒100	≒100	<100	–	–
Standards at boundary	1,000	2	20	10	5	9

The above table shows satisfactory results of deodorization with the concentration of treated odors reduced to sufficiently low level.

In the case of chemical scrubbing, while deodorization is in process, the pH meter monitors pH values of circulating liquid and operates the interlocked pump to automatically replenish chemicals to maintain the optimum pH value.

3.2.4 COSTS FOR DEODORIZATION

As is seen in Table 5, construction cost of the deodorizing facility accounts for 2-4% of the total investment for the construction of the plant. Operational cost differs greatly from one plant to another depending on the size and maintenance conditions of the plant. Power consumption of the deodorizing facility is 9 to 33.4% of the total power consumption of the plant, and service water consumption is 3.6 to 63.5% of the total service water consumption of the plant. Thus, operational cost cannot be

ignored in considering deodorization projects, and the development of deodorizing equipment which is efficient and cost effective for a long period of time is urgently called for.

Table 5. Costs for Deodorization at Major wastewater treatments

Treatment Plant	Sewarage deliered to the plant(㎥/day at average) / Treatment capacity (㎥/day at maximum)	Construction Cost (million yen)	Eletric Power Consumption (KWH/year)	Service Water Consumpion (㎥/year)	Chemicals (1000yen/year)	Remarks
Chubu	233,700 / 350,000	372.6 / 12,597.5 (3.0%)	2.08×10^{6} / 23.14×10^{6} (9.0%)	3.50×10^{3} / 96.03×10^{3} (3.6%)	11,923 / 210,576 (5.7%)	deodorization facilition / the whole plant
Tobu	53,600 / 83,000	382.0 / 10,368.6 (3.7%)	1.20×10^{6} / 11.0×10^{6} (10.9%)	4.57×10^{3} / 50.18×10^{3} (9.1%)	3,130 / 30,786 (10.2%)	Figers in brackets show the percentage of deodorization -related quantity to the total quantity
Seibu	23,400 / 37,500	97.8 / 4,862.5 (2.0%)	5.30×10^{5} / 38.16×10^{5} (13.9%)	3.52×10^{3} / 5.39×10^{3} (65.3%)	367 / 14,904 (24.6%)	
Wajiro	9,220 / 20,000	201.2 / 5,645.8 (3.6%)	7.76×10^{5} / 38.34×10^{5} (20.2%)	1.67×10^{3} / 8.36×10^{3} (20.0%)	592 / 4,556 (13.0%)	
Saitozaki	3,160 / 6,500	144.0 / 3,989.0 (3.6%)	7.80×10^{5} / 23.34×10^{5} (33.4%)	2.53×10^{3} / 3.98×10^{3} (63.5%)	575 / 1,671 (34.4%)	

4 STUDY OF SOIL DEODORIZATION SYSTEM FOR POSSIBLE APPLICATION ON THE COMPOSTING PLANT

4.1 HISTORY

In Fukuoka City, all the dewatered sludge produced at city's five major sewage treatment plants is delivered to one composting plant and fermented there. At this composting plant, daily production of compost in terms of dewatered cake amounts to 140 tons (actual figure in 1985; water content, 59%). This is the largest compost production in Japan.

When this composting plant (plant capacity, 206 ton/day in terms of dewatered cake) first started its operation, most of the surrounding areas were rice paddies or fields. With all the facilities and equipment built indoors and a combination of acid scrubbing, alkali scrubbing and activated carbon adsorption adopted for deodorization, it seemed no odor problem would occur. However, as the amount of sludge made into compost became greater, the amount of odors to be treated also increased. Since there had been no precedence of constructing a large-scale composting plant in Japan, deodorizing equipment including hoods and ducts was not designed adequately and failed to cope with all the increase in offensive odors which surpassed the initial projection. This has led to the deterioration of the working environment and caused part of the odors to leak and reach the surrounding areas. With more and more land developed for housing in recent years, offensive odors have now become a serious environmental problem. Therefore, proper assessment of odor concentration and its effects on the environment is inevitable and the application of a new type of deodorizing system is urgently needed.

A new type of deodorizing facility must meet the following conditions:

(1) Since the composting plant is located out of the sewer system service areas, there should be as little waste water as possible from the facility.

(2) The site allocated for the construction of the facility should have an area of less than 1,400m^2 per 600m^3/min of odorous air.

(3) Must be economical.

Taking the above points and characteristics of odors into consideration, soil deodorization was studied for possible application on the composting plant.

4.2 CHARACTERISTICS OF ODORS EMANATING FROM COMPOSTING PROCESSES

Compost at the composting plant is fermented by "Non-Additive Method" which does not require addition of bulking agent (Figure 3). Under this method, compost which has completed fermentation is returned and mixed with dewatered sludge, and then, after going through preliminary adjustment, fermented again with air blown into the fermentation tank. As a result, odors consist mainly of fermentation waste gas emitted by air blow. Their characteristics are as follows:

(1) Odors are complex containing many sulfur is compounds such as methyl sulfide and dimethyle disulfide. Ammonia concentration is particulary high, making the working environment hard to bear with its pungent odor. Hydrogen sulfide is negligible in amount as long as smooth fermentation is under way.

(2) Concentrations of ammonia and other constituents of odors vary in accordance with the progress of fermentation.

(3) Fermentation waste gas is hot and humid containing a great deal of carbon dioxide.

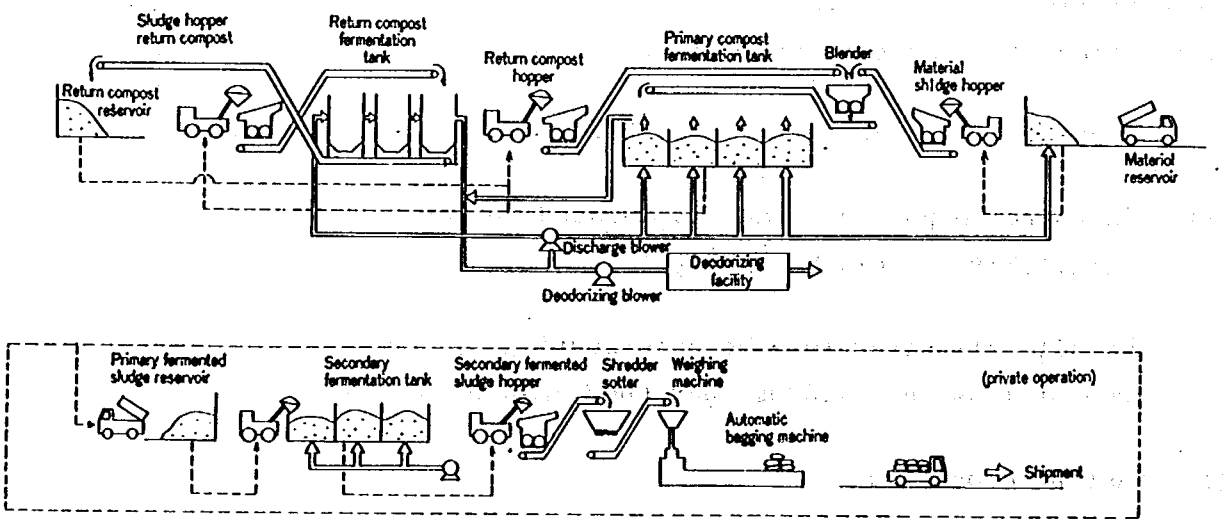


Figure 3. Compost production flow chart

Table 6. Concentrations of odor constituents at the time when ammonia concentration peaked

	Ammonia	Methyl mercaptan	Hydrogen sulfide	Methyl sulfide	Dimethyl disulfide	Trimethyl amine	Acet aldehyde	Styrene	Research date
Concentration of odors (ppm)	142	0.34	ND	0.18	0.60	ND	ND	ND	Winter Jan 18, 1985
	186	0.01	ND	0.01	0.30	ND	ND	ND	Spring Mar 18, 1985

ND : not detected

4.3 OUTLINE OF SOIL DEODORIZATION SYSTEM

Soil deodorization system sends odors to the bed of soil and filters them through it. When odors pass through the soil, the constituents of odors are dissolved into soil moisture, adsorpted by soil particles, or biologically decomposed by microorganisms in the soil.

(1) General ideas about soil deodorization in Japan are:
Kuroboku soil which is humus-made of volcanic ashes is the best and most frequently used soil for soil beds.

(2) There have been only a few examples of using soil beds to treat odors emanating from sewage treatment facilities, and in all cases the method is used to cope with low concentration odors. If the constituents of odors exceed 20ppm, pretreatment is necessary.

(3) Through past experience, it has now become common that the bed's area is 3.3m^2 per 1m^3/min of odors; the speed at which odors pass through the bed, 5.0mm/sec; soil thickness, 50cm.

4.4 POINTS TO BE CLARIFIED BY THE STUDY

The purpose of the recent experiment was to study the possibility of putting Kuroboku soil mixed with compost and zeolite (hereafter refered to as soil mixture) into practical use for soil deodorization to treat odors emanating from the city's composting plant. Points to be clarified by the experiment were its applicability to high concentration odors consisting mainly of ammonia, its adequacy for the space requirement, and its economics. Thus, through the experiment, the following items were examined by seasons.

(1) Effects of deodorization

(2) Factors that affect deodorizing efficiency

(3) Behavior of microorganisms

(4) Waste water discharge and its quality

4.5.1 THE STRUCTURE OF THE SOIL DEODORIZATION

Figure 4 shows the structure of the soil deodorization system commonly used in Japan. Hoods and ducts installed over the fermentation tank in the composting plant collect odors and send them to the diffusion layer built in the lower part of the soil bed through the duct which connects there. The diffusion layer is made up of cobble stones, fine gravels, net and others and serves to equalize the speed and pressure of odors before they are sent into the upper layers. Odors are then removed while passing through the soil. Treated odors are released into the atmospheric air. At the bottom are the waterproof sheet and drain pipe.

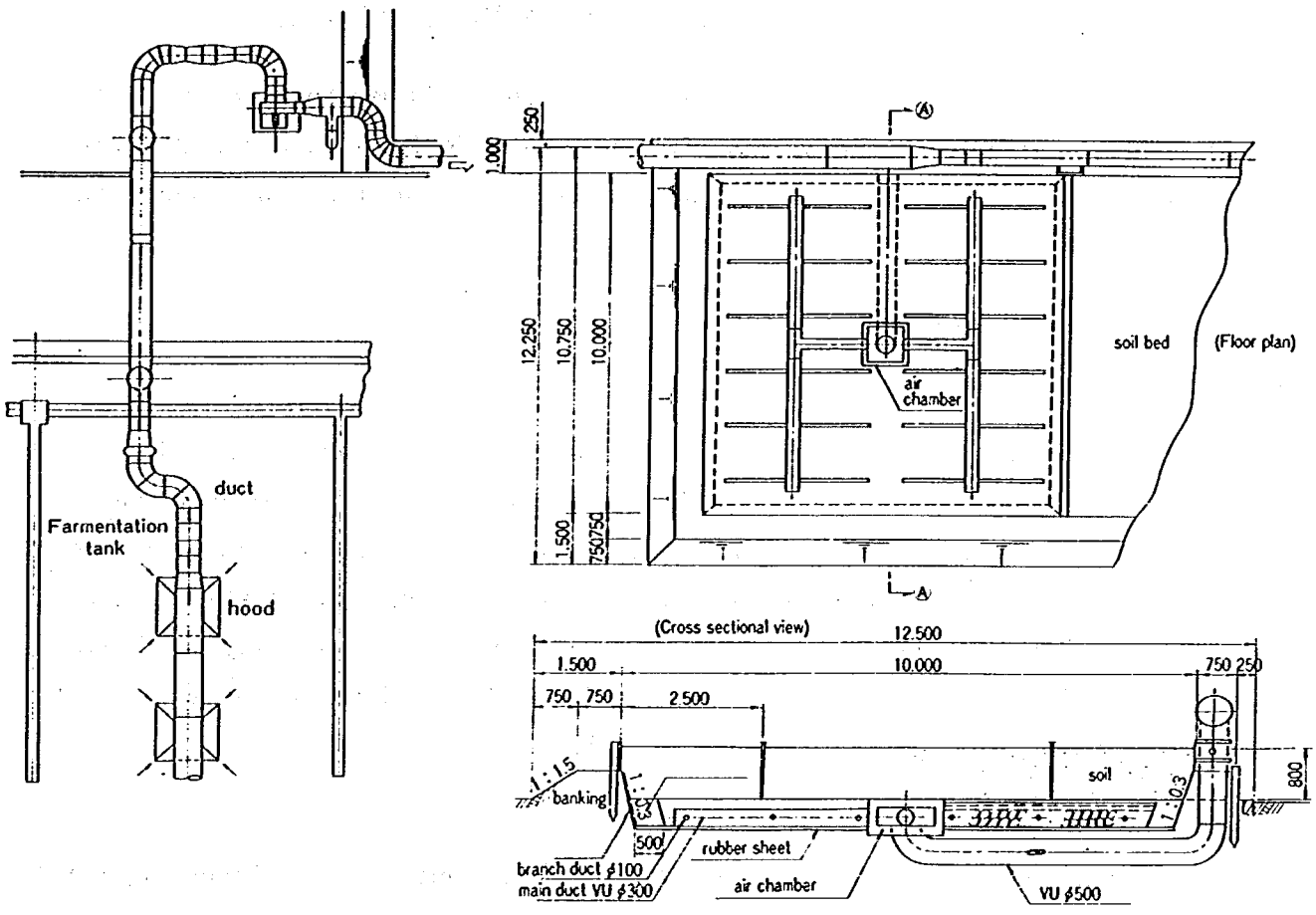


Figure 4. Soil deodorization system for experimental use

4.5.2 TEST MATERIALS, EQUIPMENT SIZE AND OPERATING CONDITIONS

In order to examine the applicability of the soil mixture to the soil deodorization system, the recent experiment was made both on unmixed Kuroboku soil (hereafter referred to as Kuroboku soil) and on soil mixture under the same conditions.

a) Characteristics of test materials

(1) Kuroboku soil

Approximately 1/6th of land in Japan is made of volcanic ash soil. Kuroboku soil which is humus made of volcanic ashes is produced when volcanic ejecta is weathered. It has high humus content and high air permeability with low viscosity because of its aggregated structure. It also has high cation exchange capacity, water retentivity, and water permeability. For these reasons, it is considered to be the best soil to be used for soil deodorization (Table 7 and 8).

(2) Sludge compost

Compared with Kuroboku soil, sludge compost contains easily-decomposable organic matter which improves physical and chamical properties of soil by increasing the gaseous phase of soil or by forming aggregated structure. Lime contained in compost also prevents soil from being acidified.

(3) Zeolite

Zeolite is inorganic and porous and known for its high cation exchange capacity which is 150me/dry 100g on the average. Therefore, it is often used as odor adsorbing agent or soil conditioner. Since Japan abounds with zeolite, its prices are very low (4,000-15,000 yen/ton). This is a favorable factor for putting it into practical use.

The use of the soil mixture aims at increasing deodorizing capability of Kuroboku soil by improving its chemical and physical properties (Table 7). The mixing ratio of Kuroboku soil, sludge compost and zeolite in the soil mixture for the experimental use was 6 to 3 to 1 in terms of volume.

Table 7. Physical and chemical properties of soil to be used for deodorization (Feb'85)

Item / Material	Water content %	Maximum water holding capacity g/100 ml	pH value (H_2O)	Organic matter %	Cation exchange capacity me/dry 100 g	total nitrogen mg/dry・Kg	C/N
Kuroboku soil	56.8	99.3	5.2	27.8	38	5,830	22
Soil mixture	47.6	98.6	7.6	21.3	49	8,110	12

Table 8. Particle size distribution in the test material

	Kuroboku soil	compost
clay less than 0.002mm	43 %	26 %
silt 0.002< ≦ 0.02	28 %	17 %
fine sand 0.02 < ≦2	24 %	37 %
coarse sand 0.2< ≦2.0	5 %	20 %

b) Size of the testing equipment and its operating conditions

Table 9 below shows the size of equipment and its operating conditions.

Table 9. Operating conditions

Item / Material		Kuroboku soil	Soil mixture
•Type		Semi-undergrornd	
Size	Surface area	100m^2 (10m×10m) each	
	Soil thickness	80 cm each	
Air flow rate		47.6m^3/min each	
Odor	Passing speed through soil	8mm/sec each	
	Residence time of odors in soil	100sec each	
Soil	Dry weight	31,400 Kg	40,600 Kg
	Wet weight	73,800 Kg	67,800 Kg
	Water content by weight	115 %	83 %

So far, "100 seconds" is the most commonly adopted residence time of odors in the soil bed for soil deodorization system now in actual operation. Therefore, the recent experiment also used "100 seconds" as the residence time.

The results of the laboratory experiment showed that when the residence time remained unchanged, deodorizing effects did not vary with the passing speed of odors as long as the speed was up to 8mm/sec. Therefore, the passing speed of odors for the experiment was determined to be 8mm/sec. As a result, it was learned that the bed's area could be reduced by about 40% compared with the soil bed to which the speed of 5mm/sec. was applied.

4.6 EFFECTS OF DEODORIZATION

As is seen in Figure 5, the original odors were gathered by the duct before they were sent into the equipment and the treated odors were gathered with the whole surface of the soil bed covered with vinyl cloth to prevent the inflow of air from outside.

Odors were measured in accordance with the "Offensive Odor Control Law" which stipulates for the mechanical analysis of the eight substances and by the human sense of smell (Triangle Air Bag Method). As for ammonia concentration, its variation with the passing of days from the start of fermentation till the completion was measured. Other substances were measured only on the day when the original odor concentration was the highest.

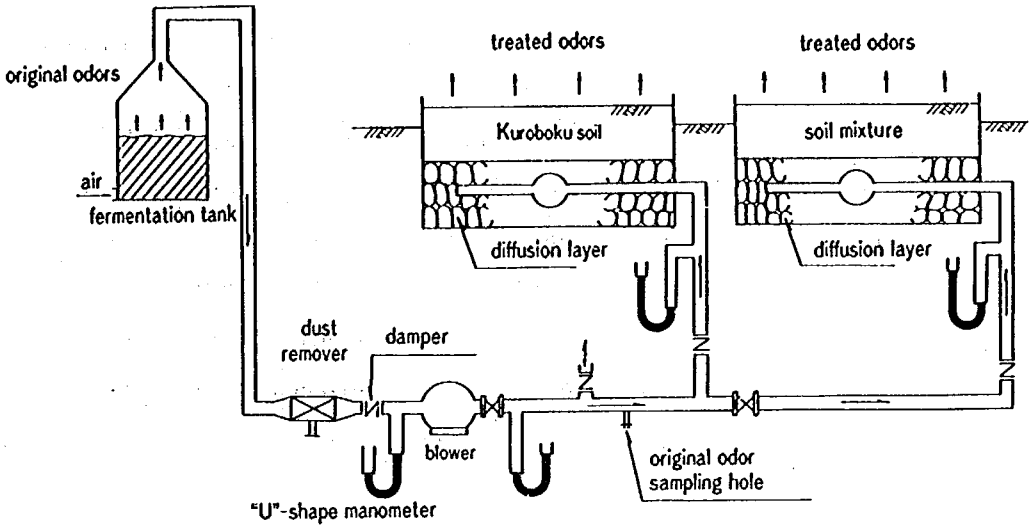


Figure 5. Flow chart.

Table 10 shows the measured values of constituents of the original odors and treated odors as well as odor removal rates. The table shows that the concentration of the original odors was high during winter. This is because the equipment was operated with the fermentation tank more hermetically sealed than in other seasons.

Table 10. Concentrations of original odors and their removal rate by season

Season	Item		Ammonia	Methyl Mercaptan	Hydrogen Sulfide	Methyl Sulfide	Dimethyl Disulfide	Trimethyl amine	Acet-aldehyde	Stylene	Odor conentration	date of measure
			NH_3	CH_3SH	H_2S	$(CH_3)_2S$	$(CH_3)_2S_2$	$(CH_3)_3N$	CH_3CHO	$C_6H_5CHCH_2$		
	Unit		ppm	ppm	ppm	ppm	ppm	ppm	ppm	ppm		
Spring	Concentration of original odors		228	0.006	< 0.002	0.064	2.57	0.0086	<0.01	<0.04	3,100	
	Concentration of treated odors	Kuroboku soil	62.2	<0.002	< 0.002	0.024	0.986	0.0068	<0.001	<0.04	720	
		soil mixture	9.2	0.003	<0.002	0.003	0.024	0.0099	<0.001	<0.04	91	May 9,'85
	removal rate (%)	Kuroboku soil	72.7	>66.7	—	62.5	61.6	20.9	—	—	76.8	
		soil mixture	96.0	50	—	95.3	99.1	0	—	—	97.1	
Rainy season	Concentration of original odors		184	25.1	0.01	4.73	46.7	<0.0005	<0.01	<0.04	9,800	
	Concentration of treated odors	Kuroboku soil	21.5	0.148	0.013	2.82	4.7	<0.0005	<0.001	<0.04	260	
		soil mixture	7.2	0.132	0.012	1.08	0.49	<0.0005	<0.001	<0.04	100	July 11.'85
	removal rate (%)	Kuroboku soil	88.3	99.4	0	40.3	89.9	—	—	—	97.3	
		soil mixture	96.1	99.5	0	77.2	99.0	—	—	—	99.0	
Summer	Concentration of original odors		272	0.185	0.072	0.135	6.54	<0.0005	<0.01	<0.04	3,300	
	Concentration of treated odors	Kuroboku soil	48.3	0.066	0.016	0.098	1.19	<0.0005	<0.001	<0.04	390	
		soil mixture	2.5	0.074	0.024	0.023	0.01	<0.0005	<0.001	<0.04	150	Aug. 22 '85
	removal rate (%)	Kuroboku soil	82.2	64.3	77.8	27.4	81.8	—	—	—	88.2	
		soil mixture	98.3	60.0	66.7	83.0	≒100	—	—	—	95.5	
Autumn	Concentration of original odors		250	0.123	0.004	0.052	8.59	<0.0005	0.02	<0.04	6,900	
	Concentration of treated odors	Kuroboku soil	145	0.003	0.004	0.059	1.97	<0.0005	<0.001	<0.04	1,400	
		soil mixture	0.2	0.009	0.004	<0.001	<0.001	<0.0005	<0.001	<0.04	20	Oct. 26,'85
	removal rate (%)	Kuroboku soil	42.0	97.6	0	0	77.1	—	>50	—	79.7	
		soil mixture	99.9	92.7	0	>98.3	≒100	—	>50	—	99.7	
Winter	Concentration of original odors		728	0.590	0.065	0.722	1.78	<0.0005	0.03	<0.04	7,200*	
	Concentration of treated odors	Kuroboku soil	280	0.059	0.038	0.286	1.69	<0.0005	<0.001	<0.04	4,200*	Feb.2,'86
		soil mixture	3.5	0.047	0.014	0.023	0.094	<0.0005	<0.001	<0.04	170*	* Gathered analyzed on Feb.4
	removal rate (%)	Kuroboku soil	61.5	90.0	41.5	60.4	5.1	—	>66.7	—	41.7	
		soil mixture	99.5	92.0	78.5	96.8	94.7	—	>66.7	—	97.6	

a) Removal of ammonia

Ammonia concentration in the original odors while fermentation was in process was 180ppm and showed little seasonal fluctuations except in winter (Figure 6).

Ammonia removal rate of soil mixture was high regardless of the season at about 98% on the average (Figure 8). It was particulary high in autumn. The concentration of treated odors was also close to nil at that time (Table 11). This is because the environment for the micro-organisms to act and the physical properties of soil came to be stabilized during the eight month lapse since the installation of the equipment.

Ammonia removal rate of Kuroboku soil varied greatly according to the season. While it was as high as 96% on the average during the rainy season, it frequently went down to 60~80% during the other seasons (Figure 7). The reason for high removal rate during the rainy season is that rain improved the chemical properties of soil and activated microorganisms.

Table 11. Summary of ammonia removal

Item \ Investigation period		Spring (May'85)	Rainy season (July'85)	Summer (Aug'85)	Autumn (Oct'85)	Winter (Feb'86)	Average
concentration of original odors (ppm)		189 (140~260)	181 (106~360)	151 (40~480)	140 (40~255)	256 (35~728)	183
Concention of treated odors (ppm)	Kuroboku soil	57 (24~80)	7 (2~21.5)	38 (15~90)	45 (17~145)	101 (25~280)	50
	Soil mixture	5.7 (1.0~10)	2.7 (0.5~7.5)	4.2 (1.5~6.5)	0.1 (0~0.3)	2.0 (0.7~4.5)	2.9
Removal rate (%)	Kuroboku soil	69.8	96.1	74.8	67.1	60.5	72.7
	Soil mixture	97.0	98.5	97.2	99.9	99.2	98.4

note : Figures abone are the average of each investigation period and those in brackets are the maximum and minimum values during the period.

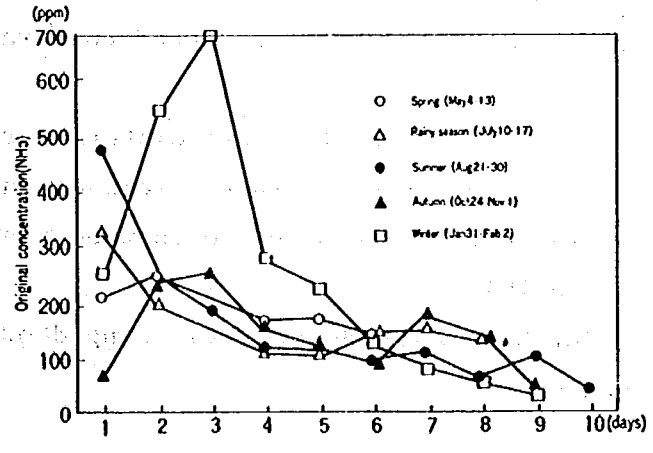


Figure 6. NH3 concentration in original odors.

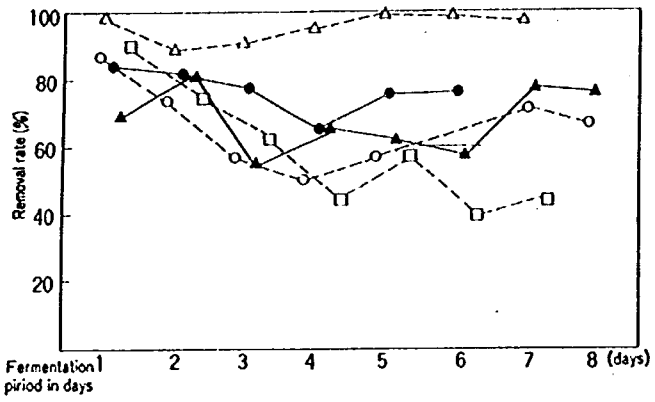


Figure 7. NH_3 removal rate by Kuroboku soil

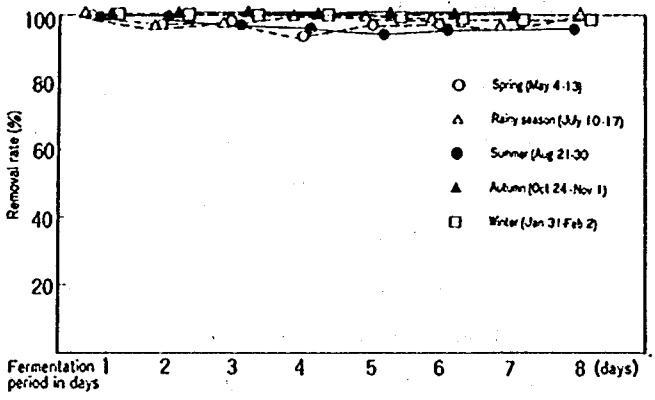


Figure 8. NH_3 removal rate by soil mixture

b) Removal of other substances

The original odors contained little acetaldehyde, stylene, or trimethylamine. However, sulfuric odors such as methyl mercaptain, methyl sulfide, and dimethyl disulfide were detected. Concentrations of these substances varied greatly in accordance with the season. In the soil mixture, dimethyl disulfide showed the most stable removal rate, which was 98.5% on the average.

Methyl sulfide removal rate was more than 96% after autumn when chemical and physical properties of soil were stabilized. Methyl mercaptan also showed the removal rate of more than 92% after autumn.

As far as the removal of sulfuric odors in Kuroboku soil is concerned, methyl sulfide removal went almost straight down and reached nil in autumn. Dimethyl disulfide showed high removal rate temporarily but this did not continue. Methyl mercaptan showed comparatively good results (Figure 9).

Concentrations of odors treated by the soil mixture were 20 to 170 when measured by the human sense. Along with the removal rate of more than 95%, these figures are quite satisfactory even when the tightest standard is referred to (Figure 10).

On the other hand, Kuroboku soil showed decreasing deodorizing capability after the rainy season.

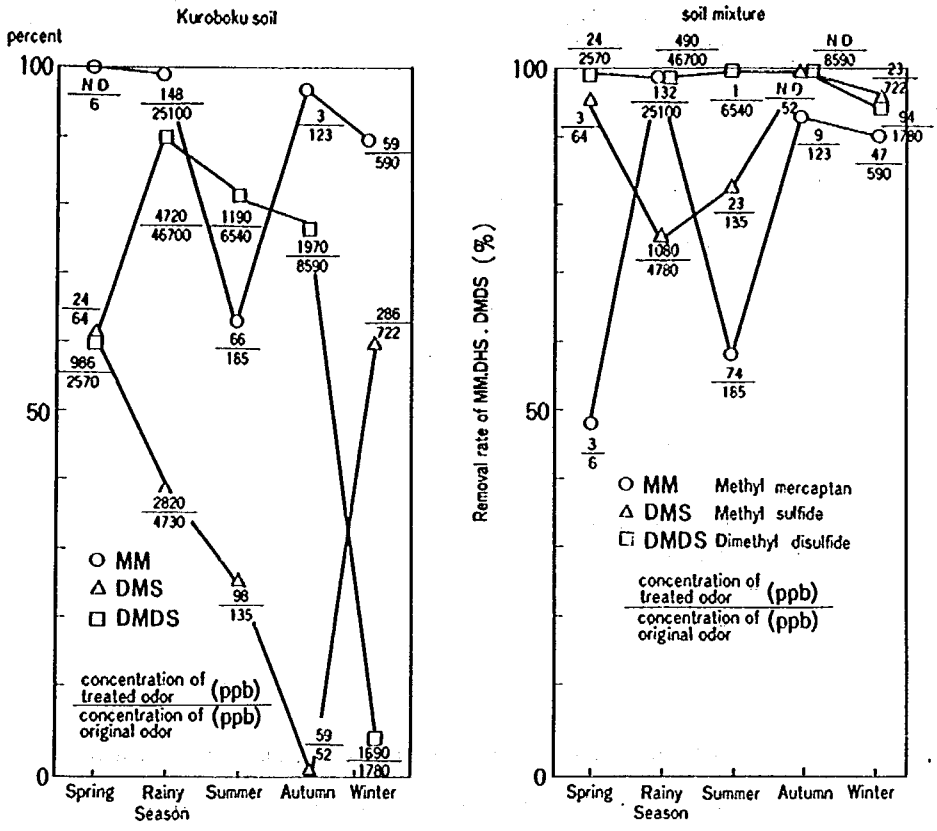


Figure 9. Variation of removal rate of methyle mercaptan, methyle sulfide, Dimethyle disulfide.

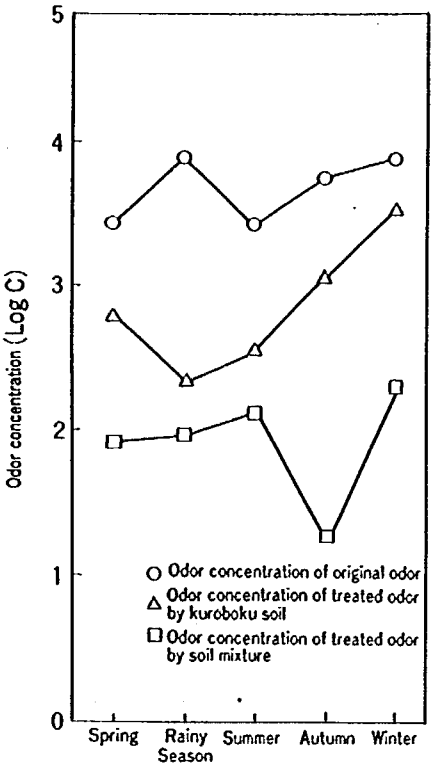


Figure 10. Variation of odor concentration

4.7 SOIL CONDITIONS

Soil conditions greatly affect the activities of microorganisms and the deodorizing capability of soil. Therefore, the conditions of both Kuroboku soil and the soil mixture were investigated every time the experiment was carried out.

a) Moisture content of soil

It is generally said that microorganisms act best when the water content of soil is approximately 60% of its maximum water holding capacity. The optimum water contents of Kuroboku soil and the soil mixture equivalent to the above-mentioned 60% are 59% and 54% respectively. Actual measured water contents of both test materials stayed close to these figures through all seasons, satisfying the water condition requirement.

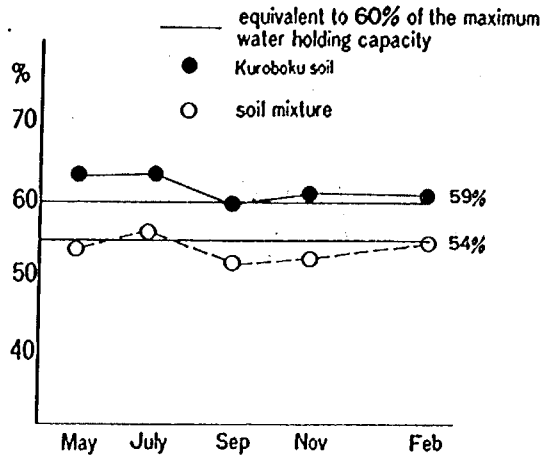


Figure 11. Variation of soil water content

b) Concentration of hydrogen ion

Since the pH value of Kuroboku soil was low, phosphate of lime was used to adjust it. The pH value of the soil mixture was more stable than that of Kuroboku soil. This is attributable to the improvement of its buffering behavior made possible by zeolite which has high cation exchange capacity, lime, and organic substances contained in compost.

The optimum pH value for microorganisms to facilitate nitrification or denitrification is approximately 7. This means that the soil mixture is more suitable for the removal of ammonia than Kuroboku soil. The pH value of the soil mixture declined as nitrate nitrogen accumulated. Meanwhile, thiobacillus sp., a microorganism known for its attribute to oxidize sulfuric compounds, acts best at the pH value of 2-3.

This shows that the pH value suitable to remove ammonia is not always good for the removal of hydrogen sulfide.

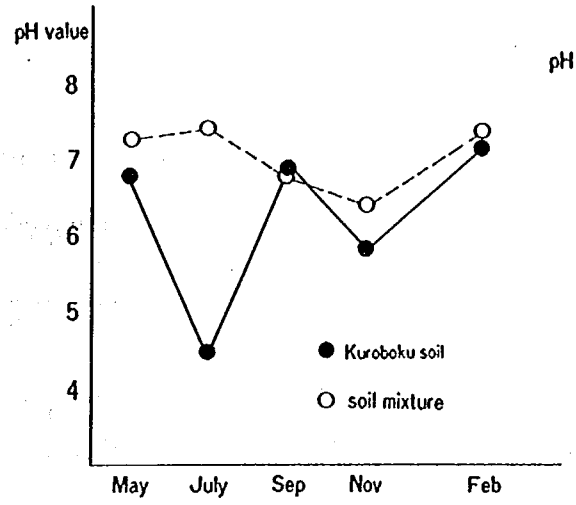


Figure 12. Variation of soil pH value

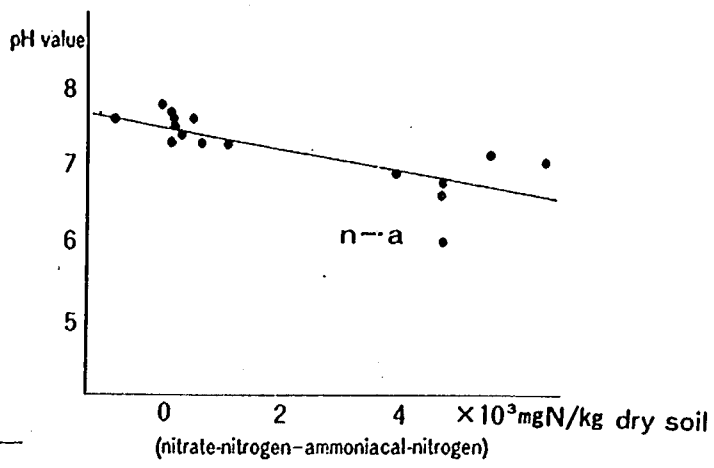


Figure 13. Decline of pH value with the accumlation of nitrate-nitrogen (soil mixture)

c) Temperature distribution

Activities of microorganisms in soil are also affected by the temperature of soil. The optimum temperature range for nitrification is 25°C - 30°C. If the soil temperature exceeds 45°C or drops below 15°C, nitrification will be slowed down.

There was little difference between the temperature of the soil mixture and that of Kuroboku soil, the highest for the two materials in the hot summer season having been 36.2°C and 36.3°C and the lowest in winter having been 11°C and 10.8°C respectively.

In winter, the temperature of the surface laywer went down below 15°C, but those of the middle and lower layers remained above 15°C (Figure 14). With these relatively high temperatures in the inner part of the soil bed, it seems that the cold does not lower the deodorizing capability of either of the two materials greatly except when it snows.

The soil temperature varies within the range between atmospheric temperature and the temperature of the influent gas. Its hourly change within the day was little (Figure 15).

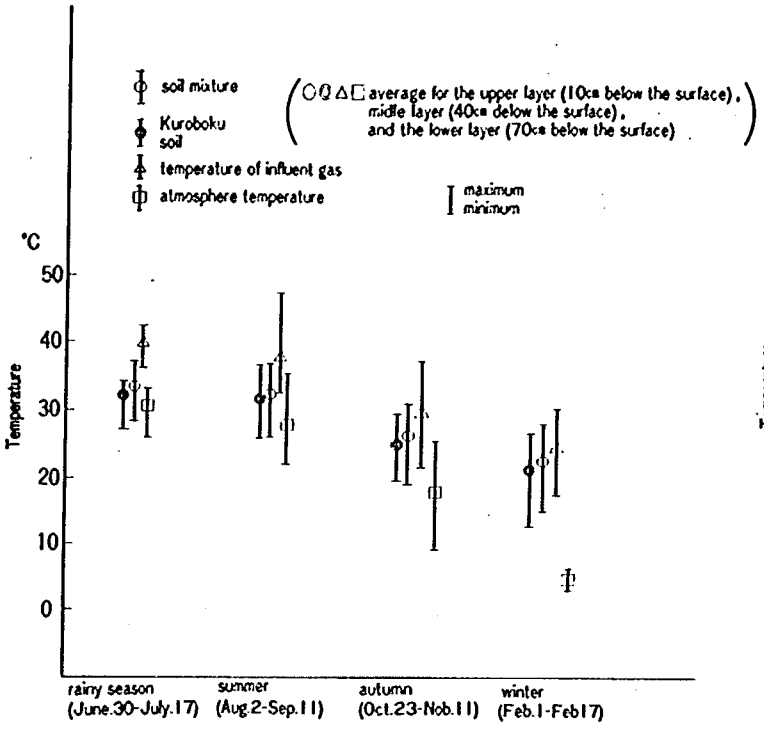


Figure 14. Seasonal variation of temperatures (from P.M.0:00 to P.M.1:00)

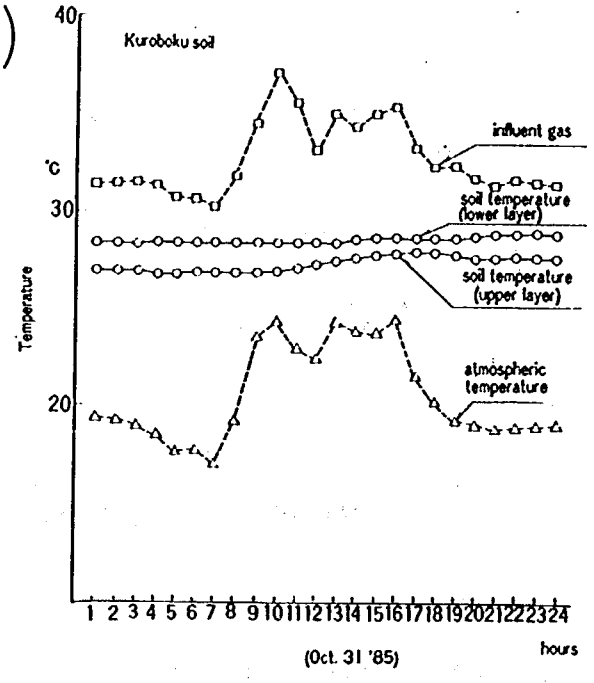


Figure 15. Hourly variation of temperatures

d) Distribution of three phases

The investigation of three phase distribution in the test materials revealed that the percentage of the gaseous phase of the soil mixture was 5-10% higher than that of Kuroboku soil and that the percentage of its liquid phase was 10% lower than that of Kuroboku soil. This is one of the reasons for the high air permeability and water discharging capacity of the soil mixture. (See Figure 16)

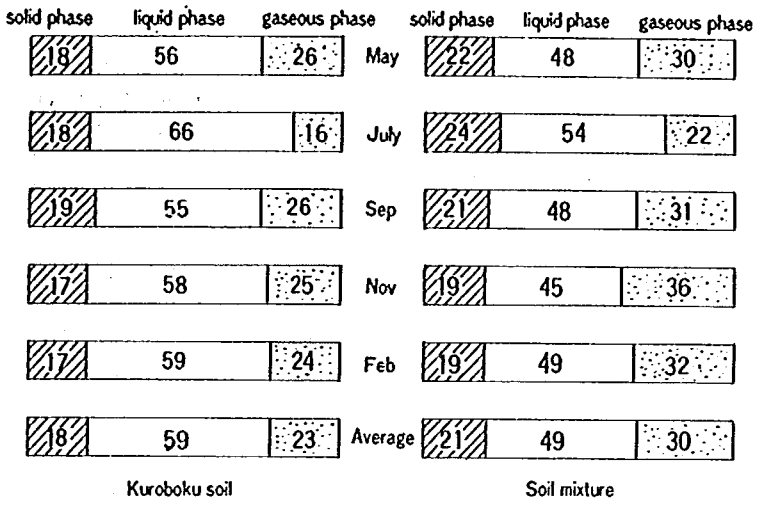


Figure 16. Variation of three phase distribution in soil (in therm of volume)

One of the operation-related problems of the soil deodorization system is the formation of short-cut passages which often takes place when the liquid phase is unevenly distributed in the soil. According to the results of the investigation on the liquid phase distribution, the lower the layer was, the higher the liquid phase percentage was. It was also learned that the difference in liquid phase percentages on the planar basis was only 4% for the soil mixture and no short-cut passages were found until November.

On the other hand, the planar difference in the liquid phase percentages of Kuroboku was more than 6%. This seems to have caused short-cut passages to form and subsequently the soil surface to be depressed. When soil surface depression occured, the soil was turned of sprinkled with water to counter it. As time went by, the planar difference in the liquid phase percentages was on the decline and no short-cut passage were seen to form.

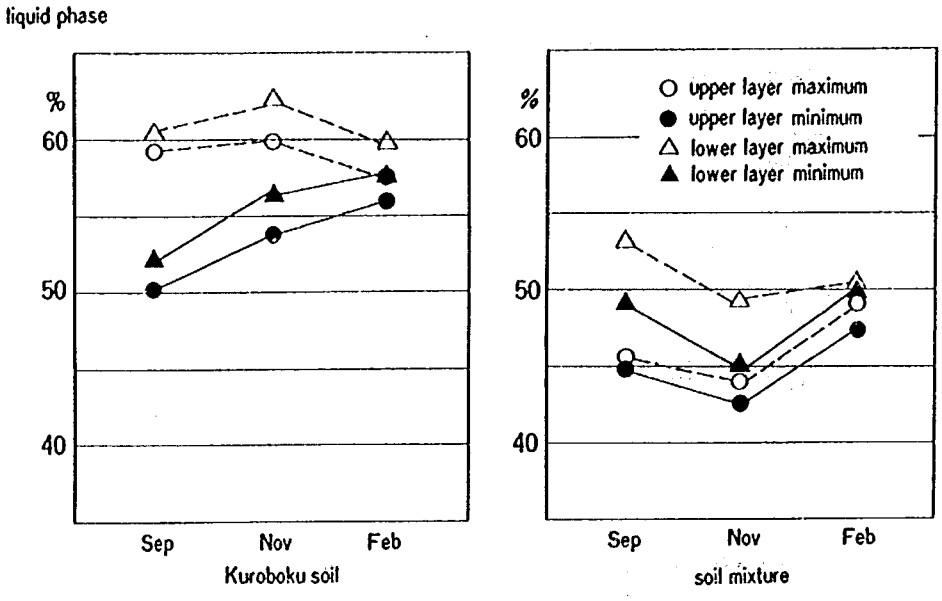


Figure 17. Plane distribution of liquid phase

4.8 INVESTIGATIONS ON NITROGEN COMPOUNDS AND MICROORGANISMS IN SOIL

Since the original odors consist mainly of ammonia, the recent experiment concentrated on the investigation of the content of nitrogen compounds in the soil and microorganisms that were closely related to the removal of ammonia.

4.8.1 REMOVAL OF AMMONIA

Ammonia which has flown into soil is first dissolved into moisture contained in the soil and adsorbed by soil colloid. Then it is oxidized and reduced. The amount of $NH4^+$ ions adsorbed by soil is affected by the cation exchange capacity of soil.

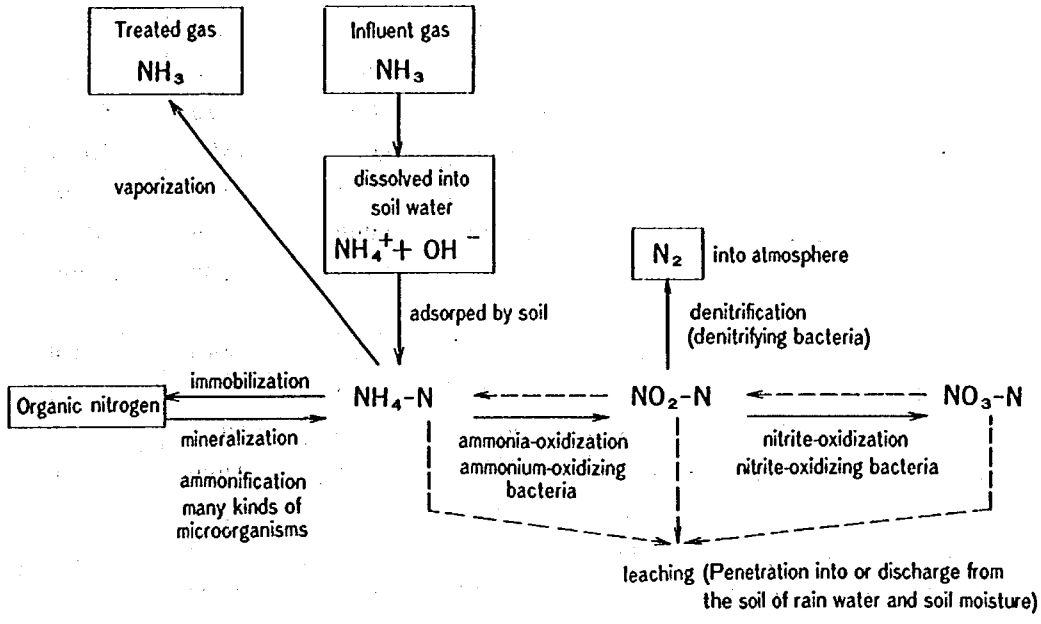


Figure 18. Ammonia removal process by deodorizing soil

4.8.2 VARIATION OF NITROGEN COMPOUNDS

Table 12 shows the results of investigation on the variation of nitrogen compounds.

Nitrite nitrogen (NO_2-N) and nitrate nitrogen (NO_3-N) in the soil mixture accumulated in May; however, as the precipitation in the latter half of June washed them away to a great extent, their accumulation turned low after July and the removal rate of ammonia increased.

In Kuroboku soil, no accumulation of nitrite nitrogen was seen until July and the ammonia removal rate was high at an average percentage of 96. However, nitrite nitrogen increased rapidly when investigated in September and so did ammoniacal nitrogen. This is attributable to the low cation exchange capacity of Kuroboku soil which increased the concentration of free

NH_4^+ in the soil solution and gave adverse effects on the nitrogen metabolism of microorganisms.

Table 12. Chemical properties of soil

	Item \ Research season	symbol	unit	Feb. befor the first odor blow	Spring (May.)	Rainy season (July.)	Summer (Sep.)	Autumm (Nov.)	Winter (Feb.)
Kuroboku soil	Moisture content	H_2O	%	56.8	62.5	62.6	58.2	59.8	59.6
	Hydrogen-ion concentration content	$pH(H_2O)$	—	5.2	6.8	4.5	6.9	5.8	7.1
	Content of organic matter	Org	%	27.8	27.2	24.5	24.4	28.1	27.9
	Organic nitrogen	Org−N	mg/kg·DS	5,820	11,500	7,090	5,920	8,050	5,670
	Ammoniacal nitrogen	NH_4−N	mg/kg·DS	<10	110	3,030	5,490	7,170	12,130
	Nitrite nitrogen	NO_2^-−N	mg/kg·DS	0.26	0.20	0.24	1.550	85	1.280
	Nitrate nitrogen	NO_3^-−N	mg/kg·DS	7.00	1,730	1,000	2,170	8,530	3,460
	Total nitrogen	T−N	mg/kg·DS	5,830	13,400	11,100	19,100	23,900	22,500
Soil mixture	Moisture content	H_2O	%	47.6	52.0	54.7	49.4	50.6	53.7
	Hydrogen-ion concentration content	$pH(H_2O)$	—	7.6	7.2	7.4	6.8	7.4	7.4
	Content of organic matter	Org	%	21.3	20.2	20.0	20.9	23.0	20.8
	Organic nitrogen	Org−N	mg/kg·DS	6,920	10,290	7,320	8,090	8,250	7,890
	Ammoniacal nitrogen	NH_4−N	mg/kg·DS	990	920	77.3	206	61	53
	Nitrite nitrogen	NO_2^-−N	mg/kg·DS	1.83	109	2.58	2.3	0.39	3.94
	Nitrate nitrogen	NO_3^-−N	mg/kg.DS	198	6,870	92.3	2,920	722	702
	Total nitrogen	T−N	mg/kg·DS	8,110	18,200	7,490	11,200	9,030	8,640

the average between the upper layer (20cm deep) and lower layer (60cm deep)

4.8.3 RESULTS OF THE INVESTIGATION ON MICROORGANISMS

This investigation was made before the first blow of odors into soil and, after that, every time the experiment was carried out.

Before the first blow, sulfur bacteria were relatively small in number both in Kuroboku soil and in the soil mixture. However, denitrifying bacteria were found in large quantities in the soil mixture numbering 100 times as many as those in ordinary soil. The presence of compost in the soil mixture is considered to be the reason for this large number.

Changes in the numbers of various bacteria with time after the first experiment showed almost the same pattern for both Kuroboku soil and the soil mixture.

Table 13 below shows the kinds of microorganisms contained in the soil and their numbers counted before and six months after (in September) the first experiment. The ammonia removal rates of the soil mixture and Kuroboku soil in September were 99.9% and 67.1% respectively.

Table 13. Excerpt of the results of investigation on microorganisms in the soil

(number of bacteria/g-dry soil)

		ammonium-oxidizing bacteria	nitrite-oxidizing bacteria	denitrifying bacteria	mold fungi	sulfur bacteria
Kuroboku soil	before the first blow (Feb.'85)	3.4×10^4	1.1×10^3	1.1×10^4	5.7×10^3	8.7×10
	after the first blow (Sep.'85)	1.7×10^7	3.0×10^5	3.7×10^4	5.3×10^3	7.5×10
soil mixture	before the first blow (Feb.'85)	7.1×10^3	1.1×10^3	1.2×10^6	7.2×10^3	1.7×10^2
	after the first blow (Sep.'85)	1.1×10^6	1.7×10^6	6.1×10^6	1.8×10^4	1.5×10^2
counting method		most probable number method	most probable number method	most probable number method	Dilution plate method	most probable number method
culti-vation		counting culture medium for ammonium-oxidizing bacteria	counting culture medium for nitrite-oxidizing becteria	giltaye culture meduim	Pose Bengal agar	Pfenning culture medium

Microorganisms which were related to the removal of ammonia were on the increase after the first experiment, but there was a difference in the number of bacteria between Kuroboku soil and the soil mixture.

In the soil mixture, the ratio of ammonium-oxidizing bacteria and nitrite-oxidizing bacteria was 2 to 3 which was well-balanced to carry out smooth nitrification whereas that in Kuroboku soil was 57 to 1. This is considered to be the main reason for the accumulation of nitrite nitrogen in Kuroboku soil. Also in the soil mixture, denitrifying bacteria which facilitate smooth denitrification were found in large numbers. Coupled with a stable pH value and a large water discharge capacity which enables nitrate nitrogen to run out, this makes the soil mixture very suitable for the removal of ammonia.

The numbers of mold fungi and sulfur bacteria didn't vary greatly throughout the year. The low numbers of mold fungi are mainly attributable to the high concentrations of carbon dioxide and ammonia which were contained in compost fermentation waste gas and served to prevent the growth of the fungi.

The reasons for the low rate of hydrogen sulfide removal are that sulfur bacteria in the soil were insufficient in number at an initial stage, that the pH value of soil was too low, and that too much $NH4^+$ adsorbed by the soil affected sulfuric metabolism of microorganisms.

4.9 PERMEATION RESISTANCE AND THE PASSING SPEED OF ODORS

Permeation resistance and odor passing speed are important design considerations for the soil deodorization equipment. The smaller and more stable the pressure loss is, the better the deodorizing equipment is for practical use. At the same time, if the speed of odors passing through the soil can be raised, the area of the soil bed can be reduced. Therefore, investigation was made on the permeation resistance and the passing speed of odors (Figure 19).

a) Permeation resistance

When odors are blown into the soil bed continuously, permeation resistance varies in accordance with precipitation, dryness of the soil, consolidation of the soil caused by the soil's self-load, and the odor amount adsorbed by the soil. When the permeation resistance is too small at a certain point, short-cut passages may form there and send the original odors directly to the soil surface.

Compared with Kuroboku soil, the permeation resistance values of the soil mixture were lower and their seasonal fluctuations were smaller. When a comparison was made between the permeation resistance values of Kuroboku soil and those of the soil mixture, a wide gap was seen between the values of the two from the start of the first experiment to July. This is attributable to Kuroboku soil's more susceptibility to rain resulting from a smaller grain diameter, larger water-holding amount, and smaller gaseous phase percentage (Table 14). The permeation resistance

value of Kuboboku soil was 14mmH_2O for a soil height of 100mm while that of the soil mixture was 6mmH_2O to 7mmH_2O:

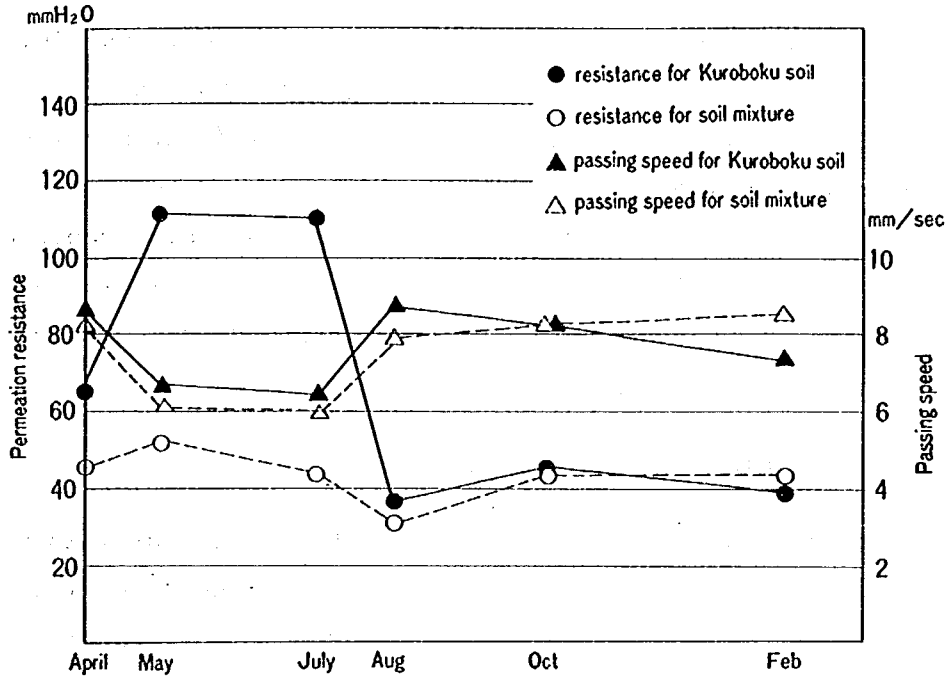


Figure 19. Permeation resistance and odor passing speed

Table 14. Variation of gaseous ratio in soil

Time of investigation	gaseous ratio (%)	
	Kuroboku soil	Soil mixture
Before the first odor blow	9.6	28.2
Spring (May 18)	26.0	31.5
Rainy season (July 5)	15.6	21.7
Summer (Sep 4)	26.0	30.2
Autumn (Nov 2)	24.7	35.8
Winter (Feb 8)	25.1	32.0
Average	21.2	29.9

b) Speed of odors

It is generally said that the soil deodorization system loses its deodorizing capability regardless of the concentration of odors to be treated when the passing speed of odors exceeds a certain limit because the soil gets too dry and short-cut passages are formed.
Through past experience, the passing speed of 5mm/sec. has generally been chosen for soil deodorization. However, the results of the recent experiment revealed that effects of deodorization were stable even at a speed of 8mm/sec. if the soil mixture was used. This raised the possibility of reducing the area for the soil bed to be installed by about 40%.

4.10 QUALITY AND AMOUNT OF WASTE WATER

Waste water from the deodorizing equipment consists of rain water and coagulated water of the influent gas in the soil. Since the drain pipe was common for the beds of the soil mixture and Kuroboku soil in the recent experiment, the investigation on the waste water discharge was unable to carry out separately for each of the two test materials. However, alkalescence detected in the waste water is believed to be attributable to the conditions of the soil mixture.

Waste water discharge did not always increase in proportion to the precipitation. This means that there was a large amount of coagulated water of the influent gas in the soil due to high temperature and humidity of the atmosphere.

The average water discharge per day was 168ℓ/200m^2. BOD, COD, and nitrogen were found in large amounts.

Table 15. Quality of waste water from the soil bed

Item checked		May 18	July 5	Sep.4	Nov.2	Dec.9	Feb.8	Average
PH	–	8.1	7.7	7.6	8.2	7.6	7.9	7.9
SS	mg/ℓ	8	13	11	9	16	8	11
BOD_5	mg/ℓ	–	–	–	52.6	124	46.3	74.3
COD	mg/ℓ	1200	748	1350	482	408	443	772
$NH_4 - N$	mg/ℓ	153	822	938	704	706	1410	789
$NO_2 - N$	mg/ℓ	870	0.17	1060	41	417	265	442
$NO_3 - N$	mg/ℓ	1860	150	1140	2540	776	1270	1289
T–N	mg/ℓ	3760	972	3680	3300	1900	2950	2760
coliform group	colony/ml	3	3	3	70	4	0	14

4.11 COST ESTIMATE

Table 16 is a comparison of the concentration of treated odors with the permissible level of odors. As is seen in this table, all the substances except hydrogen sulfide were removed to the extent that their concentrations dropped below the legally permitted values. Though the concentration of hydrogen sulfide exceeded the permissible value, the value actually measured is deemed satisfactory since the permissible values are the standars for the boundary of a plant.

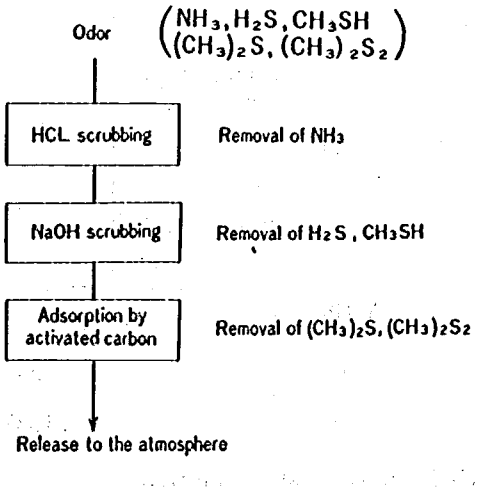


Figure 20. Flow chart of deodorizing process now in operation

Table 16. Comparison of original odors and treated odors after deodorization

	Concentration of original odors		Remoral rate of soil mixture	Concentration of treated odors	Permissible concentration
Ammonia NH_3	180 ppm		99.5 %	0.9 ppm	1.0 ppm
Methyl Mercaptan CH_3SH	10 ppb	Odor strength 3.5	92 %	0.8 ppb	2 ppb
Hydrogen Sulfide H_2S	200 ppb		78.5 %	43 ppb	20 ppb
Methyl Sulfide $(CH_3)S$	200 ppb		96.8 %	6.4 ppb	10 ppb
Dimethyl Disulfide $(CH_3)_2S_2$	100 ppb		94.8 %	5.2 ppb	9 ppb
Remarks			Measured in Jan'86		Odor strength 2.5

Figure 20 shows the flow chart of the deodorizing process now in operation at the city's composting plant and Table 17 shows the comparison of its costs with those of soil deodorization. This comparison reveals that the new system would require less construction and operational costs. As far as operational costs are concerned, it is estimated that the new system can reduce them to 1/8th of what the conventional system needs. It is also expected that operational costs can be cut down by more than 30% when the soil mixture is used instead of Kuroboku soil.

Table 17. Comparison of operational costs and estimate of construction cost

Air flow rate 570m^3/min		Conventional system			Soil deodorization method with soil mixture	
		Unit cost	Consumption	Running cost (10^4yen/year)	Consumption	Running cost (10^4yen/year)
Electric power	Deodorizing blower		75 KW		22KW	
	Circulation pump		22.5KW		—	
	Chemical injection pump		0.8KW		—	
	Sub total	20yen/KWH	98.3KW	1,722	22KW	386
Chemicals consumption	35% Hydrochrolic acid	30yen/kg	27.2kg/haur	715	—	—
	48% Sodium hydroxide	40yen/kg	18.7kg/haur	655*	—	—
	Activated coal	30×10^4yen/m^3	9.7m^3/year	291	—	—
Service water		350yen/m^3	12,719m^3/year	445	0.9m^3/day× 60days/year	2
Consumables				30		—
Iabor cost for weeding and others		15,000 yen/person·day	—	—	50 persons·days/year	75
Cost for treating waste water		300yen/m^3		—	700m^3/year	21
An nual operational costs(10^4yen/year)				3,858		484
Construction cost(10^4yen)				22,400		7,620
Site area (m^2)				400		1,200

※ 3,000ppm of CO_2 emanating from the composting plant is included in the cost calculation

5 SUMMARY

The recent experiment was aimed at studying the effects of the deodorizing method which used the soil mixture for the soil bed on offensive odors which emanated from composting processes. The soil mixture was composed of Kuroboku soil, the most frequently used soil for soil deodorization to date, sludge compost, and zeolite.

As a result, it was found that:

(1) Sludge compost increased air permeability and water permeability of the soil and improved buffering capability of the soil by increasing the number of microorganisms such as denitrifying bacteria.

(2) Zeolite improved odor adsorbing capability of the soil and helped secure the living space for microorganisms to decompose odors. Thus, the soil mixture proved to produce better results than Kuroboku soil.

(3) The experiment also showed the following results.

(i) The soil mixture can treat high concentration ammonia without treating it at a pretreatment facility to decrease odors in advance.
(ii) It is also applicable to odors whose concentration fluctuates.
(iii) It is possible to reduce the bed's area by 40% by making the odor passing speed 8 mm/sec instead of traditional 5 mm/sec if the residence time of odors in the soil remains unchanged.
(iv) Since the rate of hydrogen sulfide removal was only 78%, further efforts are needed to increase it.

As summarized, the experiment accomplished the initial purpose. From now, together with the continuation of the same experiments, investigations on the following points are now planned. The new investigations are expected to help improve the odor removal capability of the soil bed.

(1) Effects of the temperature and humidity of odors on the removal of odors

(2) Conditions of the soil, waste water, and microorganisms related to the removal of sulfuric odors

(3) Effects of high concentration odors on the activities of microorganisms which decompose low concentration odors

6 CLOSING

Biological odor treatment which makes use of microorganisms in the soil or in activated sludge has been studied in a variety of fields because of its cost effectiveness compared with physical or chemical treatment. As a result, it is known that even hydrocarbon which is said to be difficult for microorganisms to decompose can be removed to a satisfactory level by this method. Since odors emitted by sewage treatment processes are complex odors each of which is of low concentration, there are great possibilities of utilizing biological odor treatment for sewarage-related facilities. Establishment of the technology for biological deodorizing process is strongly hoped for.

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