ge

generatie rioolwater-

zuiveringsinrichtingen

rwzi
2000

R92-03

FIRST DUTCH - JAPANESE WORKSHOP ON THE TREATMENT OF MUNICIPAL WASTE WATER

8-11 April 1991, Heelsum, The Netherlands

Part I

RIZA

Rijkswaterstaat
Rijksinstituut voor Integraal Zoetwaterbeheer
en Afvalwaterbehandeling

Postbus 17, 8200 AA Lelystad

stowa

Stichting Toegepast Onderzoek
Waterbeheer

Postbus 8090, 3503 RB Utrecht

Toe oolwaterzuiveringsinrichtingen RWZI 2000

projectleiding en secretariaat: postbus 17, 8200 AA Lelystad 03200 - 70411

rwzi
2000

FIRST DUTCH - JAPANESE WORKSHOP ON THE TREATMENT OF MUNICIPAL WASTE WATER

8 - 11 April 1991, Heelsum, The Netherlands.

PART I

Edited by:

J.J. Heijnen
W. van Starkenburg
G.B.J. Rijs

RWZI 2000 92-03

Het onderzoek "Toekomstige generatie rioolwaterzuiveringsinrichtingen RWZI 2000"
is een samenwerkingsverband van de STOWA en Rijkswaterstaat (RIZA).

PREFACE

The treatment of municipal waste water and the further processing of sewage sludge presents at present a formidable task, due to the increasingly stricter emission standards for the effluent of a treatment plant and the disposal problems for the remaining sludge.
Until recently research in this field was planned at the moment when the problem was identified. The short term was considered to be more important than the rather long term. The long-term work was usually considered as being too fundamental. This is changing slowly now. Also the necessity to remove more and more pollutants makes the treatment of municipal waste water more complicated, requiring more advanced research.
Therefore it is, especially now, desirable to exchange knowledge between technologists, micro-biologists and representatives of the water authorities not only within one country but also between countries.
Japan and The Netherlands are both densely populated countries with a high degree of industrialization. It turned out that the research programmes and targets in the field of waste water treatment showed similarities. Although there are also marked differences e.g. in the approach of the problems and in the quality and quantity of municipal waste water.
For this reason it is most welcome that a workshop has been organised by prof. Y. Watanabe (Miyazaki University), prof. J.J. Heijnen (Delft University) and ir. W. van Starkenburg (projectleader RWZI 2000) to promote professional and personal contacts ánd information exchange by both presentations and discussions.

This report contains the workshop report and all presentations.

The workshop is intended to become a regular event, each two or three years. The second workshop is scheduled in Japan in autumn 1993.

Lelystad, June 1992

On behalf of the Steering
committee RWZI 2000

dr. J. de Jong

1 PROGRAMME

Sunday 7 April 1991

Arrival and welcome

Monday 8 April 1991 9.00-12.30
chairman: prof. M. Okada

Opening by prof. J.J. Heijnen

NATIONAL POLICYY AND RESEARCH PROGRAMMES

1 Water quality management in the Netherlands, by dr. J. de Jong

2 Water quality management in Japan, by prof. I. Somiya

3 "Aqua Renaissance '90" and "Biofocus" projects, by prof. H. Aya

4 Treatment of municipal waste water in the future, by ir. W. van Starkenburg

5 Biological nitrogen removal by step-feed process, by dr. I. Miyaji

12.30 - 14.00 Lunch

Monday *8 April 1991 14.00-17.20*
chairman: prof. M. Okada

TREATMENT OF SEWAGE SLUDGE

6 Gravitational thickening process for sludge, by prof. T. Kusuda

7 Some fundamental aspects of sludge dewatering, by prof. P.J.A.M. Kerkhof

8 Sludge Management in Japan, by mr. M. Ibuki

9 New techniques of sludge management in the Netherlands, by ir. W.G. Werumeus Buning

17.30 - 20.00 Dinner

20.00 - 21.00 Discussion in groups
- New techniques sludge management
- Sludge thickening
- Sludge dewatering

21.00 - 21.30 Plenary discussion
chairman: ir. E. Eggers

Tuesday 9 April 1991 9.00-12.30
chairman: ir P.C. Stamperius

REMOVAL OF NITROGEN AND PHOSPHATE

10 Two methods of nitrogen removal in Japan, by mr. Y. Hamamoto

11 N-removal: research and state of the arts in the Netherlands, by prof. J.H.J.M. van der Graaf

12 Stability of phosphorus removal and population of bio-P-bacteria under short term disturbances in sequencing batch reactor activated sludge process, by prof. M. Okada

13 P-removal: state of the arts in the Netherlands, by ir. T.W.M. Wouda

12.30 - 14.00 Lunch

14.00 - 17.00 Excursion to waste water treatment plant "Bennekom" and the research facilities of the Agricultural University Wageningen;
Introductions by ir. J.H. Rensink and ir. P.J. Tessel

17.30 - 19.00 Dinner

14 Simultaneous nitrification and denitrification for N-removal, by prof. J.G. Kuenen

15 Accumulation of polyphosphate by *Acinetobacter sp.*: physiology, ecology and application, by prof. A.J.B. Zehnder

20.00 - 21.00 Discussion in groups
- Biological N- and P-removal

21.00 - 21.30 Plenary discussion

Wednesday 10 April 1991 9.00-12.30
chairman: mr. Y. Hamamoto

ADVANCED WATER TREATMENT TECHNIQUES

16 Study on practivability of U-tube aeration, by dr. A. Toriyama

17 Multireactor and Deep-Shaft systems, feasibility for sewage treatment in the Netherlands, by ir. E. Eggers

18 Membrane bioreactors for waste water treatment, by prof. K. Yamamoto

19 Scale-up of the aerobic airlift suspension biofilm reactor, by prof. J.J. Heijnen

12.30 - 13.30 Lunch

20 Possible role of anaerobic digestion in sewage treatment, by prof. G. Lettinga

21 The influence of retention time on reactor performance and bacterial trophic populations in anaerobic digestion process, by prof. T. Noike

22 A physiological approach to waste water treatment with complete sludge retention, by prof. A.H. Stouthamer

23 Control and automation aspects in biological denitrification processes in Japan, by mr. T. Aoi

19.00 Official dinner with guests

Thursday 11 April 1991 9.00-12.40
chairman: prof. I. Somiya

MORE EXTENSIVE TREATMENT OF MUNICIPAL WASTE WATER IN FUTURE

24 Effluent 2000 Japan, by prof. H. Nakanishi

25 Needs for research in future, by ir. W. van Starkenburg

26 Municipal waste water treatment system combining physico-chemical and biological processes, by prof. Y. Watanabe

27 Bactivorous grazers: a way to reduce sludge production, by prof. S.A.L.M. Kooijman

28 Biological removal of xenobiotics, by prof. B. Witholt

12.40 - 13.30 Lunch

13.30 - 14.15 Closing of the Dutch - Japanese Workshop by ir. W. van Starkenburg

14.15 - 18.00 Excursion to municipal waste water treatment plant "Dokhaven"

2 SUMMARY

2.1 *Introduction*

From April 8 - 11, 1991 the first Dutch - Japanese workshop on the treatment of municipal waste water has been organized at Heelsum in the Netherlands.
The aim of this workshop was to exchange technical knowledge with regard to the treatment of domestic waste water and the further processing of sewage sludge in Japan and the Netherlands.

The reason for this exchange of knowledge and experience between these two countries is the great similarity with regard to the population density, resulting problems such as lack of area, intensive pollution of waste water and the generation of waste products. Both countries usually choise similar ways to solve these problems. Nevertheless, there are also several differences, for example the customs and the legislation, and also the composition of the waste water, by which the selection for using a specific treatment system will be affected.

About 35 persons attended this workshop; 13 of them from Japan, the others from the Netherlands. The participants represented different diciplines, e.g. public water authorities, industry and universities. In total 28 presentations have been given and two excursions have been organized. One excursion to the waste water treatment plant "Bennekom", including the research facilities of the Agricultural University Wageningen and one to the underground situated waste water treatment plant "Dokhaven".
The presentations were divided into the following themes:
- national policy and research programmes;
- processing of sewage sludge;
- removal of nitrogen and phosphate;
- advanced water treatment techniques;
- more extensive treatment of municipal waste water in the future.

2.2 *National policy and research programmes*

Both Japan and the Netherlands, are densely populated countries with a high degree of industrialization. Especially after World war II, rapid population growth occured and an explosive increase in industrial activity charged surface waters with intolerable loads of oxygen - consuming substances and heavy metals. Therefore, several research programs and laws, regarding this subject, were initiated.

In the Netherlands the Pollution of Surface Water Act was introduced in 1970. Its purpose was to regulate discharges and to keep the surface waters as clean as possible. For municipal waste water the activities are mainly focused on the collection via municipal sewerage systems and central treatment in biological sewage treatment plants. The result so far is that just over 90 % of municipal waste water is collected via sewerage systems and more than 85 % is treated biologically. This is a relatively high amount, certainly as compared to Japan. In Japan 96 % of night soil and 45 % of municipal and domestic waste water are treated in sewage treatment plants, Jokasou (the treatment facilities for flush toilet, or flush toilet and household waste water) and night soil treatment plants (the treatment facilities for storage tank toilet).

In Japan the national water quality policy was started in 1958 by introducing two water quality laws. One was concerned with conservation of water quality on public water areas, and the other on effluent from factories. In 1967 these both laws were unified in the form of a new reinforced Water Pollution Control Law. Besides several amendments during the following years, this law was revised again in 1990 to institutionalize the measures for household effluent.

In Japan the following four ministeries are responsible for the water quality management; Ministery of Health and Welfare (night soil, drinking water), Ministery of Construction (municipal waste water, sewage sludge), Ministery of Agriculture and Fishery (municipal waste water in rural areas) and the Ministery of Environment (Quality standards). In the Netherlands the primairy responsability for the water management, both quality and quantity, rests with the Ministery of Transport, Public Works and Water Management ("Rijkswaterstaat") as far as State managed waters and territorial waters are concerned. The water bodies are divided into large waters of national importance, such as the internatonal rivers Rhine, Meuse and Scheldt, the so-called State managed waters. The responsability for non-state waters lies with the provinces. Eight of the twelve provinces, however, delegated this responsability to regional water boards, which are functional public bodies exclusively authorized with water management tasks.

The Japan Ministery of Construction has conducted a research program "The Development of New Waste Water Treatment Systems Employing Biotechnology", which is commonly called "Biofocus". It was started in 1985 and will end in 1990. The present and future problems in waste water treatment have been approached from biotechnogical and microbiological views. It included various research projects, performed at basic and practical level, and divided into 8 subdivisions:
- composing a database with information on microorganisms that can be used in waste water treatment;
- the applicability of genetic engineering;
- methods to immobilize microorganisms;
- bioreactors in the treatment of waste water;
- bioreactors for the treatment of sewage sludge;
- biosensors
- suspended solids/water - separation methods
- systematic evaluation of investigated and newly developed technologies and their integration into novel treatment systems.

At the same central test site the development and evaluation of new concepts of waste water treatment, for example an energy-saving type, area-saving type and simultaneous nitrogen & phosphorus removal type was carried out. By using the same waste water, results of the different concepts can be easily compared. Comparison with the Dutch research program "Treatment of Municipal Waste Water in the Future" or shortly "RWZI 2000" shows a remarkable degree of similarity. Both research programs have as the main objectives:
- the decrease of energy consumption and operating costs;

- the development of compacter systems;
- the improvement the effluent quality;
- the recycling of reusable components.

The difference between these researchprogrammes is found in the involvement of the industry (more in "Biofocus") and the universities (more in "RWZI 2000")

RWZI 2000 started in 1988 and will continue till 1993 in a joint effort of the "Institute of Inland Water Management and Waste Water Treatment" ("RIZA") and the "Foundation for Applied Waste Water Research" ("STORA"). This research program consists of a number of studies in the field of waste water treatment and sludge management in different gradation; such as long-term fundamental research, practical research and evaluation study.

The "Aqua Renaissance '90" project is another Japanese government directed research program in the field of water reclamation and methane gas recovery from industrial and domestic waste water, supported by the Ministery of International Trade and Industry. The objective is to develop a treatment system of low cost, low energy consumption and small space requirement, employing combination of anaerobic process and membrane separation. The objective for the membrane separation devices is to develop membrane modules with a power consumption of less than 1.5 kWh/m^3 at sludge concentrations of 10 g/l, and of less than 0.3 kWh/m^3 at biomass concentrations of 0.1 g/l. For the anaerobic bioreactor the target value was a gas conversion rate of more than 80 % at a volumetric organic loading rate of at least 5 kg BOD/reactor m^3.day at BOD concentrations $\geq$ 2,000 mg/l of the waste water; and a gas conversion rate $\geq$ 60 % at a volumetric loading rate $\geq$ 2 kg BOD/reactor m^3.day with a BOD level of the waste water between 200 - 1000 mg/l.
For this program (1985-1991) a special organization, the Aqua Renaissance Research Association, which consists of several national research institutes and more than twenty private companies, has been founded.

2.3 *Treatment of sewage sludge*

In densely populated countries, like Japan and the Netherlands are, the disposal of sewage sludge becomes more and more a problem. The stricter requirements of heavy metals content for reusable sludge in the future will bring an end to the use in agriculture or in compost and soil production. This means that sewage sludge will change from a "fertilizer" into a waste product. It is thus becoming more and more important to minimize the volume of sewage sludge.
It will be necessary to improve the mechanical dewatering of sewage sludge and reduce the quantity of the remaining solids by composting, drying or incineration.

In the past sludge in the Netherlands was mostly used to produce compost in the form of fertilizer or for use in soil production. But by the stricter standards the purpose of composting changed into a dewatering technique only, which can reduce the volume and increase the dry matter content of sewage sludge. Composting in Japan has only a 4 % share of the total amount of generated sewage sludge; in the Netherlands it still is 30 %.

Another method by which the volume of the sludge can be reduced is thermal drying. Thermal drying increases the dry matter content, but the amount of organic matter remains nearly the same. Two examples of the so-called indirect drying systems, where there is no direct contact between hot air and the sludge, are the "Exergy" system and the "Carver Greenfield" process. They are both multi-stage drying processes, which use a carrier, steam or oil, as a transport medium and heat supply. In Japan there is some experience with a heavy oil variant of the Carver Greenfield process. The dried sludge contains a high percentage of oil (up to 35 %), which will be used as waste fuel for incineration. Drying is adapted as a pre-treatment for incineration.

Volume reduction by incineration of sludge has been used for a number of years in the Netherlands. About 3 % of the total amount of sewage sludge is treated by incineration in the Netherlands; in Japan more than 50 %. The incineration technique used depends on the fuel characteristics of the starting material, which is determined by the degree of dewatering and the ash or lime content. In Japan two types of incinerators are commonly used, the "Multiple Hearth Furnace" and the "Fluidized Bed Furnace". In the Netherlands the most practicable incineration systems for sewage sludge are fluidized bed, grate incinerator and powder burner.
The treatment of the flue gas and the leaching of heavy metals from ash are becoming more and more important in relation to incineration. For this reason a new type incinerator, the so-called melting furnace, is taken into consideration in both countries. In the melting process the dried sludge is brought into a liquid form at a temperature of 1400 - 1600 °C. At this temperature the organic material will be incinerated and the inorganic part of the sludge melted. Depending on the method of cooling a glassy or basaltlike slag is formed. When the cooling takes place quickly in water, a glassy endproduct is produced; cooling at the air results in a basaltlike slag. In both cases the slags are stable. Tests indicated that the leaching of the heavy metals in the slag can be brought back to a minimum. In Japan several types of melting furnaces are developed; for example "cyclone melting furnace", "film surface melting furnace", "coke bed melting furnace" and the "arc melting furnace".

The first full scale Vertech wet oxidation installation in the world will be completed in the Netherlands in 1992. In a wet oxidation process, organic contaminants are oxidized in a water phase with oxygen, at a temperature of at least 175 °C and under high pressure to prevent too much evaporation of water. In the Vertech system the oxidation reaction is performed in a 1200 m, vertical undergroud pipe reactor. The necessary pressure is achieved by the weight of the overlying gas/liquid mixture at the bottom of this pipe.

Besides optimizing these techniques for treatment of sewage sludge, new waste water treatment systems have to be developed, which will produce less (secondary) sewage sludge.
One way to reduce sludge production is to select micro-organisms with a high maintenance energy requirement. Reducing the biomass/substrate ratio is a possible selection mechanism. Under these conditions the bacterial growth rate is low and most utilized substrate is spent on maintenance processes. As a result biomass production is low. This can be carried out in reactors which permit biomass retention either by the use of carrier material or by membranes. In Japan as well as in the Netherlands

experiments show that much lower sludge production can be obtained.
Another option is to introduce organisms which predate on bacteria. Through predation the sludge will be converted into energy, water and carbon dioxide. Under optimum conditions the loss of energy will be maximized and the biomass production minimized.

It should be noted that despite the above mentioned research on low sludge production, sewage sludge will always be generated; for example the supply of primairy sludge from the sewer systems.
Therefore another fundamental research was started in the Netherlands. This study was ment to increase the knowledge and understanding of the problems regarding dewatering of sewage sludge. It should lead to the development of techniques to increase the dry solids content and to reduce the volume of sewage sludge. Products of this research should be:

- knowledge of the relation between various characterisation methods and physical parameters;
- theoretical simulation models by which filtration and compression behaviour can be predicted.

2.4 *Removal of nitrogen and phosphate*

In the Netherlands extensive nitrogen reduction is mainly achieved in oxidation ditches and low-loaded activated sludge plants. Nitrification (the oxidation of reduced nitrogen compounds, generally ammonia, to nitrite, nitrate or other nitrogen oxides) and denitrification (the reduction of oxidized nitrogen compounds to gases; N_2O, N_2) are the main processes. In oxidation ditches denitrification occurs in the anoxic zones between the aerobic zones. In activated sludge plants there are a number of possible configurations whereby the nitrate-containing water from the nitrification zone is brought into contact with BOD in an anoxic area. One of the configurations is pre-denitrification in a separate tank, to which nitrate-containing effluent is recycled to the first stage of the process.
The so-called "multistage step-feed process" claims to result in a reduced energy demand for the recycling of effluent to the denitrification tank and in reduced demand for carbon sources. In this system denitrification and nitrification compartments are arranged in alternation and waste water is equally divided into each of them. This system is being evaluated in Japan. It was found that treatment of night-soil, using the step-feed process with four steps, resulted in decreased operational costs and increased N-removal, when optimal methanol dosage and recycling ratio were chosen. An overall nitrogen removal rate of 99 % could be maintained.
Another configuration is a system with alternating aeration, where nitrification and denitrification occur consecutively in the same tank, which is alternately aerobic and anoxic. Examples of this system are the "Bio-Denitro"-process, the "Intermittent Cycli process" and the "Tricycle-Verfahren".

Besides the conventional autotrophic aerobic nitrification and the anaerobic denitrification there could be a possibility to eliminate ammonium in nitrogen gas in one processtep by simultaneous heterotrophic nitrification and aerobic denitrification.

In the Netherlands micro-organisms (e.g. *Thiosphaera panthatropha*) are identified who are able to oxydise simultaneously the organic carbon and ammonium and to reduce nitrate and nitrite to N_2-gas.

But also other systems for nitrogen reduction are feasable, like integrated and separate biofilm systems and methods of physiochemical removal such as ion exchanging, stripping and chemical precipitation of ammonium.

It can be concluded that in both countries a need for compact systems exists, in which the activity and the number of micro-organisms for nitrogen reduction remains high. Mentioned was the possibility of dosage of nitrifying micro-organisms or immobilized in polymers. More knowledge is needed about the production of NO_x during biological N-removal and about the combination of biological N- and P-removal.
There was also attention for (fuzzy)control and automation aspects in a biological denitrification reactor by directly monitoring of NH_4-N and NO_x-N in a mixed liquid. The problem of pollution of the monitoring equipment was solved by introducing an ultrafiltration treatment prior to the determination. There was also a possibility to control the feed/air ratio in a biological denitrification reactor by NH_4-N itself.

For P-removal a large number of techniques can be applied, either separately or combined, such as chemical precipitation, crystallisation in a pellet reactor, and magnetical and biological P-removal. In both countries chemical precipitation is mostly used, but the attention for biological P-removal is growing every day.

The biological P-removal techniques are based on the principle that, given the right conditions, certain bacteria are able to remove phosphates from municipal waste water. Roughly two different processes of biological P-removal have to be distinguished; biological P-removal in the "mainstream"- and "sidestream".
In the biological P-removal in the mainstream the phosphate is concentrated in the surplus sludge by passing anaerobic and aerobic zones. This phospate-rich surplus-sludge needs an aerobic sludge treatment, in order to prevent phosphate release back to the waterline. In Japan the effluent quality is influenced negatively by the low P-concentration in municipal waste water (3-5 mg/l compared to 9-11 mg/l in the Dutch situation). This influence is affected by the large amount of rainwater inflow during the rainy season (increase in dissolved oxygen and decrease in organic concentration) and by an increase in P-loading by sidestreams from sludge treatment. To cope with these problems primary sludge or methanol is added to the rainwater inflow with the purpose to increase the organic concentration. Otherwise the negative influence by the sidestream from sludge treatment is reduced by combination of biological P-removal and chemical precipitation, and by introduction of a improved sludge treatment process; the BEST-system, a sludge pelletizing thickener.
The P-removal in the sidestream is characterized by keeping the generated phosphate-rich sludge under anaerobic conditions in a stripper tank in a sidestream of a waste water treatment plant, so that the phosphates can be released from the sludge in a controlled way. The stripped sludge is sent back to the aeration basin for phosphorus uptake, while the enriched supernatant can be treated by chemical precipitation, pellet reactor or magnetic separation.

These three P-removal techniques can also be used in the waterline. Chemical precipitation is simple and reliable, but it has the disadvantage that it increases the salt load in effluent and produces phosphate sludge that can not be reused. Crystallisation in a pellet reactor, magnetic separation or biological P-removal do not have these disadvantages. In the Netherlands there is one full scale pellet reactor in operation and two magnetic P-removal installations.

2.5 *Advanced water treatment techniques*

It may be expected that the conventional waste water treatment systems will not be changed so much in the near future. On the other hand several configurations on the conventional activated sludge system will be developed. Besides aims as better effluent quality and less sludge production, these new treatment systems are also characterized by a denser construction e.g. to restrict the nuisance for the neighbourhood.
For the last mentioned purpose the following techniques have been discussed during this workshop: "U-tube aeration", "Deep Shaft" and "Multireactor", "membrane bioreactors" and the "airlift suspension biofilm reactor".

The "U-tube aeration", "Deep Shaft" and "Multireactor" processes are all three based on the principle that aeration for the aerobic degradation of organic substances takes place in an underground shaft; above the ground is a unit for water/sludge separation. The advantage of this configuration of activated sludge system is the increased solubility of oxygen at the bottom of the shaft by the high hydrostatic pressure of the above situated liquid column and the prolonged gas-liquid contact time. The depth of the shafts of the different systems varies from 20 m (Multireactor) to 30 - 100 m. The inlet of air varies also; "U-tube aeration" at the top of the shaft, "Deep Shaft" one-third from the top and the "Multireactor" at the bottom of the shaft. Besides this the Multireaktor sytem is characterized also by the flotation system for the sludge/water separation. At the moment 40 of the 50 "Deep Shaft" installations in the world are situated in Japan. Space limitations and very high ground costs, together with the immanent urgence of water conservation and recycling are responsible for this Japanese interest. In the Netherlands these factors are - until now - still quite different. For example the larger buildings and office complexes in Japan have a separate collection systems for kitchen waste water and toilet water. The kitchen waste water is treated locally and the purified waste water is used as flush water for the toilet system. Also membrane bioreactors are used for this purpose. In large cities in Japan the operation and maintenance costs of a ultrafiltration reclamation system are less than the total charges for tap water usage and waste water discharge (2 $\$/m^3$ compared to 3 - 5.5 $\$/m^3$). It is well-known that Japan has a famous reputation on the field of development of membrane bioreactors; already mentioned is the "Aqua Renaissance '90" project. But the great energy demand for cross-flow filtration and the high costs for the investment in filtration systems, prohibits further application for treatment of domestic waste water. Research efforts are focused on achieving a significant reduction in the costs of membranes, a drastic increase in membrane fluxes and thus a decrease in the energy demand.

Another new water treatment technique, which is developed for domestic wastewater

in the Netherlands is the "airlift suspension biofilm reactor". This system is comparable to the three above-mentioned deep-aeration processes, but distinguishes itself by the attachment of biomass to an inert carrier material; usually sand or basalt. This reactor is also vertical and consists roughly of two concentric tubes, the inner and outer tube, and a separator for the three-phase mixture of air / waste water / carrier material.
With domestic waste a stable process at lab-scale is obtained at a hydraulic retention time of 1,5 - 2 hours. Dissolved components are removed efficiently with a COD-load of 10 kg COD/(m^3.day) and a biomass concentration of 10 - 30 kg organic matter per m^3 reactor volume. The nitrification was very good, but the suspended solids are not removed. This system has been applied for several years in the anaerobic and aerobic treatment of industrial waste water and has to be scaled-up for domestic waste water.

2.6 *More extensive treatment of domestic waste water*

In both countries, Japan and the Netherlands, the attention will be focused this decade on the removal of phosphate and nitrogen from municipal waste water. Within a few years the effluent standards for sewage treatment plants in the Netherlands will be roughly: BOD 20 mg/l, suspended solids 30 mg/l, total-P 1-2 mg/l and total-N 10-15 mg/l. In Japan there will be the same situation. These levels are attainable with techniques now available or techniques still to be developed.
In view of the seriousness of the eutrophication on specific inland waters, the requirements for N and P will become even tighter, depending on the location situated.

Besides these stricter standards for the effluent of sewage treatment plants, it will be necessary to treat more municipal waste water by means of collecting and biological treatment process. For example the treatment of the rest percentage of municipal and domestic grey waters (55 %) in Japan is desirable. In Japan there is a tendency for treating domestic waste water in many smaller waste water plants at several locations in stead of collecting via municipal sewerage systems followed by central biological treatment.

For the near future it is expected that new effluent standards will be discussed for heavy metals and organic micropollutants for sewage treatment plants. Heavy metals and organic micropollutants (xenobiotics) should not be tolerated in surface water, not even in low concentrations. Most of these components in effluent of sewage treatment plants are largely bound to the suspended matter still present in the effluent. Removal of the suspended matter by sand filtration with or without coagulation seems to be sufficient to meet the requirements imposed.
For the further future complementary requirements for sulphate and pathogens are feasable.

If these requirements imposed on the effluents have to be met, attention should not not only be given to the method of treatment, but also to the quality of the municipal waste water supplied and the source of the pollutants. In a number of cases the source will be industrial, but sometimes exclusively domestic discharges are concerned.
Certain products can be responsable for not mating specific requirements. The only remaining possibility in those cases is to ban the product or to replace it by other

products. An example is to omit sulphur compounds in detergents.

Future research will not be restricted to meeting the effluent requirements in the strict sense, but will approach the treatment of municipal waste water as a whole, including the source of the pollutants, the collection and transport via a municipal sewerage system, the treatment technique in relation to the imposed requirements of the effluent, the use of energy and chemicals, re-use of by-products such as sewage sludge, gaseous compounds (methane) and phosphate/ammonium-rich sludge (fertilizer).

3 CONCLUSION

In this workshop a wide scope has been covered by presentations and discussions. Several items, which are of interest have emerged.

- The environmental situation in Japan and the Netherlands seems similar with regard to the population, the small area and the high degree of industrialization. Therefore the research programmes and the targets for the treatment of municipal waste water are comparable.

- However, some remarkable differences exist, e.g.:
  - the quality and quantity of sewage is in Japan:
    - more diluted
    - less P (3-5 mg P/l and P/BOD ratio of 0.02 in Japan; 9-11 mg P/l and P/BOD ratio of 0.05 in the Netherlands)
    - higher ratio of wet and dry weather flow due to the season rains;
  - Still a large fraction of human waste is treated as night soil:
  - Much more treatment facilities in the Netherlands for municipal and domestic grey waters;
  - In contrast to the Netherlands there is a tendency to increase the treatment capacity by the construction of many relative smaller treatment plants;
  - Water reclamation e.g. in large office buildings can be done economically in Japan, especially in big cities by applying biological treatment in membrane reactors. This is due to the high charges for tap water usage and waste water discharge;
  - In Japan more than 50 % of the sewage sludge is incinerated, in the Netherlands only 3 %.

- Up to now, research is planned at the moment when the problem is identified. The short term investigations were considered more important than the rather longterm research. The long-term work was usually dismissed as being too fundamental. In both countries this point of view is slowly changing, in view of the research programmes "Biofocus", "Aqua Renaissance" and "RWZI 2000". Still there are also great differences in the research approach:
  - In the Japanese research programmes the industry participates very much. The industry is absent in the Dutch research project, but there is more involvement by universities and water authorities;
  - In Japan different treatment techniques are usually tested at one reseach location, so that the results e.g. the removal efficiency are comparable. In the Netherlands this does not occur.
  - In contrast to the Netherlands, there is not much involvement of microbiologists in this field of research in Japan.

- For all people involved in municipal waste water treatment, including the politicians for their financial support, it must be a challenge to have the needed technology available at the right time in the next decade.

- The format of this workshop has aimed at promoting personal contacts and exchanging information between the people involved, such as technologists, microbiologists and representatives of the water authorities from both countries. This has been realized to a considerable extent by the given presentations, followed by discussions and social events.

This workshop is intended to become a regular event each two to three years. The second Japanese - Dutch workshop on the treatment of municipal waste water and the further processing of sewage sludge is scheduled in Japan in autumn 1993.

4 LIST OF PARTICIPANTS

name	institute
Japan:	
prof. Y. Watanabe	Department of Civil Engineering, Miyazaki University, Miyazaki 889-21
prof. H. Aya	Department of Civil Engineering, Musashi Institute of Technology, Tamatsutsumi 1-28-1, Setagaya, Tokyo 158
prof. K. Yamamoto	Department of Urban Engineering, University of Tokyo, Hongo 7-3-1, Bunkyo-ku, Tokyo 113
mr. T. Aoi	Sumitomo Environtech, Sumitomo Heavy Industries Ltd., Makamiya 7-1, Morinosato, Atsugi 243-01
mr. M. Ibuki	Environmental Plant Division, Nihon Gaishi, Sudamachi 2-56, Mizuho-ku, Nagoya 467
dr. A. Toriyama	Environmental Plant R & D Dept., Kubota Corporation, Shikitsuhigashi 1-2-47, Naniwa-ku, Osaka 556-91
dr. I. Miyaji	Kurita Central Laboratories, Kuriuta Water Industries Ltd., Makamiya 7-1, Morinosato Atsugi 243-01
prof. T. Noike	Department of Civil Engineering, Tohoku University, Aramaki Aoba, Semdai 980
prof. I. Somiya	Department of Sanitary Engineering, Kyoto University, Yoshidahonmachi, Sakyo-ku, Kyoto 606
prof. T. Kusuda	Department of Civil Engineering, Kyusyu University, Hakozaki 6-10-1, Hogashi-ku, Fukuoka 812
mr. Y. Hamamoto	Nishihara Environmental Sanitation Research Corporation Ltd., Shibaura 3-6-18, Minato-ku, Tokyo 108
prof. M. Okada	Dept. of Chemical Eng., Tokyo University of Agriculture and Engineering, Nakamachi 2-24-16, Koganei 184
prof. H. Nakanishi	Department of Civil Engineering, Yamaguchi University, Tokiwadai 2557, Ube 775
The Netherlands:	
prof.dr.ir. J.J. Heijnen	Department of Biochemical Engineering, Delft University of Technology, P.O. Box 5057, 2600 GB Delft
ir. W. van Starkenburg	RIZA, Institute for Inland Water Management and Waste Water Treatment, P.O. Box 17, 8200 AA Lelystad From 01-06-'91 Haskoning, Royal Dutch Consulting Engineers and Architects, P.O. Box 151, 6500 AD Nijmegen
ir. P.C. Stamperius	STORA, Foundation for Applied Waste Water Research, P.O. Box 80200, 2508 GE Den Haag
dr. J. de Jong	RIZA, Institute for Inland Water Management and Waste Water Treatment, P.O. Box 17, 8200 AA Lelystad
prof.dr.ir. P.J.A.M. Kerkhof	Dept. of Chemical Engineering, Eindhoven University of Technology, P.O. Box 513, 5600 MB Eindhoven
ir. W.G. Werumeus Buning	Water Authority "Hollandse Eilanden en Waarden", P.O.

	Box 469, 3300 Al Dordrecht
prof.ir. J.H.J.M. v/d Graaf	Witteveen & Bos Consulting Engineers, P.O. Box 233, 7400 AE Deventer
ir. T.W.M. Wouda	Water Authority "De Dommel", P.O. Box 10001, 5280 DA Boxtel
ir. J.H. Rensink	Dept. of Environmental Techn., Agricultural University Wageningen, P.O. Box 8129, 6700 EV Wageningen
ir. P.J. Tessel	Water Authority "Veluwe", P.O. Box 9030, 7300 EN Apeldoorn
prof.dr. J.G. Kuenen	Kluyver Laboratory for Biotechnology, Delft University of Technology, P.O. Box 5057, 2600 GB Delft
prof.dr. A.J.B. Zehnder	Dept. of Microbiology, Agricultural University Wageningen, H. van Suchtelenweg 4, 6703 CT Wageningen
ir. E. Eggers	DHV Water BV, P.O. Box 484, 3800 AL Amersfoort
prof.dr.ir. G. Lettinga	Dept. of Environmental Techn., Agricultural University Wageningen, P.O. Box 8129, 6700 EV Wageningen
prof.dr. A.H. Stouthamer	Dept. of Microbiology, Free University Amsterdam, de Boelelaan 1087, 1007 MC Amsterdam
prof.dr. S.A.L.M. Kooyman	Dept. of Theoretical Biology, Free University Amsterdam, de Boelelaan 1087, 1007 MC Amsterdam
prof.dr. B. Witholt	Dept. of Biochemistry, Groningen Biotechnology Center, Nijenborgh 16, 9747 AG Groningen
dr.ir. W.C. Witvoet	DHV Water BV, P.O. Box 484, 3800 AL Amersfoort
ir. A. Mulder	TNO Institute of Environmental Sciences, P.O. Box 6011, 2600 JA Delft
ir. H.J.W. Postma	Grontmij NV, P.O. Box 203, 3730 Ae De Bilt
dr.ir M.C.M. van Loosdrecht	Department of Biochemical Engineering, Delft University of Technology, P.O. Box 5057, 2600 GB Delft
ing. G.B.J. Rijs	RIZA, Institute for Inland Water Management and Waste Water Treatment, P.O. Box 17, 8200 AA Lelystad

WATER QUALITY MANAGEMENT IN THE NETHERLANDS

J.S. Hoornstra and J. de Jong
Institute of Inland Water Management and Waste Water Treatment (RIZA)

WATER QUALITY MANAGEMENT IN THE NETHERLANDS

J.S. Hoornstra and J. de Jong,
Institute for Inland Water Management and Waste Water Treatment

1. INTRODUCTION

This paper describes water management in the Netherlands with an emphasis on water quality aspects. First some features of the country are presented, underlining the importance of water and indicating the need for international cooperation on water quality matters. In the third paragraph, the water management structure is discussed, such as relevant laws and authorities, management instruments (planning, licensing) and funding methods. More detailed information is provided on water use and purification. Paragraph four addresses the main problems in water quality management like eutrophication, micropollutants and sediments. Finally some conclusions are drawn.

Water quantity and quality management are interrelated strongly in the Netherlands, resulting in the adoption of policies aiming at integrated water management, considering water systems as a whole, including quantity, sediments, banks, physical, chemical and biological characteristics.

2. THE NETHERLANDS

2.1. Size and economy.

The area of the Netherlands is about 42.000 km^2 including 3.000 km^2 large freshwater reservoirs. As a result of land reclamation, polder development and subsidence, nowadays about one third of the country is below sea level, needing permanent protection against flooding. Moreover, large areas of the country have to be protected against temporary flooding by the sea or by rivers. A dense system of pumping stations, canals and sluices is employed for this important goal.

The Netherlands is a densely populated country. During the 20th century population has risen from 6 million to 15 million inhabitants. This increase in population has been accompanied by a process of industrialization, urbanization and modernization of agriculture. The country as a whole can be regarded as a prosperous one, its Gross National Product per caput (about $ 18000) ranking among the 15 largest in the world.

2.2. Water.

The Netherlands have a temperate, maritime climate, wet deposition (760 mm, corresponding to some 900 m^3/s), being rather constant during the year. Due to evaporation a water deficit occurs in summer. However, the rainfall input is small as compared to the influx of the transboundary rivers Rhine (2200 m^3/s) and Meuse (230 m^3/s). Therefore, apart from its economic importance, the Rhine has large impact on the Netherlands water management, though the river may seem small in comparison with rivers such as the Danube or the Wolga (6500 - 9000 m^3/s).

Because of the countries' development, increasing amounts of waste water from urbanized and industrialized areas call for adequate purification. Furthermore the increased application of pesticides and fertilizers and the increased manure production from intensive stock-breeding in agriculture are a serious non-point threat to the aquatic environment.

2.3. International perspective.

Because these processes have taken place in neighbouring countries as well, the transboundary pollution contributes considerably to environmental pollution in the Netherlands. The Rhine, Meuse and Scheldt contribute substantially to water quality in Dutch surface waters and in the North Sea. Conversely, emissions from Dutch industries, agriculture, transport and consumers affect the environment in other countries, as well as the North Sea. Hence, the national environmental objectives set cannot be achieved without international cooperation. The Netherlands make large efforts to reach such agreements on pollution reduction measures.

The **European Community** is the most important international framework in this respect. Unanimous agreements are laid down in directives, that must be implemented in national laws. Since directives sometimes cover the scope of several national laws, including laws in preparation, they are never easily implemented in the existing legislative structure of member states.

Besides, in the last decades several other **international water conventions** have been adopted, with specific aims such as reduction of chemical and salt pollution of the river Rhine and reduction of pollution of the North Sea by land-based sources and by dumping from ships. The international commissions established to prepare programmes, measures and standards have reached various agreements since.

Due to the time-consuming process of negotiating on specific items within international fora, in the mid-1980's European countries realized progress was slow. Therefore, in addition, more political agreements were reached to accelarate the work in combating river and sea pollution. This has lead to concrete **action plans**, containing objectives, reduction percentages for specific substances and deadlines. Both for the **Rhine** and the **North Sea** such plans have been established. The Netherlands' government is of the opinion similar plans are necessary for the Meuse and the Scheldt.

3. WATER MANAGEMENT STRUCTURE

3.1. Legal basis.

The four main Acts relating to water are the Water Management Act, Pollution of Surface Waters Act, the Groundwater Act and the Drinking Water Supply Act. The former three laws are framework laws that are largely implemented on a decentralised basis. As for the management of all environmental media the General Environmental Hygiene Act applies.

The **Water Management Act** has a twofold function. On the one hand it aims at a coordinated and effective policy (including planning) and management as regards the water regime as a whole, while on the other hand it provides instruments for water quantity management. In view of the relationship between water quantity and quality management this Act is of major significance for water quality management.

The purpose of the **Pollution of Surface Water Act** is to regulate discharges and to keep the surface waters as clean as possible. According to the explanatory note, surface waters should be suitable for the preparation of drinking water, for fisheries, for industrial, agricultural and recreational purposes. Therefore, the Act regulates discharges of materials that affect the quality of surface water. The Act does not describe or even state any particular way of combating pollution. It provides a framework and instruments to regulate the discharge of harmful substances into the surface waters. The Act does not apply to groundwater nor to discharges into the seas unless the discharge is made through 'works' from the Dutch coast.

The intention of the **Groundwater Act** is to provide tools for the administration of groundwater. It contains instruments concerning the abstraction and infiltration of groundwater. Apart from that, the protection of groundwater abstraction areas is regulated by the Soil Protection Act.

Finally, the management of drinking water supplies - including planning, organization and supervision - is regulated by the **Drinking Water Supply Act**. The quality of drinking water has in turn been predescribed by statutory instrument issued under that Act: the Decree on Water Supply. This Decree has been amended to comply with EC legislation.

3.2. Responsibilities.

3.2.1. Introduction.

Before a description of the responsibilities of distinct institutions can be given, a brief overview of the Dutch administrative structure will be given. The Netherlands has 12 *provinces* and some 700 *municipalities*. Both provinces and municipalities have democratically elected governing bodies that are responsible for all matters in their area. These in turn have appointed executive bodies, drawn from their midst, to deal with day to day organisation.

Water bodies are divided into large waters of national importance, such as the Rhine and the Meuse, and other waters. The former are managed by the central government and therefore they are generally referred to as state (-managed) waters. The primary responsibility for non-state waters lies with the provinces. The provinces, however, can delegate this responsibility to *regional water boards*, which are functional public bodies exclusively authorised with water management tasks. Most the provinces has done so.

Finally 60 *water supply companies* are engaged in water supply. These are public enterprises, of which about one-third is run by municipalities.

3.2.2. Planning.

The Water Management Act provides the regulations for water management planning. Plans have to be drawn up on a national level, on a provincial level and on the level of the regional water boards.

A survey of plans, including those with regard to drinking water, and items to be covered is given in table 1.

Table 1. Survey of planning responsibilities

LEVEL	NAME OF PLAN	TYPE OF PLAN	ITEM		
			surface waters	groundwater	drinkingwater
Central Government	. Policy Document Water Management	Policy	Yes	Yes	No
	. Management Plans for state waters	Operational	Yes	No	No
	. Water Supply Plan	Policy	No	No	Yes
Province	. Water Management Plan	Policy	Yes	Yes	No
Water Boards	. Water Management for non-state waters	Operational	Yes	No	No
Water Supply Companies	. Water Supply Plan	Operational	No	No	Yes

3.2.3. Licensing and Enforcement.

As with planning, licensing and enforcement take place at several governmental levels, depending mainly on the laws concerned, the nature of intended or executed activities and the type of receiving water bodies. In general, licensing and enforcement activities are executed at the same administrative level.

Table 2 Survey of licensing and enforcement responsibilities.

Act concerned	Activity	Specification	Responsible Authority
Water Management Act	• Surface water abstraction 1)	. State waters . Non-state waters	Central Government Regional Water Board
Groundwater Act	• Groundwater 2) abstraction		Province
Pollution of Surface Waters Act	• Direct discharges	. State waters . Non-state waters	Central Government Regional Water Board
	• Indirect discharges (sewer)	. Specified industrial sectors . All industrial discharges	Regional Water Board Municipality
Drinking Water Supply Act	• Hygienic supervision • Organization	 . Domestic supply	Central Government Province

1) For small quantities instead of a license permission suffices.
2) For small quantities abstracted a license is not required.

3.2.4. Policy on waste water discharges.

From 1975 onwards, national policy on waste water discharges has been formulated in successive 5-year scope Water Action Programmes. The current approach has been laid down in the latest Policy Document on Water Management (1989). Two basic principles of this policy are (1) emission reduction and (2) the stand-still principle.

In general, the *emission reduction principle* implies that pollution should be minimized, irrespective of the types of substance concerned. Industries should select processes and conduct operations accordingly ("good housekeeping"). However, where major remedial efforts are required, and particularly if purification plants are to be constructed, a distinction should be made between different pollutant types.

In the case of **black-listed substances** the objective is the elimination of pollution, or at least to come as close as possible to a zero-discharge situation. Control must involve the use of the *best technical means available*. However, even using these means it will not always be possible to eliminate the discharges concerned completely. In such cases it should be considered whether residual discharges lead to unacceptable concentrations of the substance concerned in the aquatic environment. If so, further measures may be needed, including total prohibition of discharges.

"Other substances" include a large number of pollutants, ranging from substances of low toxicity that occur naturally in surface waters to xenobiotic substances, showing relatively high levels of toxicity, persistence and mobility. The action taken in respect of such substances depends on the degree of damage they are likely to cause: the greater the hazard the more control efforts required.

For hazardous non-blacklisted substances (including nutrients) control efforts are required that do not depend directly on the quality objectives for the surface waters into which they are discharged, and in this respect the approach resembles the approach for black-list substances. However, instead of the best technical means available, the *best practicable means* should be applied. Water quality objectives are used as a back-up check. Failure to achieve standards may lead to the imposition of more far-reaching measures.

Secondly there are the relatively harmless pollutants, substances with a low degree of toxicity which occur naturally in surface waters (such as sulphates and chlorides). The degree to which measures are needed to limit discharges of such substances mainly depends on the quality objectives applying to the receiving waters. No general rule can be laid down for the techniques to be applied.

The second policy objective, *the stand-still principle*, for **black-listed substances** means that the total discharges in a particular administrative area (provinces and administrative areas of the central government) should not increase. This may have impact in particular for plant extensions and new dischargers. This principle can be departed in exceptional cases only, requiring solid arguments.

For **other substances**, water quality may not deteriorate significantly. The water quality standards should not be exhausted. Since water quality is influenced by seasonal and upstream conditions also, this requirement does not necessarily affect discharges. The main objective of the principle here is water authorities should monitor water quality, trace causes of deterioration and take measures if necessary.

This section is summarized in annex 1.

3.2.5. Construction and Operation of Works.

The *water supply companies* are responsible for the construction and operation of drinking water production and transportation works. It is the responsibility of municipalities to collect and transport waste water via sewers to collection points, usually pumping stations. Hence sewer construction, operation and maintenance are *municipal* tasks. The pumping stations, the pipework needed to transport the waste water to treatment plants and the treatment plants themselves are generally owned and operated by the *regional water boards*. They also operate the pumping stations and other works for regional water management. National government is in charge of construction, operation and maintenance of works in the national water management infrastructure.

3.3. Water Use.

3.3.1. Drinking water supply.

Nearly the entire dutch population has been connected to drinking water supply networks. The domestic water consumption has risen steadily to approximately 665 million m^3 per year, corresponding to 125 litre per capita per day, at a mean price of $ 1,- per m^3. Water supply companies also partly serve to industries and other business-sectors. In 1987 a total amount of 1190 million m^3 was delivered to all users.

Two-thirds of the drinking water produced originates from groundwater, whereas one-third is produced from surface water, viz. Rhine, Meuse and Lake IJssel. The entire installed production capacity amounts to 950 million m^3 per year for groundwater (from 250 groundwater pumping stations) and 430 million m^3 for surface water (1988). The latter can be subdivided into direct purification, infiltration and storage basins, with annual capacities of 30, 185 and 215 million m^3 of water respectively.

Table 3 Drinking water production by the supply companies

million m3/y	1981	1986	2000
surface water	322	410	437
groundwater	711	780	927
total	1033	1190	1364

3.3.2. Industrial and agricultural water use.

Industrial water consumption has fallen since 1970 because of both economic recession and the Pollution of Surface Waters Act that urged industry to economize water use. It now amounts to some 4000 million m^3 per year, originating mainly (over 90 %) from surface water. Cooling purposes cover the larger part of industrial water consumption (95 %). Furthermore, 10000 million m^3 per year (surface water only) is used for cooling purposes in power stations.

Non-industrial private groundwater exploitations (several sectors; surface irrigation and sprinkling in agriculture excluded) cover a water need of about 50 million m^3 per year.

Table 4 Direct-intake water by industries.

million m3/y		1981	1986	2000*
surface water	cooling	3205	3666	---
	other purposes	123	134	73
subtotal		3328	3800	---
groundwater	cooling	179	185	---
	other purposes	130	133	---
subtotal		309	318	309

* Freshwater only

Agricultural water use amounts to 340 million m^3 groundwater and 280 million m^3 surface water per year for surface irrigation and sprinkling in a 10 % dry year. In a very dry year, such as 1976, both groundwater and surface water need rise with 200 million m^3.

3.3.3. Funding water supply.

The *supply of water* by the water supply companies is funded by direct payments by the users. Water prices are based on the principle of cost recovery. The latter refers to cost for purification, storage and distribution, as well as to cost for research, analysis, claims and administration. Domestic consumers are charged in form of a fixed fee to cover standing charges and an amount per m^3 on the basis of other charges. Larger consumers mostly pay an operating hours charge; the longer the operating time, the lower the charge.

3.4. Waste water purification.

3.4.1. Extent and efficiency.

Two types of sewer systems are used, of which the "combined" system is most common (90 %). Waste water and storm water are led together to treatment plants. This is feasible since rainfall occurs during the entire year with moderate intensities; moreover, inclined surfaces are scarce. "Separate systems" account for the smaller part of the sewer systems. In these systems storm water is led directly into surface water.

In the Netherlands 92 % of all households are connected to the sewer system. This is relatively high, certainly as compared to Japan (40 %). Almost 90 % of the sewage is purified in waste water treatment plants. On the average 90 % of BOD, 46 % of nitrogen and 42 % of phosphorus are removed in these plants. It is planned that in 1995 nitrogen and phosphorus will be removed with an efficiency of 60 and 75 % respectively. In table 5 the development of treatment capacity is given.

Table 5 Development of sewage treatment capacity (in 10^6 population equivalents).

Year	1975	1980	1985	1990	1995
Number of biological plants	403	439	469	461	445
Treatment capacity (mill.p.e.)	10,9	16,6	20,7	23,9	25,3
Number of mechanical plants	117	65	34	14	0
Treatment capacity (mill.p.e.)	2,8	2,0	1,8	0,2	0

3.4.2. Funding purification.

Municipalities fund the extension and upkeep of the *sewerage system* in several ways although the most common means is levying a sewerage tax or including it in a general tax based on property. The level of *sewerage tax* (for the benefit of sewerage construction/maintenance) depends on the municipality concerned. On average this tax amounts to some $ 60,- per year for each household; the tax is likely to double in the year 2000 due to sewerage back-maintenance. The tax is usually paid on a yearly basis.

The regional water boards cover general costs and cost for water quantity management by an apportionment for interest groups (owners of urban and rural real estate). Cost for *water quality management* is funded by a levy. Everyone discharging oxygen consuming substances into the sewer system or into non-state waters has to pay a charge. This charge is based on the discharged quantity of "population equivalents". The annual revenue from this levy amounts to $ 0,7 billion.

A similar charging scheme for discharges is in operation for the State-managed waters. Dischargers, including sewage treatment plants discharging their effluent into state waters, pay levies. The revenues are used to grant money to dischargers to alleviate specific pollution problems. This primarily comprises providing grant aids for *sewage treatment plants* and for *industrial waste water treatment equipment* of industries discharging into state-managed waters. Finally, a part of the administrative cost is funded by these charges. The total annual revenue from this charging scheme is of the order of $ 60 million; approximately $ 1.2 billion has been granted so far.

Effluent charges for all waters are levied on the basis of "population equivalents" (pe). This has been defined as the average amount of oxygen consuming material produced by one person in one day from a household. Rules are then defined to allow the expression of the pollution load of industrial effluents in terms of pe, viz. 1 p.e.= 136 g/d oxygen consuming material = 100 g COD + 4.57 * 8 g Kjeldahl nitrogen. In addition, regional water boards can impose an extra charge for heavy metals. It is defined that 100 g mercury or cadmium, or 1 kg copper, lead, nickel, lead or zinc corresponds to 1 pe. In the near future heavy metal-based levies will probably also be imposed on discharges into state-managed waters.

Pollution loads are determined by standard methods. Domestic effluent households are generally assessed at 3 pe, except for one person households (1 pe). Small businesses producing less than 5 pe are also charged at 3 pe. Industries discharging less than 1000 pe are charged according to a table of coefficients to avoid disproportionate costs arising from sampling and measurement. These coefficients assume a linear relationship between e.g. production and pollution load. For different industries different parameters are used in order to characterize production volume. Specified industrial sectors (such as chemical industry and paint manufacturing) discharging more than 100 pe or any other industries discharging more than 1000 pe are obliged to measure the actual pollution load in their effluents and are charged accordingly.

The present levy for discharges into state-managed waters is $ 20,-/pe for discharges into freshwater and $ 17 for discharges into seawater, though for 1995 a $ 12,- rise is probable because of additional cost, e.g. for phosphate removal. Levies for discharges into non-state waters, imposed by the regional water boards vary considerably because of differences in size, area, equipment and so on. For 1990 the charge per pe varies from $ 20,- to $ 65,-.

Most dischargers pay monthly based on their estimated pollution load. However, all dischargers whose charge is calculated according to actual pollution load also have to submit returns at the end of the year to the appropriate authority, accounting the actual pollution load discharged from their sampling and measurement programme. Any surplus or deficit in their payments will be adjusted at that time.

4. WATER MANAGEMENT PROBLEMS.

3.1. Surface water quality.

In order to evaluate water quality, actual data should be compared to standards. In the Netherlands, the quality objectives for the year 2000 for both water and water sediment have been drawn up based on eco-toxicological considerations, taking into account no-effect-levels and combination toxicity, for over 100 parameters relevant to the aquatic environment. Annex 2 provides a short survey of objectives for regularly measured parameters.

Furthermore standards have been laid down in EC directives for surface waters with specific important functions. A function is considered an objective, in a water management sense, of surface and groundwater, bearing in mind the interests involved. These objectives are:

- surface water intended for the preparation of drinking water;
- bathing water;
- water for cyprinids;
- water for salmonoids;
- shellfish water.

Function-oriented legal quality requirements for groundwater have not been developed, however, soil quality guidelines apply as quality objectives in this case.

Three main problems emanate in evaluating actual water quality against standards mentioned, viz. eutrophication, organic and inorganic micropollutants and contaminated sediments. These are discussed in the next paragraphs.

3.1.1. Eutrophication.

Eutrophication of fresh waters has resulted in large algal blooms, hampering the production of drinking water and making the water unfit for recreational purposes. It furthermore results in a general impoverishment of the aquatic ecosystem. In coastal waters eutrophication also causes serious problems.

The most important sources of nitrogen and phosphate are industry, households and agriculture at home and abroad. Eutrophication policy in the last decade, mainly aiming at phosphate reduction, has shown poor results. The replacement of phosphate in detergents is, however, a favourable development. Manure legislation will also have positive effects. Nevertheless at three-quarters of the measurement sites show excess phosphate while in 90 % of the sites nitrate standards are not met and 20 % of the sites the levels of ammonia are too high.

The eutrophication policy objectives for the year 2000 are a 75 % emission reduction of phosphate and a 70 % reduction of nitrogen as compared to 1985; as an interim goal emissions should be cut by half in 1995. Means to achieve these goals are (besides those mentioned in the groundwater section):

- complete replacement of phosphate in detergents;
- nutrient removal at municipal waste water treatment plants with an average efficiency of 75 % and 70 % for phosphate and nitrogen respectively;
- application of best practicable technology to reduce industrial nutrient discharges including halving the phosphate discharges from fertilizer industry.

In addition, the development of new technologies for nutrient removal, such as fluidized bed crystallization and magnetic separation, is sponsored.

3.1.2. Micropollutants.

Micropollutants adversely affect the life and reproduction of aquatic organisms, hinder drinking water production and recreation and they can pose a serious threat to public health when present in aquatic organisms used for consumption.

Though in the decade 1975 - 1985 an 80 percent reduction of industrial heavy metal discharges has been achieved, water quality standards are still not met at 10 % of the measurement sites, whereas water sediment quality exceeds standards in more than 60 % of the sites. Mercury, cadmium, zinc and copper are the most important problems.

Organic micropollutants are a source of even more concern because of their multitude, their diversity and the partial lack of knowledge with respect to their environmental behaviour and effects. Furthermore many of these substances cannot be detected properly. For both water and water sediment standards are not met at 70 % of the sites. PCB, PAH and chlorinated pesticides in particular cause problems.

The micropollutant policy objectives for the year 2000 are reductions of emissions up to 90 % relative to 1985, though for some metals a 50 % reduction may suffice because of far-going efforts in the past. For 1995 a 50 % reduction is aimed at, but specific organic compounds should already be cut by 90 %.

Measures to reach this goals are for an important part aimed at diffuse sources:

- continuation of cleaning up industrial discharges by application of best available technology;
- special sector-aimed measures to decrease mercury emissions from dentists and heavy metal emissions from grit blasting;
- central partial softening of drinking water resulting in lower copper emissions;
- restricted pesticide use and pesticide emission reduction by best environmental practice in agriculture;
- restricted use of tar-impregnated wood as bank protection material;
- restricted use of organotin as an antifouling agent.

Furthermore research is performed on new production and abatement technologies; the latter includes improvement of heavy metal removal in sewage plants.

3.1.3. Sediments.

In the Netherlands sediments in many rivers, lakes and canals are polluted. This is largely due to the situation of the Netherlands at the downstream end of the Rhine, the Meuse and the Scheldt. Though the extent of pollution varies from site to site it is obvious that the contaminated sediments are a serious risk to the environment. Hazardous substances are absorbed in biological food chains and they are distributed into vulnerable regions such as the North Sea and the Wadden Sea. In addition groundwater is threatened by pollution from these sediments.

For watercourse maintenance about 60 million m^3 of spoil is dredged annually in the Netherlands. Two-thirds, mainly originating from salt and brackish waters, can be tipped back elsewhere since it is only slightly polluted. The other part, however, is severely contaminated and has to be stored. Since storage facilities of this size are not available serious problems have to be faced. Maintenance programs for watercourse dredging have already been cut drastically influencing navigability.

Nevertheless Netherlands' policy is aimed at ecologically safe sediments, reusability of dredging spoil and cleanup of those sites that represent a serious danger to human health or ecosystems. Before 1995 the following steps must be taken:

- development and application of environmentally safe dredging and processing techniques;
- completion of provincial plans on removal, processing, cleaning and storage of dredging spoil;
- clean-up of most dangerous sites;
- construction of two large-scale storage facilities
- construction of installations for treatment of 2 million m^3 of dredging spoil annually in order to facilitate reuse.

3.1.4. Temporary and local surface water problems.

Accidental spills

Apart from regular discharges considerable amounts of hazardous substances are emitted due to accidental discharges. It is estimated that about 1500 tonnes of products, mainly mineral oil, reach dutch surface waters in this way (1985). The 2000 policy goal is termination of disturbances in aquatic ecosystems by these emissions. As a first step it is the objective to cut drinking water intake closure time by half and the amounts discharged by three-quarters in 1995, relative to the average situation of the period 1984 - 1988. Moreover, provisions must be made to extend the time lapse between accident and discharge to 2 hours, so that relevant measures can be taken in due time. Finally accidental spill reporting procedures will be made more stringent.

Measures necessary in order to achieve these goals are:
- implementation and modification of relevant legislation, e.g. the Post Seveso Directive, and measures internationally agreed upon (mainly within the framework of the International Rhine Commission);
- licensing coordination between the acts on water pollution and nuisance;
- completion of a decision support system for inland calamity procedures;
- drawing up and implementing of shipping calamity prevention regulation.

Urban drainage.

In recent years a large-scale research project has been executed in this field in the Netherlands. The most important findings regarding storm water overflow are presented below.

a) A relatively small amount of peak loads largely determines the annual pollution discharged into surface waters; this is true for both combined and separate systems, while the mean pollution concentration is lower in separated systems. As for all systems a vast variation occurs due to construction, geographical area and industrial intensity.

b) It is estimated that 1.4 % of all pe discharged annually, viz. some 200000 pe, can be attributed to storm water overflow. In addition, 0.3 tonne of cadmium, 1 tonne of mercury and 1 to 40 tonnes of other heavy metals are discharged in this way (1985). However, 85 % of storm water overflow is discharged in small (semi)stagnant surface waters, leading to considerable local problems.

c) Pollution can be reduced significantly using improved separate systems. In these systems the larger part of rainwater is sent to treatment plants. Other technical measures to reduce storm water overflow pollution are:
   - storage sedimentation basins, retaining sewer sludge and sewage thus lowering the storm water overflow frequency (65 - 70 % efficiency);
   - swirl concentrators, retaining sewer sludge only (20 - 40 % efficiency);

At present planning with respect to implementation of these measures is in a preliminary stage only so it is not possible to give cost indications yet.

4.2. Groundwater pollution.

Apart from quantity problems, groundwater in the Netherlands is threatened by nitrates and pesticides. The principal source of **nitrates** in groundwater is the use of manure and mineral fertilizers in agriculture. In particular in sandy soils nitrate leaches relatively fast (2 - 3 times faster when compared to clay soils). This nitrogen load on groundwater is a threat to drinking water production in the Netherlands. At present the 50 mg/l EC-wide standard is exceeded at 2 of the 255 pumping stations. Extrapolation based on current policy would lead to sincere problems at about 35 pumping stations in the year 2050. For the time being closure is not necessary since deeper groundwater is extracted. If, in future, additional nitrate removal will be employed by the water supply companies, the drinking water price will rise with approximately $ 0.12 per m^3.

Nitrogen from agriculture also contributes significantly to eutrophication of surface waters, through leaching and run-off. Measures to combat the nitrogen emissions are therefore taken in order to protect groundwater as well as surface water. With respect to the latter, decisions made on the International Conferences on Protection of the North Sea (1987, 1990) have had large impetus.

In order to facilitate implementation and (provincial) enforcement of measures in agriculture special Manure Legislation has been developed. The current set of measures comprises:

- limitation of the period during which manure is applied;
- limitation of the amounts of manure/fertilizer applied (phosphate-based);
- assignment of soil and groundwater protection areas;
- technical provisions to prevent atmospheric emissions.

Other measures contributing to nitrogen reduction are the use of catchcrops during wintertime, fertilization plans and mineral accounting on farm level, erosion prevention, public instruction programmes.

Dichloropropane has been detected in groundwater some years ago, and later on several other **pesticides**, formerly believed to remain in upper ground layers or to be degraded, have been found at several locations in concentrations up to 100 ppb. Examples are organo-phosphorus compounds, organo-chlorine compounds and in particular triazenes, and persistent metabolites of these products. Emissions should be reduced at the source, in order to combat this type of pollution by pesticides, as well as the related surface water and air pollution. Measures to this end are :

- prohibition of pesticides harmful to the aquatic environment;
- restricted pesticide use and pesticide emission reduction by best environmental practice in agriculture;
- cleaning-up of point sources emissions from greenhouse horticulture;
- extension of research programmes on emissions and distribution of pesticides.

4.3. Water quantity aspects.

4.3.1. Different interests.

Although the Netherlands is known as a "wet" country and water seems to be abundantly available in many areas, shortages of water occur, particularly in dry periods. Under these conditions grassland and agricultural crops suffer from drought damage. For flushing in order to control the salinity of the surface water in dry periods, more than 150 m^3/s of water must be available. To limit saline intrusion at the Rotterdam harbour area huge amounts of water are needed (ca. 650 m^3/s).

Examples of more indirect water use include navigation, fisheries and recreational purposes. Various must be met for these user categories. A sufficient water depth in rivers and canals is necessary for navigation. For fisheries and recreation the emphasis is on the water quality.

Water also has an important role in **nature conservation** and general amenity. Changes in natural hydrological regimes, for example, will affect both the terrestrial and the aquatic environment. Wetlands, forests and other natural landscapes are threatened by the lowering of the groundwater table due to improved agricultural drainage, brook and river corrections and, in particular, groundwater abstraction. Agricultural water needs for sprinkling and surface irrigation purposes are large (some 800 million m^3 in a dry year) and have a large economic impact. Agricultural damages of the order of several hundreds of million guilders are prevented in this manner. Groundwater extraction is employed to cover 40 - 50 % of this demand, causing, however, serious dehydration problems. Groundwater level reductions in these areas amount to more than one metre. Out of a total of 450.000 ha of nature reserves and forest sites about 8 % proves to be moderately or severely dehydrated. Apart from ecological functions, agricultural groundwater abstractions are also threatening future drinking water supply. Therefore policy is aimed at a reduction of dehydration and groundwater abstractions.

4.3.2. Periods of drought.

In table 6 water balances for both average and very dry summers are given, summer being defined as the period from April to September. From these figures it can be concluded that:

- supply is dominated by river inflow (indirect precipitation) and direct precipitation;
- dry summers reduce water supply by 45 %;
- evapotranspiration by vegetation is the most important item for water demand;
- since evapotranspiration and domestic and industrial water use cannot easily be reduced, river outflow reduces by 70 % in a very dry year.

Table 6 Water balances (million m^3 per summer).

Demand side			Supply side		
Item	normal	very dry	Item	normal	very dry
Evapotranspiration	16500	17000	Precipitation	14900	8100
Domestic/industrial	2300	2300	Surface storage	500	500
Flushing	600	1200	Subsurface storage	4800	3800
River outflow	36700	11000	River inflow	34700	17900
--			Recycling	1200	1200
Total	56100	31500	Total	56100	31500

In periods of drought salt intrusion can become a serious problem, because 10.000 million m^3 is needed and supply may be irregular. Furthermore shortages can occur, mainly in supply areas in the higher parts of the Netherlands, such as the southern, eastern and north-eastern sandy areas. Water is transferred by the main river system. Thus Rhine water is distributed over a large part of the country by a intricate secondary and tertiary network of water courses. However, it is not economically feasible to meet all shortages in these periods.

4.3.3. Flood protection.

Too little water on the one hand, too much on the other. As stated in the introduction, half of the country would be permanent or temporary under water without a comprehensive water drainage and supply system. The protection against flooding planned for in the year 2000 amounts to 1/1.250 year^{-1} for river-influenced areas and 1/4.000 to 1/100.000 year^{-1} for areas threatened by storm surges. The greenhouse effect, however, is expected to cause a sea level rise of about 60 cm in the century to come, lowering the protection by a factor 10. Additional reinforcement of dykes and hydraulic works will therefore be necessary.

It should be noted, however, that the construction of dams, dykes, barriers, weirs and shiplocks for flood protection for navigation and agricultural purposes has undesired side-effects. The migrating fish such as salmon, salmon-trout, twaite and sturgeon have already disappeared. Spawning grounds, shore plants and organisms have vanished because of dams, hard bank constructions and navigational structures. These developments, in combination with water pollution, have impoverished the quality of aquatic ecosystems in our country.

5. CONCLUSIONS.

Water plays an important role in the Netherlands, protection of the country against water requiring more attention than shortages. Because of the high abundance of water and the high population density, a rather complex administrative structure and a sophisticated technical structure are necessary for water management. The central government manages the most important surface waters and determines the general policy e.g., towards municipal and industrial discharges. Several types of local authorities or public bodies are responsible for regional waters, local water quantity management, drinking water supply, sewer systems and sewage treatment. Water quantity management is primarily paid for by interest parties, whereas for water quality management "the polluter pays".

Major environmental problems in the Netherlands relating to water are:

- Pollution of surface waters by nutrients, heavy metals and organic micropollutants;
- Pollution of sediments by organic micropollutants and heavy metals;
- Pollution of groundwater by pesticides and nitrate;
- Accidental spills, in particular of oil and pesticides;
- Flood protection, needing hydraulic works affecting aquatic ecosystems.

Because of their scale and complexity, most of these problems need an integrated approach, covering measures in several distinct policy sectors. Water quality management instruments as well as water quantity management instruments should be used. Both direct discharges into water and atmospheric emissions should be reduced at source. Industry, as well as agriculture and consumers have to adopt different behaviour. Moreover, the decision-making level should become more and more international, since water quality is substantially influenced by activities to windward or upstream.

This "integrated" approach increasingly determines the Netherlands' policy on water management problems. International action programmes have been adopted to reduce the inputs of micropollutants and nutrients to the Rhine and the North Sea. These programmes include measures both for point and non-point sources. Implementation of the measures agreed upon will positively affect pollution of sediments. Nitrate pollution of groundwater is dealt with partially by existing by EC-legislation, while specific measures for nitrate from diffuse sources are in preparation. Accidental spills are discussed in the International Rhine Commission, covering both industry and shipping. Finally global water problems such as sea level rise inherently require international cooperation.

Annex 1 Overview of the application of concepts and technologies in relation to the substances emitted

type of substance	black-listed substances	other pollutants	
	specific organohalogen compounds, dioxins, mercury, cadmium etc.	heavy metals oxygen-consuming substances, nutrients (P & N)	sulphate, chloride, heat
clean-up primarily on the basis of	emission approach	emission approach	water-quality objectives approach
treatment method	best technical means	best practicable means	acceptability of discharges and steps to be taken depending on the water-quality objectives aimed at
any further requirements based on	unacceptable concentrations in the aquatic environment	water-quality objectives	

Annex 2 Short survey of water quality objectives.

Table 1 Inorganic parameters

Parameter	Regular		Occasional	
	Water (µg/l)	Sediment (mg/kg)	Water (µg/l)	Sediment (mg/kg)
Oxygen	5.000			
Total-P	150			
Total-N	2.200			
Ammonia	20			
Chloride	200.000			
Fluoride			1.500	
Bromide			8.000	
Sulphate	100.000			
Cadmium	0,2	2		
Mercury	0,03	0,5		
Copper	3	35		
Nickel	10	35		
Lead	25	530		
Zinc	30	480		
Chromium	25	480		
Arsenic			15	85

Annex 2 Short survey of water quality objectives (continued).

Table 2 Organic parameters

Parameter	Regular		Occasional	
	Water (µg/l)	Sediment (mg/kg)	Water (µg/l)	Sediment (mg/kg)
Oil		1000		
sum 6 PAH		0,6		
VOX	5			
Trichloroethylene			2	
Hexachlorobenzene		0,004		
PCB (each of 7)		0,004		
gamma-HCH	0,01	0,001		
PCP	0,05	0,02		
Parathion-E			0,02	
Malathion			0,03	
Tributyltins			0,01	0,0015
Triphenyltins			0,01	0,001
Dinoseb			0,02	
DNOC			0,3	
2,4-D			11	
Atrazine			0,1	
Simazine			0,4	
Permethrin				0,8

WATER QUALITY MANAGEMENT IN JAPAN

I. Somiya
Department of Sanitary Engineering, Kyoto University

Water Quality Management In Japan

Depart. of Environ. & Sanitary Engg.,
Kyoto University
Professor, Dr. Isao Somiya

1. History of Water Pollution Control in Japan

After the World war II, there have been very rapid industrial development and city infrastractural reconstructions, following the disputes concerning water pollution problems. Some local ordinances were enacted to enable local governments take measures against water pollution. In 1958, the central Government set out to legally control water pollution with two water quality laws. One was concerned with conservation of water quality on public water areas, and another was a law regulating effluent from factories.

Since the decade from around 1960, where rapid economic growth and urbanization of suburbs of big cities have been achieved, water pollution problems became more widespread and severe on the aspects of not only organic pollution, but also heavy metal problems. In 1967, the Government enacted the Basic Law for Environmental Pollution Control to promote comprehensive measures against various environmental pollution. Subsequently, the former two laws were unified in the form of a new reinforced Water Pollution Control Law. In addition, a new law concerning prevention of marine pollution and maritime disaster was enacted.

Nevertheless, serious water pollution occurred frequently. For example, destruction of marine including cultured yellowtail as a result of large-scale red tide within the Seto Inland Sea in 1972, spillage of heavy metal from a large oil refinery factories in 1974, and the appearance of hexavalent chromium contamination caused by successive dumping of industrial waste. To cope with such situation, the Interim Law for conservation of the environment of the Seto Inland Sea was enacted in 1973. the Law was revised in 1978 to include new policies and renamed the Law concerning Special Measures for Conservation of the Environment of the Seto Inland Sea, under which comprehensive measures have been taken.

In recent years, the situation of water quality in public water area has been reaching to rather steady condition , so that remarkable trend of improvement have not been attained in water quality concerned with organic pollution especially in semiclosed water area(such as narrow frontage bays, mediterranean seas, lakes and reservoirs), and rivers in urban areas.

In 1984, special action has been taken to enact the Law concerning Special Measures for Conservation of Lake Water Quality, and the water Pollution Control Law was revised in 1989. The Water Pollution Control Law was revised again in 1990 to institutionalize the measures for household effluent.

History of National Water Quality Management Policy is summarized in Table -1.

2. Legal System for Water Quality Management and Environmental Quality Standard (EQS) for Water

1) On Legal system

Schematic diagram of legal system for water quality management in Japan is shown in Figure-1. Basic Law for Environmental Pollution Control is the most fundamental Law concerning with Environmental Quality Management, which is juridically controlled by the Environment Agency. To reduce and inspect contaminant loads to public water area, around 16 Laws shown in this Figure were enacted. The competent Ministry of each Laws are different. Some of them are governed by the Environment Agency and another by the Ministry of Construction, and the other by the Ministries of Industry and Trade, Agriculture, Transportation and so on. While, there is a Law called "Johkasou (private sewer) Law" which is controlled by the Ministry of Health and Welfare, and mainly regulating the small-scale treatment facility installed at each residential house in the area of sewerage system uncovered.

2) According to Basic Law, article 9, Environmental Quality Standards for water were determined. EQS for water pollution are the target levels of water quality to be achieved and maintained by the water quality control administration for public water areas. These standards consists of two major categories, one for the protection of human health and the other for the conservation of living environment.

In the former category, national uniform standards are set forth, which are applied for public waters, such as rivers, lakes, and sea areas. EQS values are classified for each classes to meet for their water usage purposes in each watershed.

EQS concerning with the protection of human health are originally stipulated for nine items; namely, cadmium, cyanide, organic phosphorus,lead, chromium(VI), arsenic, total mercury, alkyl mercury, and PCB's. In most cases, the values adopted are the same as those specified by water quality standard for water supply. In 1989, two items were added in this category, that re Tricholoethylene and Tetrachloroethylene. The values for mercury and PCB's are set taking into consideration for the possibilities of health hazards due to their accumulation and concentration in fish and shellfish.

Meanwhile, the second category provides a set of standards including BOD, COD and dissolved oxygen (DO) for preservation of the living environment. In addition, EQS for total nitrogen and total phosphorus are set forth specially for lakes and reservoirs to prevent eutrophication. Regarding the removal of sediment containing toxic substances, provisional removal standards have been set for sediments contaminated by mercury or PCB's. Table-2 and 3 are examples of EQS Values for those two categories, respectively.

3. Present State of Water Quality in Public Water Area

According to 1987 nationwide field survey on water quality in public water areas, only 0.02 % of the samples analyzed contained cadmium or other substances known to cause health risks in higher concentrations than the environmental quality standards (EQS) as shown in Figure -2. It is easily recognized that the non-compliance ratio is sharply decreasing. While, the compliance ratio related to the water quality Items on Living Environment is shown in Figure-3. As for BOD and COD, and other indices related to the preservation of the living environment, it was found that of the 3,070 water areas (2369 rivers, 116 lakes and reservoirs, and 585 sea areas) grouped under the categories in EQS up to 1986, only 70.1 % of the total water areas met the EQS for BOD and COD. From the types of water areas, it was found that 68.3 % of the total rivers , 43.1 % of the total lakes and reservoirs and 82.6 % of the total sea areas, and the closed water areas and the medium and small rivers in cities registered low compliance ratio as before.

As for other forms of water pollution, there are pollution of underground water by trichloroethlene and other toxic and recalcitrant substances, natural acidification of rivers, lakes and reservoirs in the volcanic regions, and environmental impact of thermal effluents from large-scale power generation plants.

4. Water Pollution Control Measures

1) More Stringent Prefectural Effluent Standards

In an effort to preserve water quality, the Water Pollution Control Law lays down uniform national effluent standards for specified facilities from which effluents are discharged into public water areas. For those water areas where it is judged that the uniform national standards are insufficient to attain EQS, the Law provides that stricter effluent standards can be set under Prefectural ordinances. Since 1975, all the prefectures have put more stringent prefectural standards into force.

2) Expansion of Control Targets

The Water Pollution Control Law , originally enforced in 1971, covered some 500 out of 1,100 industrial sectors under the subcategories of the Japanese Standard Industrial Classification. the scope of application wa steadily expanded later on through a series of ordinance amendments. As of the end of fiscal 1988, approximately 600 sectors had been placed under control.

3) Studies on Items Not Under Regulation

Trichloroetrhylene, tetrachloroethylene, and 1,1,1-trichloroethane have newly been designated as toxic substances, and the standards for their discharge have been established in water areas for public use, under the survey on the present efflux status of those chemicals from factories and other business establishments.

While, the survey on thermal effluent problem has been achieving under the Water Quality Committee of the Central

Council for environmental Pollution Control, so as to figure out an environmental permissible limit to thermal effluents.

4) Counter Measures for Water Pollution Accidents

After the accident of cyanogen flowing into the Iruma River in April, 1988. the environment Agency drafted guideline to be used by local public entities for the instruction and guidance they would give to enterprises engaged in electroplating work using cyanides, to prevent their leakage. The Environment Agency also submitted to the 114th Diet session the "Bill to Amend Part of the Water Pollution Control Law" for the purpose of the establishing the legal provisions for the measures to be taken by enterprise's when water pollution accident occur.

5. Improvement of Sewerage

It is quite obvious that Sewerage is an essential element of infrastructure in the sound development of cities. betterment of public hygiene, maintenance of good living environment. and water quality control in public water areas. Sewer system has been constructing under five-year programs. The on-going 6th program planned to be continued from 1986-1990 seeks to perform the following works with its total working of 12.2 trillion. At the end of fiscal 1990, around 42% coverage ratio on sewer system in urbanized area is roughly figured. Actually in 1988. the following construction of sewerage systems has been conducting at each domestic governments. such as 977 public sewerage systems, 93 drainage basin sewerage systems, 681 urban drainage systems. 3 specified public sewerage systems and 172 specified public sewerage systems for environmental conservation (including 15 simple public sewerage systems).

While taking into account the effects of the development of sewerage systems on the public water quality, the "come-back aquatopia" concept designated to build cities with a view to foster contact between inhabitants and clear water was stepped up 28 cities across the nation. Model Projects were carried out at 4 places (of which new projects were carries out at 2 places) for disposal of miscellaneous household waste water in the urban sewage systems in the areas where sewerage systems were not yet developed.

6. Example of Water Quality Monitoring in Yodo River

Yodo River is one of most typical and famous river in Japan in the sense of multi-stage water use. As Figure 4 shows the location of this river in Kansai District. more than 15 million inhabitants has been dwelling around this river and more large number of peoples beyond the border of watershed are getting water from this watershed through domestic water works. At most upstream of this river. there is a biggest lake in Japan, which is called " Lake BIWA " and has a sole and natural outlet through Seta River. and is directly connected to Yodo River. Along the flow down route to Osaka Bay. there is two big cities. Kyoto City and Osaka City which have populations of more than 1.4

million and 2.5 million, respectively. Furthermore, at just west side of Osaka City, there is also another big city of Kobe with the population of about 1.4 million. In this Yodo river watershed, there are also lots of cities located, in which each populations are more than 50 thousand. Those municipalities are getting water from this Yodo River and also discharging secondary effluent to this river. Hence, on the occasion of utilization of this river water, certain kinds of troubles are inevitable on water qualities between those upstream and down stream towns and cities.

Figure 5 shows the distribution of water quantity monitoring stations for flood control in this watershed and Figure 6 shows the places of water quality monitoring stations. From those information, we could recognize the kinds and situations of water pollution and develop the actual management and maintenance measures of river quality. Unfortunately, now-a-day, even in this area, those information has not yet been combined effectively and conveniently under a program governed by a special authorized bureau for the estimation of water pollution and its control, because the governing board of those two items are completely different; quantity is mainly governed by the branch of Ministry of Construction and quality is by each domestic authority on Environment issues.

Figure 7 is the schematic diagram to express the actual situation of uptake and discharge places in Yodo River basin. You will easily imagine that some cities located at upstream and middle part of Yodo River are discharging the secondary effluent to this river and downstream cities have to intake water from this river.

On the water quality of river and lake in water course of Yodo River Basin, Figure 8 to 11 show annual average concentrations on BOD, COD, T-N and T-P, respectively. In Table 4, the typical water quality characteristics concerning to each branch rivers in Yodo River Basin are summarized.

Table 1. History of National Water Quality Management Policy

- from water pollution control to integrated water quality management

(1) Unintegrated regulation under specific laws

(2) Prefectural ordinances (1949 Tokyo-)

(3) Water Quality Conservation Law, Factory Effluent Control Law (1958)

(4) Basic Law for Environmental Pollution Control (1967)

(5) Water Pollution Control Law (1970)

(6) Inauguration of the Environment Agency (1971)

(7) Cabinet agreement on environmental impacts of public works (1972)

(8) Law concerning Special Countermeasures for Conservation of Seto Inland Sea (1973)

(9) Amendment to the Water Pollution Control Law and the Law concerning Special Measures for Conservation of Seto Inland Sea (1978)

(10) Law concerning Special Countermeasures for Lake Water Quality (1984)

(11) Amendment to the Water Pollution Control Law for Prevention of Groundwater Contamination and Prevention of Pollution by Toxic Substances in case of Accident (1989)

(12) Amendment to the Water Pollution Control Law for Promotion of Countermeasure for Household Waste Water (1990)

Environmental Water Quality Standards (Dec. 28, 1971, Amendments 1974, 1975, 1982, 1985)

Table 2

Standards related to the Protection of Human Health

Item	Standard values[1]
Cadmium	0.01 mg/ℓ or less
Cyanide	Not detectable
Organic phosphorus[2]	Not detectable
Lead	0.1 mg/ℓ or less
Chromium (hexavalent)	0.05 mg/ℓ or less
Arsenic	0.05 mg/ℓ or less
Total mercury	0.0005 mg/ℓ or less
Alkyl mercury	Not detectable
PCB	Not detectable

Notes : 1. Maximum values. But with regard to total mercury, standard value is based on the yearly average value.

2. Organic phosphorus includes parathion, methyl parathion, methyl demeton and E. P. N.

3. Standard value of total mercury shall be 0.001 mg/ℓ in case river water pollution is known to be caused by natural conditions.

Table. 3 Standards related to the Coservation of the Living Environment

a. Rivers

Category	Item / Purposes of water use	Standard values[1]				
		pH	Biochemical Oxygen Demand (BOD)	Suspended Solids (SS)	Dissolved Oxygen (DO)	Number of Coliform Groups
AA	Water supply, class 1 : conservation of natural environment, and uses listed in A—E	6.5—8.5	1 mg/ℓ or less	25 mg/ℓ or less	7.5 mg/ℓ or more	50 MPN/100mℓ or less
A	Water supply, class 2 ; fishery, class 1 ; bathing and uses listed in B—E	6.5—8.5	2 mg/ℓ or less	25 mg/ℓ or less	7.5 mg/ℓ or more	1,000 MPN/100mℓ or less
B	Water supply, class 3 ; fishery, class 2; and uses listed in C—E	6.5—8.5	3 mg/ℓ or less	25 mg/ℓ or less	5 mg/ℓ or more	5,000 MPN/100mℓ or less
C	Fishery, class 3 ; industrial water, class 1; and uses listed in D—E	6.5—8.5	5 mg/ℓ or less	50 mg/ℓ or less	5 mg/ℓ or more	—
D	Industrial water, class 2 ; agricultural water[2], and uses listed in E	6.0—8.5	8 mg/ℓ or less	100 mg/ℓ or less	2 mg/ℓ or more	—
E	Industrial water, class 3 ; conservation of the environment	6.0—8.5	10 mg/ℓ or less	Floating matter such as garbage should not be observed	2 mg/ℓ or more	—

Notes : 1. The standard value is based on the daily average value. The same applies to the standard values of lakes and coastal waters.

2. At the intake for agriculture, pH shall be between 6.0 and 7.5 and dissolved oxygen shall not be less than 5 mg/ ℓ. The same applies to the standard values of lakes.

3. Conservation of natural environment : Conservation of scenic spots and other natural resources.

4. Water supply, class 1—Water treated by simple cleaning operation, such as filtration.
Water supply, class 2—Water treated by normal cleaning operation, such as sedimentation and filtration.
Water supply, class 3—Water treated through a highly sophisticated cleaning operation including pretreatment.

5. Fishery, class 1— For aquatic life, such as trout and bull trout inhabiting oligosaprobic water, and those of fishery class 2 and class 3.
Fishery, class 2— For aquatic life, such as fish of the salmon family and sweetfish inhabiting oligosaprobic water and those of fishery class 3.
Fishery, class 3— For aquatic life, such as carp and crucian carp inhabiting β-mesosaprobic water.

6. Industrial water, class 1—Water given normal cleaning treatment such as sedimentation.
Industrial water, class 2—Water given sophisticated treatment by chemicals.
Industrial water, class 3—Water given special cleaning treatment.

7. Conservation of the environment—Up to the limits at which no unpleasantness is caused to people in their daily life including a walk by the riverside, etc.

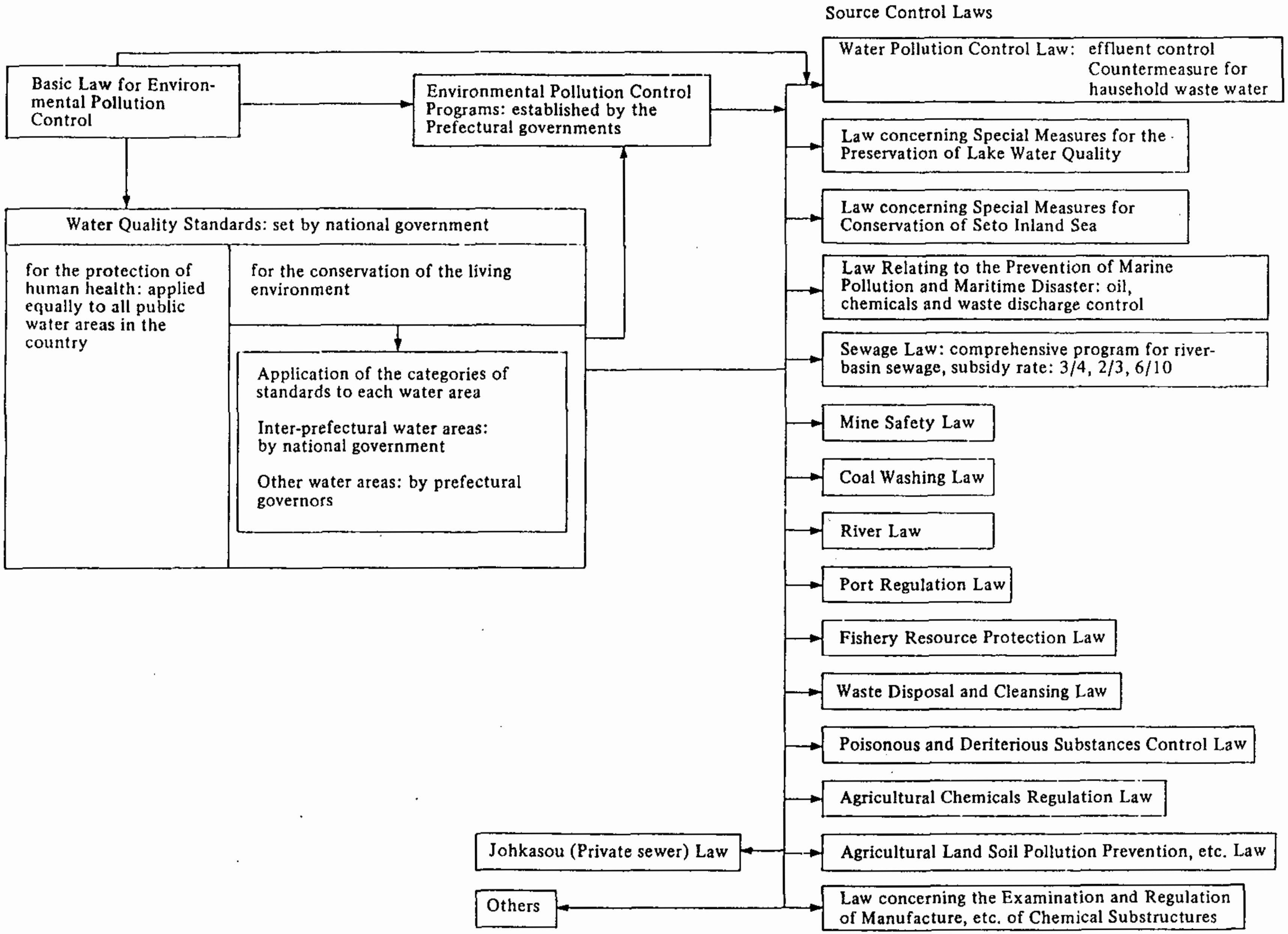


Fig. 1. Legal System for Water Quality Management

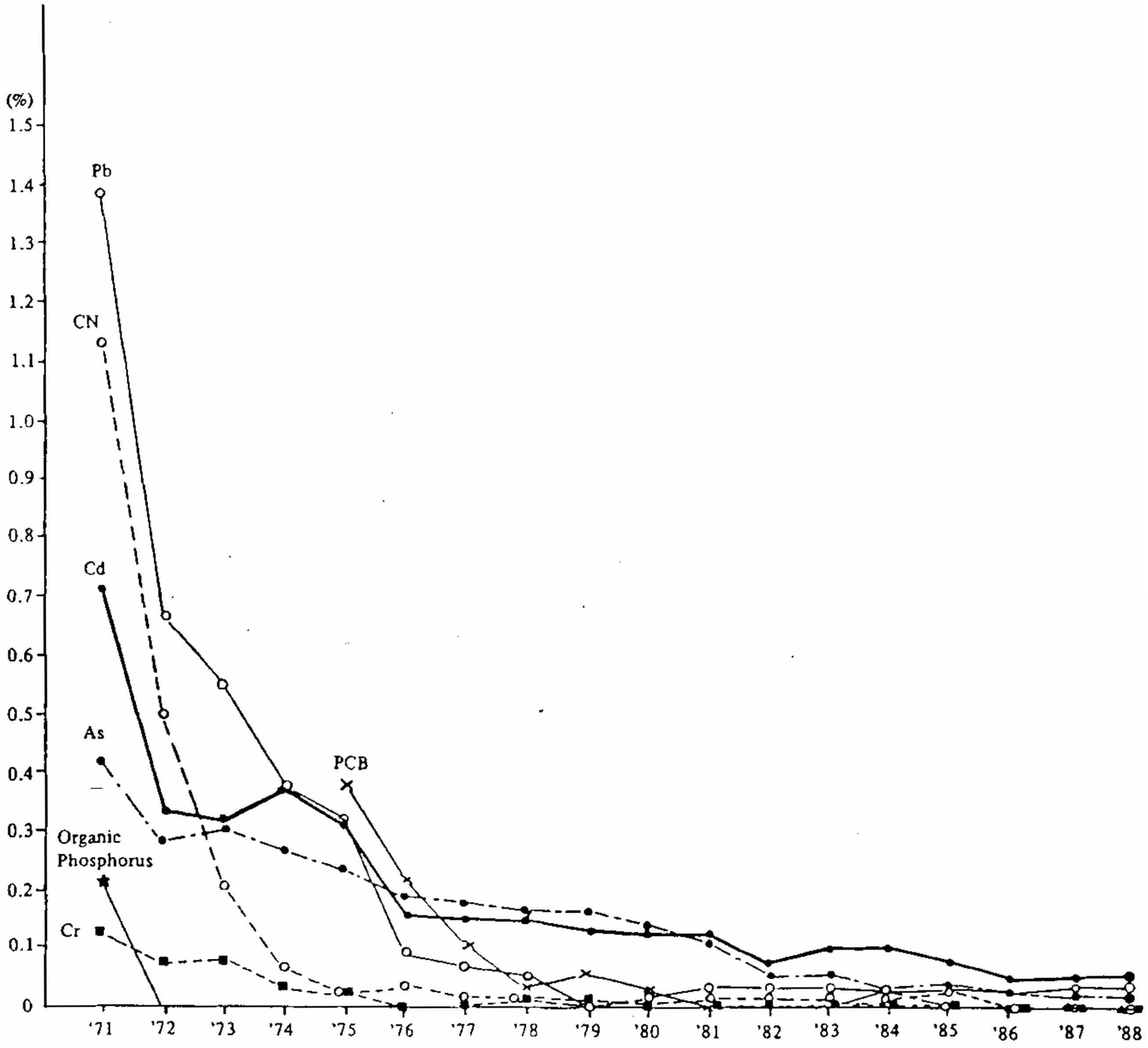
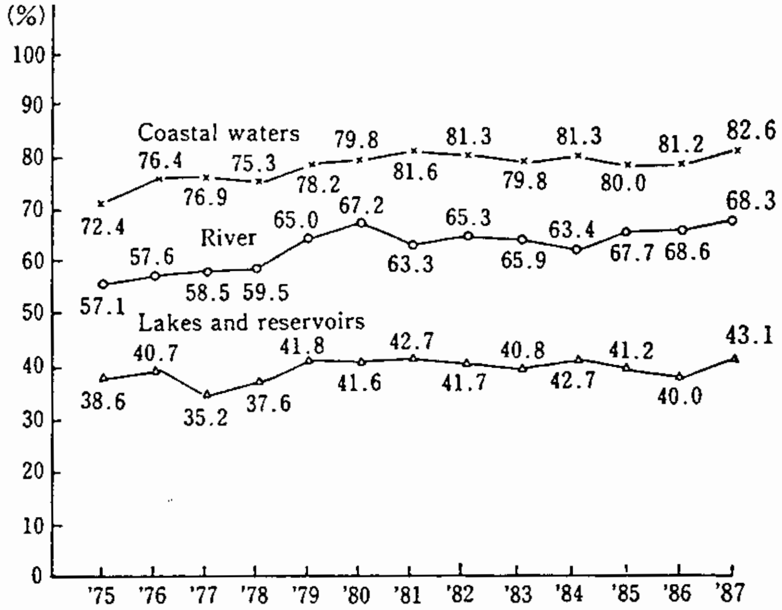


Fig. 2 Changes in the Rate of Non-Compliance with Water Quality Standards (ratio of samples exceeding the standards)

Fig. 3 Compliance Ratio with Environmental Quality Standards on Items Relating to the Living Environment

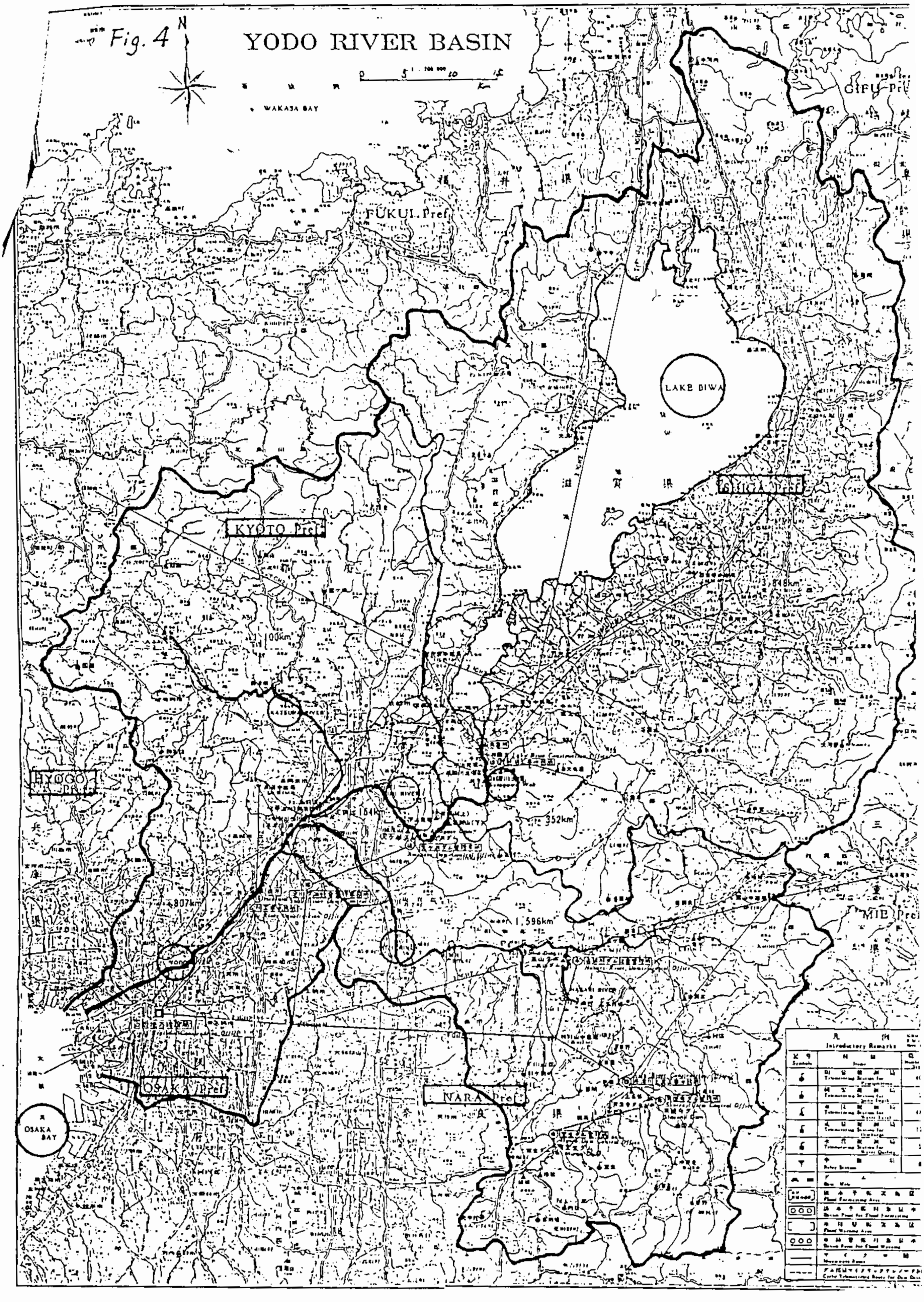


Notes : 1. The compliance ratio in obtained by :

$$\frac{\text{No. of water bodies complying with EQS}}{\text{No. of water bodies to which EQS is applied}} \times 100(\%)$$

2. EQS on items relating to the living environment, as classified according to uses to which different kinds of bodies of water are put, fall under six categories for rivers, four categories for lakes and reservoirs, and three categories for coastal waters.

Fig. 4


YODO RIVER BASIN
N
0 5 10 15 Km
1 : 200,000
WAKASA BAY
FUKUI Pref
GIFU Pref
LAKE BIWA
SHIGA Pref
KYOTO Pref
HYOGO Pref
MIE Pref
OSAKA Pref
NARA Pref
OSAKA BAY
KATSURA RIVER
RIVER
YODO
3,848km²
100km²
154km²
352km²
807km²
1,596km²
凡例
Introductory Remarks
記号
Symbols
項目
Items
雨量観測所
Telemetering Station for Precipitation
積雪観測所
Telemetering Station for Snow Depth
水位観測所
Telemetering Station for Water Level
流量観測所
Telemetering Station for Discharge
水質観測所
Telemetering Station for Water Quality
中継所
Relay Station
ダム
Dam Weir
Flood Forecasting Area
Datum Point for Flood Forecasting
Flood Warning Area
Datum Point for Flood Warning
Microwave Route
Cycle Telemetering Route for Dam Data

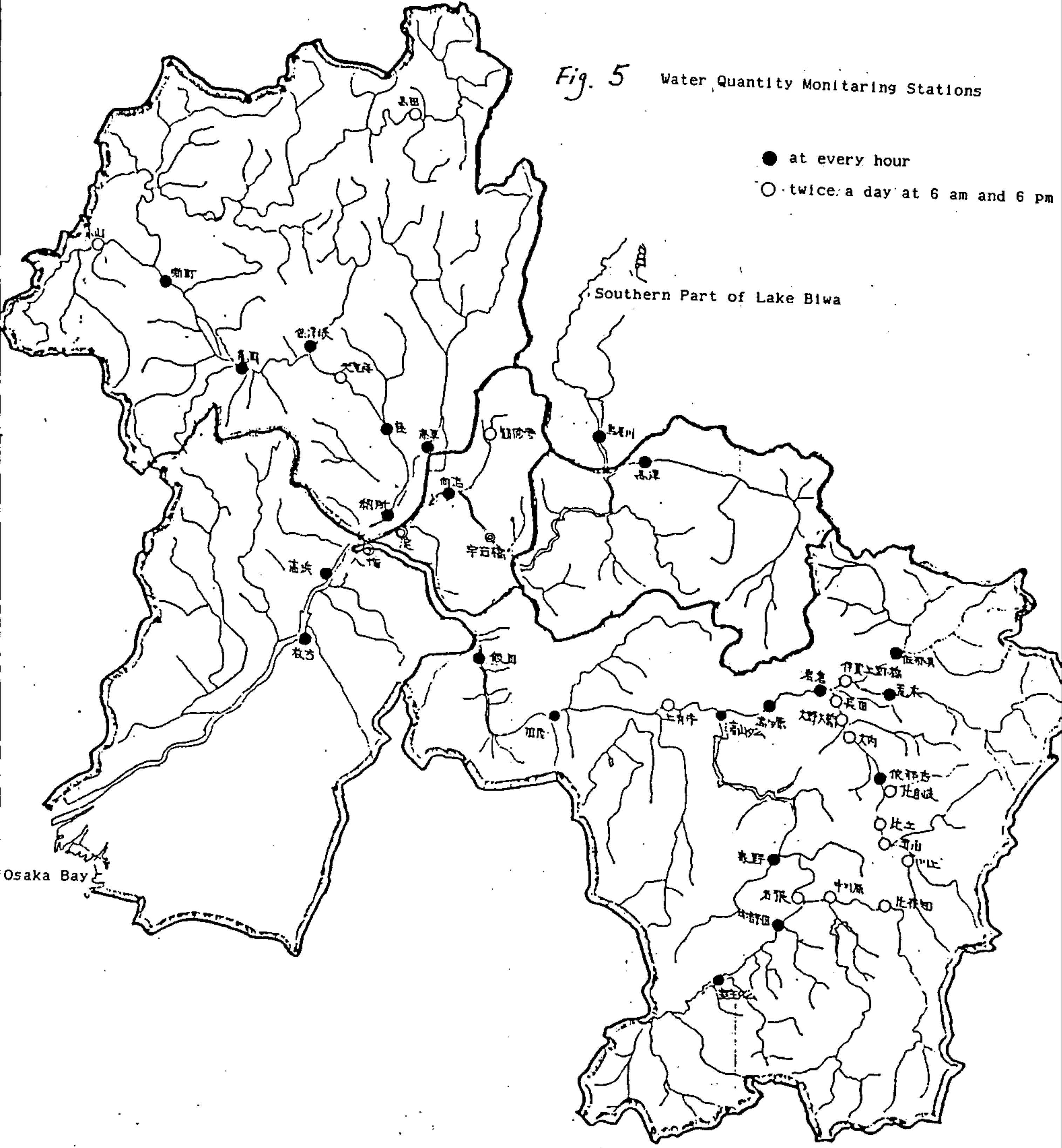
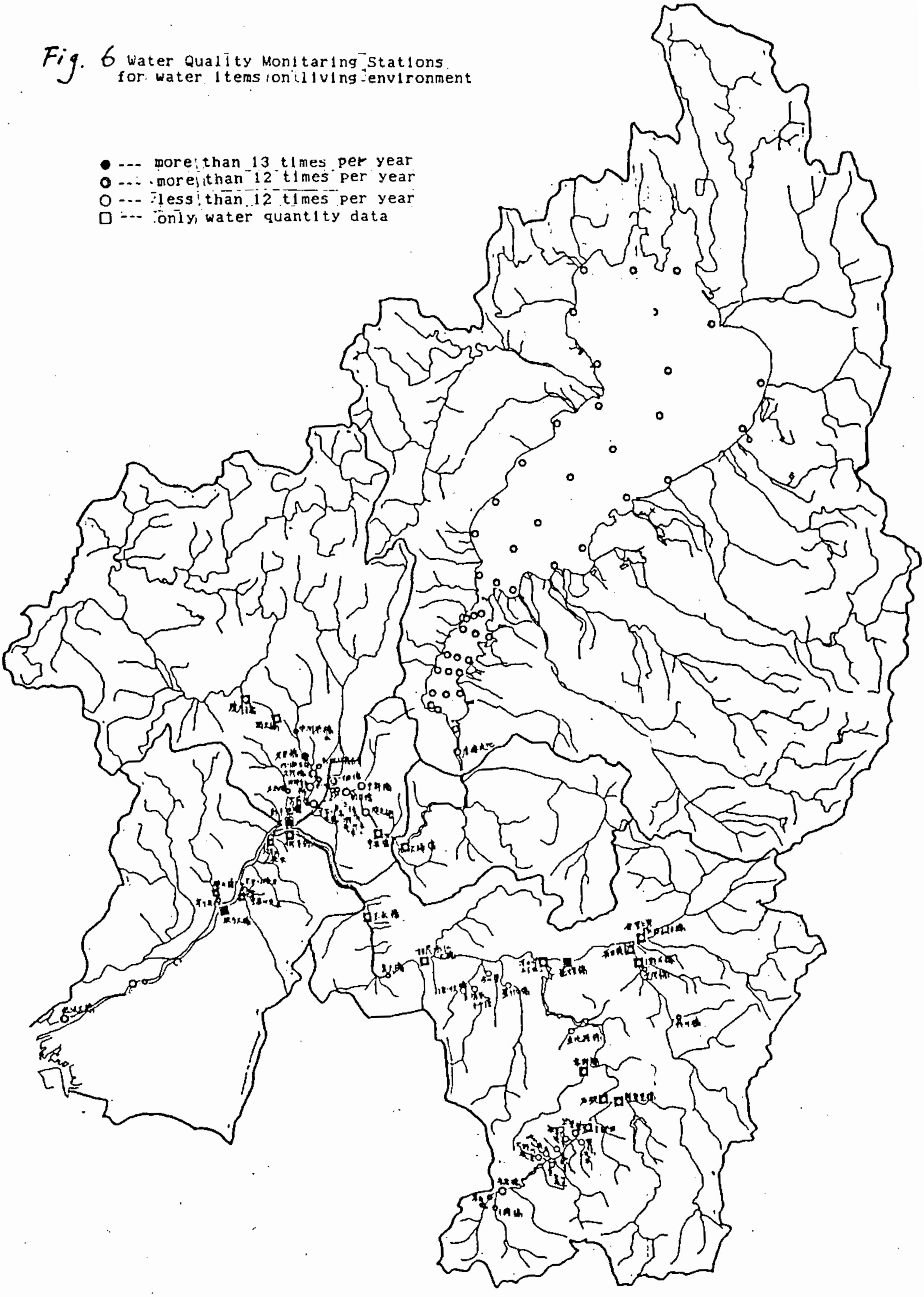


Fig. 5 Water Quantity Monitaring Stations

● at every hour

○ twice a day at 6 am and 6 pm

Fig. 6 Water Quality Monitaring Stations for water items on living environment



● --- more than 13 times per year

o --- more than 12 times per year

○ --- less than 12 times per year

□ --- only water quantity data

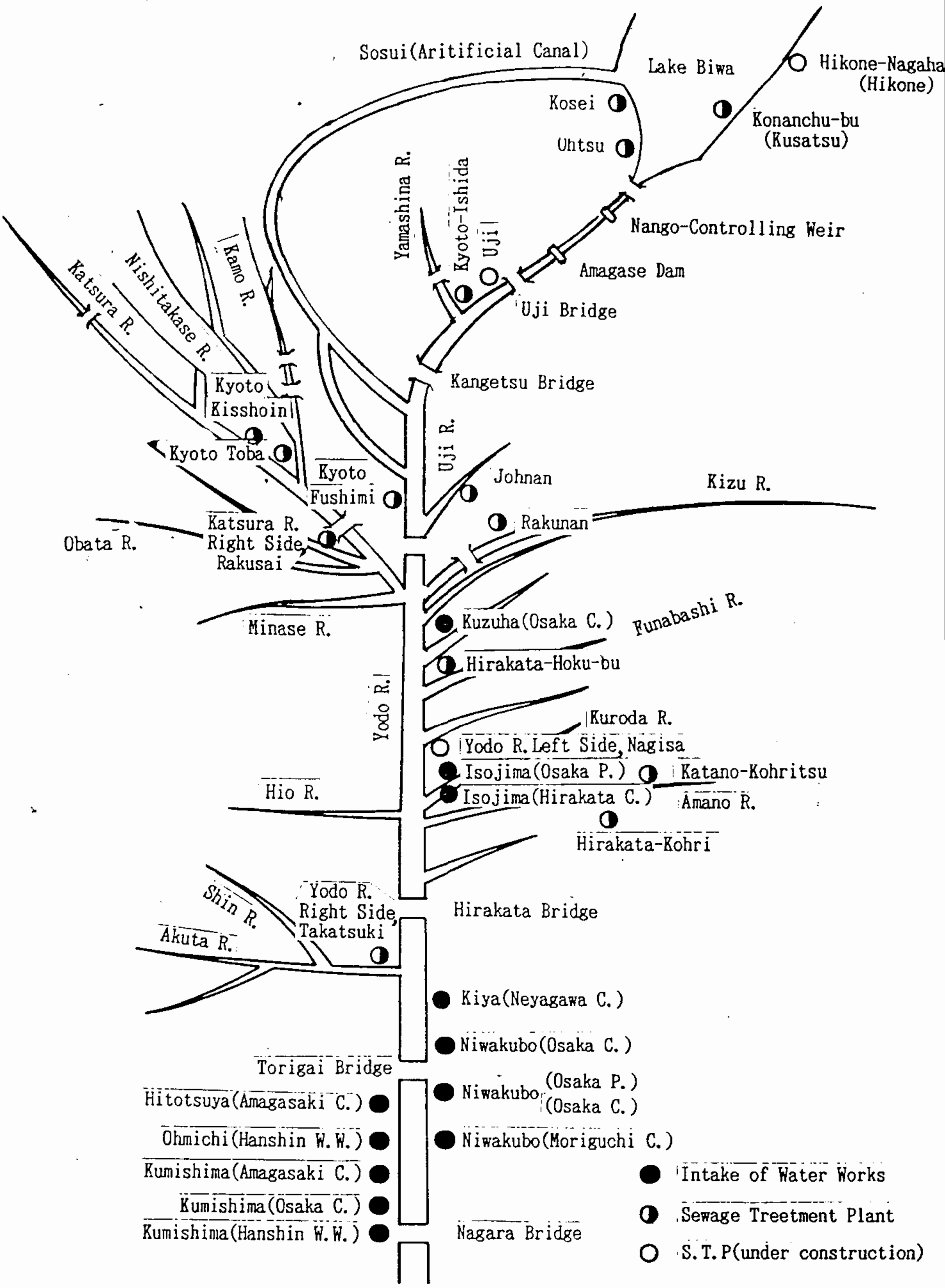


Fig. 7 Intake and Discharge Points

Fig. 8 Concentrations of BOD

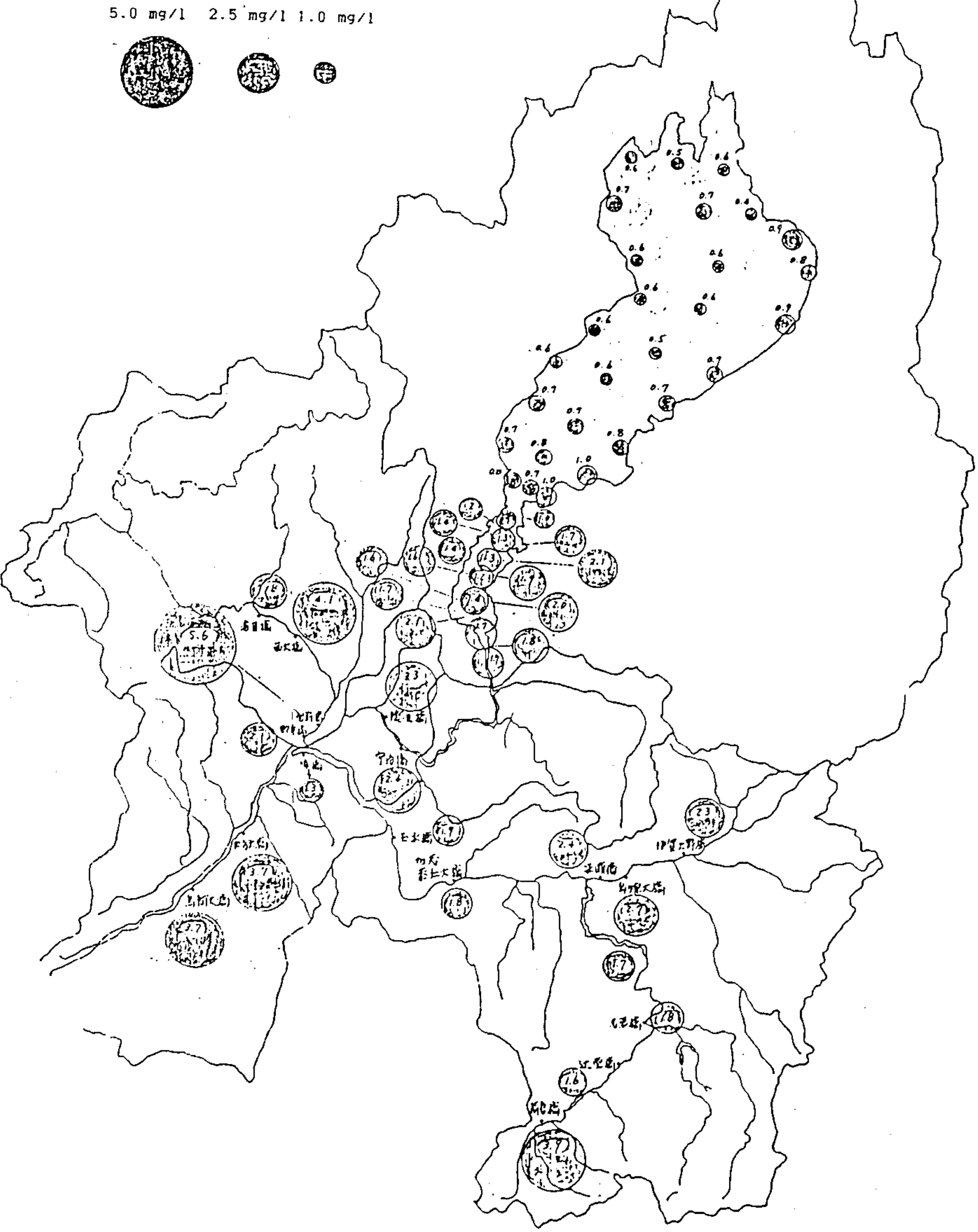


Fig. 9 Concentrations of COD

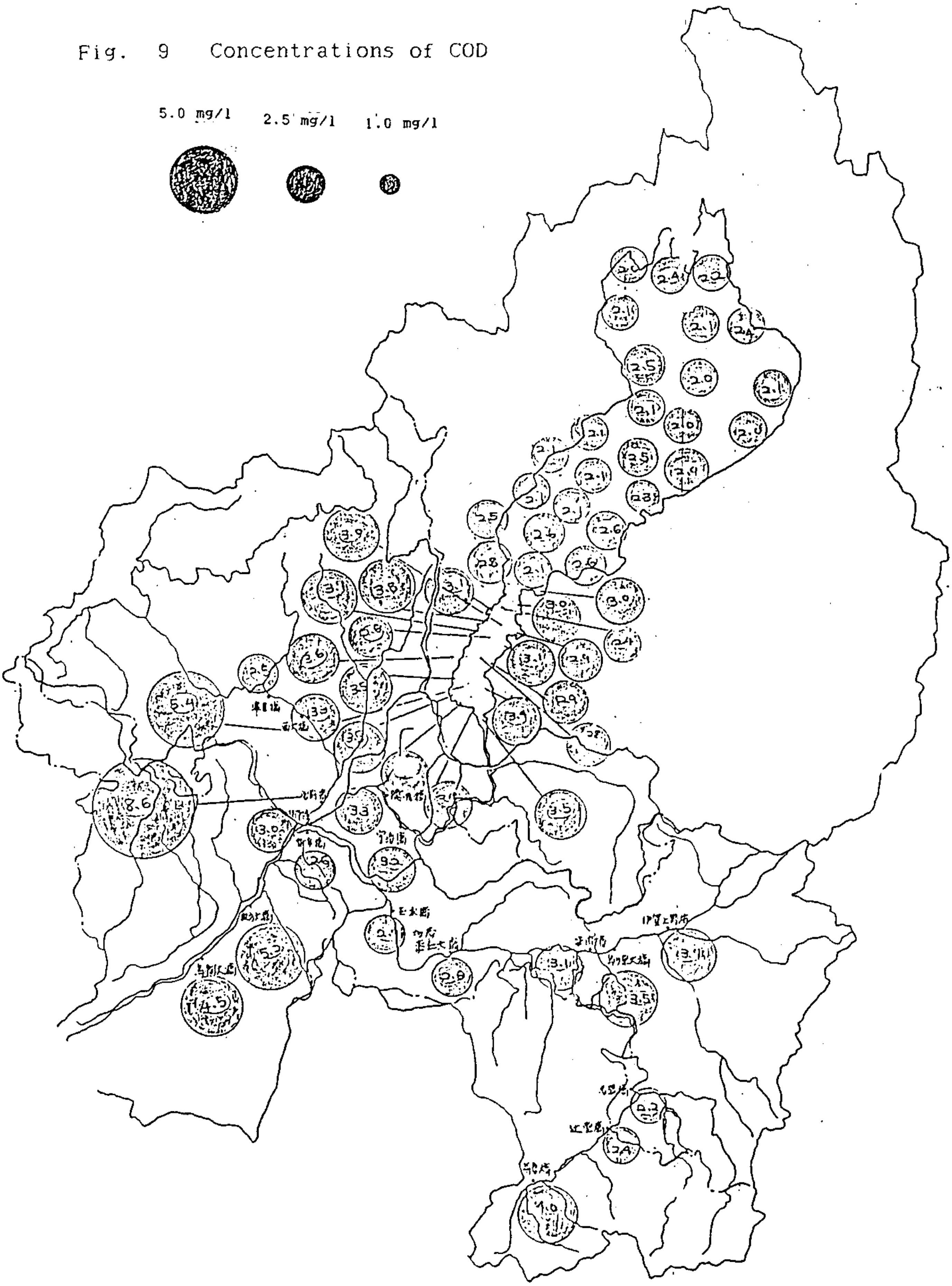


Fig. 10 Concentrations of T-N

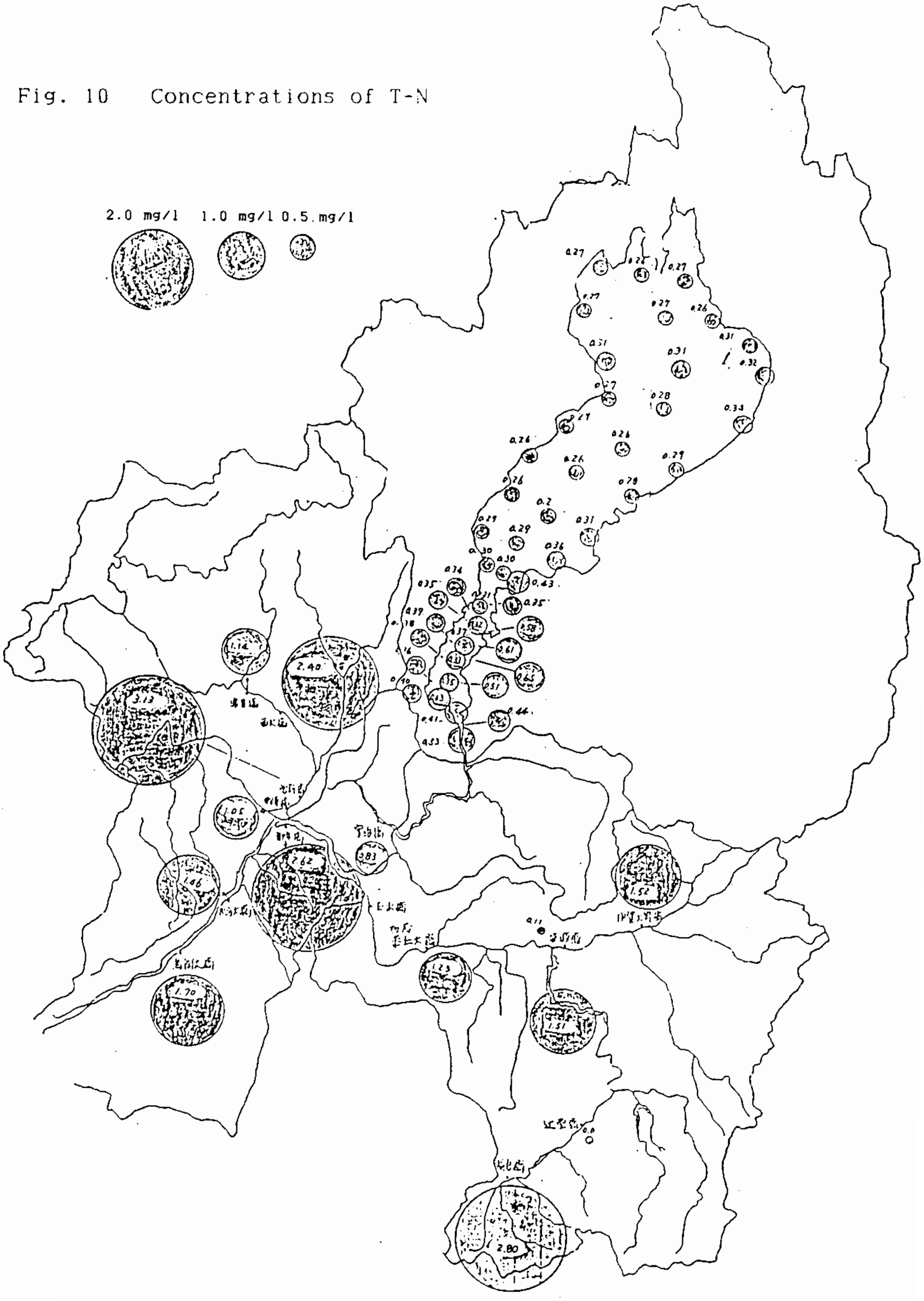


Fig. 11 Concentrations of T-P

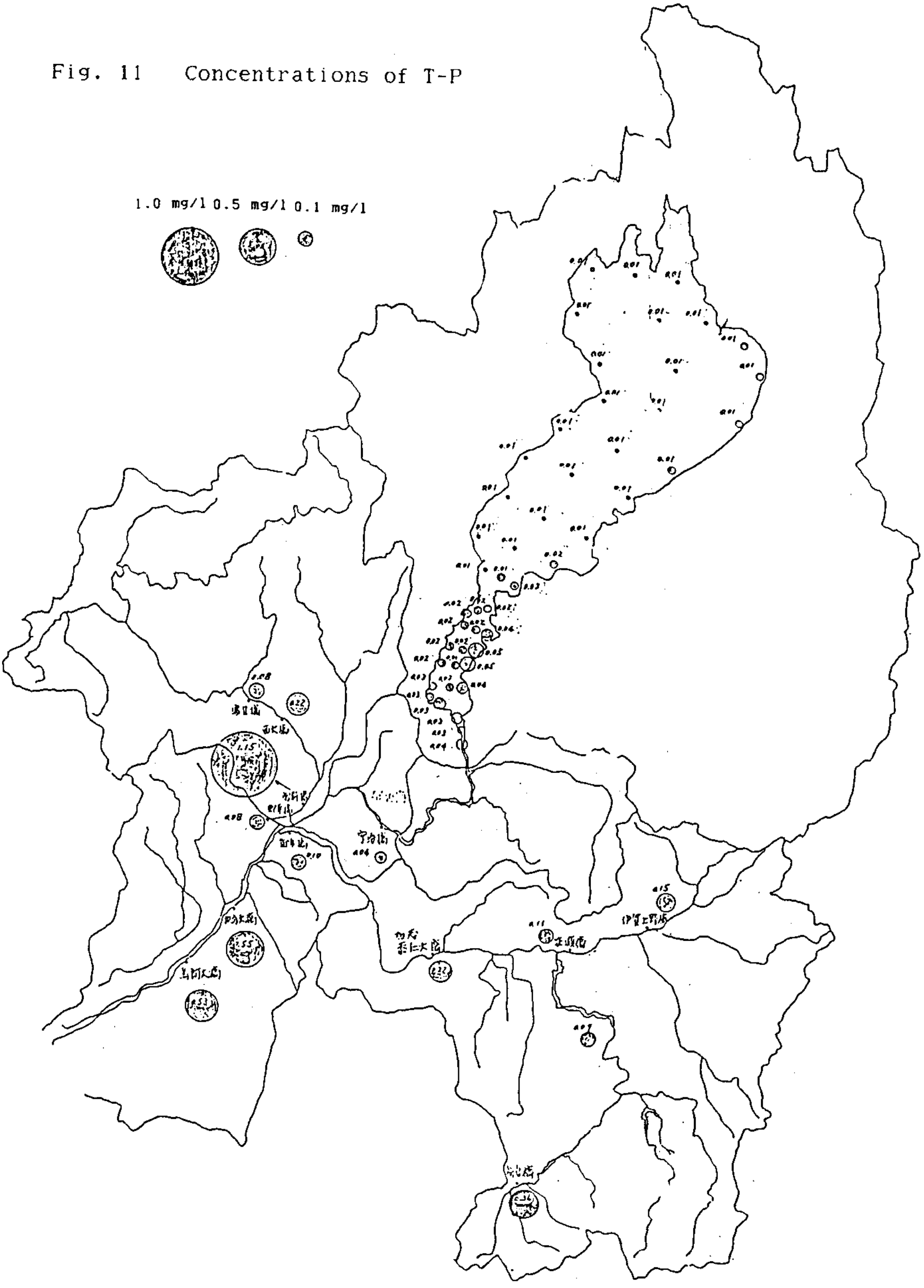


Table 4 Summary Distribution of Water Quality in Yodo River

Items	Lake Biwa Northen Part	Lake Biwa Southern Part	Uji River Uji B.-Ujimiyuki B	Katsura River Togetu B.-Miyamae B.	Kizu River till Miyuki B.	Yodo River Hirakata-Torigai B
DO	Surface layer 10 mg/l almost saturation		almost 10 mg/l	average 8 mg/l	10-11 mg/l almost saturation	almost 9 mg/l
SS	1.2-8.4 mg/l Higher at east southern part	3.5-12.5 mg/l High at east side	6-10 mg/l Increase with flow	13-18 mg/l Increase downstream	8-32 mg/l Max at Igaueno B.	21-25 mg/l upper≧lower
BOD	0.6-1.0 mg/l Max at east-south	0.9-2.1 mg/l Max at east side	2.1-3.3 mg/l Max at Ingenn B.	1.8-5.6 mg/l Increase downstream	1.3-3.9mg/l Max at Takakura B.	3.7 mg/l Hirakata=Torigai
COD	2.1-2.8 mg/l Slightly higher at eastern part	2.5-4.1 mg/l Higher at east	3.0-3.3 mg/l Slightly high at Ingenn Bridge	2.5-8.6 mg/l Increase downstream	2.2-4.0 mg/l Max at Takakura B.	4.5-5.2 mg/l upper≧lower
E. Coli	3-1300/100ml High at Hikone	17-5700/100ml Max at Sina-Sugie	3700-10000/100ml Max at Miyuki B. (downstream)	4000-68000/100ml High at middle part	5300-48000/100ml High at middle part	1400-3100/ml Hirkara=Torigai
T-N	0.2-0.43 mg/l	0.31-0.65 mg/l	0.83-1.05 mg/l	1.14-3.13 mg/l	0.07-2.80 mg/l	1.46-1.70 mg/l
NH4-N	0.02 mg/l	0.01-0.09 mg/l	0.08-0.14 mg/l	0.06-1.80 mg/l	0.06-0.31 mg/l	0.58-0.66 mg/l
T-P	0.01 mg/l	0.01-0.05 mg/l	0.04-0.08 mg/l	0.08-1.15 mg/l	0.09-0.36 mg/l	0.5.-0.55 mg/l
remarks	1)The north part of Lake Biwa is still remained clear, but the southern part is rather eutrophicated. Especially, the east side of southern lake is rather heavily polluted by the inflow of nutrient from the watershed. 2)The water quality of Uji River is basically composed by that of southern lake, but rather deteriorated by inflow loads 3)The water quality of Katura River deteriorates downstream from Togetsu Bridge. 4)The water quality of Kizu River tends to increase at the middle part, such as near Takakura Bridge and Igaueno Bridge, where is the inflow of domestic sewage from some towns/cities. Max values of T-N are often detected at Miyuki Bridge. This reason is estimated as the wash-out of fertilizer applied for tea plantation garden. 5)The basic water quality of Yodo River is composed by two rather clear River waters, such as Kizu River and Uji River, and one from the comparatively polluted Katsura River. 6)The health concerning pollution such as heavy metal an so on happens only once a year or twice a year in this Basin.					

"AQUA RENAISSANCE '90" AND "BIOFOCUS" PROJECTS

H. Aya
Department of Civil Engineering, Musashi Institute of Technology

AQUA RENAISSANCE '90 PROJECT
A National R & D Project
on Wastewater Reclamation and Resources Recovery by Combination of Anaerobic Digestion and Membrane Separation

Hidenori AYA
Department of Civil Engineering, Musashi Institute of Technology
1-28, Tamazutsumi, Setagaya, Tokyo 158 Japan

ABSTRACT

Aqua Renaissance '90 is a research and development project for water reclamation and methane gas recovery from industrial and domestic wastewater supported by the Ministry of International Trade and Industry. The objective is to develop treatment system of low cost, low energy consumption and small space requirement, employing combination of anaerobic process and membrane separation. Several national research institutes and more than twenty private sectors have been combining their efforts to establish its goal from 1985 to 1991. Significant developments have been observed in membrane application and bioreactor design. The project will be concluded on the end of this year.

KEYWORDS
Bioreactor, membrane, membrane module, anaerobic treatment, wastewater reuse, resources recovery

INTRODUCTION

The original concept of developing innovative wastewater treatment system was proposed by Aya according to the request of the Ministry of International Trade and Industry (MITI) and was elaborated through discussions with a working group. It was named as "Aqua Renaissance '90" project as for a new age wastewater treatment system. The main idea was introduction of phase separated anaerobic digestion with the first phase separated by membrane producing clear permeate which might be treated by biological or physical and chemical method for resources recovery.

Since the R and D project was accepted by the government in 1985, a consortium was organized by twenty four private companies to conduct required research. The Aqua Renaissance Research Association (ARRA) formulated research program and has performed experiments. The budget was mainly supplied by NEDO (New Energy Development Organization) and the remaining fundamental researches were supported by MITI. The original concept was modified accordingly to recover methane gas and also reusable water from wastewater as for major issue.

ARRA has performed development of practical systems in three phases program. The first phase was development of unit opera-

tions, the second was small scale experiments and the last stage was pilot plant studies which has been operating.
Feasibility study and development of data base are conducted by a few research organization such as Water Reuse Promotion Center. Most of fundamental researches were conducted by national research institutes accepting visiting researchers from private companies supported by the government budget. Research activities were; 1) organic membrane suitable for aerobic and anaerobic treatment, 2) development of ceramic membrane, 3) immobilization technic of useful bacteria, and 4) oil production by high pressure and high temperature reaction.
A planning committee and evaluation committees, which were organized by researchers of national institutes and universities, set up the course of developments, evaluated the performances, and gave advises.

As the objectives of the project covers very wide fields and the results conclude yet, an outline of the field studies is reviewed.

TARGET and SCHEDULE

The planning committee proposed the targets of R and D through discussion on current capabilities of bioreactor and membrane module, and about desirable power consumption based on conventional wastewater treatment. Targets of bioreactor and those of membrane were set separately (Table 1, and 2). The other targets for land requirement, excess sludge production and flax of membrane are also presented as options.
Selected private companies began development of their assigned theme on 1985 independently. After intermediate evaluation of proposed bioreactors, membrane modules and control systems, combinations of them were examined. Seven small scale pilot plants (referred as bench scale plants) were constructed for seven different wastewater in 1988, and operated until the end of fiscal year of 1989.
From 1990 two large scale pilot plants have been in operation, as for municipal sewage and wheat starch wastewater. The tests will be extended until the middle of 1991. The actual R and D were running behind the original schedule by one year.

SMALL SCALE PLANTS

Process configuration and raw wastewater of seven small scale plants were all different. Table 3 shows seven kind of wastewaters with combinations of private sectors involved in. Flow diagram and representative performances of each plant are shown in Figure 1 to Figure 7.

LARGE SCALE PILOT PLANTS

Experimental results of small scale plants were evaluated and two target wastewater which were municipal sewage and wheat starch wastewater were selected to be tested by larger pilot plants. The system configurations of both plants were modified from the

smaller plants from the results of experiments.
The wheat starch wastewater plant was a phase separated anaerobic digestion. The first stage was a fixed film bioreactor combined with hollow fiber membrane modules. The second stage was a up flow sludge bed without membrane separation which was thought unnecessary. Its capacity was 50 m^3/day. It produced effluent of BOD less than 100 mg/L according to a current report.
The large scale sewage treatment system was built at a sewage treatment plant in the suburb of Tokyo. The design average capacity was 240 m^3/day and maximum hydraulic loading 20m^3/Hr. The specifications and flow diagram are shown in both English and Japanese in Table 4 and Figure 8 - 9.
Both plants will be operated until middle of 1991.

DISCUSSION

Most of experiments succeeded to clear given targets. However, feasibility of each system developed are to be subjected for further evaluations. Tentative findings are as follows:
Phase separated anaerobic digestion Advantage of the process is reported on fat, oil and protein wastewater. The role of membrane separation is clear on removing suspended solid completely from the feed to second stage, which will become very easy to operate. But accumulating of high molecular weight fatty acid has not discussed in this project. Single stage digestion is likely much better for sewage sludge treatment. So far the large pilot plant and the smaller plant have been converted to single stage digestion, which produced excellent permeate of less than 100 mg/L BOD.
Membrane A lot of know-how were obtained on application of membrane. 1) A variety of inorganic membrane module produce high flux cooperated with back washing or slugging by gas flow. 2) Hollow fiber membrane modules succeeded to operate on liquid of very high concentration of suspended solid. Effectiveness of back wash by water or gas was confirmed. 3) Ultra filtration membrane and micro filtration membrane are produced similar results. However, scale up of modules and flow control are not well studied.
Methane gas production Most of plants were proved to convert organic matter to gas efficiently. But gas harvest from weak wastewater was poor and difficult as expected, even it was possible.
As the project has been conducted by the joint efforts of the government research institutes and private sectors, a lot of know how have been obtained on those new systems. Accepted number of patents accepted was 22. New designs of practical use (lesser patent) were 70 in 1991. They are belong to both of the government and private sectors.

ACKNOWLEDGEMENT

This report was produced under the permissions of MITI and NEDO. Most of materials were supplied by ARRA.

Table 1. Target value of membrane modules.

Microbe concentration of mixed liquor	Power consumption for unit flux
10,000mg/ℓ	$\leqq 1.5$ kwh/m^3
100mg/ℓ	$\leqq 0.3$ kwh/m^3

Table 2. Target value of bioreactor.

BOD level of waste water	Volumetric organic loading rate	Gas conversion rate
$\geqq$ 2,000mg BOD/ℓ	for heated reactor $\geqq$ 15kg BOD/reactor m^3 · day for non-heated reactor $\geqq$ 5kg BOD/reactor m^3 · day	$\geqq$ 80%
200–1,000mg BOD/ℓ	for non-heated reactor $\geqq$ 2 kg BOD/reactor m^3 · day	$\geqq$ 60%

Table 3. Summary of seven bench-scale plants.

	Type of waste water	Capacity (m^3/day)	Companies in charge	Location of plant
①	Large-scale (Municipal) Sewage	20	Ebara Corporation, Kawasaki Heavy Industries, Ltd., Nitto Denko Corporation, Toshiba Corporation	Fujisawa-city, Kanagawa Pref.
	Small-scale (Rural) Sewage	10	Water Re-use Promotion Center, Ishikawajima Harima Heavy Industries Co., Ltd. DIC-Degremont Co., Ltd., Mitsubishi Rayon Engineering Co., Ltd., Hitachi Plant Engineering & Construction Co., Ltd., Mitsubishi Electric Corporation	Chigasaki-city, Kanagawa Pref.
	Fat-and-Oil and Protein	7.5	Chiyoda Corporation, Nitto Denko Corporation	Yokohama-city, Kanagawa Pref.
②	Wheat Starch	5	Kobe Steel, Ltd., Mitsubishi Rayon Engineering Co., Ltd., NGK Insulators, Ltd. Mitsubishi Electric Corporation	Yamasaki-cho, Shisou-county, Hyogo Pref.
	Alcohol Fermentation	5	Sanki Engineering Co., Ltd., Kurita Water Industries Ltd.	Kanoya-city, Kagoshima Pref.
	Pulp and Paper	10	Kurita Water Industries Ltd., Shimizu Corp., TOTO Ltd., Toshiba Corporation	Iyomishima-city, Ehime Pref.
	Night Soil	0.5	Water Re-use Promotion Center, Nishihara Environmental Sanitation Research Corporation, Nitto Denko Corporation	Higashiura-cho, Chita-county, Aichi Pref.

① Low-concentration waste water ② Mid.-to-high concentration waste water

パイロットプラント構成設備仕様
Table 4 Summary of pilot plant equipment

前処理設備 Pretreatment unit	バイオリアクター設備 Bioreactor Process			膜モジュール設備 Membrane module	データ処理システム Plant monitoring and control unit
	可溶化メタン発酵槽 Hydrolyzation unit	嫌気性リアクター Anaerobic bioreactor	好気性リアクター Aerobic bioreactor		
・原水タンク×1 Raw water tank ×1 有効容積：7 m^3 Effective Vol. : 7 m^3 ・予備スクリーン×2 Pre-screen ×2 ハイドラシーブ形 Hydrosieve type 傾斜スクリーン Screen slot 目開：1 mm Slit : 1 mm 横幅：600 mm Total width : 600 mm ・懸濁物分離機×3 S. S. Separator ×3 立型円筒スクリーン式 Vertical drum screen ピンホール径：20 μm Pin hole opening : 20 μm スクリーン径：1 m Drum dia : 1 m スクリーン高：2 m Drum depth : 2 m 最大ろ過速度 : 75 $m^3/m^2 \cdot d$ Max. capacity : 75 $m^3/m^2 \cdot d$	・リアクター×1 Reactor ×1 立形円筒ガスかくはん式 Circular tank with gas mixing & steam injection 有効容積：12m^3 Effective Vol. : 12 m^3 加温温度：35℃ Liquor temp. : 35℃ 蒸気直接吹込 Steamdirect injection ・ボイラ×1 Boiler ×1 パッケージ形 Package type メタン・LPG 切換燃焼方式 Dual fuel with methane and LPG 相当蒸発量 : 100 kg/h Capacity : 100 kg/h	・リアクター×1 Reactor ×1 角形 UASB 式 固気液分離装置付 UASB type with solid & gas separator 全容積：77 m^3 Total Vol. : 77 m^3 反応部容積：24 m^3 Reaction Vol. : 24 m^3 流出水ガス分離装置付 Degassifire ・ガス脱硫器 Desulfurization 乾式脱硫剤 Dry type 充填塔 Colamn ・ガスホルダ 円筒形水封式 有効容積：20 m^3 Effective volume : 20 m^3 余剰ガスバーナー付 Gas tank	・リアクター×4 Reactor ×4 粒状付着体充てん生物膜ろ過方式 Biofilm type 角形下向流二床エアレーション式 Square tank with two stage media 有効容積：2 m^3 Tank Vol. : 2 m^3 ・処理水槽×1 Treated water tank ×1 ・逆洗排水槽×1 Backwashing drain tank ×1	・前処理器×2 Strainer ×2 自動洗浄式インラインストレーナー メッシュ：1 mm Self cleaning mesh type ×1 mm ・膜モジュール×10 Membrane module ×10 キャピラリー形有機膜（限外ろ過） Capillary organic membrane 外圧片端固定式 External pressure. one-end fixed 分画分子量：15 000 Molecurer cut-off : 15 000 ろ過面積：5 m^2/本 Filtration Area : 5 m^2/tube ・洗浄装置×1 Washing device ×1 透過水逆圧洗浄 Backwash ・窒素ガス洗浄 Gaswash (N_2) ・薬品洗浄 Chemical wash	・知識工学用計算機 Knowledge engineering computer ・プラント監視制御用計算機 Plant instrumentation controller ・プラント計装制御装置 Plant monitoring and controlling ・計装機器 Computer
川崎重工業(株) Kawasaki Heavy Industries, Ltd.	(株)荏原製作所 Ebara Corporation			日東電工(株) Nitto Electric Industrial Co., Ltd.	(株)東芝 Toshiba Corporation

下記の支援・附帯の設備は(株)荏原製作所の所管である。

(注) 1. 管理棟, 水質分析棟, 受変電設備, 電気配線, 給排水設備, その他共通設備

2. 処理水の高度処理による工業用水・業務用水化のための限外ろ過膜・逆浸透膜モジュール試験装置

Note : Control room. loboratory. power station. electric divice. water supply and RO·UF test equipment for water reuse are covered by Ebara Corporation.

Table Operating Parameters of Membrane Modules Used

Types of Waste Water	Module used	Memb. Area (m^2)	Appl. Press. (kg/cm^2)	Feed Vel. (m/s)	Washing Method	Washing Interval	Temp. (°C)
Large-Scale Sewage	1	100	1.5	0.7	Backwash by Perm. Flushing by Gas	10s/2min 150s/3 hr.	26
Small-Scale Sewage	2	54	1.1	1.0	Backwash by Perm.	30s/7.5min	26
Fat/Oil and Protein	1	50	1.3	0.7	Backwash by Perm. by Gas	10s/2min 150s/3hr.	33
Wheat Starch	2	54	0.79	0.9	Backwash by Perm.	25s/6.7min	37
	4	10	0.3-0.5	0.76	Stop Perm.	5min/ 8-20min	33
Alcohol Fermentation	3	12	1.3	1.0	Phys. & Chemical Cleaning	1.5/month	30
Pulp & Paper	3	20	0.45	1.0	Phys. & Chemical Cleaning	1/2-3weeks	35
	5	24	0.6	1.75	Stop Perm. Chemical Cleaning	12s/5min 1/1month	48
Night Soil	1	30	1.2	0.9	Backwash by Perm. by Gas	10s/2min. 150s/3hr.	30

Table Specifications of Membrane Modules Used

No.	Configuration (Manufacturer)	Dimension (mm)	Packing Density (m^2/m^3)	Material	Mol.Wt. Cut Off or Pore Size(μ)	PWP*
1	Capillary (Nitto Elc.)	O.D. 1.3 I.D. 0.8 length 1,000	714	PVA/PS	15,000	8.4
2	Hollow Fiber (Mitsubishi Rayon Eng.)	O.D. 0.39 I.D. 0.28 length 690	1607	PE	0.1	3.5**
3	Plate & Frame (Kurita Water Ind.)	590x1,430	64	PS	200x10^4	20
4	Plate & Frame (NGK Insu. Ltd.)	500x160 3t	49	Alumina	0.2	11
5	Tubular (TOTO Ltd.)	O.D. 5.2 I.D. 3.7 length 500	130	Alumina	0.16	25

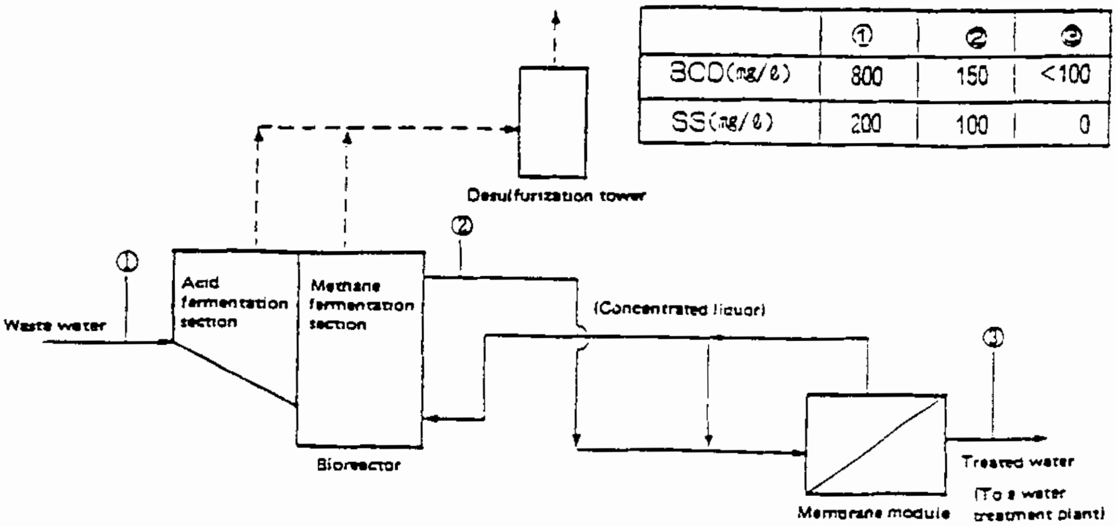
*: Pure water flux (m^3/m^2 day) measured at 1 kg/cm^2, 25 °C

**: Pure water flux (m^3/m^2 day) measured at 0.5 kg/cm^2, 20 °C

Table Performances of Membrane Modules Tested

Types of Waste Water	Feed Water			Permeate		Press. Loss (2)	Power Con. (3)	Module No.
	SS (mg/l)	Visc. (cP)	COD (mg/l)	COD (mg/l)	Flux (1)			
Large-Scale Sewage	483	1.0	1,144	70	0.38	0.2	0.74	1
Small-Scale Sewage	1,370	1.0	2,187	34	0.58	0.29	0.72	2
Fat/Oil and Protein	4,283	1.12	9,677	863	0.2	0.1	0.56	1
Wheat Starch	16,900	5.3	36,000	8,800	0.39	0.5	0.96	2
	16,900	4.4	28,600	5,700	0.25	0.18	0.94	4
Alcohol Fermentation	10,000	5	18,000	11,000	0.33	0.75	3.67	3
Pulp & Paper	15,000	6	28,000	1,100	0.3	0.4	1.41	3
	12,100	4.2	18,500	1,420	0.64	0.075	1.60	5
Night Soil	2,600	6	25,500	2,000	0.16	0.3	2.80	1

(1):(m^3/m^2 day)
(2):(kg/cm^2/module)
(3):(kwh/m^3 permeate) calculated value assuming pump efficiency is 60 % and motor efficiency is 85 %.



	①	②	③
BOD(mg/ℓ)	800	150	<100
SS(mg/ℓ)	200	100	0

Fig. 1 Field Test Plant for Fat-and-Oil and Protein Type Waste Water

	①	②	③
BOD(mg/ℓ)	15.000	12.000	200
SS(mg/ℓ)	3.000	10	10

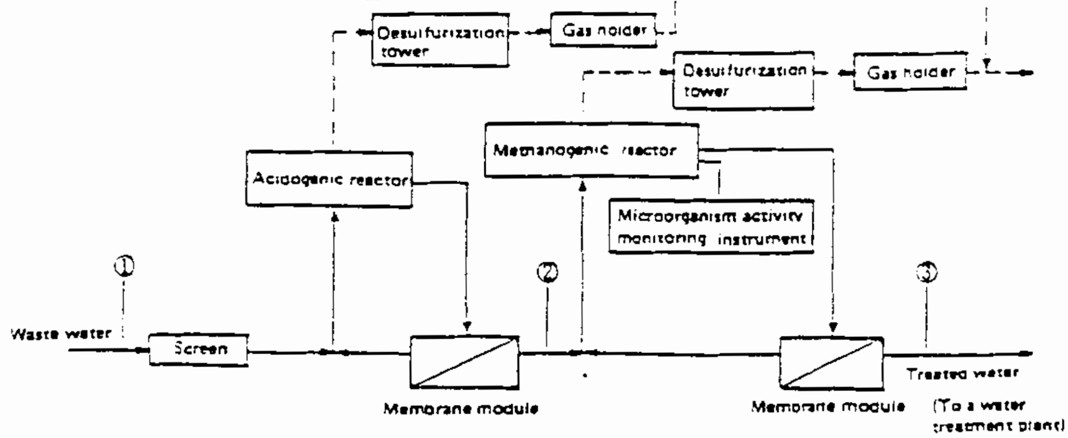


Fig. 2 Field Test Plant for Wheat Starch Waste Water

	①	②	③
BOD(mg/ℓ)	40.000	4.000	2.000
SS(mg/ℓ)	2.000	2.000	0

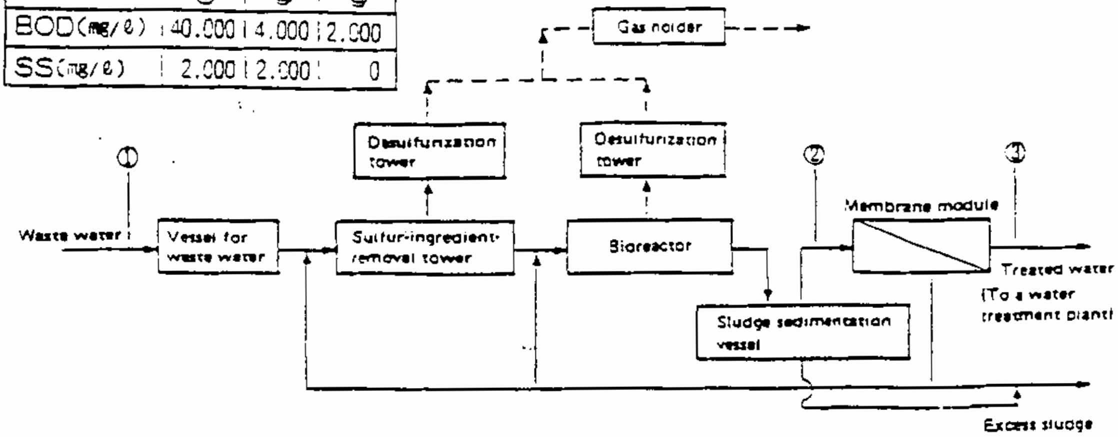
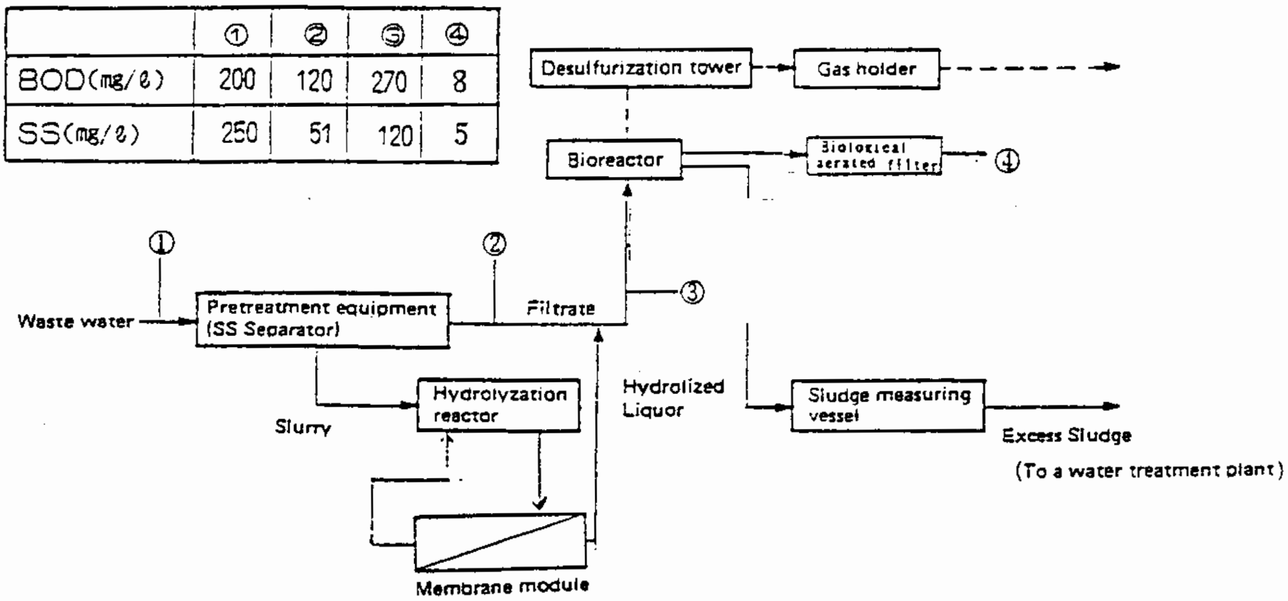
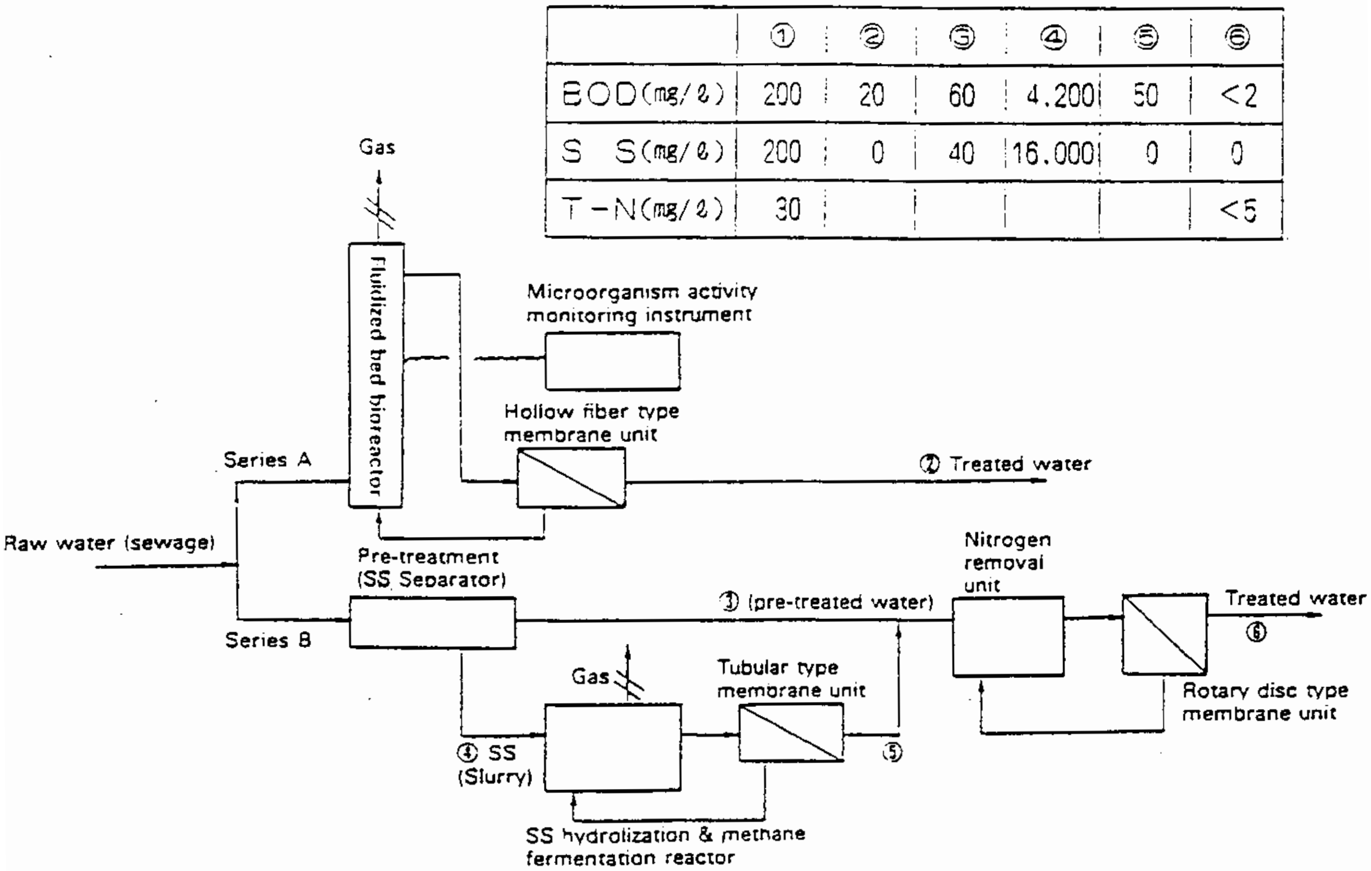


Fig. 3 Field Test Plant for Alcohol Fermentation Waste Water



	①	②	③	④
BOD(mg/ℓ)	200	120	270	8
SS(mg/ℓ)	250	51	120	5

Fig. 4 Field Test Plant for Large-scale (Municipal) Sewage



	①	②	③	④	⑤	⑥
BOD(mg/ℓ)	200	20	60	4.200	50	<2
S S(mg/ℓ)	200	0	40	16.000	0	0
T-N(mg/ℓ)	30					<5

Fig. 5 Field Test Plant for Small-scale (Rural) Sewage

	①	②	③	④
BOD(mg/ℓ)	10.500	10.500	1.200	1.200
SS(mg/ℓ)	<10	<10	16.000	<10

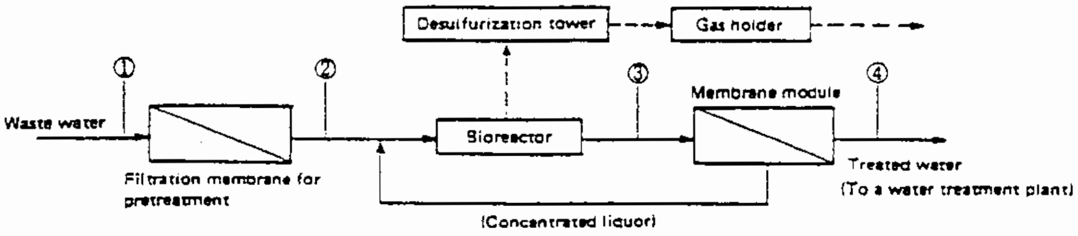
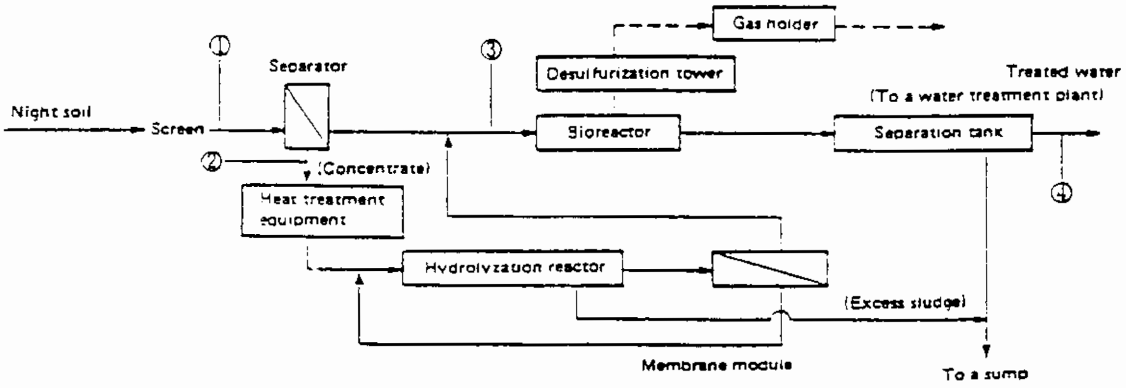
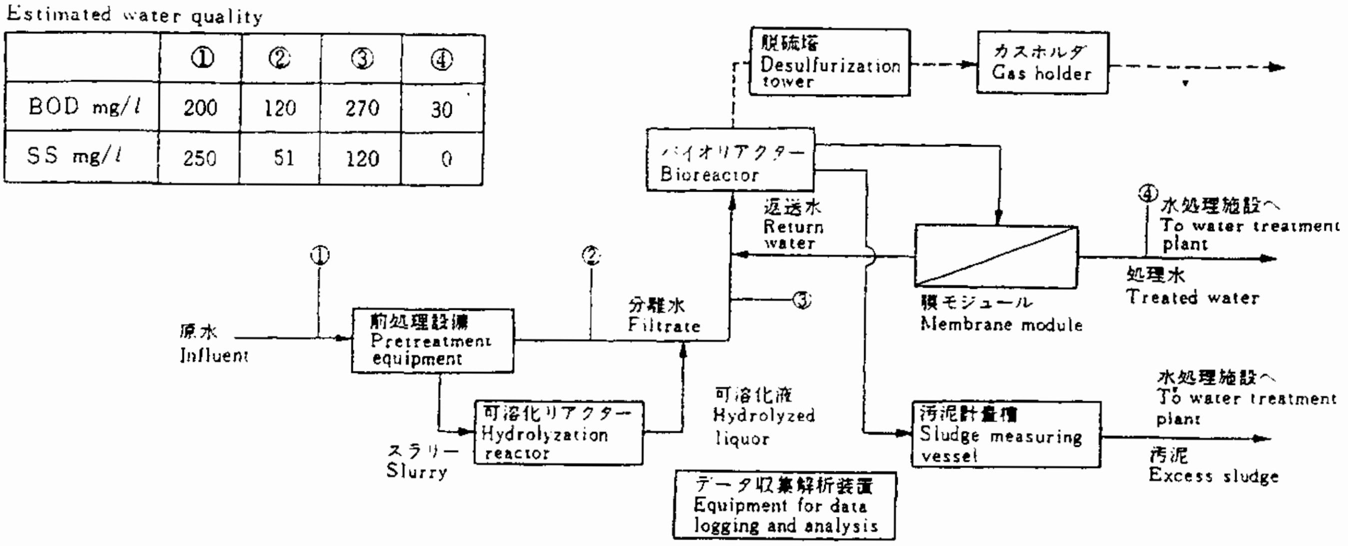


Fig. 6 Field Test Plant for Pulp and Paper Waste Water



	①	②	③	④
BOD(mg/ℓ)	11.000	21.000	14.000	300
SS(mg/ℓ)	12.000	30.000	0	200

Fig. 7 Field Test Plant for Night Soil

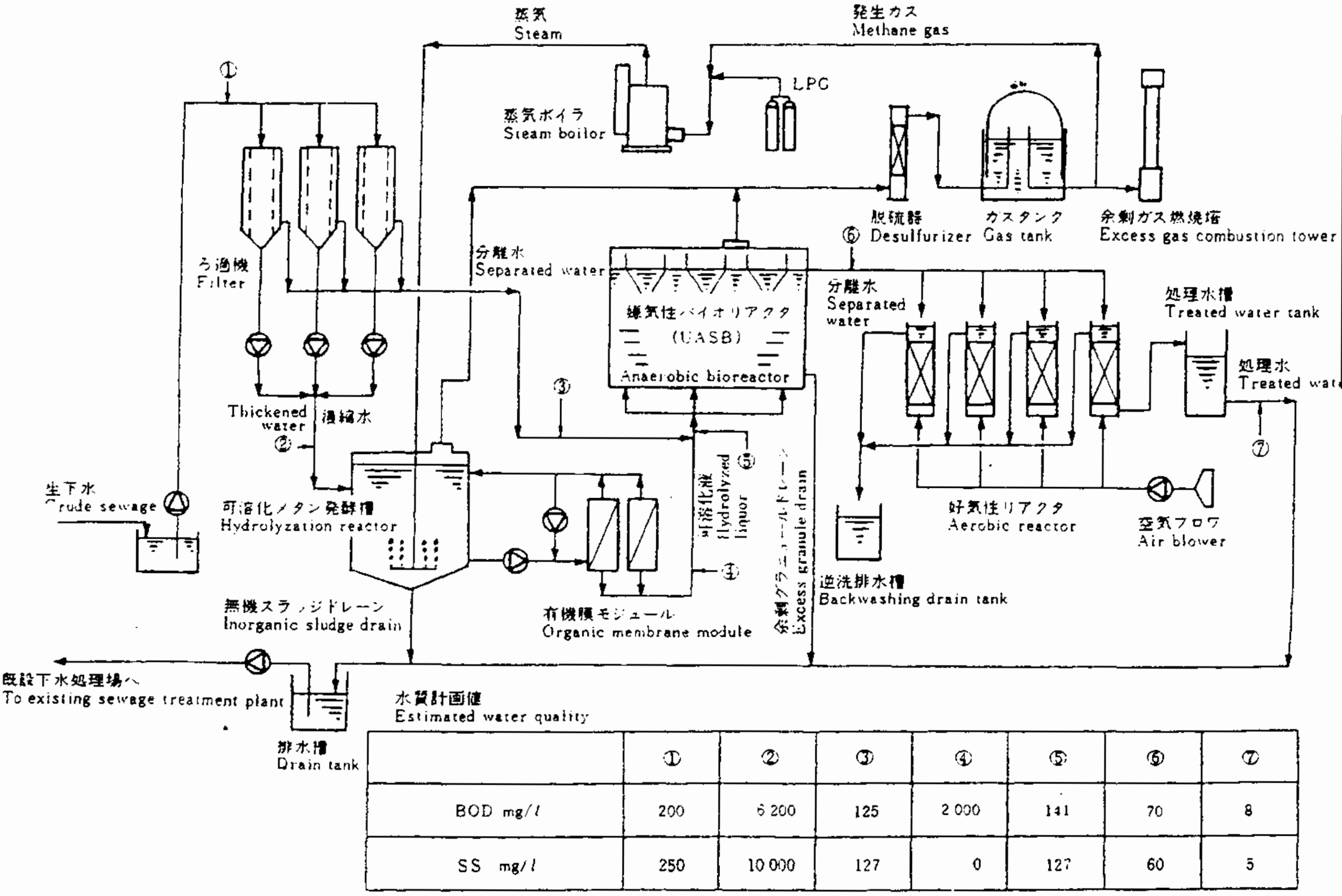


Estimated water quality

	①	②	③	④
BOD mg/l	200	120	270	30
SS mg/l	250	51	120	0

ベンチプラントプロセスフローシート

Fig. 8 Process flow-sheet of bench plant



水質計画値
Estimated water quality

	①	②	③	④	⑤	⑥	⑦
BOD mg/l	200	6 200	125	2 000	141	70	8
SS mg/l	250	10 000	127	0	127	60	5

パイロットプラントフローシート

Fig. 9 Flow sheet of pilot paint

APPLICATION OF BIOTECHNOLOGY TO MUNICIPAL WASTEWATER TREATMENT

T. Matsui, S. Kyosai and M. Takahashi

Water Quality Control Department, Public Works Research Institute, Ministry of Construction, Asahi, Tsukubashi, Ibarakiken 305, Japan

ABSTRACT

Application of biotechnology to wastewater treatment for its improvement and greater understanding was investigated in the Japanese national project named Biofocus WT as to a microorganism bank, genetic engineering application, immobilization of microorganisms, bioreactors for wastewater treatment, bioreactors for sludge treatment, SS separators for raw wastewater, biosensors and new wastewater treatment systems.

KEYWORDS

Biofocus WT; microorganism bank; genetic engineering; immobilization of microorganisms; bioreactors for wastewater treatment; bioreactors for sludge treatment; SS separators for raw wastewater; biosensors.

INTRODUCTION

The Japan Ministry of Construction conducted a research project, "The Development of New Wastewater Treatment Systems Employing Biotechnology", which was commonly called Biofocus WT, from fiscal year 1985 to 1989. By broadly applying biotechnology to wastewater treatment, Biofocus WT intended to solve the present and future problems in wastewater treatment. The Public Works Research Institute (PWRI) and the Building Research Institute of the Ministry of Construction were the main organizations carrying out Biofocus WT. PWRI was mainly in charge of matters related to publicly owned treatment works (POTW). Table 1 shows the subjects of research and development in Biofocus WT carried out by PWRI. This paper presents the interim results of the 8 subjects.

TABLE 1 Subjects of Research and Development in Biofocus WT Conducted by PWRI

a. Study on Microorganism Bank for Wastewater Treatment
b. Study on Application of Genetic Engineering to Microorganisms for Wastewater Treatment
c. Study on Immobilization Methods of Microorganisms for Wastewater Treatment
d. Development of Bioreactors for Wastewater Treatment
e. Development of Bioreactors for Sludge Treatment
f. Development of Solid-Liquid Separators for Raw Wastewater
g. Development of Biosensors for Water Quality Measurement
h. Development of New Wastewater Treatment Systems

STUDY ON MICROORGANISM BANK FOR WASTEWATER TREATMENT

Recently, environmental microbiology has achieved great progress. A microorganism bank for wastewater treatment was investigated in order to apply the results of microbiology research to wastewater engineering. The bank consists of two parts, a data bank and a preservation bank. The data bank accumulates data sheets in which microbiological and ecological data on microorganisms related to wastewater systems are summarized. The preservation bank is an ordinary one.

Data bank. Most microbiological data on any individual strain are obtained from pure cultures. However, most microbiological phenomena in biological wastewater treatment processes are observed in mixed cultures of bacteria, protozoa and other microorganisms. The results of the pure cultures are usually different from the phenomena in the mixed cultures. This understanding impedes the utilization of microbiological information for wastewater treatment systems. *The data bank would reconcile this situation by providing the ecological* properties of each strain. In addition, all strains are classified by their relationship to wastewater systems. Table 2 is the data sheet.

TABLE 2 Data Sheet of Data Bank

1. CODE
   (1) Bank Code
   (2) Scientific Classification: 1) Family 2) Species 3) Common Name
2. RELATION TO WASTEWATER TREATMENT
   (1) Functional Name
   (2) Relationship to Wastewater Treatment
3. DATA SHEET WRITER
   (1) Name
   (2) Affiliation
   (3) Date of Register
4. RESEARCHER
   (1) Name
   (2) Affiliation
5. PRESERVATION
   (1) Organization
   (2) Code Number in the Organization
6. ORIGIN OF STRAIN
   (1) Purification from Natural Environment
      a) Purification Data: Source of Strain, Date of Purification, Researchers
      b) Identification Data: Researchers, Affiliation
   (2) Genetic Engineering
      a) Applied Genetic Engineering Method
      b) Conditions: Host, Vector, Other
      c) Characteristics of Strain
   (3) Other
      a) Obtained from Other Researcher
      b) Characteristics of Strain
7. CHARACTERISTICS OF STRAIN
   (1) Physiological: Growth Rate, Substrate, Products, Genetic Data
   (2) Ecological: Environmental Condition for Growth, Competition, Symbiosis
8. NOTES FOR EXPERIMENT
   Epidemic Notes, Hazardous Products, Patents, so forth
9. CONDITION FOR PURIFICATION
   (1) Cumulative Cultivation: Temperature, Media, Method
   (2) Purification: Temperature, Media, Method
10. PRESERVATION OF VIABLE CELL: Temperature, Media, Subculturing Method
11. PRESERVATION AND REACTIVATION OF FREEZING CELL
   (1) Preservation: Temperature, Media, Period of Preservation
   (2) Reactivation: Temperature, Media, Reactivation of Specified Nature
12. RELATED LITERATURE

STUDY ON APPLICATION OF GENETIC ENGINEERING TO MICROORGANISMS FOR WASTEWATER TREATMENT

In addition to genetic engineering itself, there are several problems in the application of engineered microorganisms to wastewater treatment systems, such as their stability in the mixed culture and a biohazard problem. Basic experimental studies were conducted as to ecological and evolutionary stability of microorganisms using long-term cultures of enteric bacteria by serial transfer cultures. The results are shown in Figures 1 and 2, and summarized as follows: (a). The fitness of bacterial populations, which is expressed by the specific growth rate in the culture, increased with the number of transfers. This is considered to be caused by the appearance and increase of mutants with higher fitness in the culture, (b). The fitness of a bacterial population eventually came to a plateau level, (c). The plateau values of the fitness of bacteria were not changed by the treatment of a chemical mutagen with NTG, (d). The plateau values were different in species, and in case of C. freundii the values varied in strains, (e). Genetically identical bacterial strains which were derived from the same clone had the same value for the plateau level, (f). From the experimental results of mixed cultures consisting of two kinds of bacteria, both of which had reached each plateau level, it was revealed that the population with the lower plateau level decreased and became extinct in the mixed cultures, while that with the higher plateau level remained, and as a conclusion, (g). From the view point of the application of engineered microorganisms to wastewater treatment, it is important to use the bacterial species or strains with higher plateau levels for treatment.

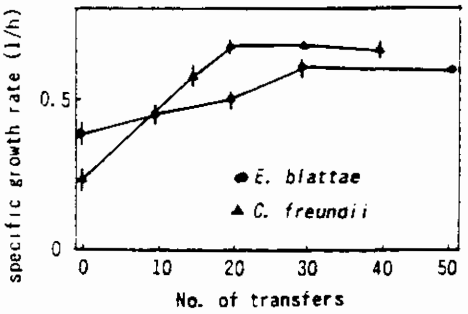


Fig. 1. Changes of specific growth rate of E. blattae and C. freundii IFO13545 isolated from the serial transfer cultures (i.e. from their single specie culture)

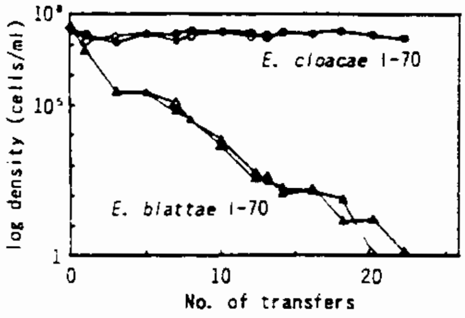


Fig. 2. Changes in population densities of E. cloacae and E. blattae in their mixed culture (Both were isolated from the serial transfer culture of 70 times.)

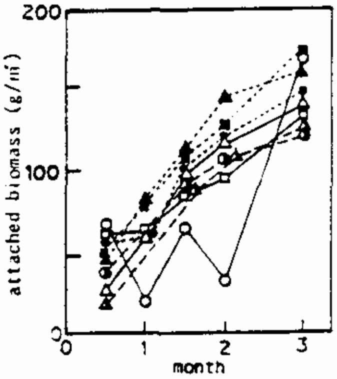
STUDY ON IMMOBILIZATION METHODS OF MICROORGANISMS FOR WASTEWATER TREATMENT

The immobilization of microorganisms in bioreactors for wastewater treatment and sludge treatment is useful to increase the concentration of microorganisms in the reactor, and to maintain a large amount of microorganisms for specifically removing particular pollutants. Three immobilization methods were investigated: binding method, entrapping method, and self-immobilization method. Microorganisms themselves attach to the surface of media in the binding method. Microorganisms are immobilized by chemicals in the entrapping method. Immobilization is made by microorganisms themselves with no other materials in the self-immobilization method.

Binding Method

Suitable shape and materials of attached media depend on the features of reactors in which the media are used. The reactors can be classified into fixed bed type or fluidized bed type and aerobic or anaerobic. The results for fixed bed type reactors are summarized.

Attached media for fixed-bed type reactor for aerobic treatment. Attached biomass was compared among eight different media. Figure 3 shows the results. Though attached biomass was different between media, sufficient biomass grew after 3 months on all media tested. The media with more specific surface area accumulated more biomass but required regular washing due to clogging. The media with diameter less than 1 cm removed SS, then additional SS removal was unnecessary, though washing was required. On the other hand, washing was not necessary for media with bigger diameters but solid separation from the effluent was essential.



Material	Surface Characteristics
◑ Stainless Steel	smooth
● Vinylchloride(a)	smooth
▲ Vinylchloride(b)	roughed with fine sand paper
■ Vinylchloride(c)	roughed with coarse sand paper
○ Ceramics(a)	pore size: 0.1 - 0.2 mm
△ Ceramics(b)	pore size: 0.01 - 0.1 mm
□ Ceramics(c)	pore size: 0.045 mm
▲ Polyethylene	smooth

Fig. 3. Comparison of growth rate of biomass attached on media (aerobic)

Attached media for fixed-bed type reactor for anaerobic treatment. A comparison test of attached media for anaerobic treatment revealed that porous media produce gas more than media with smooth surfaces, showing that microorganisms attach to the porous media more than the smooth media. As no aeration results in insufficient mixing in anaerobic treatment compared with aerobic treatment, it was necessary to select and arrange media so that adequate influent distribution was attained.

Entrapping Method

Polyvinyl alcohol (PVA), acrylic amide (ACAM) and polyethylene glycol (PEG) were examined as chemicals to entrap microorganisms. The PVA results are discussed here. Two PVA methods were examined: a PVA-boric acid method and a PVA-refrigeration method.

PVA-boric acid method. A mixture of PVA and activated sludge is dropped into saturated boric acid solution, then activated sludge is entrapped in lattices of pellets made of PVA and boric acid. Though the respiration rate of the entrapped activated sludge was low at the beginning of cultivation, the rate increased rapidly to 3 - 4 times the initial value after 10 days (Figure 4).

PVA-refrigeration method. The mixture of PVA and activated sludge is refrigerated and becomes a gel. The PVA concentration of 10 % was not enough to make pellets with enough strength. Pellets of 22.5 % showed a lower respiration rate than that of 15 %, thus PVA concentration of 15 % was suitable (Figure 5).

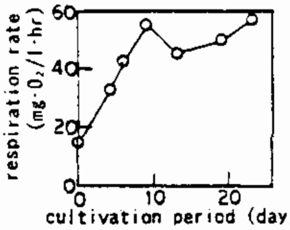


Fig. 4. Transition of respiration rate of microorganisms immobilized by PVA-boric acid method

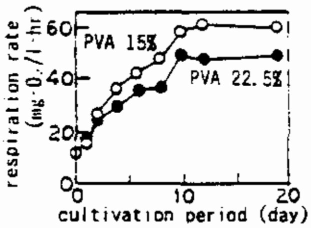


Fig. 5. Transition of respiration rate of microorganisms immobilized by PVA-refrigeration method

Self-Immobilization Method

UASB (upflow anaerobic sludge blanket) process applied for treatment of wastewater with high organic concentration has been well known as a process utilizing this method. In Biofocus WT, three kinds of self-immobilization were investigated. They are an UASB process, an AUSB (aerobic upflow sludge blanket) process, and MRB (multi-stage reversing-flow bioreactor). The three methods will be presented in the next topic.

DEVELOPMENT OF BIOREACTORS FOR WASTEWATER TREATMENT

Twenty-three bioreactors were investigated in pilot-plant scale by joint researches with private companies. They are divided into aerobic bioreactors, anaerobic bioreactors, MRB and bioreactors for nitrogen removal. All of the bioreactors incorporated immobilization methods. The main development target of aerobic bioreactors was to reduce the retention time in reactors to 1/2 to 1/10 compared with the conventional processes. That of anaerobic bioreactors was to reduce the energy requirement and/or sludge production to 1/2 compared with conventional processes. And that of bioreactors for nitrogen removal was to reduce the retention time in reactors for biological nitrogen removal to 6 to 12 hours so that existing plants could be modified to remove nitrogen. An AUSB process, an UASB process and MRB are summarized as examples of the developments.

AUSB process. Figure 6 is a flow diagram of the AUSB process. Influent flows into a pressurized oxygen tank. Effluent from the tank with 50 to 70 mg/l of DO was introduced to the bottom of the upflow reactor. The high DO and liquid recirculation made it possible to keep the sludge blanket aerobic. Within several weeks after the start-up, aerobic biological granules formed in the reactor. Effluent was good with HRT of 1.72 hrs.

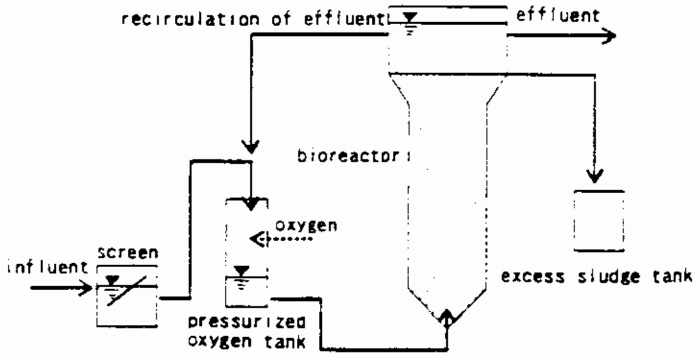


Fig. 6. Flow diagram of AUSB process

TABLE 3 Operational Conditions and Performance of AUSB Process

CONDITIONS		
HRT in sludge blanket, hr	1.72	
Total HRT, hr	4.00	
Recirculation Ratio of Effluent	1.57	
Up-Flow Velocity in Reactor, m/day	125	
Influent DO mg/l	63	
Effluent DO mg/l	4.8	
PERFORMANCE	**Influent**	**Effluent**
T-BOD mg/l	112.6	19.9
S-BOD mg/l	42.4	10.2
SS mg/l	91.3	9.6
T-N mg/l	30.9	24.6
T-P mg/l	2.7	1.8

Fig. 7. Self-granulated sludge in AUSB process

UASB process. Figure 8 is a flow diagram of UASB process. This system installed a modified trickling filter for post-treatment. The filter media were cubic sponges which catch fine suspended solids. Biological granules were formed in the anaerobic sludge blanket after 6 month acclimation. Though they were small in diameter of less than 1 mm, they were basically the same as biological granules generally found in the UASB process treating wastewater with high organic concentration. Effluent from the system was good with HRT of 8 hrs for UASB, 2 hrs for the trickling filter even in winter (Table 4). The system has neither liquid recirculation, nor aeration devices. If there is enough water head difference available between influent and effluent in a plant, this system can be an energy saving system.

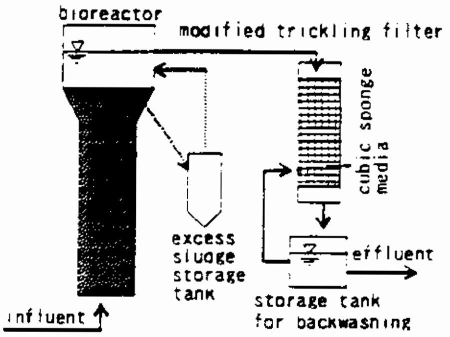


Fig. 8. Flow diagram of UASB process

TABLE 4 Operational Conditions and Performance of UASB Process

CONDITIONS			
Hydraulic Retention Time of Reactor, hr			8
Surface Loading of MTF, m/day			45
Water Temperature, C°			13-16
PERFORMANCE	Inf.	Eff.(UASB)	Eff.(MTF)
BOD mg/l	71-111	24-40	7-17
SS mg/l	45-99	8-15	2-7
Sulfide as S mg/l	-	6-8	0

MTF: Modified Trickling Filter for post-treatment

Fig. 9. Self-granulated sludge in UASB process

MRB. MRB has several stages as shown in Figure 10. Each stage consists of a downflow aeration vessel and an upflow biological reaction vessel. Successful accumulation of self-granulated sludge (SGS) was achieved in biological reaction vessels. Most SGS was covered with a thin white film. The film consisted of filamentous bacteria *Beggiatoa*. The inside of SGS was black and anaerobic, including sulfate-reducing bacteria. DO in the aeration vessel was limited to the saturation concentration. DO supplied from the aeration vessel was consumed rapidly in the biological reaction vessel if organic substrate in the wastewater was high. Consequently, near-anaerobic conditions allowing the growth of the sulfate-reducing bacteria were developed in the biological reaction vessel. Sulfide produced from this reaction diffused into bulk liquid through the SGS surface. Though the oxygen supply was limited in the biological reaction vessel, there was a chance of the microorganisms contacting oxygen on the SGS surface. Because an oxygen consumption rate of sulfide-oxidizing bacteria is much higher than that of organic substrate oxidizing bacteria, most the oxygen was utilized by the sulfide-oxidizing bacteria which are microaerophilic. *Beggiatoa*, which is one of the most common bacteria existing in such an environment, increased on the SGS surface in a thin film. A prototype plant study was conducted to demonstrate the performance of the MRB. Though the influent BOD was sightly low, the effluent quality was as good as that of a conventional activated sludge process with HRT of 4.5 hrs. MRB development has been under the stage of a pilot plant with the maximum treating capacity of 450 m^3/d.

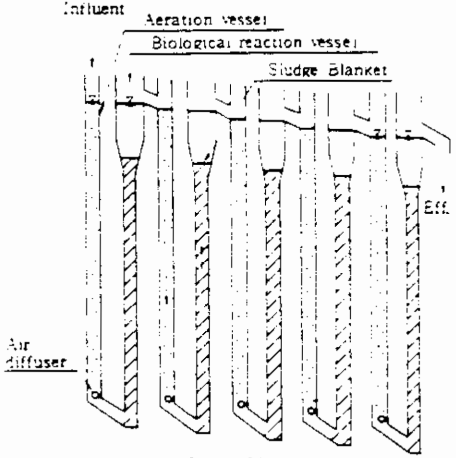


Fig. 10. Flow diagram of MRB

TABLE 5 Operational Conditions and Performance of Prototype MRB

CONDITIONS	
Influent: Settled municipal wastewater	
BOD volumetric load for total volume	0.42 kg/m^3.day
for BV volume	0.73 kg/m^3.day
Hydraulic retention time	4.5 hour
Dissolved oxygen concentration in AV	5-9 mg/l

PERFORMANCE					
No. of BV	No.1	No.2	No.3	No.4	No.5
Total Solids(mg/l)	21,000	14,600	8,900	5,900	14,000

		Influent	Effluent
BOD	(mg/l)	80	14
D-BOD	(mg/l)	50	9
SS	(mg/l)	30	4

Fig. 11. Self-granulated sludge in MRB

DEVELOPMENT OF BIOREACTORS FOR SLUDGE TREATMENT

There were two objectives in the development of anaerobic sludge digestion processes: to shorten the sludge digestion period to less than 10 days and to increase the solid decomposition percentage by 1.5 times that of the conventional process. Two techniques were investigated in order to attain the objectives: liquefaction of feed sludge and immobilization of bacteria related to anaerobic sludge digestion by the binding method. It is well known that liquefaction of feed sludge, especially excess biological sludge, is the most limiting factor in the digestion. As attempts were unsuccessful to find bacteria and enzymes which accelerate the liquefaction of sludge, the direction was changed to use physical-chemical methods. The following three methods were investigated: (a) Crushing excess sludge by a mill with 0.05 N NaOH (Table 6), (b) Heat treatment of excess sludge with conditions of 60°C for 1 hour, and (c) Heat treatment of excess sludge with conditions of 90°C for 1 hour and protease addition to the heat-treated sludge flowing to an acidification process.

TABLE 6 Liquefaction of Excess Sludge by Crushing with NaOH Addition

Items (mg/l)	CRUSHING		CRUSHING WITH NAOH		
	Control	Crushed	Control	NaOH Addition	Crushed with NaOH
TS	24,100	23,900	24,800	25,000	24,800
VS	19,700	18,800	19,500	19,300	18,400
VSS	18,400	12,500	18,100	16,400	8,100
pH	6.0	6.2	6.1	7.1	7.0
Sol. Sugars	199	458	128	232	554
Sol. Proteins	250	2,420	205	1,060	3,030
VFA	1,836	3,070	1,361	3,222	3,847
formic	-	114	-	28	51
acetic	580	2,082	460	1,824	3,162
propionic	425	-	346	350	-
isobutyric	84	348	250	99	-
butyric	401	-	327	239	-
isovaleric	157	526	119	169	-
valeric	189	-	109	513	168

Attached media for bacteria in sludge digestion are different from those for bacteria in wastewater treatment, because sludge is much more viscous and contains more SS than wastewater. Cement balls as attached media for a fluidized bed type reactor, and ceramics as attached media for a fixed bed type reactor, were used. Three pilot plants were operated under the joint researches with private companies.

All bioreactors for wastewater treatment developed in Biofocus WT incorporate immobilization methods in the reactors for microorganism increase. A problem may exist in reactor clogging due to SS in influent for such reactors, especially fixed bed type reactors. In addition, as dissolved organics are decomposed by microorganisms faster than particulate organics, removal of the bigger particles from raw wastewater makes such bioreactors more efficient.

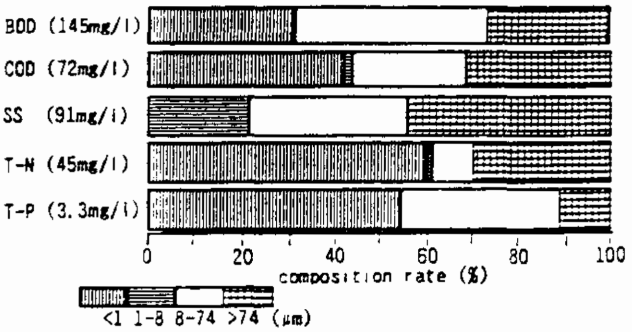


Fig. 12. Distribution of particulate contaminants in raw wastewater

Distribution of particulate contaminants in raw wastewater. Size distribution of SS in raw wastewater in relation to contaminants was examined. Figure 12 is an example of the results. Percentage of particulate BOD in raw wastewater was higher than those of particulate nitrogen and phosphorus.

Biodegradability of particulate TOC. Biodegradability of particulate TOC was examined under aerobic and anaerobic conditions. Figures 13 and 14 are examples in an aerobic condition and an anaerobic condition. Initial biodegradation rate of particulate TOC of particle size more than 1 micrometre was much lower than that of smaller than 1 micrometre in both conditions.

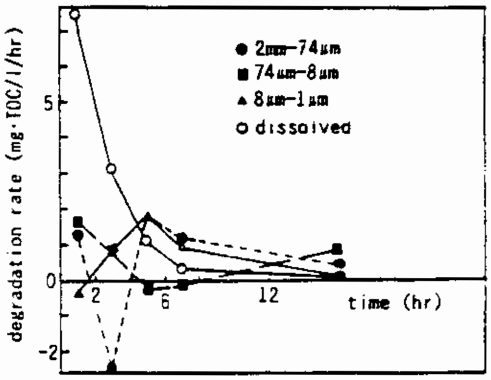


Fig. 13. Difference of biodegradation rates of TOC due to particle size of contaminants in aerobic condition

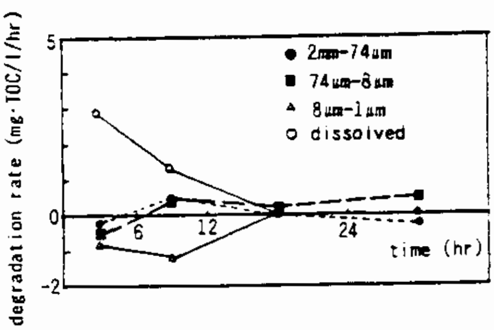


Fig. 14. Difference of biodegradation rates of TOC due to particle size of contaminants in anaerobic condition

Solid-liquid separators for raw wastewater. Five solid-liquid separation processes for raw wastewater were developed partially by the joint researches with private companies. (a). Upflow sedimentation process, (b). Floating-bed filtration process, (c). Filtration process with tube-shaped filtration media, (d). High rate bioflocculation process with a lamella settler, and (e). Two stage filtration process with an upflow moving bed and a downflow fixed bed. Figure 15 is an example of SS removal obtained by b., c. and d. Removal rates of SS larger than 8 μm in raw wastewater were more than 80 % in these three processes. On the other hand, SS smaller than 8 μm were difficult to remove by these processes.

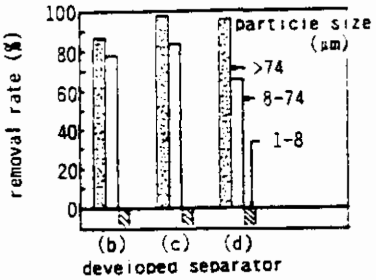
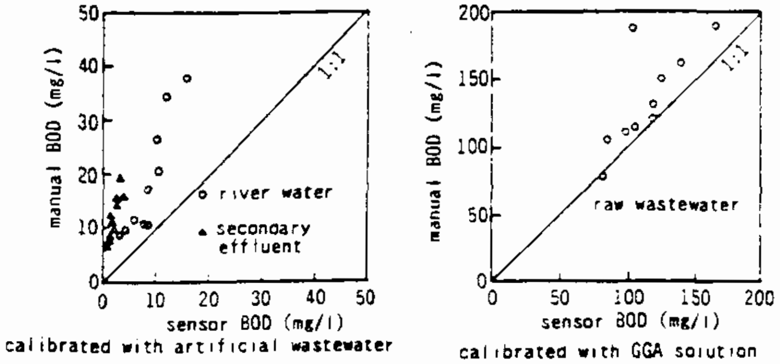


Fig. 15. SS removal from raw wastewater

DEVELOPMENT OF BIOSENSORS FOR WATER QUALITY MEASUREMENT.

Biosensors, which use enzymes or microorganisms as detectors to measure objective matters, were developed for BOD, ammonia-nitrogen and organic acids.

BOD sensor. Figure 16 shows the relationship between the sensor BOD and manually analyzed BOD_5. In case of raw wastewater, the sensor BOD showed enough output current. In cases of secondary effluent and river water, however, the current difference was so small that the sensor BOD showed much lower than the manual BOD_5.



calibrated with artificial wastewater

calibrated with GGA solution

Fig. 16. Relationship between sensor BOD and manual BOD

Ammonia sensor. Figure 17 shows an example of the ammonia biosensor performance. The relationship between ammonia nitrogen concentration below 10 mg/l and a current difference was linear. According to the biomass increase, the upper sensible concentration became lower, but the resolution became better. As a nitrification bacterium is very sensitive to toxic substances, application of the developed ammonia biosensor to a toxic sensor has been expected.

Organic acid sensor. Figure 18 shows an example of the performance of the organic acid biosensor. A yeast was used as a receptor. A linear relationship was observed between an acetic acid concentration below 600 mg/l and the current difference. But the sensor had a problem in detecting total organic acids, because each organic acid had different response from the sensor.

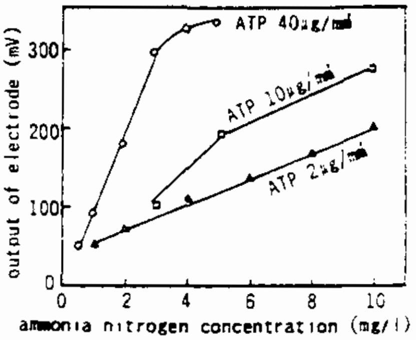


Fig. 17. Relationship between sensor ammonia and manual ammonia

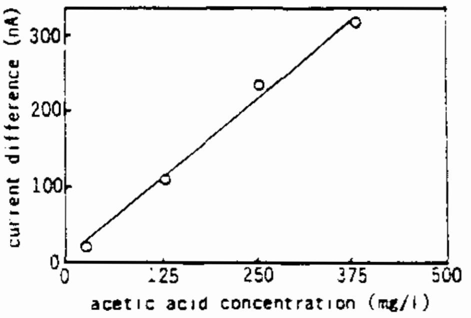


Fig. 18. Calibration curve of organic acid sensor for acetic acid solution

DEVELOPMENT OF NEW WASTEWATER TREATMENT SYSTEMS

Bioreactors for wastewater and sludge treatment, solid-liquid separators and biosensors were developed in addition to the basic studies on the microorganism bank and the genetic engineering application in Biofocus WT, as described above. Systematizing the individual development to propose new wastewater treatment systems was considered. Table 7 summarizes new wastewater treatment systems to be proposed based on results of pilot-plant studies.

TABLE 7 New Wastewater Treatment Systems to be Proposed in Biofocus WT

a. Energy-saving Type Wastewater Treatment System
b. Area-saving Type Wastewater Treatment System
c. Nitrogen Removal Type Wastewater Treatment System
d. Simultaneous Nitrogen & Phosphorus Removal Type Wastewater Treatment System
e. Wastewater Treatment System for Small Flow

A self-sustaining energy system has been proposed as an ultimate system of an energy-saving type wastewater treatment system. The system is illustrated in Figure 19. Most SS in raw wastewater are removed by a high-efficiency solid-liquid separator. Energy consumption in the secondary treatment can be greatly reduced by two means. One is influent to the secondary treatment containing not only less organics, but less particulate organics which are more difficult to decompose than soluble ones. The other is adopting an energy-saving type bioreactor. In sludge digestion, the feed sludge has a large amount of primary sludge which has higher potential for generating digestion gas than that of excess sludge. Also, an energy-creating type bioreactor for sludge digestion is employed, so that a greater amount of energy can be recovered. A key point of the system development is how energy consumption of a high-efficiency liquid solid separator can be reduced.

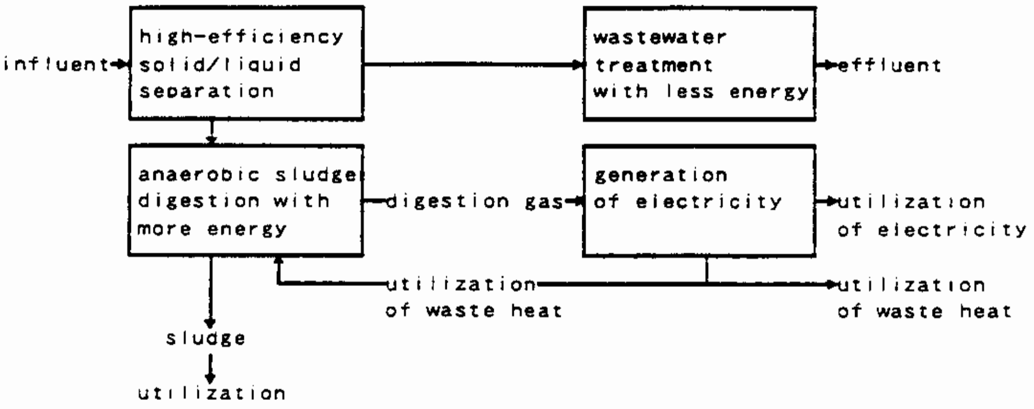


Fig. 19. Self-energy sustaining wastewater treatment system

POSTSCRIPT

As Biofocus WT was a research project which intended to broadly apply biotechnology to biological wastewater treatment for its improvement and greater understanding, it included various research subjects from very basic ones to practical ones, as mentioned above. Though the project was finished last fiscal year, the final report is being prepared at present. The results of Biofocus WT will be varied, from those of identifying further research needs to practically feasible technologies which may break through present technologies. We look forward to having an opportunity to present the final results.

JAPAN'S AQUA RENAISSANCE '90 PROJECT

Shoji Kimura

Department of Chemical Engineering, University of Tokyo, 7-3-1 Hongo, Bunkyo-ku, Tokyo 113, Japan

ABSTRACT

Aqua Renaissance'90 is a six year R & D project for water reuse and energy recovery supported by the Japanese Ministry of International Trade and Industry. The objective is to develop low cost treatment processes utilizing bioreactors coupled with membrane separation units to produce reusable water from industrial waste water and sewage. Polymeric and ceramic membranes in capillary, hollow fiber, tubular and plate and frame modules have been tested in conjunction with bioreactors on a number of actual waste water and sewage streams. This paper will focus on membrane performance and power consumption. Recent results from test plant operation indicate that treatment of concentrated waste water by membrane separation units uses less than a third of the conventional power requirement.

KEYWORDS

Bioreactor, membrane, tubular module, capillary module, hollow fiber module, plate and frame module, waste water, sewage, water reuse, anaerobic bioreactor.

INTRODUCTION

Aqua-Renaissance'90 is a six year R & D project for water reuse and energy recovery, which was planned by the Ministry of International Trade and Industry (MITI) as one of its National Research and Development Programs for Large-Scale Industrial Technology.

The fundamental objective of this project was to find ways to meet middle- and long-range water demand and, at the same time, solve water pollution problems and recover energy from pollutants. The particular objective was to develop energy-saving and space-saving water treatment processes utilizing bioreactors coupled with membrane separation units to produce reusable water from industrial waste water and sewage. The conceptual flow of this project is shown in **Fig. 1**. An anaerobic bioreactor decomposes suspended matter in various kinds of waste water and produces methane gas and water, which is clarified by a membrane separation unit to produce reusable water.

Following this fundamental scheme, several actual processes for treating waste water and sewage were proposed by the planning committee of the Aqua-Renaissance Research Association under MITI, whose members are called from universities, governmental institutions and private companies. With consideration given to the capabilities of the presently available bioreactors and the power consumption of conventional sewage plants, objective targets for

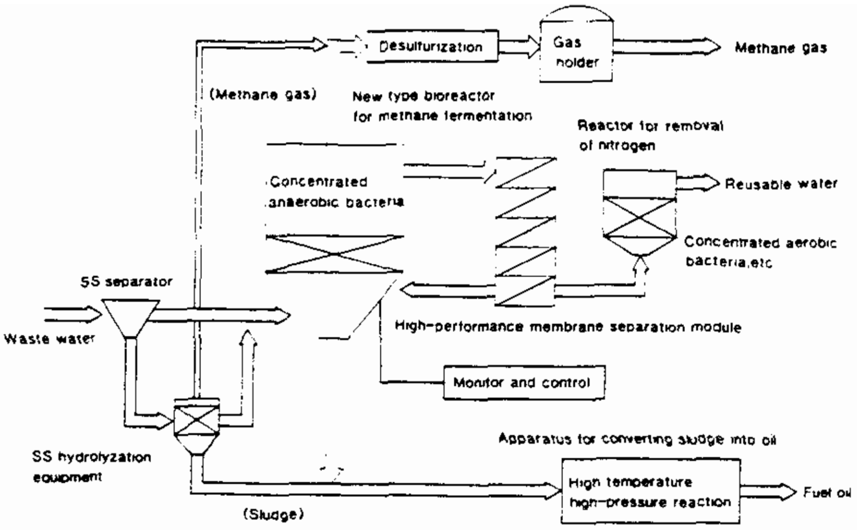


Fig.1 Conceptual Flow Chart of Aqua-Renaissance'90 Project

reactors and membrane modules were set, as shown in **Tables** 1 and 2.

Table 1 Objective Targets of Bioreactors

BOD level of waste water	Volumetric organic loading rate	Gas conversion rate
>2,000 mg BOD/l	for heated reactor >15kg BOD/m^3 day for non heated reactor >5 kg BOD/m^3 day	≥ 80 %
200-10,000 mg BOD/l	for non heated reactor >2 kg BOD/m^3 day	≥ 60 %

Table 2 Objective Targets of Membrane Modules

Microbe concentration of mixed liquor	Power consumption per unit flux
10,000 mg/l	≤ 1.5 kwh/m^3
100 mg/l	≤ 0.3 kwh/m^3

The R & D schedule of this project is shown in **Table 3.** The project got under way in 1985, and from 1986, various companies began to develop their own bioreactor and membrane module facilities, proposals for which were accepted by the planning committee. After intermediate evaluation of various reactors and membrane modules, 7 different bench scale field test plants which treat different kinds of waste water and sewage effluents were built in 1988 and are now being run until the end of fiscal 1989 to demonstrate their ability. The project is now running one year behind schedule as shown by the dotted lines in **Table 3.**

Table 3 Research and Development Schedule of the Project

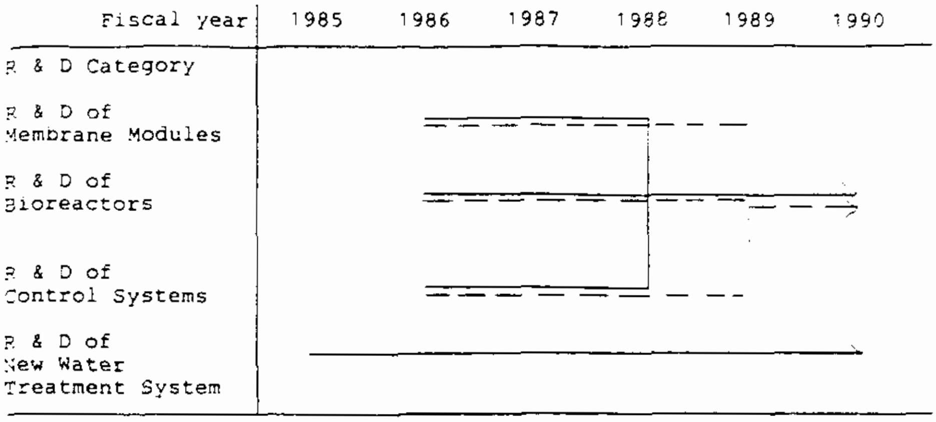


Table 4 lists kinds of feed water and their capacities. In 1989 two systems were selected with a view to building and running larger pilot plants for final demonstration and evaluation by the end of fiscal 1990.

Table 4 Test Units for Various Kinds of Waste Water and Companies in Charge

	Type of Waste Water	Capacity (m^3/day)	Companies in Charge	Location
1*	Large-Scale (Municipal) Sewage	20	Ebara Corp., Kawasaki Heavy Ind., Nitto Denko Corp., Toshiba Corp.	Fujisawa city
	Small-Scale (Rural) Sewage	10	Water Re-use Promot'n Center Ishikawajima Harima Heav.Ind. DIC-Degremont Co., Mitsubishi Rayon Eng., Hitachi Plant Eng., Mitsubishi Elec. Corp.	Chigasaki city
	Fat/Oil and Protein	7.5	Chiyoda Corp., Nitto Denko Corp.	Yokohama
2**	Wheat Starch	5	Kobe Steel Ltd., Mitsubishi Rayon Eng., NGK Insulators Ltd., Mitsubishi Elec. Corp.	Hyogo Pref.
	Alcohol Fermentation	5	Sanki Eng. Co., Kurita Water Ind.,	Kanoya city
	Pulp & Paper	10	Kurita Water Ind., Shimizu Corp., TOTO Ltd., Toshiba Corp.	Iyo-mishima city
	Night Soil	0.5	Water Re-use Prom. Center Nishihara Env.Sanit.Eng., Nitto Denko Corp.	Aichi Pref.

1*: Low-concentration waste water
2**: Mid.-to-high-concentration waste water

DESCRIPTION OF VARIOUS UNITS

As seen from **Table 4** the various units are divided into two groups, depending on the feed water BOD value. There are three units for low BOD feed and 4 units for high BOD feed. They will be explained in the order shown in **Table 4**.

Field test unit for large-scale (municipal) sewage

This unit is for municipal sewage effluent, and the design capacity is 20 m^3/day. It consists of pretreatment equipment for the separation of suspended solids, a hydrolyzation reactor for organic solids, an anaerobic bioreactor for methane fermentation, a membrane module unit, equipment for treating gas and a data processing system. The flow scheme is shown in **Fig. 2**. Waste water is fed through a preliminary screen and separated by a solid separator into water and solid slurry, which is sent to a hydrolyzing reactor. The hydrolyzed solution and water from the solid separator are mixed and sent to a bioreactor where organic matter is converted to methane and carbon dioxide. The reactor used is of the sludge blanket type and has a volume of 5.4 m^3. The supernatant from the reactor is fed to a membrane unit, whose permeate is discharged for reuse, while the concentrate is recycled to the reactor for concentrating bacteria. Inorganic matter tends to accumulate and is periodically discarded.

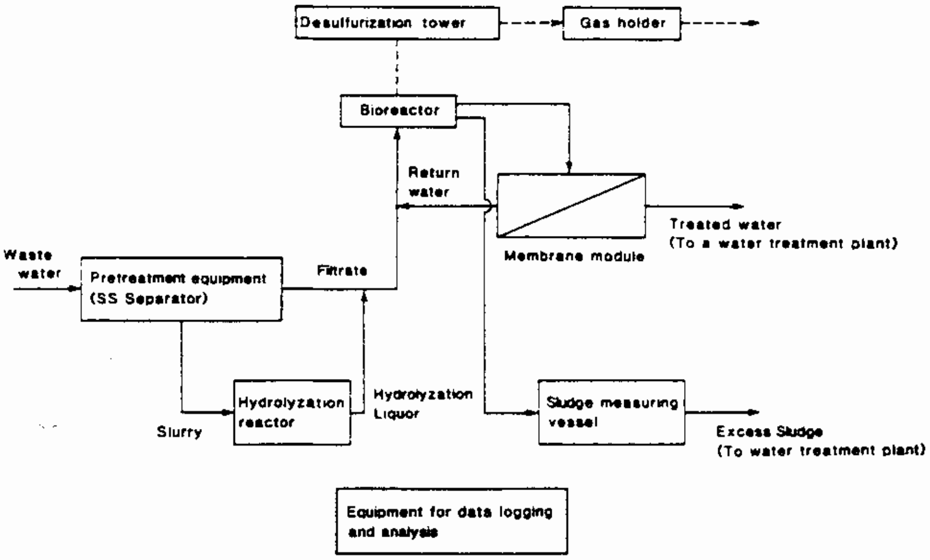


Fig.2 Flow Scheme of a Field Test Unit for Large-scale Sewage

Field test unit for small-scale (rural) sewage

This scheme is shown in **Fig.** 3 and is very similar to the previous one. The reactor used is of the fluid-bed type and has a capacity of 1 m^3. There are three membrane modules. One is used after the hydrolyzing unit, the second is used after the reactor and is the main one, and the third one is used after nitrogen removal and is the subsidiary one. The first one uses tubular membranes and the last one uses rotating disc membranes. Detailed data on these modules are not reported here.

Field test unit for effluent from a soybean oil refinery

This unit is tested for an effluent from a soybean oil refinery, which contains fat, oil and protein. The bioreactor used here is of the fixed bed type and has two parts, namely acid and methane fermentation sections. The scheme shown in **Fig. 4** was the original one, but later a membrane unit was moved between the acid and the methane section to promote hydrolyzation of soybean solids.

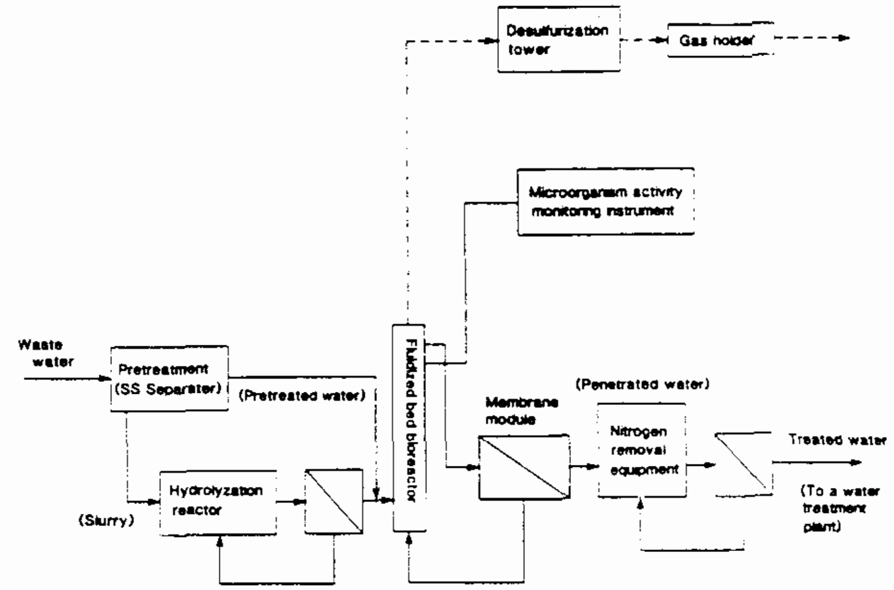


Fig.3 Flow Scheme of a Field Test Unit for Small-scale Sewage

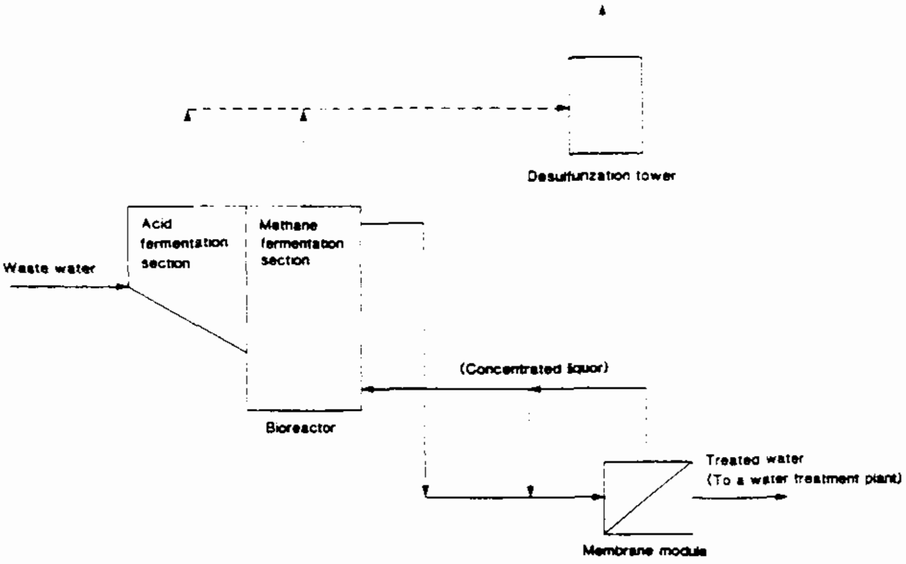


Fig.4 Flow Scheme of a Test Unit for Effluent from an Oil Refinery

Field test unit for wheat starch waste water

This unit is for an effluent from a factory producing starch from wheat. As shown in Fig. 5 it consists of two bioreactors, namely, a fixed bed acidogenic one and a sludge-blanket methanogenic one. A hollow fiber and a ceramic membrane unit are located after both the acidification and the methanization apparatus.

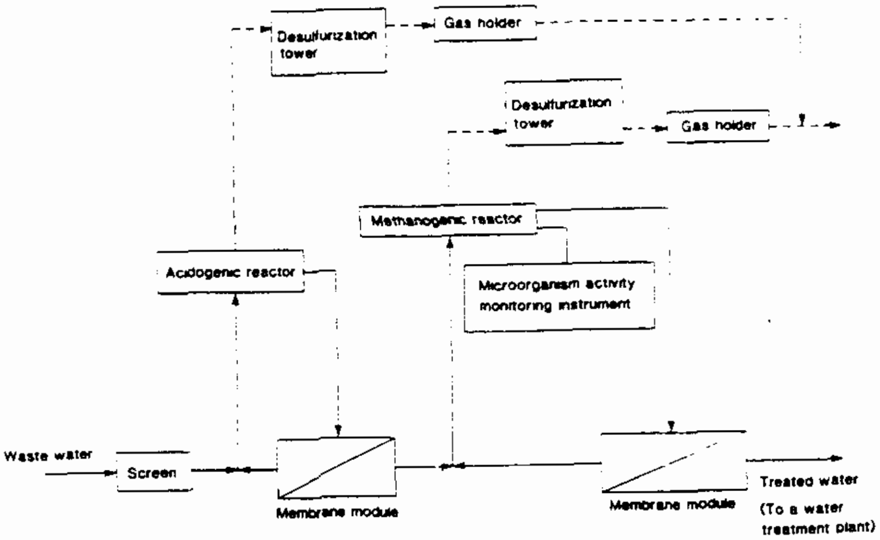


Fig.5 Flow Scheme of a Test Unit for Wheat Starch Waste Water

Field test unit for alcohol fermentation waste water

This unit is used for an effluent from an alcohol fermentation factory, where alcohol is produced by fermentation from molasses and distilled, leaving a very high BOD effluent. The flow scheme is shown in Fig. 6. Before a 8 m^3 fluidized-bed-type bioreactor, there is a desulfurization tower to remove sulfur in the feed, and after the reactor there is a membrane unit to purify water and to concentrate MLSS in the reactor.

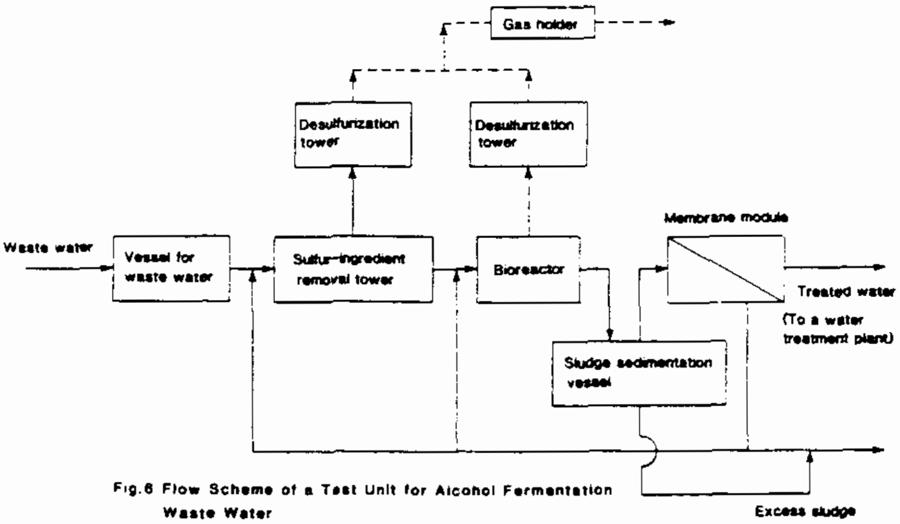


Fig.6 Flow Scheme of a Test Unit for Alcohol Fermentation Waste Water

Field test unit for pulp and paper waste water

This unit is to process waste water coming out as evaporator condensate from a Kraft paper plant, which contains high BOD and sulfur. A 7 m^3 fixed-bed-type bioreactor is used, and two different membrane units are tested. The flow scheme is shown in **Fig. 7**.

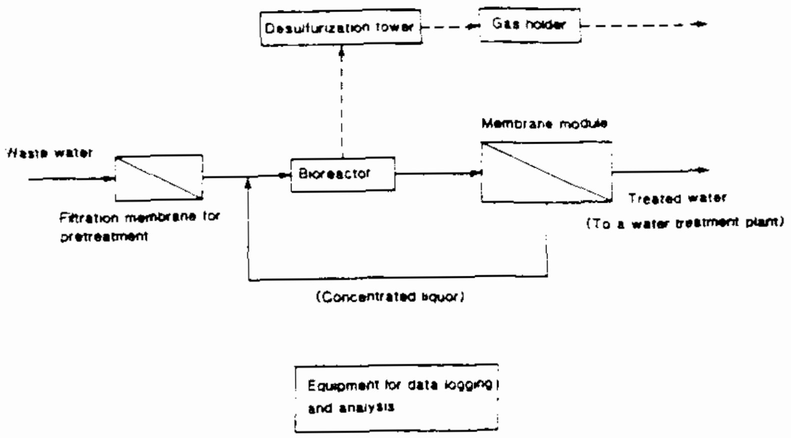


Fig.7 Flow Scheme of a Test Unit for Pulp and Paper Waste Water

Field test unit for night soil

A flow scheme is shown in Fig. 8. Night soil, which is screened to remove coarse solids, is fed to a liquid-solid separator, and the solid fraction is crushed, heat-treated and then fed to a hydrolyzation reactor. The effluent from here is treated by a membrane unit, and the permeate is mixed with the liquid fraction and fed to a bioreactor to recover methane. This reactor is of the sludge blanket type and its volume is 0.4 m^3.

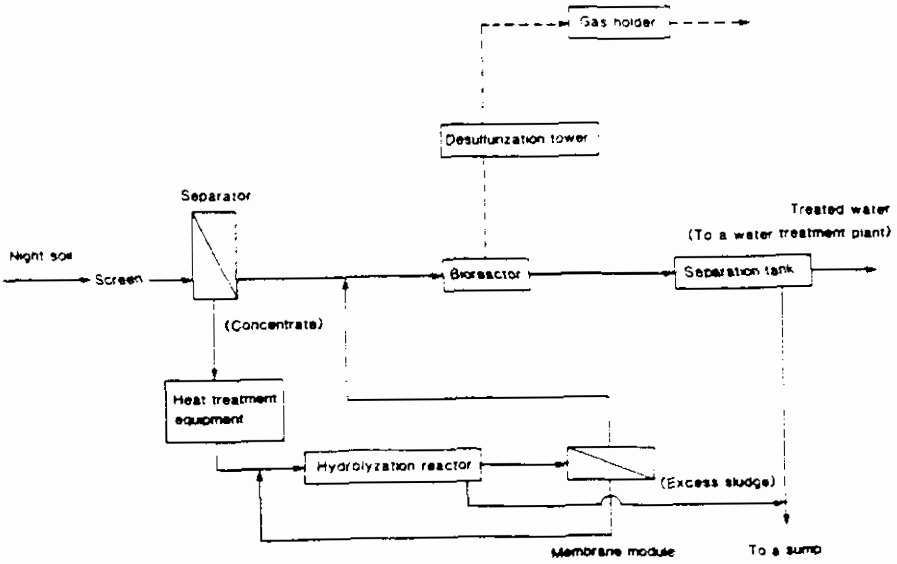


Fig.8 Flow Scheme of a Test Unit for Night Soil

DESCRIPTION OF MEMBRANE UNITS

The membrane modules tested in this project are listed in Table 5. Three organic and two ceramic membranes were used. Number 1 is a capillary-type module, made by Nitto Denko Corp., whose basic membrane consists of polyvinyl alcohol coated on a porous poly-sulfone substrate. Capillaries are contained in a pressure vessel whose dimensions are Φ100x1,000L and total area is $10m^2$.

Number 2 is a hollow-fiber-type module, made by Mitsubishi Rayon Co., which is made of polyethylene and was originally developed as a microfiltration membrane. The size of the pressure vessel is Φ 65x1,000L. Large modules using this element are presently used for purification of cooling water in atomic power stations in Japan.

Number 3 is a plate & frame-type module using commercial flat sheet polysulfone membranes. Module design factors such as water inlet, outlet, gap length and configuration of spacers were the main factors for optimization against high SS feed. The development was investigated by Kurita Water Ind. A final module consists of 28 plates, each of which has a 0.8 m^2 membrane area, so its total area is $22.4m^2$.

Number 4 is a plate & frame design using flat ceramic plates. About 84 plates were packed into one module having a 5.46 m^2 membrane area. This module was developed by NGK Insulator Ltd.

Number 5 is a tubular-type module using ceramic tubes, where feed flows inside tubes and permeate comes outside. One module consists of 200 tubes and has a total membrane area of [illegible] m^2. Six modules are connected vertically in series and four in parallel, and feed water is circulated by gas lift flowing inside tubes. The combined gas and liquid flow is intended to simulate washing by gas flushing to reduce fouling. Preliminary data were taken by using model solutions and were openly published (1).

Table 5 Specifications of Membrane Modules Used

No.	Configuration (Manufacturer)	Dimension (mm)	Packing Density (m^2/m^3)	Material	Mol.Wt. Cut Off or Pore Size(μ)	PWP*
1	Capillary (Nitto Elc.)	O.D. 1.3 I.D. 0.8 length 1,000	714	PVA/PS	15,000	8.4
2	Hollow Fiber (Mitsubishi Rayon Eng.)	O.D. 0.39 I.D. 0.28 length 690	1607	PE	0.1	3.5**
3	Plate & Frame (Kurita Water Ind.)	590x1,430	64	PS	200×10^4	20
4	Plate & Frame (NGK Insu. Ltd.)	500x160 3t	49	Alumina	0.2	11
5	Tubular (TOTO Ltd.)	O.D. 5.2 I.D. 3.7 length 500	130	Alumina	0.16	25

*: Pure water flux (m^3/m^2 day) measured at 1 kg/cm^2, 25 °C

**: Pure water flux (m^3/m^2 day) measured at 0.5 kg/cm^2, 20 °C

OPERATING CONDITIONS OF MEMBRANE MODULES

These modules had been tested to find appropriate modular designs, operating parameters, washing methods and intervals using model solutions and/or effluents from their own bioreactors, before they were combined in each test unit. From fiscal 1988, the units shown in **Table 4**, including bioreactors and membrane modules, have been running to provide performance data, and they are giving quite interesting results. But since these test units are still running and the personnel in charge are trying to improve their equipment, it would be premature to report their performance data now. At the same time, the objective of this test run is to obtain the best performance data of the system as a whole, not only of individual pieces of equipment.

In the following, examples of presently available data on the operation parameters and performances of membrane modules are shown. In **Table 6**, operating parameters and washing methods are listed. It is seen that most modules are cleaned at very short intervals by flushing or backwashing, while the plate & frame- type module is run without cleaning for a long period of time.

Table 6 Operating Parameters of Membrane Modules Used

Types of Waste Water	Module used	Memb. Area (m^2)	Appl. Press. (kg/cm^2)	Feed Vel. (m/s)	Washing Method	Washing Interval	Temp. (°C)
Large-Scale Sewage	**1**	100	1.5	0.7	Flushing by Gas	150s/3 hr.	26
Small-Scale Sewage	**2**	54	1.1	1.0	Backwash by Perm.	30s/7.5min	26
Fat/Oil and Protein	**1**	50	1.3	0.7	Backwash by Perm. by Gas	10s/2min 150s/3hr.	33
Wheat Starch	**2**	54	0.79	0.9	Backwash by Perm.	25s/6.7min	37
	4	10	0.3-0.5	0.76	Stop Perm.	5min/ 8-20min	33
Alcohol Fermentation	**3**	12	1.3	1.0	Phys. & Chemical Cleaning	1.5/month	30
Pulp & Paper	**3**	20	0.45	1.0	Phys. & Chemical Cleaning	1/2-3weeks	35
	5	24	0.6	1.75	Stop Perm. Chemical Cleaning	12s/5min 1/1month	48
Night Soil	**1**	30	1.2	0.9	Backwash by Perm. by Gas	10s/2min. 150s/12hr.	30

PERFORMANCES OF VARIOUS UNITS AND MODULES

Performance data of membrane modules are summarized in **Table 7**. In this table, power consumptions per m^3 permeate were calculated from pressure losses and flow rate, assuming that pump efficiency is 100%. These figures should be checked carefully with the target values listed in **Table 2**, but it is possible to say that these target values are almost cleared by the demonstration data.

Table 7 Performances of Membrane Modules Tested

Types of Waste Water	Feed Water SS (mg/l)	Feed Water Visc. (cP)	Feed Water COD (mg/l)	Permeate COD (mg/l)	Permeate Flux (1)	Press. Loss (2)	Power Con. (3)	Module No.
Large-Scale Sewage	483	1.0	1,144	70	0.38	0.2	0.37	1
Small-Scale Sewage	1,370	1.0	2,187	34	0.58	0.29	0.38	2
Fat/Oil and Protein	4,283	1.12	9,677	863	0.2	0.1	0.41	1
Wheat Starch	16,900	5.3	36,000	8,800	0.39	0.5	0.87	2
	16,900	4.4	28,600	5,700	0.25	0.18	0.56	4
Alcohol Fermentation	10,000	5	18,000	11,000	0.33	0.75	2.4	3
Pulp & Paper	15,000	6	28,000	1,100	0.3	0.4	0.72	3
	12,100	4.2	18,500	1,420	0.64	0.075	0.91	5
Night Soil	2,600	6	25,500	2,000	0.16	0.3	1.58	1

(1):(m^3/m^2 day)
(2):(kg/cm^2/module)
(3):(kWh/m^3 permeate) calculated value assuming pump efficiency is 100 %.

CONCLUSIONS

Many schemes of water reclamation from various kinds of waste water and sewage by means of various membrane technologies have been tried throughout the world. The most important problem, which should be taken into account, is the scheme of pretreatment. Excessive use of pretreatment leads to high-cost performance, while simple treatment leads to severe fouling of membranes and modules and low capacity of equipment. In this Aqua-Renaissance Project, membranes and modules are used without any pretreatment, and in this sense the project is new and bold. At the same time, the project aims for reducing the power consumption of the membrane modules, which has been limiting their applications to conventional waste water treatment. Fortunately, the results obtained so far and reported here show that the membranes and modules have been working very well and some of them are clearing the target.

ACKNOWLEDGMENT

All data reported here were supplied by the Aqua-Renaissance Research Association (ARRA), and the author is grateful for this cooperation. As mentioned in the text, data are too premature to be published at this time and the official report will be published by ARRA in the near future.

As a member of the Aqua-Renaissance Research Association, the author was commissioned to conduct this investigation by the New Energy and Industrial Technology Department Organization.

REFERENCE

1) Imasaka, T.,N.Kanekuni,H.Wajima & S.Yoshino,
KAGAKU KOGAKU RONBUNSHU 15 299-305,306-312,633-644(1989).

TREATMENT OF MUNICIPAL WASTE WATER IN THE FUTURE

W. van Starkenburg[*], G.B.J. Rijs[*] and P.C. Stamperius[**]
[*] Institute for Inland Water Management and Waste Water Treatment
[**] Foundation for Applied Waste Water Research

TREATMENT OF MUNICIPAL WASTE WATER IN THE FUTURE

W. van Starkenburg*, G.B.J. Rijs*, and P.C. Stamperius**
* Institute for Inland Water Management and Waste Water Treatment
** Foundation for Applied Waste Water Research

Waste water treatment in the Netherlands

Waste water treatment in the Netherlands started over a hundred years ago. Towards the end of the 19th century, it became evident that the natural regeneration capacity of surface water was overstretched. Surface water, receiving at that time all waste water discharges untreated, turned into malodorous, dead water.
At first, this effect was counteracted by dispersing receiving water through flushing or by moving discharge points to larger expanses of surface water. It was not until the early 20th century that the Netherlands saw its first biological sewage treatment plants (RWZI's). These were fairly simple constructions. Voorburg RWZI, for example, built in 1906 to serve a population of 8000, consisted of septic tanks and trickling filters (figure 1). Main object of the treatment was the abatement of the odour nuisance accompanying waste water release.

Figure 1 Voorburg sewage treatment plant; trickling filters (1906).

Up to the early 1970s, this was as far as sewage treatment went, even though as early as the immediate postwar years, rapid population growth and an explosive increase in industrial activity charged surface waters with intolerable loads of oxygen- consuming substances. The 1970 Pollution of Surface Waters Act (WVO) marked a turning point. The act aimed at the eventual treatment of all waste water, mainly by means of collection via municipal sewerage systems and central treatment in RWZI's. The result so far is that just over 90% of municipal wastewater is collected via sewerage systems and more than 85% treated biologically. At the beginning of 1991, about 470 RWZI's were in operation, with a total treatment capacity of almost 24 million population equivalents (p.e.).

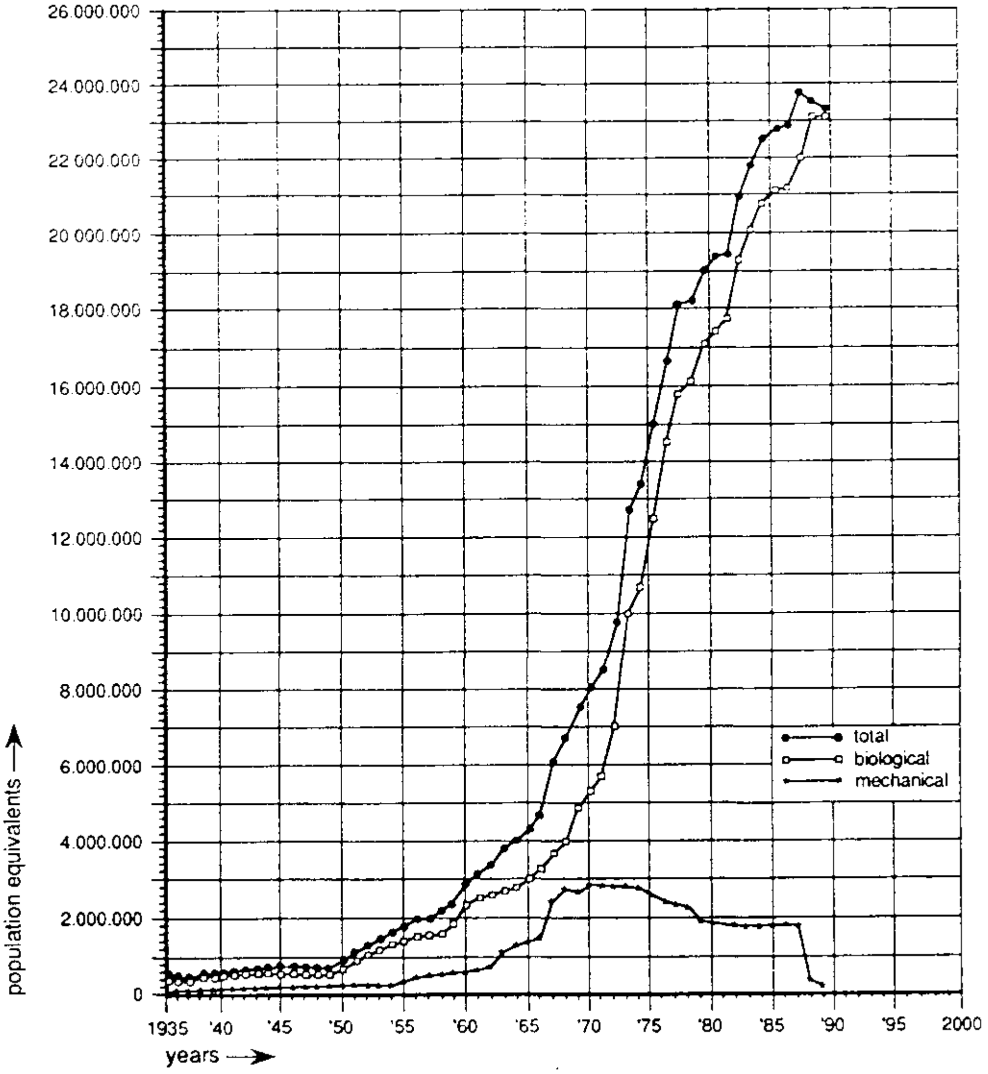


Figure 2 Current estimated capacity of RWZI's in the Netherlands.

Problems

Costs

The annual costs of water quality management increased gradually to one miljard guilders, the larger part of it merging into investment and energy costs. This has resulted in a more critical and scientific look at the minimising of the costs of treatment. Not only new construction, but also renovation and partial replacement of existing plants offer opportunities to apply entirely new techniques which might be more cost-effective and also be able to solve other problems, as environmental nuisance, stricter effluent requirements, and sludge disposal.

Environmental nuisance

Until recently, a fair distance from residential areas was one of the main requirements for new RWZI sites. However, urban expansion has pushed built-up areas out to the doorstep of many a once-remote RWZI. As a result, while the capacity of RWZI's to expand has been virtually eliminated, environmental requirements have become stricter. Future RWZI's, physically more compact with protective roofing, will make noise and odour easier to control.

Effluent requirements

Effluent quality is one of the areas where regulations have been gradually tightened up over the years. Following considerable reductions in the load of oxygen-consuming organic pollutants on surface water over the past two decades, government and water-authorities now concentrate more on drastically reducing the quantities of nutrients - phosphate (P) and nitrogen (N) compounds - discharged.

A large number of techniques for phosphate-removal can be applied, either apart or combined. They include chemical precipitation, crystallisation in a pellet reactor, and the biological and magnetic removal of phosphate. Ways of combining P-removal techniques both with one another and with biological N-removal techniques involving nitrification and denitrification are currently topics of studies.

Effluent quality can be improved in the early future by reducing the amounts of organic micropollutants, heavy metals, and suspended solids. Other effluent components - such as pathogenic organisms, sulphate, and substances that are either not readily biodegradable or highly soluble - may be tackled later.

Sludge disposal

Municipal RWZI's together produce 280,000 tonnes of dry solids per year, not including sludge produced by chemical phosphate-removal. Figure 3 presents an overview of sewage sludge disposal in 1983, 1986, and 1988.

In 1988, about half of all sewage sludge was used in agriculture and processed into fertilizer soil. Quality requirements for reusable sludge are to be tightened further in the years to come. This, and the decreasing possibilities of agricultural disposal because of the gigantic current surplus of animal manure, will shift the focus of attention towards better processing methods and treatment processes producing less sludge.

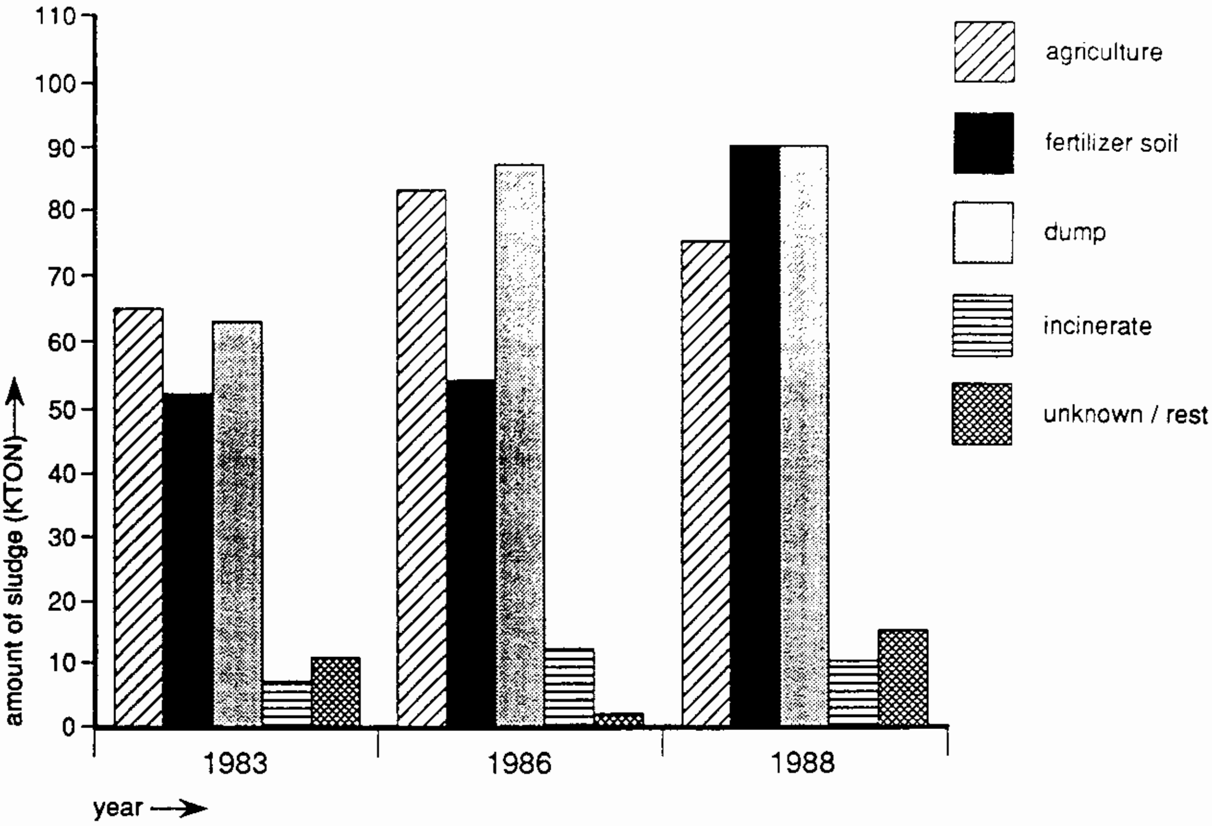


Figure 3 Disposal of sewage sludge in 1983, 1986 and 1988.

Neither these problems nor those likely to develop in the future can be solved using today's technology only. New needs will demand new techniques for the next generation of Netherlands sewage treatment plants.

Feasibility study

In order to draw up a balanced programme of research into new techniques, a feasibility study was conducted to find out which existing techniques could be applied in the Netherlands and how they might contribute in solving the problems mentioned above. The tackling of the feasibility study is explained in chart form in figure 4.
An inventory was made of all the techniques potentially applicable in the treatment of wastewater, varying from vague ideas to concrete measures. Out of these techniques, about 90 were deemed suitable for use in the Netherlands. This means that they could handle Dutch wastewater temperatures and the rain water/dry weather discharges of the sewerage systems and comply with Dutch legal requirements. About half this number was at a closer look eliminated because of their insufficient potential for further technological development. Factors taken into account were the state of the art of each technique, the simplicity of its integration in the treatment process, its

reliability, need for space and sludge handling.

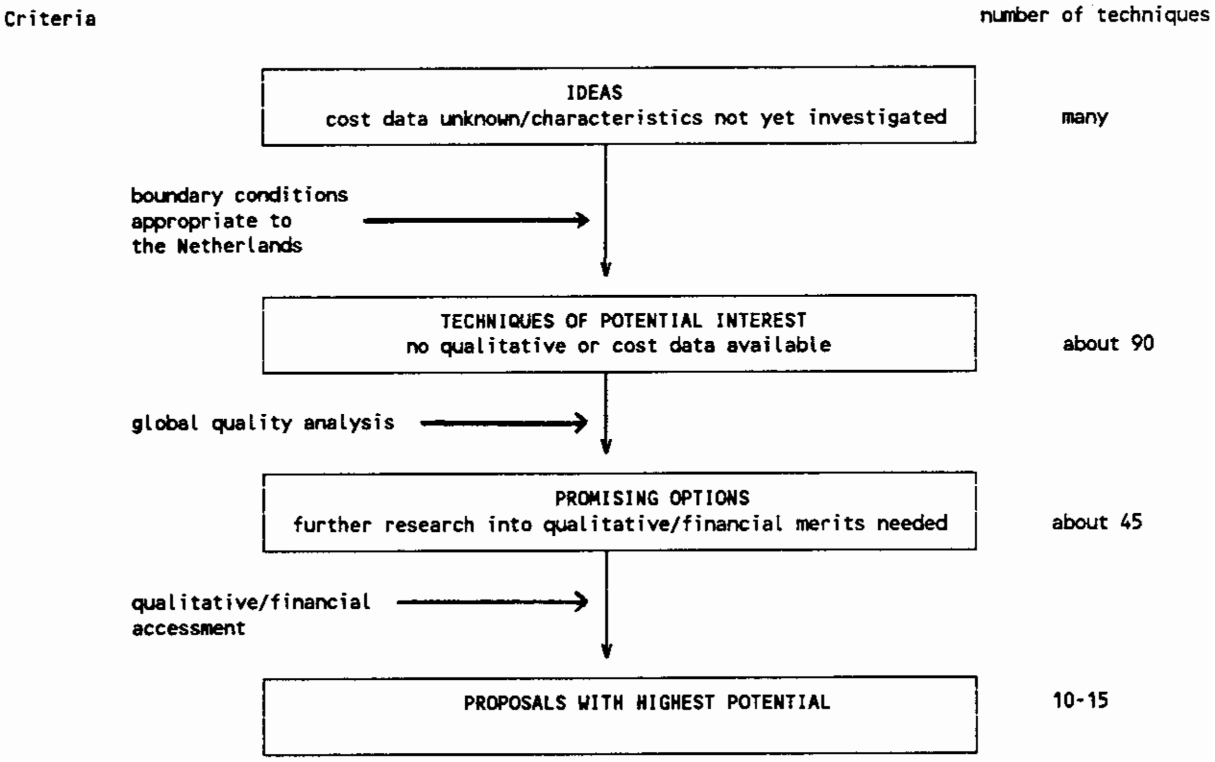


Figure 4 Strategy of the feasibility study.

A final list of 15 potential techniques deserving further consideration was arrived at by means of an analysis of environmental and cost criteria. It was also concluded that savings of between 10 and 20 percent were possible on municipal wastewater treatment costs. A research programme was drawn up for the 15 finally selected techniques.

RWZI 2000 research programme

The research programme entitled "Future Treatment Techniques for Municipal Waste Water", or "RWZI 2000" for short, was set up to develop technological solutions to most of the problems outlined above and is carried out now in a joint effort of the Institute of Inland Water Management and Waste Water Treatment (RIZA) and the Foundation for Applied Waste Water Research (STORA).

The programme was launched in 1988, with a 10-million-guilders budget. It has until 1993 to attain two main objectives:
- the development of more efficient treatment techniques for wastewater and sludge;
- the development of techniques which match current techniques for efficiency and environmental effects but which are cheaper.

Such techniques have to meet four essential criteria:
- treatment costs have to be limited;
- environmental impact, such as odour and noise nuisance, have to be reduced;
- effluent quality must be improved;
- the sludge processing component must have the potential for future further development.

The programme distinguishes three types of research, which vary according to the state of the art of the treatment technique under investigation:
1 - long-term fundamental research based on ideas and principles that are different from the old tried-and-trusted ones;
2 - practical research based on successful applications abroad or knowledge from other fields such as industrial wastewater treatment; this type of research can be carried out in either pilot plants or full-scale plants;
3 - evaluation study: techniques from both home and abroad are examined theoretically for their suitability in specifically Dutch conditions with a view to wider-scale application.

The research programme consists of a number of studies, divided into two main categories: waste water treatment and sludge managenment studies. There is an important difference between techniques applied at the two major stages of waste water treatment: water purification and sludge processing. Table 1 presents an overview of these studies, categorised according to the stage at which they take place.

Table 1 Classification of RWZI 2000 studies.

	1 Fundamental research	**2 Practical research**	**3 Evaluation study**
WATER	- reduced sludge production through loss of energy or high maintenance energy - high performance reactors - biological P-removal - simultaneously heterotrophic nitrification and aerobic denitrification	- sludge-on-carrier system - "Linpor" process - "Multireactor" system - automatic measurement and adjustment of sludge content - biological P-removal, combined with chemical precipitation, crystallisation in a pellet reactor, and magnetic separation	- "Deep Shaft" system - "AB" system
SLUDGE	- sludge/water separation	- thermophilic sludge digestion - "SSP"system	- "Carver Greenfield"-system - "Vertech" process - hydrolysis of sludge using ammonia - heavy metals removal from sludge using electrolysis

It is noteworthy that international agreements influenced strongly the choice of the studies and their character. This is particularly evident from the large number of studies concerned with P- and N-removal. It is also noteworthy that next to the RWZI 2000 studies on P- and N-removal, other application-oriented research is being carried out as part of STORA's PNs 1992 research programme. Elsewhere, demonstration projects are carried out with pellet reactors and magnetic phosphate removal apparatus.

Waste water treatment

Reduction of sludge production

The basic research is focused on reducing sludge production. In principle, this can be achieved in several ways.

The development of treatment systems using types of bacteria specifically selected for their low biomass reproduction and maintenance energy requirements leads to lower sludge production. In existing sewage treatment plants, bacteria with all sorts of characteristics are selected at random.

In another possible strategy superfluous biomass is to be consumed by organisms normally found in sewage treatment plants. It is well known that some flagellates, many ciliates, and rotifers will consume large amounts of suspended bacteria, whereas small oligochaeta such as Nais and Aelosoma will feed on sludge flakes. During the transformation of bacterial biomass into other biomass, energy - and thus biomass too - is dissipated. By actively managing the feeding behaviour of these organisms in a way that does not affect substrate degradation - rather than leaving it to chance - waste water engineers can reduce sludge production. Another possible benefit is that the presence of higher organisms helps the remaining sludge to settle.

Alongside these above-mentioned lines, RWZI 2000 also includes a feasibility study into the possible application of high performance reactors for the treatment of municipal waste water. This idea originates from biochemistry. In certain biochemical processes, it turns out to be possible to process very large amounts of organic matter in very small reactors with high bacterial matter content.

In existing municipal wastewater treatment plants, activated biomass per cubic metre of aeration volume is significantly lower than in such high performance reactors. By using membranous filters rather than the existing secondary sedimentation tanks, it is possible to work with higher concentrations of biomass in the aeration area. Containing the biomass more efficiently in the treatment plant also offers the possibility to work with sludge that has been in the aeration area for a longer time, leading to lower sludge production.

Water treatment techniques

Another treatment system offering the advantage of lower sludge production is the three-stage airlift reactor. This system has been applied for several years in the anaerobic and aerobic treatment of industrial wastewater. The treatment process is similar to that of conventional activated sludge systems, but distinguishes itself by the attachment of biomass to an inert carrier material: sand. Figure 5 shows an outline diagram of the three-stage airlift reactor.

Figure 5 Principle of a three-stage airlift reactor; biofilm on a carrier.

The reactor stands vertically and consists roughly of two concentric tubes and a secondary sedimentation tank. The waste water to be treated, whether pre-settled or not, first flows through the outer tube from top to bottom. Oxygen, in the form of air bubbles, is introduced into the bottom of the inner tube. There is an upward stream of air, water and carrier covered with biofilm. Directly above the aeration area, separation takes place in stages. The air that has been introduced escapes, while the carrier material settles. The settled carrier material is transported with the influent via the outer tube back to the aeration area. The effluent flows to the secondary sedimentation tank and catches any suspended material.
The pilot plant study with 25-litre reactors showed promising results. As had been the case with industrial waste water, the formation of stable biofilms proved possible with municipal waste water, despite the turbulent conditions in the airlift reactor. Dissolved components are removed very efficiently; with a hydraulic retention time of 1.5 - 2 hours, there is a good chance that complete nitrification will take place. However, very

little suspended solids are removed; this has to be separated either beforehand or afterwards. Surplus sludge production is similar to that in an activated sludge plant with a very low load.

As well as sand, other types of material can be used as carriers. In the Linpor process, small cube-shaped polyurethane pads (2.5 cubic centimetres, 97% porous) are used as carriers in order to increase the sludge content in the aeration area. Such pads can also be used in a separate tank for effluent nitrification, since there slowly-growing micro-organisms such as nitrifiers are flushed out less easily owing to their firmer consistency.

Results from pilotplant research shows that the pads enhance the stability of the process. The rate of chemical oxygen demand removed (COD) improves also, especially as a result of an improvement in the removal of suspended solids. Furthermore, the sponged sludge makes a noticably greater contribution to the nitrification of suspended sludge.

It can be expected that the Linpor process can be used in specific circumstances. For instance, it can reduce the sludge load in overloaded activated sludge systems and improve nitrification. However, its relatively high cost constitutes a considerable impediment.

In another study the Multireactor system for treating municipal waste water is investigated. This system consists of a 20-metre-deep underground shaft for the aerobic degradation of organic substances, and above ground, a flotation unit for water-sludge separation. Figure 6 shows an outline diagram of the system. A mixture of wastewater and returned sludge flows countercurrent with the air bubbles in the outer tube to the bottom of the shaft. In contrast with the sludge-on-carrier system, the sludge here is in floc form.

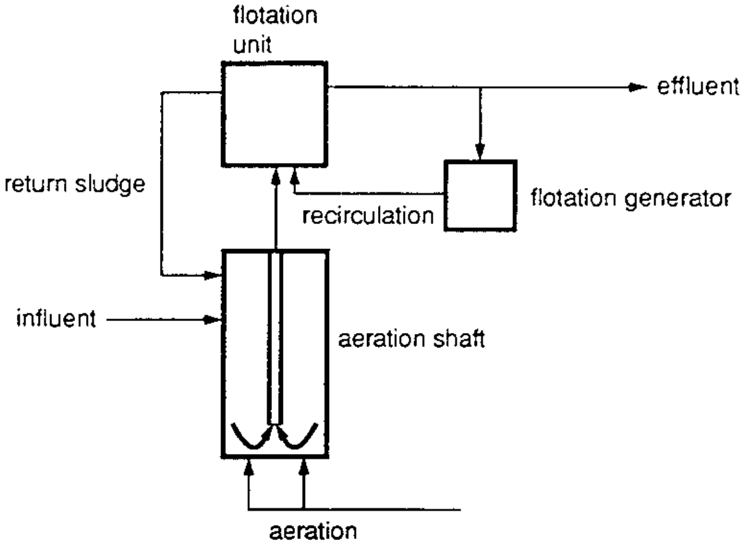


Figure 6 Principle of the Multireactor system.

Because of the high hydrostatic pressure in the liquid column (above), more oxygen will dissolve at the bottom than in conventional activated sludge systems. The aerated mixture of sludge and water flows upwards through the inner tube and is separated in the flotation unit into effluent and returned sludge. The flotation process is succoured by the pressure drop as the mixture rises. As a result, the dissolved air escapes in small bubbles which will attach to the sludge flocs.

A similar system is the Deep Shaft system. The most noticeable differences are the deeper shaft (50 to 100 metres), the alternation of the downward and upward flow via the inner and outer tube respectively, and the conventional sludge-water separation in a secondary sedimentation tank. On the basis of practical experience abroad, a study is evaluating the possible benefits and disadvantages of using the Deep Shaft system in the Netherlands.

A evaluation has taken place of the adsorption-activated-sludge process (AB-system). This is a two-stage activated sludge system with a high-load first stage - "Adsorption" - and a low-load second stage - "Belebung". The main activity at the A stage is adsorption; the B stage mainly involves substrate degradation and further mineralisation.
Typical features of the AB system are the absence of presettling, the high sludge load at the A stage - at least 2 kg of biochemical oxygen demand (BOD) per kg of dry solids per day (d.s.d.) - and the strict separation of activated sludge flows into A and B stages.
None of the three Dutch wastewater treatment plants studied comply with the minimum sludge load for the AB system, and for this reason do not benefit from the system's specific advantages, such as lower space and energy requirements.
It cannot be expected that the AB system will be applied on a large scale in the Netherlands, due to the high RWA/DWA (rain/dry weather supply) ratios and unfavourable BOD/N ratio, as well as the thorough N-removal it involves. The advantages of reduced space and energy requirements would be minimal.

Two subjects that also receive substantial attention in the RWZI 2000 research programme are automation and the removal of nutrients.

Automation
In sewage treatment plants, effective control of the sludge content in the aeration area is essential. Automatic control of this and of surplus activated sludge improves the stability of the process, helps to save energy, and makes its operation easier. This in turn leads to savings in operating and analysis costs.

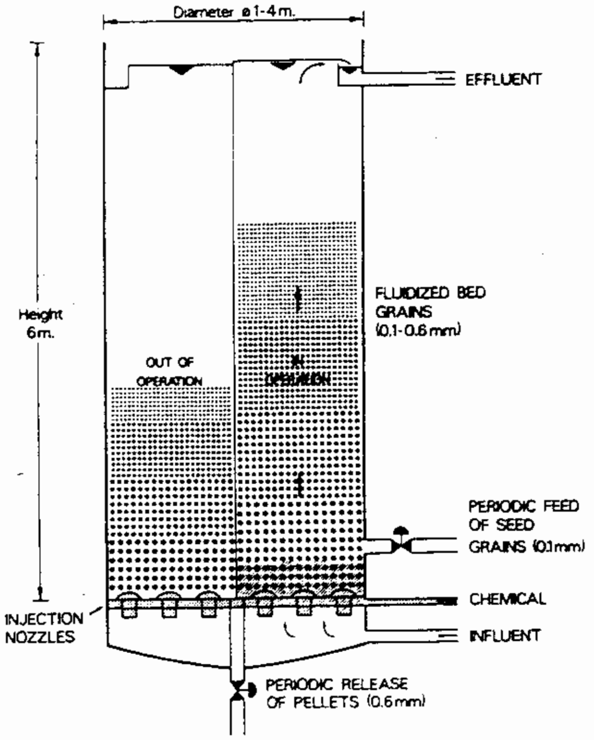
The study of automation evaluated the scope for automatic measurement and adjustment, using sludge concentration meters commercially available. These meters are based on optical principles and are particularly sensitive to pollution. The most promising meter was tested under practical conditions. However, it did not lead to discernible improvement in the treatment process nor to any savings in energy costs. Besides this, the research programme looks for the possibility of developing new sludge concentration meters, less sensitive to pollutants.

Nutrient removal

Simultaneous removal of the nutrients phosphate and nitrogen constitutes an important part of the RWZI 2000 research programme.

The state of the art of all current P-removal techniques - some already operational, others under development - has been compiled to obtain an accurate scheme of all possible problems/difficulties at the implementation of P-removal techniques in practice. The techniques can be divided in following categories:
- chemical precipitation;
- techniques in development, as crystallisation in a pellet reactor, magnetic separation and biological P-removal.

Chemical P-removal is simple and reliable, but it increases the salt load in effluent and produces phosphate sludge that cannot be reused.
Crystallisation in a pellet reactor and magnetic separation do not have these disadvantages, but they are both sensitive to policy developments on sludge disposal, effluent requirements, and reduced phosphate ratios in influent as a result of the introduction of phosphate-free detergents.



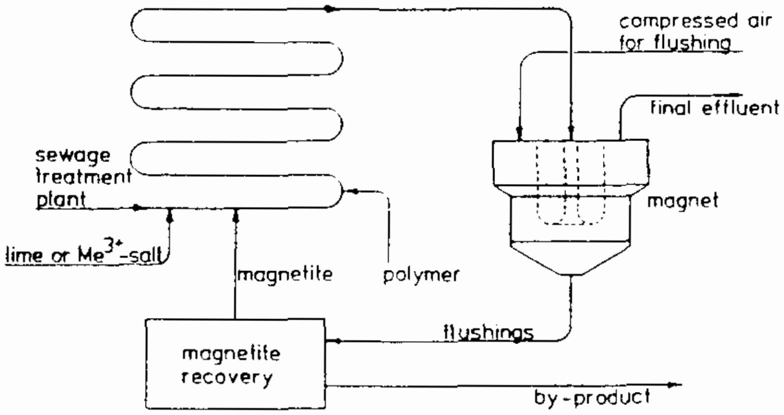


Figure 7 Phosphate removal by a pellet reactor and magnetic separation.

The most promising technique would seem to be a combination of biological P-removal with one of the three above.
Biological P-removal is based on the ability of certain micro-organisms (acinetobacter) to absorb more phosphate from the sewage water than is necessary for cell growth and to release it under anaerobic circumstances. Phosphate is discharged from saturated sludge in a so-called stripper tank. The concentrated flow of phosphate thus released can be further treated by chemical precipitation, by a pellet reactor or magnetic separation. The RWZI 2000 research programme investigates all three methodes on full scale, with special attention to the interaction between biological P-removal and denitrification.

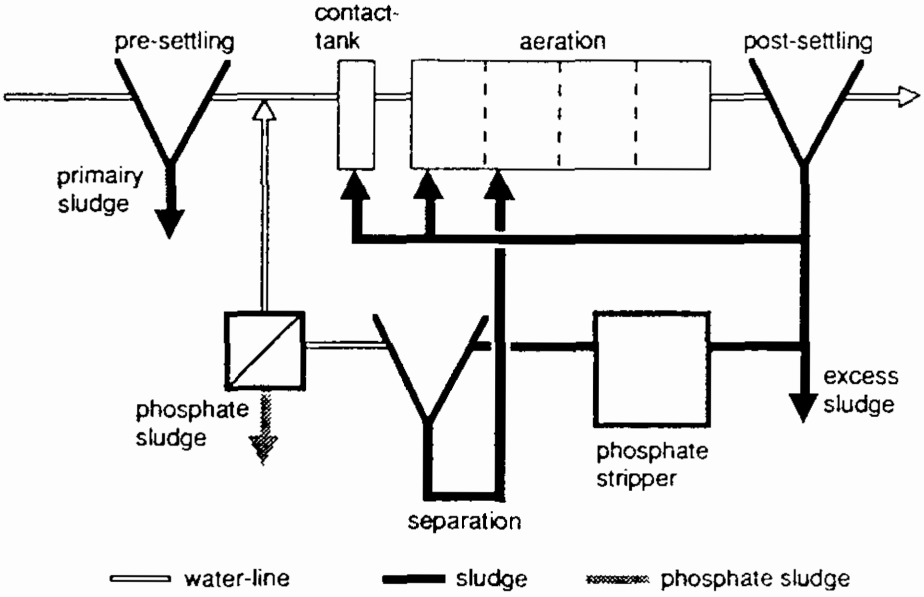


Figure 8 An outline of the principles of biological P-removal.

Biological P-removal is subject to more fundamental research, aimed at filling up missing knowledge in bacteriology and combining knowledge from microbiology, biological processing, and water treatment technology to achieve a well-founded design model for biological P-removal. Figure 8 shows an outline of the principles of biological P-removal.

The present results of biological N-removal in sewage treatment plants do not generally meet expected future standards. The low rates of growth of the nitrifying micro-organisms, their low competitiveness, and their sensitivity to extreme conditions are all responsible for this. The immobilisation and concentration of nitrifying organisms may improve results.

The processing of sewage sludge

The Netherlands will have in future a sharply diminishing capacity for the disposal of sludge - dewatered or not - whether directly in agriculture, or in the manufacture of fertilizer soil, or in waste dumps. The reprocessing of sewage sludge is therefore bound to account for an increasing share of the costs of waste water treatment in the future. Little fundamental is known about sludge-water binding and the improvement of sludge dewatering. A remarkably small amount of basic research has been conducted in this problem.
RWZI 2000 is spending about 25% of its research budget on this problem. The main objective of this basic research on sludge-water separation is to achieve that the improvement in knowledge and understanding of the dewatering of sewage sludge can be develope or optimise techniques to raise its dry solids content and reduce its volume or mass considerably.
The following two studies are being conducted: one to identify the main properties of sewage sludge, the other to create a physical-mathematical model of solid-liquid separation in compressible media.
The properties of sludge to be identified are those of relevance to the dewatering process: its composition, colloid-chemical variables, particle properties, sludge-water binding, and the time dependence of these properties.
The setting up of a physical-mathematical model involves a number of factors including filter cake properties and model parameters such as specific filtration resistance, the compressibility of sludge cake, capillary suction time (CST), and the modified filtration test (MFT).

An examples of practical research into the suitability of integrating sludge processing techniques in the Netherlands is thermophilic sludge digestion.

Thermophilic sludge digestion (at a temperature of 54°C) has been studied by means of comparative semi-technical research into mesophilic digestion (at 33°C). The main object was to get a better understanding of fermentation rates, methane production,

the degradation rates of organic material, the reduction of pathogenic bacteria, the dewaterability and the suitability for further processing of digested sludge. In addition, attention has been paid to process stability, especially the sensitivity of processes to toxic substances and temperature fluctuations.
It can be concluded that the initially mooted advantages of thermophilic sludge digestion over its mesophilic counterpart - such as cost-effectiveness and easier dewatering - cannot be confirmed. It is true that thermophilic sludge digestion can produce 10 to 25 percent more gas and kill many more pathogenic micro-organisms. However, regarding to this research the economic feasibility of thermophilic sludge digestion is largely hampered by the poor dewaterability of digested sludge.
In addition, study results indicate that the time needed for normal mesophilic sludge digestion can be reduced quite easily from between 20 and 30 days to 10 days for sludge contents of about 2.5 kg of organic dry solids per cubic metre per day. The results show that the decay of organic materials is sufficient; a disadvantage is the poor dewaterability.

RWZI 2000 evaluation studies into sludge processing - some exploratory - have investigated the following techniques:
- the Carver-Greenfield drying process;
- the Vertech wet air oxidation process;
- the hydrolysis of sludge using ammonia, followed by ammonia recovery and anaerobic digestion of the hydrolysate;
- the removal of heavy metals from sewage sludge using electrolysis.

The Carver-Greenfield drying process is based on the principle of multi-stage vacuum evaporation. A carrier liquid, which is not soluble in water and with a high boiling pont (usually oil), is added to the sludge to be dried. The sludge mass remains liquid throughout evaporation because of the carrier. Once the water has been evaporated, the carrier liquid is separated from the dried sludge in order to be reused.
The evaluation study focused on the principle and its implementation, its current capacity for drying sewage sludge, energy and environmental aspects, the scope for its integration into existing wastewater treatment plants, and the process costs with an accompanying sensitivity analysis.
The assumptions are that the Carver-Greenfield system should be located in an existing wastewater treatment plant of 200,000 p.e.; dewatered sludge (20% dry solids) coming from 500,000 p.e., is treated five days a week, 24 hours a day; the dried sludge (95% dry solids) is to be dumped and the polluted condensate is to be taken back to the RWZI.
The study showed that the Carver-Greenfield process can be integrated into an existing RWZI of sufficient capacity. The condensate is not heavily polluted and can be released at the RWZI without pretreatment. In the considered situation, this means an extra chemical oxygen demand (COD) and N-Kj load of one and six percent respectively. The autothermal drying of sludge is possible if the sludge's oil and fat contents are high (10% or more on a dry solids basis). Sludge oil is then separated

and used as an energy source.
More than 60% of the annual costs are spent on capital investment. For this reason, the process needs to be operated on a fully continuous basis, and increases in process capacity can yield large reductions in costs.

A second proces is the Vertech wet air oxidation process. An evaluation of environmental and cost criteria has been conducted for this process, using the above mentioned assumptions and an initial dry solids content of 5 %.
In the Vertech system, wet air oxidation takes place in a vertical tube reactor (depth of 1200 m) under high pressure and high temperature. The weight of the gas-liquid mixture in the tube leads to high pressure (about 80 bars) at the bottom; a high temperature (260-300°C) is obtained by the reaction heat released.
By means of the wet oxidation of fermented sewage sludge, Vertech expects a COD reduction of 70 to 75 percent and a 90% degradation rate for suspended organic matter. The decantate - ashfree effluent - should be pretreated before it is carried back to the RWZI. If no pretreatment takes place COD and N-Kj loads will jump by 20 and 45 percent respectively.
A significant share (55%) of the Vertech system's annual costs are spent on capital investment. The initial dry solids content and the price of the liquid oxygen have little effect on process costs.
A Vertech plant will be built in the Netherlands, with a maximum oxidation capacity of 13,400 tonnes of COD per year and a capacity of 22,800 tonnes of dry solids per year.

In conclusion

This article has attempted to shed some light on the RWZI 2000 research programme. A start has been made on all the separate studies. The intention is to present the (intermediate) results of the programme at an international congress during Aquatech 1992 in Amsterdam.

The future tightening-up of emission regulations will require new efforts in the technical field. Experience has shown that each specific group of substances requires its own specific approach. However, a more integrated approach, concentrating on the removal of several pollutants and focusing on future effluent quality targets, will be desirable in the future. This approach can reduce the probable risk that the benefits of any innovation may cancel out benefits of previous ones, if the adaptations to RWZI's follow one another over the years. Besides this, it will be probable cost-effective.

A last general remark on the sett-up of the RWZI 2000 programme is that it seems a very useful concept for more countries than the Netherlands alone.

References

1. "The future treatment of municipal waste water. A feasibility study I. Final report II. Draft report".
DBW/RIZA, Netherlands Organisation for Applied Scientific Research (TNO), and Witteveen and Bos engineering consultancy (July 1986)

2. "Future treatment techniques for municipal waste water. Research programme".
DBW/RIZA, STORA (January 1988)

BIOLOGICAL NITROGEN REMOVAL BY STEP-FEED PROCES

I. Miyaji, M. Iwasaki and Y. Sekigawa
Kurita Central Laboratories, Kurita Water Industries Ltd.

BIOLOGICAL NITROGEN REMOVAL BY STEP-FEED PROCESS

Y. Miyaji, M. Iwasaki and Y. Sekigawa

Kurita Central Laboratories, Kurita Water Industries Ltd.,
1723 Bukko-cho, Hodogaya-ku, Yokohama, Japan

ABSTRACT

To reduce chemicals and power consumption which constitute major operational costs in biological nitrogen removal, a new multi-stage "step-feed process" was evaluated, in which denitrification and nitrification compartments are alternately arranged in series and wastewater is fed, equally divided, into each of the denitrification compartments. Nitrogen removal efficiency (η) for the conventional one-stage (denitrification tank followed by nitrification tank) recirculation process can generally be expressed as $\eta = 1 - 1/(1 + r + r_n)$, where r = recycling ratio, r_n = return sludge ratio, and intense recycling of nitrification tank mixed liquor to the denitrification tank is needed to achieve a high degree of nitrogen removal. In the present multi-stage step-feed process, the nitrogen removal efficiency can be described theoretically as $\eta = 1 - 1/n(1 + r + r_n)$, where n = number of stages, and the residual nitrogen can be reduced to 1/n as compared to the one-stage recirculation process. The recycling ratio can be greatly reduced, and therefore the power cost is lowered extensively. A night-soil treatment plant by the step-feed process has achieved more than 90% removal of nitrogen without using methanol and NaOH. The operating data have proved the above theoretical considerations true. This process is also applicable to municipal wastewater treatment where the amount of recirculation has a significant meaning in the power cost.

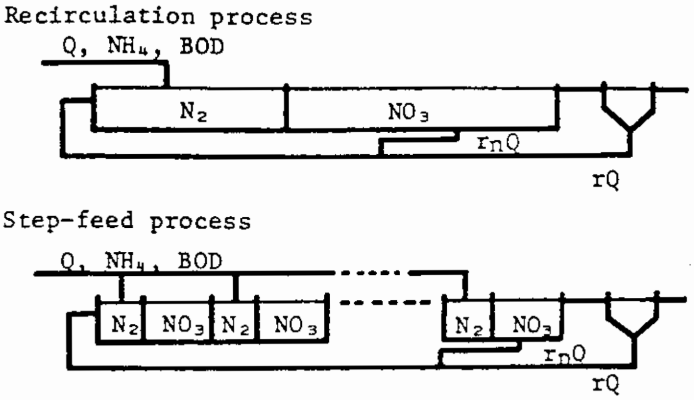
INTRODUCTION

Microorganisms involved in biological nitrification-denitrification reactions include nitrifying bacteria, denitrifying bacteria and BOD-decomposing bacteria. The so-called single sludge system 1), 2), where these bacteria are utilized, mixed in a one unit process, has been appreciated to be most economical in running cost 3). The principal running costs of biological nitrogen removal by nitrification-denitrification reactions are concerned with alkali required to neutralize acid formed in nitrification reaction, carbon sources for denitrification reaction, and aeration power for oxygen supply for BOD decomposition and nitrification reaction. In the single sludge system proposed by Barnard 1) and Toya 2), carbon sources contained in wastewater is utilized in the denitrification tank while alkali formed there is effectively used in the nitrification tank by forced recirculation of a part of the mixed liquor in the nitrification tank (hereafter we call this process the recirculation process).

However, since the amount of nitrogen to be removed in the denitrification tank is nearly equivalent to that of the recycled nitrogen in the mixed liquor, massive recirculation is needed to achieve a high degree of nitrogen removal and therefore the cost for pumping power for recycling the mixed liquor may be considerable for a large scale wastewater treatment plant.

With this in minds we have evaluated a power-saving "step-feed process", a modification of the recirculation process in which nitrification and denitrification tanks in the recirculation process are equally divided into several compartments, and the denitrification and the nitrification compartments are arranged alternately in series, with wastewater being fed, equally divided, to each of the denitrification compartments. By this method, recirculation of mixed liquor can be greatly reduced to achieve the same degree of nitrogen removal as the recirculation process does. Flow diagrams of the recirculation and the step-feed process are shown in Fig. 1.

This report discusses economic considerations and characteristics of the step-feed process as well as operational results of an actual night-soil treatment plant by this process.


Recirculation process

Step-feed process

N_2: denitrification tank
NO_3: nitrification tank

Fig. 1. Flow diagrams of recirculation process and step-feed process

BASIC CONSIDERATION OF STEP-FEED PROCESS

In the step-feed process shown in Fig. 1, wastewater is fed equally divided to each of the denitrification compartments. Wastewater flows by gravity through the nitrification and the denitrification compartments, successively. A part of mixed liquor of the final nitrification compartment (n-th tank), and the settling tank sludge is forced to return to the first denitrification compartment. Denitrification and nitrification reactions are repeated in each compartment. Therefore, after the first denitrification compartment, a complete removal of nitrogen components may be expected except for the final stage, if BOD in the wastewater is in a sufficient ratio to nitrogen.

The nitrogen removal rate can be simply expressed by a material balance equation (1) as follows, by assuming 100% efficiency for nitrification and denitrification reactions in each compartment.

$$\eta = 1 - \frac{1}{n(1 + r + r_n)} = 1 - \frac{N_T}{No(1 - \eta_s)} \qquad (1)$$

where n = number of division (number of steps), r = return sludge ratio, r_n = mixed liquor recycling ratio, N_T = NH_4-N + NO_3-N in the treated water, η_s = ratio of nitrogen utilized for bacterial cell synthesis, No = total-N in the influent raw wastewater.

The relationship between the nitrogen removal rate and the recycling ratio is shown in Fig. 2. The nitrogen removal rate is greatly influenced by the number of steps (n) particularly in the lower recycling ratio. When the BOD/N ratio in wastewater critically limits the denitrification reaction or when a large number of steps may be permitted, the recirculation of mixed liquor may be omitted.

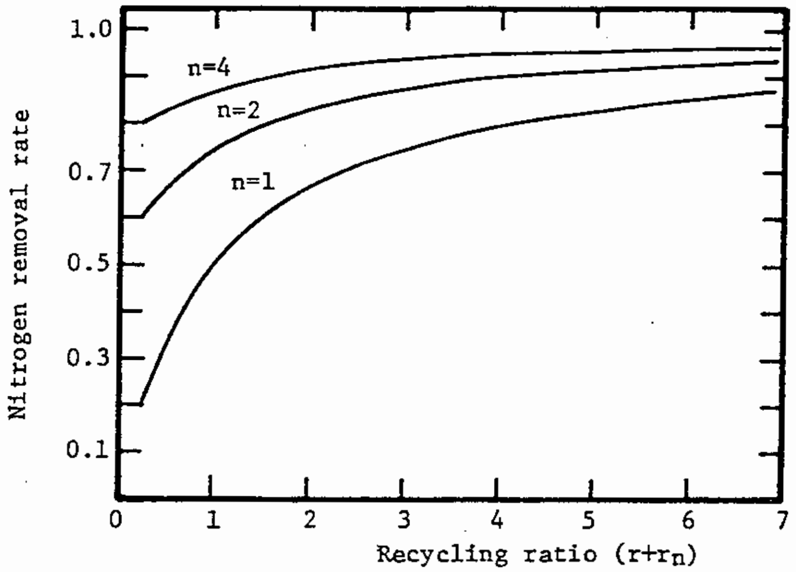


Fig. 2. Relationship between nitrogen removal rate and recycling ratio

A CASE STUDY FOR RUNNING COST EVALUATION

The running cost of nitrogen removal by biological nitrification-denitrification is mainly determined by the power cost for aeration (Pg) and that for the recirculation of return sludge and mixed liquor (Pl) as well as by chemicals such as sodium hydroxide and methanol as required.

A case study was made on the relationship among the number of steps (n), power consumption, and alkali requirement in the step-feed process as compared with the recirculation process. Conditions for the study are as shown in Table 1.

Blower and Pumping Power

Blower power is calculated from equation (2), where power efficiency is estimated at 60%.

$$Pg = 0.164\ Qg\ [(P/1.034)^{0.286} - 1] \qquad (2)$$

Amount of air required (Qg) can be expressed as follows.

$$Qg = C_1\ (BOD - BOD_d)q + C_2 \cdot NH_4\text{-}N \cdot q + C_3 \cdot MLSS \cdot V$$

TABLE 1. Conditions for Case Study

Flow rate Q = 50,000 m^3/d	
Influent quality (after primary settling)	
BOD	140 mg/l
Suspended solids	125 mg/l
NH_4-N	25 mg/l
Alkalinity	110 mg/l

where P = blower pressure, q = flow rate (m^3/hr), C_1 - C_3 = constants, V = volume of nitrification tank, BOD_d = BOD removed by denitrification reaction.

Pumping power required for mixed liquor recirculation and return sludge is calculated as follows;

$$\text{Pumping power (Pl)} = 3.88 \times 10^{-3} \times (r_n q + r_q) \cdot H \qquad (3)$$

where, power efficiency is estimated at 70%, $r_n q$ = amount of return sludge (m^3/d), r_q = amount of mixed liquor recirculation (m^3/d), H = hydraulic head (mH_2O), estimated at 6 meters.

The results of these calculations are summarized in Fig. 3. As can be seen in Fig. 3, power concumption decreases temporarily as the nitrogen removal rate increases. However, with a further increase in nitrogen removal rate, power consumption shows a steep increase at a certain point owing to an increased recycling ratio there. The temporary decline in power consumption is due to increased reduction of BOD at the denitrification stages, resulting in reduced aeration requirement. The tendency toward power requirement reduction is more pronounced as number of steps increases. As can be understood from equation (1), the required recirculation decreases with an increase in the number of steps for a definite nitrogen removal rate.

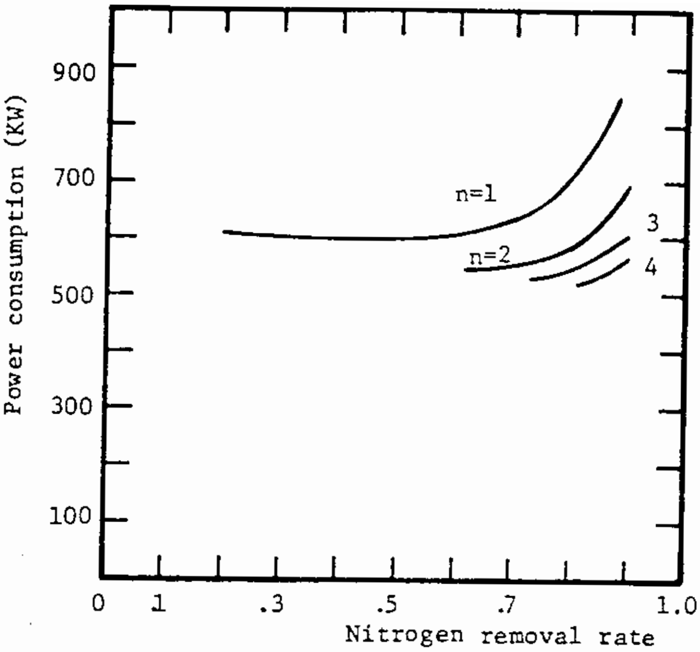


Fig. 3. Effect of number of steps on power consumption

Balance of Alkalinity

Stoichiometrically, 7.14 mg (as $CaCO_3$) of alkalinity is decomposed as 1 mg of NO_3-N is produced in the nitrification reaction, while 3.57 mg (as $CaCO_3$) of alkalinity is formed as 1 mg of NO_3-N is reduced to nitrogen gas in the denitrification reaction. However, slightly lower values are reported for them as actually obtained values [4]. An alkalinity balance can be represented as equation (4) by using the stoichimetric values.

$$\frac{QMn}{QNo} = \frac{1}{n}\left\{ Mo/No - 7.14 + 3.57\,\frac{r_n + r}{1 + r_n + r} \right\} + \frac{n-1}{n}\,(Mo/No - 3.57) \qquad (4)$$

where, Mo = M-alkalinity as $CaCO_3$ in influent, M_n = M-alkalinity as $CaCO_3$ in effluent.

When the effluent alkalinity Mn is negative, alkalinity adjustment in the first nitrification compartment is required. Figure 4 illustrates the relationship of equation (4), where it is clear that an increased number of steps decreases the amount of alkali required for neutralization.

From equation (1) and (4) is obtained equation (5) which indicates that the alkalinity requirement is proportional to nitrogen removal and hence to the number of steps.

$$\frac{QMn}{QNo} = 3.57 \times \eta + Mo/No - 7.14 \qquad (5)$$

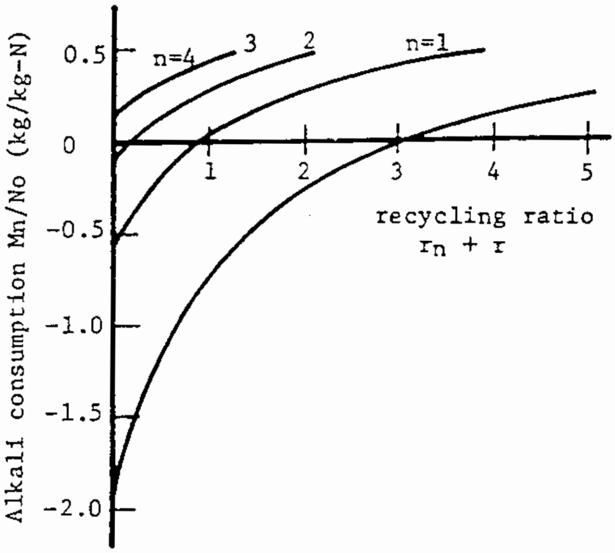


Fig. 4. Effect of recycling ratio on alkali consumption

PLANT PERFORMANCE

Six commercial plants by the step-feed process are in operation and some others are under construction for night-soil treatment. For small-scale sewage treatment, several plants of this process are also under construction. Operational results of an actual night-soil treatment plant by this process will be discussed below particularly with respect to its running costs.

The flow diagram and design criteria of this plant are shown in Fig. 5 and Table 2, respectively. In addition to biological treatment, this plant includes advanced treatment based on dissolved air flotation assisted by alum coagulation for phosphorus removal, filtration and ozonation for color removal and sterilization.

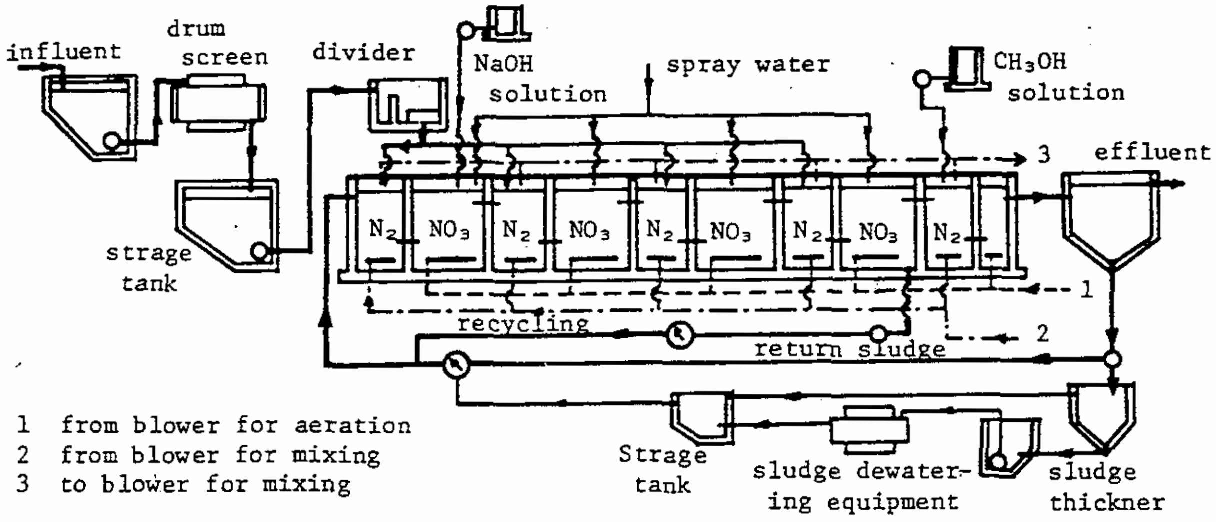


Fig. 5. Flow diagram of night-soil treatment plant

TABLE 2 Design Criteria

Projected wastewater quality	
BOD	12,000 mg/l
Suspended solids	18,000 mg/l
Total-N	4,500 mg/l
NH_4-N	3,500 mg/l
Projected flow rate	75 m³/day
Design Criteria	
Minimum water temperature	15°C
Mixed liquor suspended solids	5,000 mg/l
Sludge retention time	7.7 days
Specific nitrogen loading for nitrification	0.05 kg-N/kg-ss·d
Specific nitrogen loading for denitrification	0.06 kg-N/kg-ss·d
Volume of nitrification tank V_N	1,012 m³
Volume of denitrification tank V_D	934 m³
Blower for nitrification tank	Qg = 55 m³/min at 4,900 mmAq
Blower for denitrification for mixing	Qg = 11.5 m³/min at 5,000 mmAq

The total volumes of the nitrification and denitrification tanks were designed by calculation of the sludge retention time (SRT) in the nitrification tank, and by the denitrification reaction rate for the expected lowest water temperature, respectively. The tank for nitrification was divided equally into four compartments and that for denitrification into five equal rooms, and they were arranged alternately in series. In the 5th denitrification tank, methanol supply equipment was provided to ensure removal of remaining NO_2^- and NO_3^-. An automatic pH control system was provided in the 1st nitrification tank which was capable of adjusting pH by using NaOH solution.

Operational Results

The average influent wastewater quality over a year of operation and operating conditions are shown in Table 3 and 4. The actual flow rate was about 97% of the projected value and the nitrogen loading was about 76% of the projected. The sludge retention time was 20.7 days on an average, a high value compared to the projected value, and a very stable nitrification resulted throughout the year. A probability distribution of NH_4-N as well as of organic nitrogen is shown in Fig. 6. Concentration of NH_4-N in the effluent was 1.0 mg/l or below at 92% probability. Water sprayed for the purpose of suppressing foaming at the biological stages dilutes the inflowing waste 3.75 times (as measured by Cl^- concentration). Further, waters from sludge thickening, sludge dewatering and from other miscellaneous use, which enter the biological system dilute the waste up to 5.93 times in volume in actual operation. The effluent qualities listed in Table 5 represents results obtained under these operating conditions.

To maintain effluent nitrogen as low as possible CH_3OH was added at a rate 2 - 3 times the residual NO_3-N (on weight basis) in the 5th denitrification tank.

TABLE 3 Influent Wastewater Quality

Item	Average $\bar{x}$	Variance σ_{n-1}
pH	7.64	0.22
BOD	9,150	1,030
T-N	3,390	339
NH_4-N	2,760	230
T-P	375	81
Alkalinity	7,270	811
Cl^-	2,990	450
S.S.	8,440	2,990

(Unit in mg/l without pH)

Operational results during the winter season are shown in Fig. 7. The effluent NO_2-N and NO_3-N concentrations were lower than those expected by equation (1), owing to CH_3OH addition. The mixed liquor water qualities at the 4th nitrification tank before CH_3OH addition can be regarded as those expectable from equation (1) which represents a material balance in the step-feed process, and therefore those water qualities are used in the following analyses.

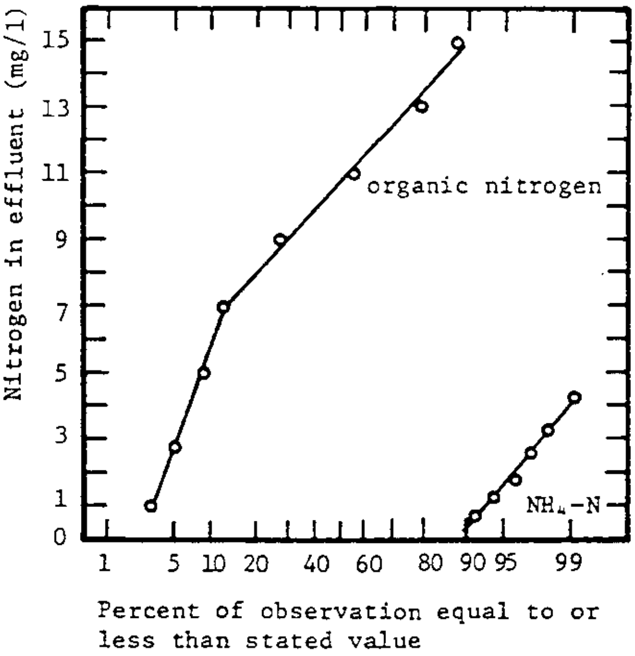


Fig. 6. Probability distribution for effluent nitrogen

Operational results obtained over a year by 18 runs at varying mixed liquor recycling ratios (r_n) and sludge return ratios (r) are listed in Table 5. Figure 8 summarizes relations between these data. Nitrogen used for cell synthesis can be obtained by the excess sludge production rate (4.18 kg/m³·d) shown in Table 4, and the nitrogen content in activated sludge measured previously (9.2% on an average). It is indicated that 11.3% of the influent total-N (3,390 mg/l) was used for cell synthesis on an average. The nitrogen removal rates shown in Table 5 were calculated from the following equation where a correction is made for nitrogen reduction due to cell synthesis.

TABLE 4 Operating Conditions

Item	Average $\overline{x}$	Variance σ_{n-1}
Flow rate (m³/d)	73.0	2.3
MLSS (mg/l)	5,180	584
Excess sludge production rate (kg-ss/m³·d)	4.18	1.76
SRT (day)	20.7	10.1
Specific loading for nitrification (kg-N/kg-ss)·d	0.038	0.008
Dilution rate		
as measured by Cl⁻	3.75	0.44
as measured by flow rate	5.93	0.96

$$\eta = 1 - \frac{(NO_2 + NO_3) - N \times \text{dilution rate}}{T - N \times (1 - 0.113)} \qquad (6)$$

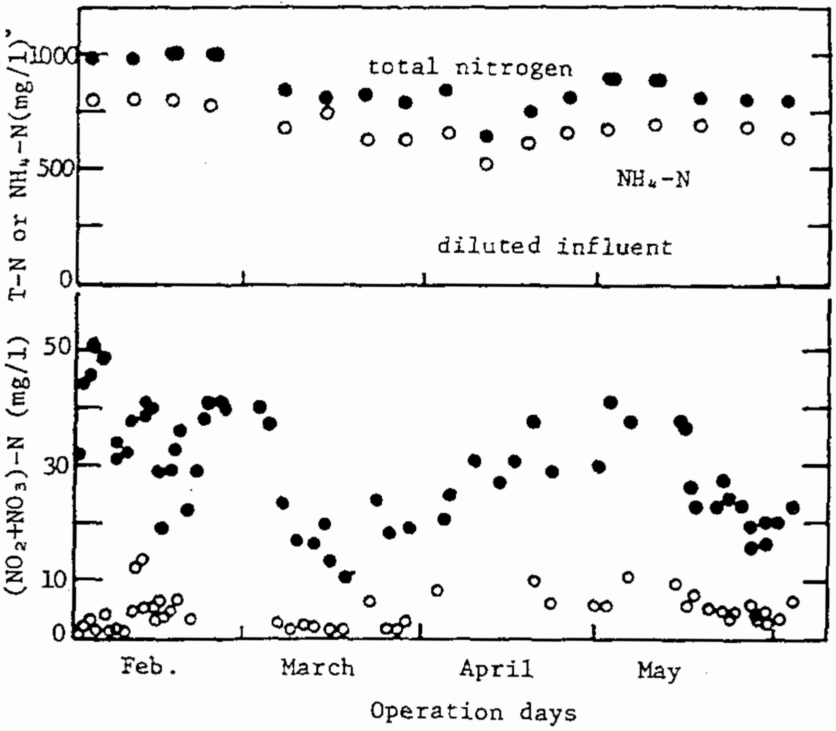


Fig. 7. Operational results during a winter season

● effluent before addition of methanol

○ effluent after addition of methanol

As can be seen in Fig. 8, the operational results agreed well with the calculated values from the material balance equation (1), giving about 93% removal of nitrogen without addition of CH_3OH. In Fig. 8 is also shown a curve obtained by the material balance equation for n = 1, for reference. The amount of NaOH dosage varied depending on the amount of the mixed liquor recirculation The test results are shown in Fig. 9. From the operational results the following equation describing a relationship between alkali consumption and the nitrogen removal rate can be obtained by using equation (5) with a correlation factor of 0.995

$$QM_n/QNo\ (1 - \eta s) = 3.44\eta - 3.438 \qquad (7)$$

A large part of the BOD components in night-soil is constituted of lower fatty acids, and NH_4^+ in the waste is rated to be converted to NH_4HCO_3 in the biological process [5]. In consideration of these facts, the coefficient obtained by the operational data will be deemed as extremely approximate to an estimation from theoretical reaction formulas. In the application of the step-feed process for night-soil treatment, more than 99% removal of nitrogen is needed if neutralizing NaOH is to be made zero.

Operating costs including NaOH for neutralization, CH_3OH for denitrification and pumping power for recirculation are closely related to a nitrogen removal rate to be attained, and an optimum combination of these factors was obtained for the plant. The cost of methanol dose is given as follows.

$$C_{CH_3OH} = @ \times No \times (1 - \eta_s) \times b \times Q\,(\eta - \eta_o)$$

where b is the amount of methanol needed for NO_3-N reduction and 2.5 is given for night-soil treatment [5], η_o denotes the basic removal rate of nitrogen in the step-feed process, attainable without addition of CH_3OH.

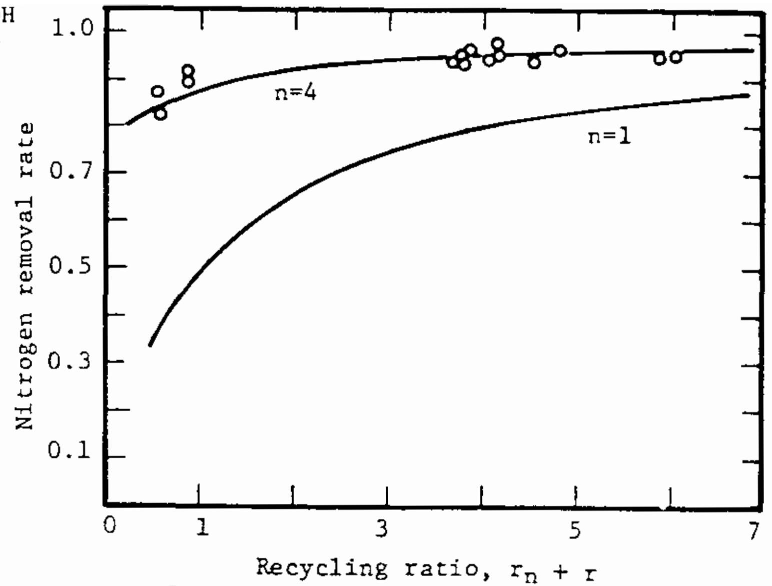


Fig. 8. Relation between nitrogen removal rate and recycling ratio

The amount of NaOH dose for neutralization will be represented as follows by using equation (7) obtained by the operation data.

$$C_{NaOH} = -@ \times 0.8 \times No\,(1 - \eta_s)(3.447\eta - 3.438)Q \qquad (9)$$

TABLE 5 Operational Results for Nitrogen Removal Rate

Run	Flow rate (m^3/d)	Return sludge ratio r	Recycling ratio r_n	$r+r_n$	Influent total-N (mg/l)	Dilution rate in volume	Effluent NH_4-N	Quality * NO_2, NO_3-N	** Nitrogen removal rate η
1	75.4	0.77	3.30	4.07	3,530	5.37	0.2	39.6	93.2
2	71.2	0.79	3.78	4.57	3,350	5.48	0.4	29.0	94.6
3	74.9	0.75	3.56	4.31	3,400	5.57	0.3	33.2	93.9
4	71.1	0.77	3.53	4.30	3,460	5.69	1.7	34.3	93.6
5	72.9	0.79	3.38	4.17	3,340	6.08	1.2	15.1	96.9
6	74.0	0.65	3.19	3.84	3,340	6.92	0.9	20.7	95.2
7	75.0	0.71	3.11	3.82	3.160	6.86	1.2	25.5	93.8
8	74.1	0.61	3.17	3.78	3,830	6.71	0.9	31.5	93.8
9	73.0	0.73	3.06	3.80	3,910	6.89	1.0	29.1	94.2
10	74.1	0.66	3.11	3.77	3,520	7.00	1.1	36.4	91.8
11	71.2	0.70	3.03	3.73	3,460	6.52	tr.	34.2	92.7
12	74.8	0.84	3.99	4.83	2,940	5.83	tr.	21.9	95.1
13	65.3	1.11	4.80	5.91	3,720	4.57	tr.	45.3	93.7
14	73.0	1.11	4.95	6.06	3,870	4.28	2.2	48.6	93.9
15	72.2	0.56	0	0.56	2,550	3.00	tr.	108	82.0
16	73.2	0.54	0	0.54	2,580	2.79	tr.	104	87.3
17	73.6	0.84	0	0.84	2,710	1.91	tr.	107	91.5
18	74.6	0.82	0	0.82	2,930	2.74	tr.	108	88.6

* effluent means mixed liquor in the 4th nitrification tank before addition of CH_3OH

** η is calculated by using equation (6)

The negative sign in the equation is because of calculation of reduction in alkalinity as NaOH required. Power costs for the recirculation pumps can be described as follows by using the material balance equation (1) and the power calculation equation (3);

$$C_{Pump} = @ \times 3.88 \times 10^{-3}\left\{\frac{1}{4(1-\eta)} - (1+r)\right\} Q \times D \times H \qquad (10)$$

where D denotes the dilution rate, 5.93 on an annual average. The unit prices of methanol, NaOH and electric power were estimated at ¥60/kg, ¥90/kg and ¥13/kwh, respectively. The operating factors for these calculations were estimated at;

$No = 3{,}390 \times 10^{-3}$ kg/m^3, $\eta_s = 0.113$, $Q = 73$ m^3/d, $r = 0.78$, respectively, according to the average operational data. The results of calculations are shown in Fig. 10. In the calculations the sludge return ratio was assumed 78%, and the expectable nitrogen removal rate as calculated from equation (1) will be 87.5% for the basic value.

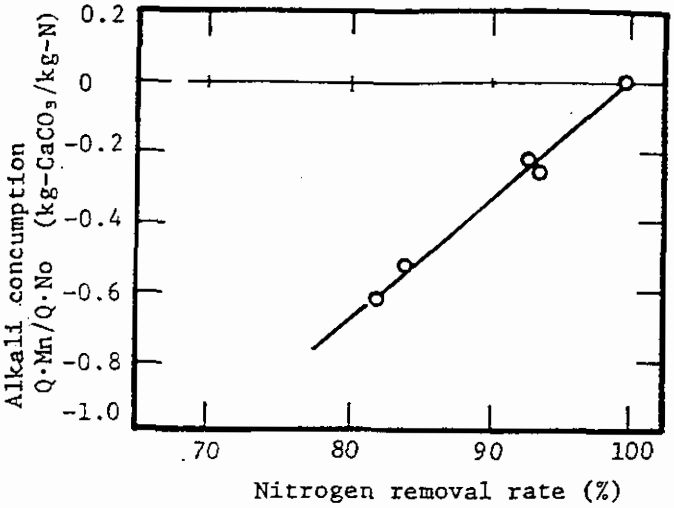


Fig. 9. Relationship between alkali consumption and nitrogen removal rate

The amount of NaOH required decrease linearly with an increase in the nitrogen removal rate as can be seen in Fig. 10. As is clear from Fig. 10, the required amount of methanol rises in proportion to the nitrogen removal rate, whereas the pumping cost sharply picks up at the nitrogen removal rate of 95%. Therefore, it would be better to improve the nitrogen removal rate by increasing the recycling ratio up to a removal rate of 95%, and thereafter by adding methanol to further improve the removal rate. The total running cost is obtained as shown in the Fig. 10 from the bold line which shows it decreases with an increased nitrogen removal rate owing to lowered NaOH costs.

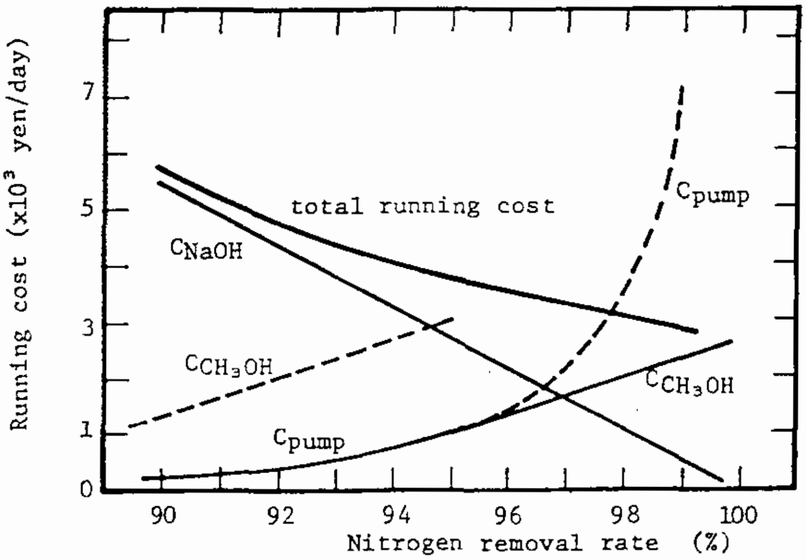


Fig. 10. Optimum running cost in step-feed process

CONCLUSION

An economical biological nitrogen removal process, the step-feed process, was studied theoretically and evaluated on the basis of actual plant operation data. The following is a conclusion obtained from these observations.

(1) The following equation was verified by the actual plant operation data concerning the relation among the nitrogen removal rate, the number of steps n and the recycling ratio of mixed liquor r_n, and a great reduction in the mixed liquor recycling ratio could be achieved compared to the conventional processes with a comparable nitrogen removal rate.

$$\eta = 1 - \frac{1}{n(1 + r + r_n)}$$

(2) In the case of night-soil treatment, the running cost of the step-feed process with 4 steps was proved to decrease as the nitrogen removal rate increases through an optimum selection of methanol dosage and the recycling ratio. An overall nitrogen removal rate of 99% could be maintained.

REFERENCE

1) Barnard, J. L. (1973). Biological denitrification. Water Pollution Control, 6, 705-720.
2) Toya, Y., Matsuo, Y., Suzuki, T., (1974). Biological process for nitrogen removal. 10th Annual Conference of Sanitary Engineering, Japan Society of Civil Engineering 116-121.
3) Miyaji, Y. (1977). Nitrogen removal by biological nitrification-denitrification. Kagaku-kojo, 21, December, 36-41.
4) Process Design Manual for Nitrogen Control, U. S. Environmental Protection Agency 3-4, (1975).
5) Miyaji, Y., Sekigawa, Y. (1974). Plant operation of nitrogen removal. Journal of Japan Sewage Works Association, 11, (120) 30-38.

GRAVITATIONAL THICKENING PROCESS FOR SLUDGE

T. Kusuda
Department of Civil Engineering, Kyushu University

Gravitational Thickening Process for Sludge

Tetsuya Kusuda
Department of Civil Engineering
Kyushu University
Fukuoka 812
JAPAN

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.

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 a 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 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

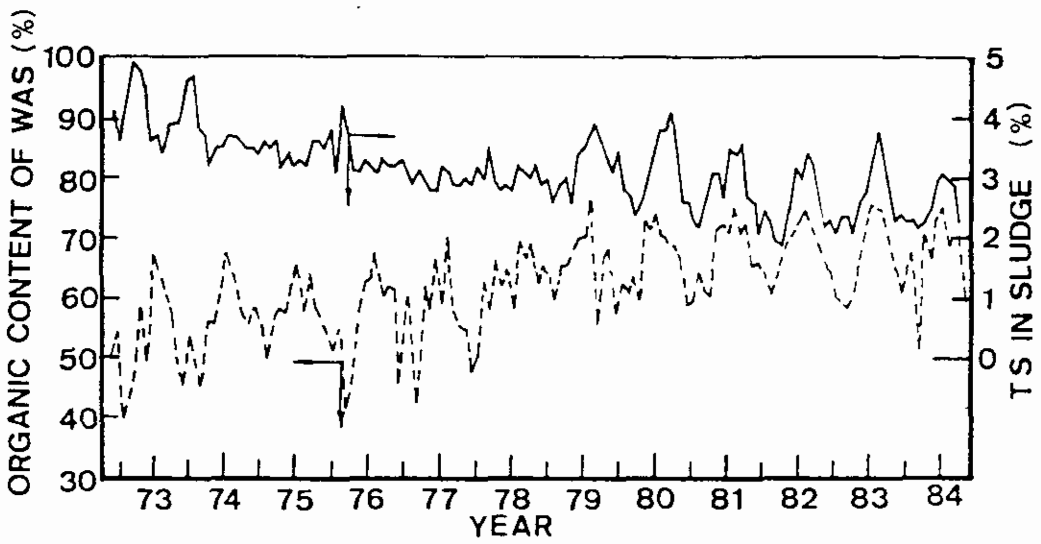


Fig. 1 Changes of TS in thickened sludge and organic content of WAS.

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 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 5 *l* 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.

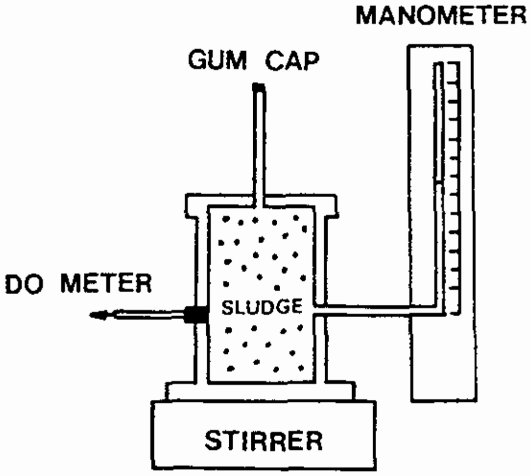


Fig. 2 Gas sampling device.

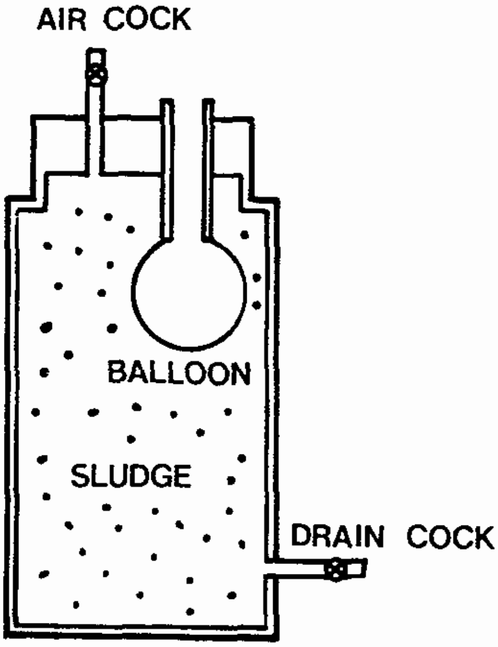


Fig. 3 Sludge tank.

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 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 Figs. 8 to 12. 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

Table 1 Properties of sludge

Series	RUN	SS (mg/l)	SVI	Ig-Loss (%)	Wet Sludge Density ρ_s (t/m^3)	Fluid Density ρ_f (t/m^3)	Wet Solid Fraction ε_s	Aeration Time(h)	Added NO_3-N (mg/l)	Alkainity (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	5,230	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

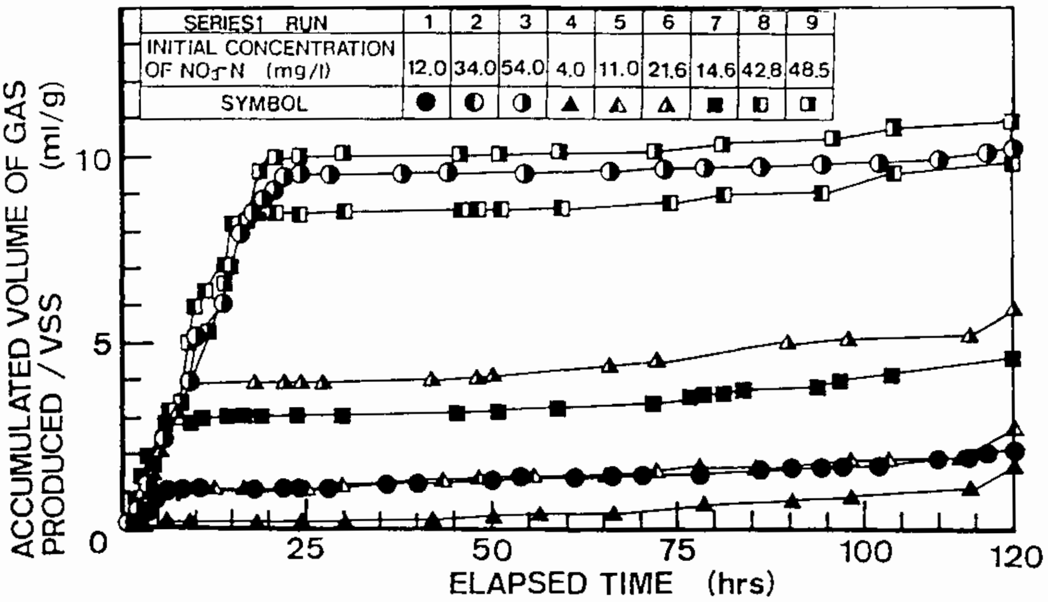


Fig. 4 Accumulated volume of gas produced (Series 1).

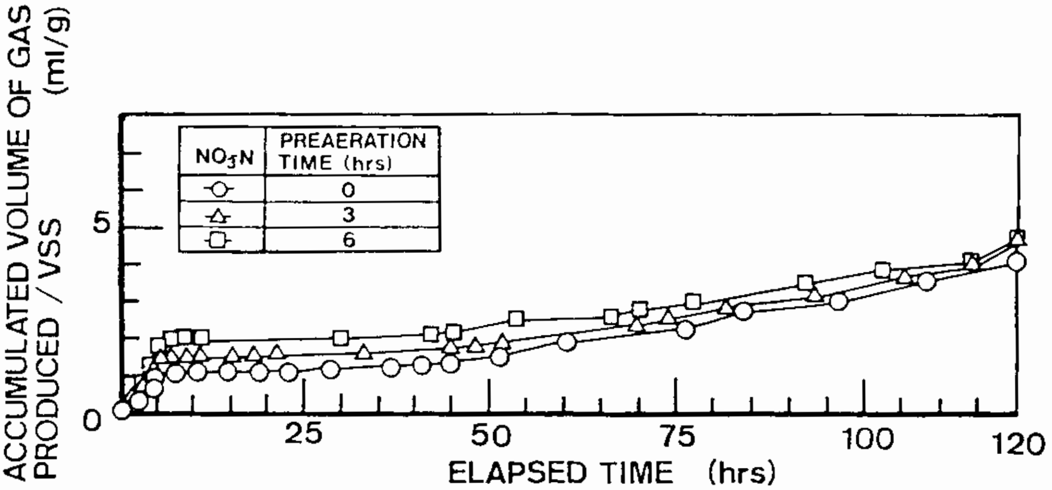


Fig. 5 Accumulated volume of gas produced (Series 2).

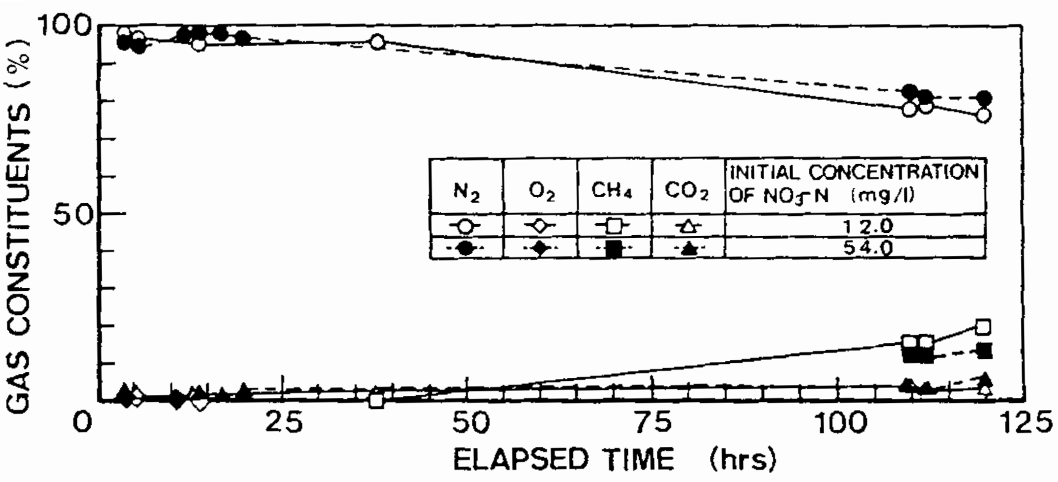


Fig. 6 Change of gas constituents (Series 1).

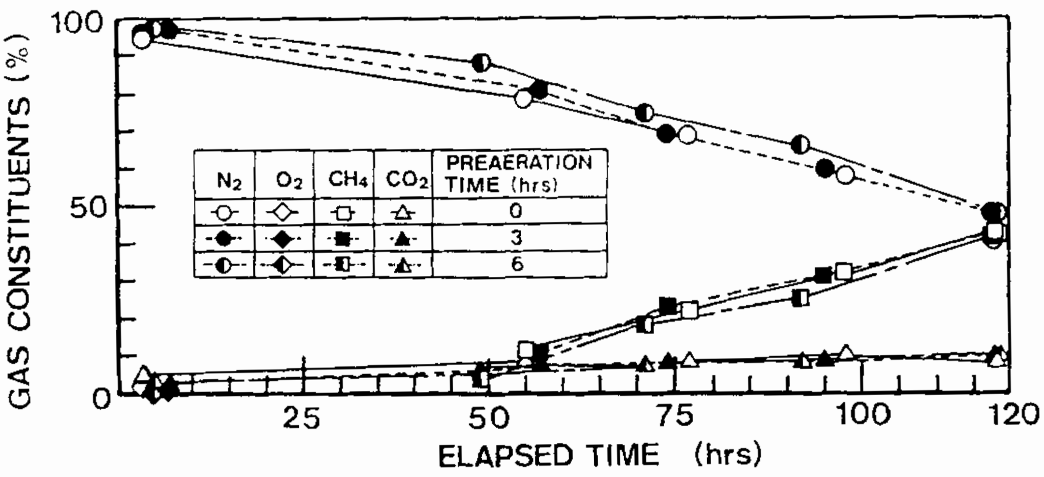


Fig. 7 Change of gas constituents (Series 2).

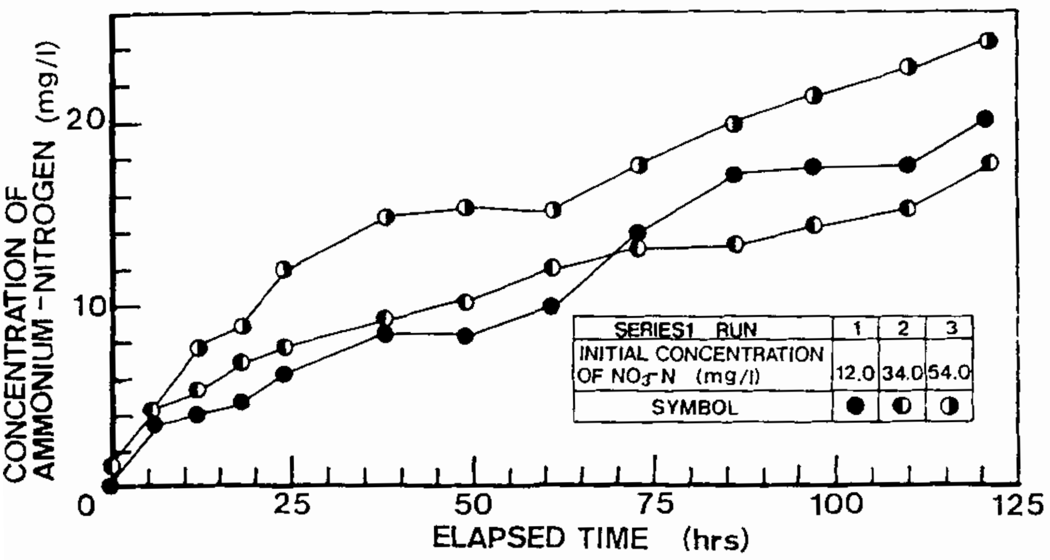


Fig. 8 Ammonium nitrogen concentration in interstitial water (Series 1).

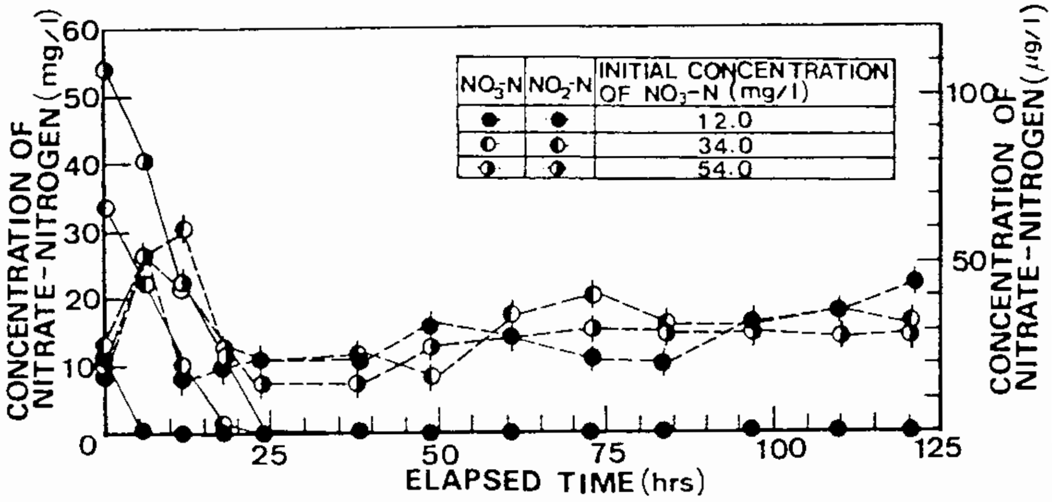


Fig. 9 Nitrate- and nitrite- nitrogen concentrations in interstitial water (Series 1).

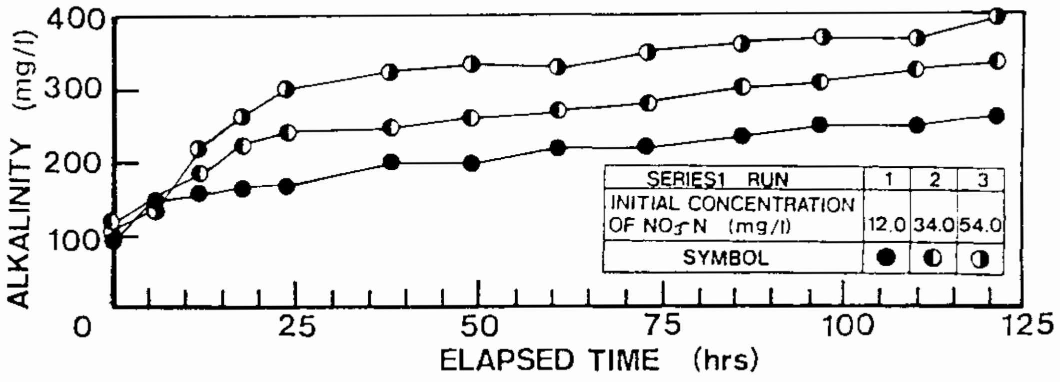


Fig. 10 Alkalinity in interstitial water (Series 1).

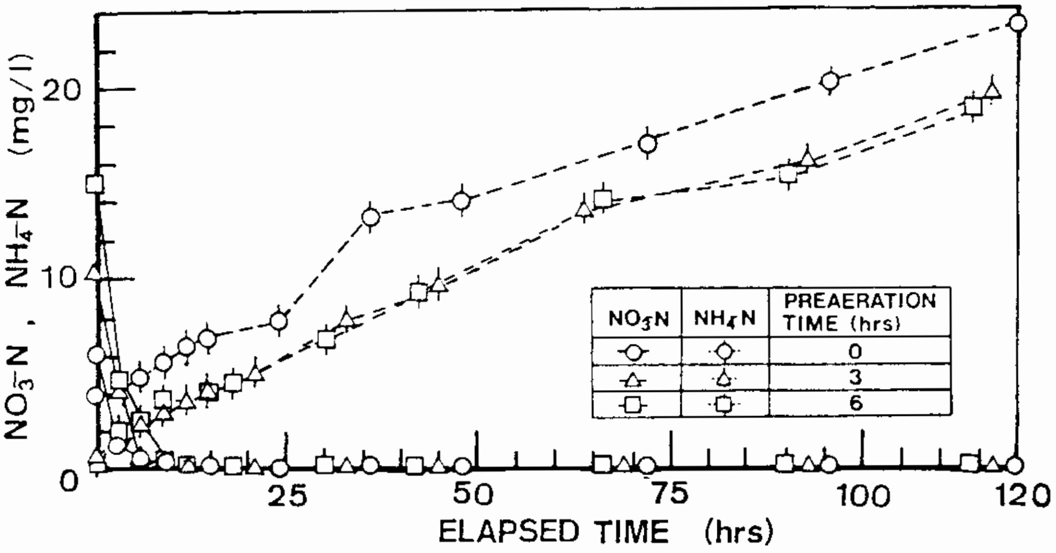


Fig. 11 Ammonium and nitrate nitrogen in interstitial water (Series 2).

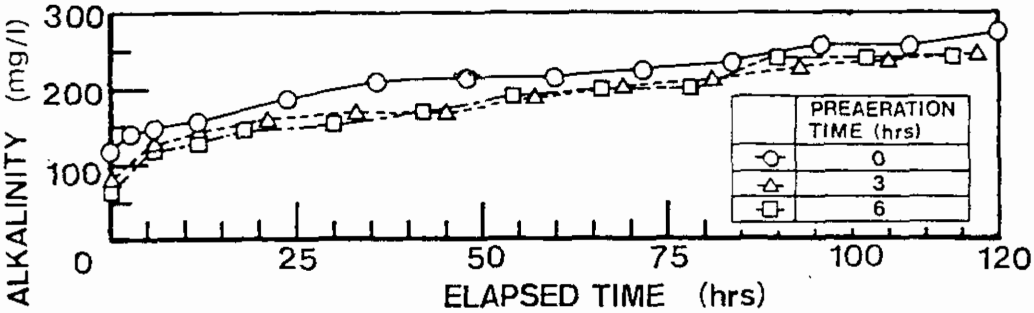


Fig. 12 Alkalinity in interstitial water (Series 2).

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 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 g 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 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

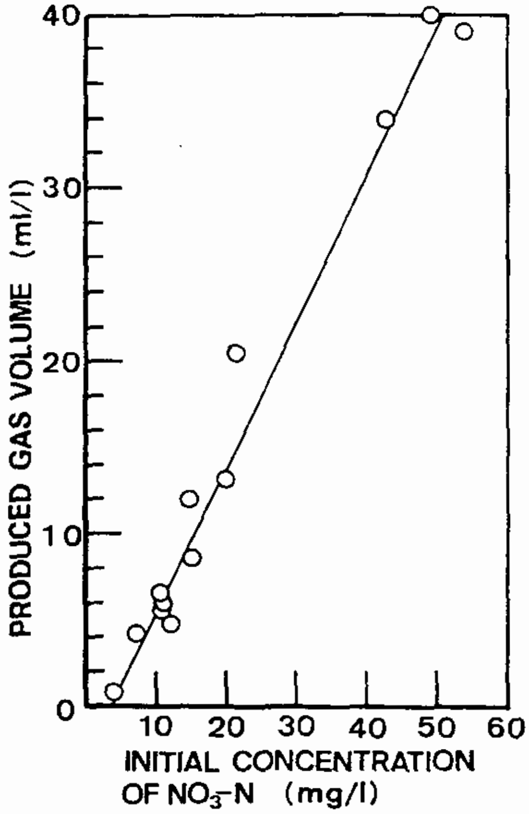


Fig. 13 Produced gas volume.

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 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 bubbles of gas produced. Figure 15 illustrates the relationship between the period of no floatation of sludge and the initial concentration of NO_3-N. The minimum 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.

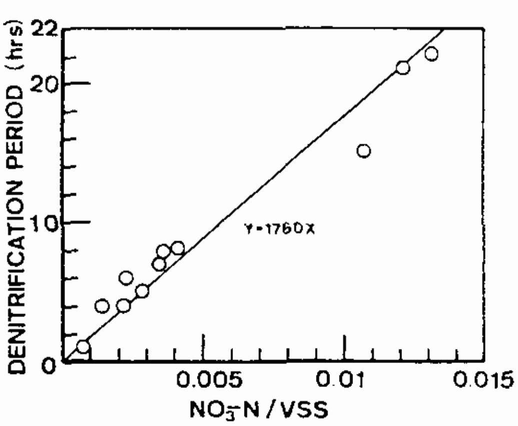


Fig. 14 Denitrification period.

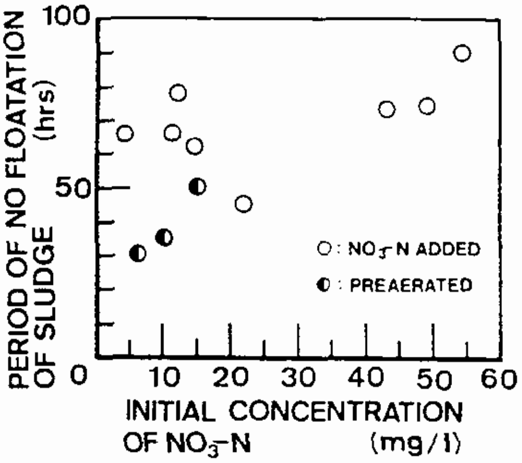


Fig. 15 Sludge floatation time in the period of reduced rate of gas production.

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

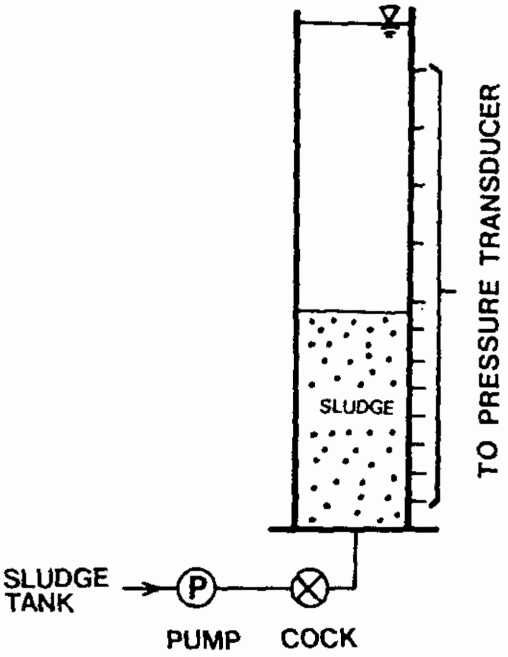


Fig. 16 Settling column.

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.

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 :

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

SOLID :

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

LIQUID :

$$\frac{\partial P_l}{\partial x}+\frac{\mu}{K}\left(V_l-\frac{\varepsilon_l}{\varepsilon_s}V_s\right)=0 \tag{3}$$

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

TOTAL :

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

SOLID :

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

LIQUID :

$$\frac{\partial \varepsilon_l}{\partial t}+\frac{\partial V_l}{\partial x}=0 \tag{6}$$

GAS :

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

$$\varepsilon_g+\varepsilon_l+\varepsilon_s=1 \tag{8}$$

$$q_g=\varepsilon_s\rho_s\alpha \tag{9}$$

$$V_l=\varepsilon_l v_l \tag{10}$$

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

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

where V: Volume flux per unit area, v: real velocity, ε : solid fraction, ρ : density, suffixes 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 (w, t), where w is mass of sludge, based on the following equation.

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

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

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

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

3. 3. Experimental Results and Discussion

The properties of sludge employed in experiments are listed in Table 2. Since μ is given, P_l and v_s are directly measured, K, coefficient of 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.

Table 2 Properties of sludge

Series	RUN	SS (mg/l)	SVI	Ig-Loss (%)	Wet Sludge Density ρ_s (t/m^3)	Fluid Density ρ_l (t/m^3)	Wet Solid Fraction 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

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

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

before Run 2 of Series 3,

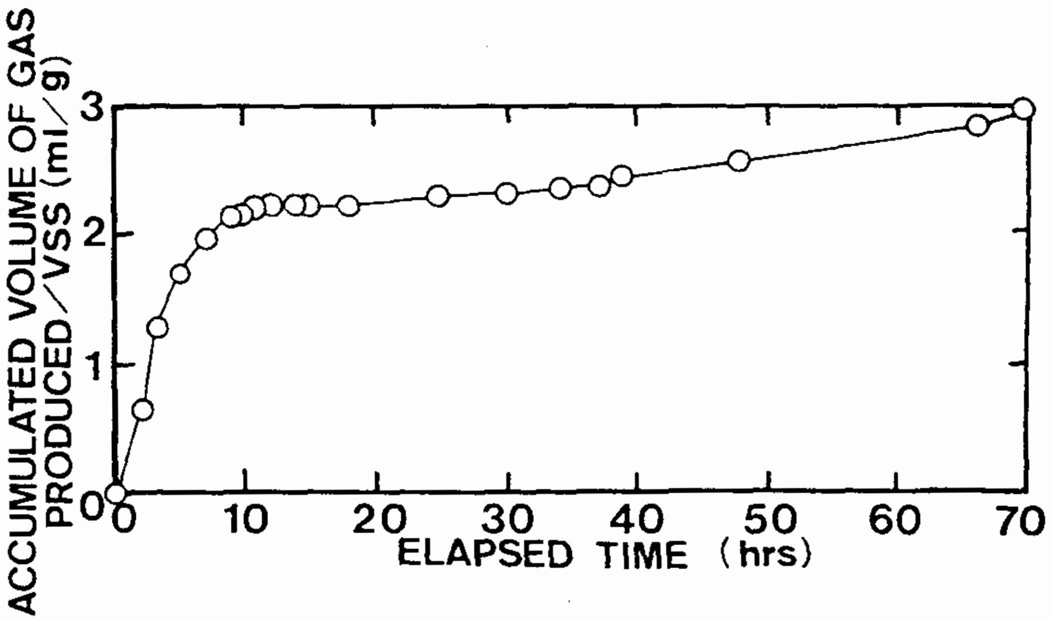


Fig. 17 Accumulated volume of gas produced (Series 3, Run 1).

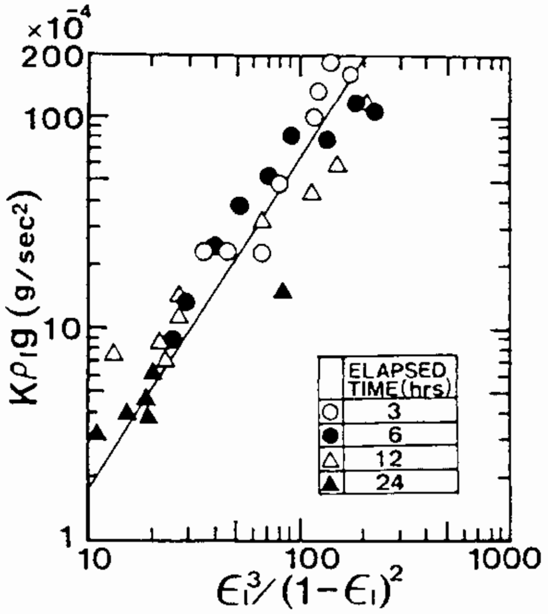


Fig. 18 Coefficient of permeability (Series 3, Run 1).

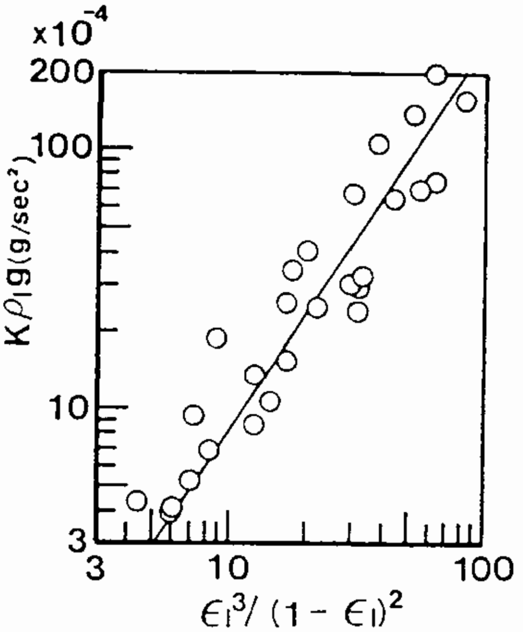


Fig. 19 Coefficient of permeability (Series 3, Run 2).

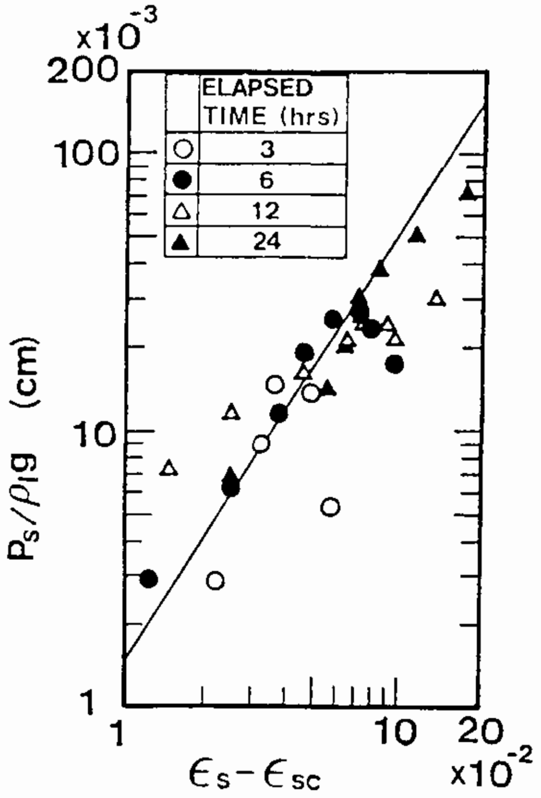


Fig. 20 Effective pressure (Series 3).

$$\rho_l g K = 2.43\times10^{-5}\{\varepsilon_l/(1-\varepsilon_l)\}^{1.52} \tag{18}$$

for both

$$\frac{P_s}{\rho_l g} = 2.1(\varepsilon_s - \varepsilon_{sc})^{1.6} \tag{19}$$

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,

$$\alpha = 7.718 \times 10^{-8} \qquad (\mathrm{ml/gs}) \tag{20}$$

for Run 2 of Series 3,

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

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 (w, t).

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

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

$$q_g = \varepsilon_S \rho_S \alpha \tag{22}$$

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

$$\int_0^w q_g\,dt = \alpha \triangle w N \tag{23}$$

where N is the total division number of sludge layers.

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

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

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 whole with that in experiment, however, the experimental value becomes larger around the consolidation point and smaller

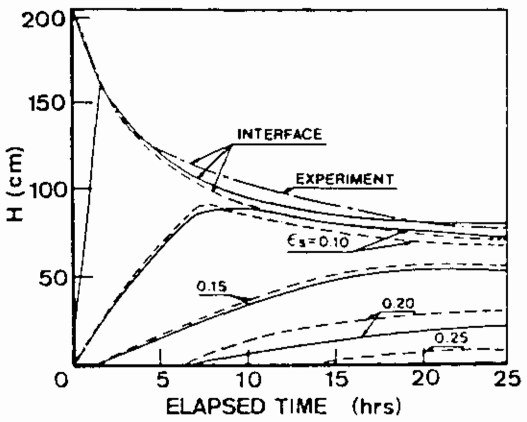


Fig. 21 Numerical simulation results on batch test (Series 3, Run 1).
—·— observed,
—— Simulation with gas production,
----- Simulation without gas production

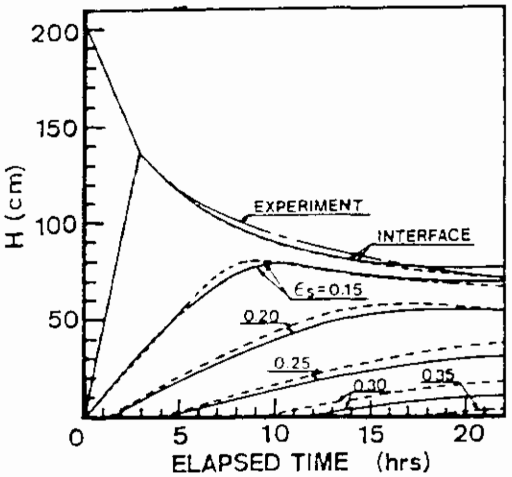


Fig. 22 Numerical simulation results on batch test (Series 3, Run 2).
Lines: refer to Fig. 21

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:

$$\triangle \varepsilon_S = \frac{d\varepsilon_S}{dP_S}\left[\frac{\mu}{K}\left(q_b + \int_0^x q_g dx - \frac{V_S}{\varepsilon_S}\right) - \{\varepsilon_S(\rho_S - \rho_l) + \varepsilon_g(\rho_g - \rho_l)\}g\right] \triangle x \tag{25}$$

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

In a steady state, V_S and V_l 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 q_b 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 thickened before the completion of denitrification. When sludge

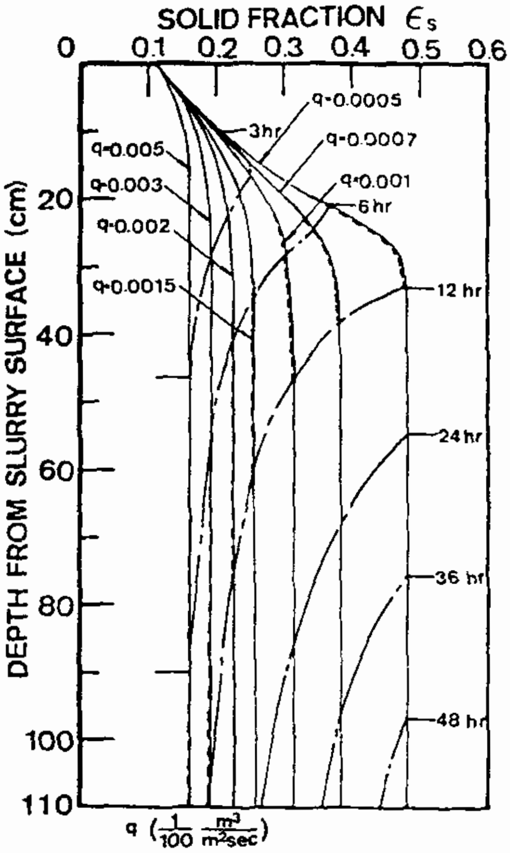


Fig. 23 Numerical simulation results in a continuous flow.
—— No gas produced, ----- Gas produced

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 the 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 a 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.

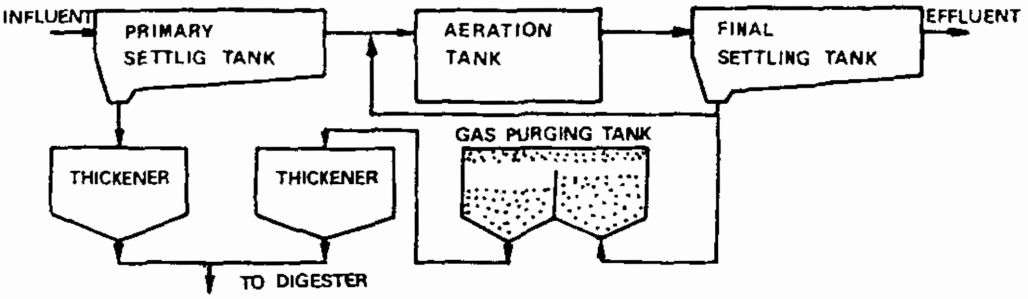


Fig. 24 A proposed process with a gas purging tank.

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 denitification, a period of reduced rate 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 conducted;
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.

This research was partly supported a grant from the Ministry of Construction.

References

1) Japan Sewage Works Agency: Report on Improvement of Sludge Management, p. 12, 1983.
2) Santhanam, C. J. and J. F. Zieyers: Dewatering of Sludges, In Sludge Treatment, Marcel Dekker, New York, pp. 141–209, 1981.
3) Kouzeli-Katsiri, A.: Treatment of Sewage Sludge from Major Athens Area, Proc. Third International Symposium on Processing and Use of Sewage Sludge, Brighton, pp. 97–103, 1983.
4) Ohwaki, H., K. Ishikawa, and H. Miwa: Know-how in Waste Water Treatmen Plants, Jour. Sewarage, Vol. 8, pp. 18–23, 1985.
5) Japan Sewage Works Agency: Survey of Function Improvement on Gravitational Thickeners, Annual Report, pp. 1–21, 1984, and pp. 1–12, 1985.
6) Baba, K., S. Nogita, and S. Watanabe: Some Considerations on the Gravity Sludge thickening Process by Gas Production Model of Anaerobic Bacteria, Jour. Japan Sewage Works Association, Vol. 22, No. 258, pp. 48–56, 1985.
7) Tohya, T.: Biological Denitrification, Jour. Japan Sewage Works Association, Vol. 7, pp. 19–32, 1970.
8) Winfrey, M. R. and J. C. Zeikus: Microbial Methanogenesis and Acetate Metabolism in a Meromitic Lake, Appl. and Environ. Microbial., Vol. 37, p. 213, 1979.
9) Van Velesen, A. F. M.: Adaptation of Methanogenic Sludge to High Ammonia-Nitrogen Concentration, Water Research, Vol. 13, p. 995, 1979.
10) Tanaka, K. et al.: Automatic Control of Biological Denitrification Process by Redox Potential, Water Treatment Technology, Vol. 23, No. 8, pp. 36–41, 1982.
11) Kusuda, T., K. Koga, and Y. Awaya: Gravity Thickening of Sludge, Trans. Japan Soc. Civil Eng., No. 12, pp. 117–120, 1980.

SOME FUNDAMENTAL ASPECTS OF SLUDGE DEWATERING

P.J.A.M. Kerkhof
Department of Chemical Engineering, Eindhoven University of Technology

SOME FUNDAMENTAL ASPECTS OF SLUDGE DEWATERING

Piet J.A.M. Kerkhof

(Laboratory for Chemical Process Engineering, Dept. of Chemical Engineering, Eindhoven University of Technology, P.O.Box 513, 5600 MB Eindhoven, the Netherlands)

SUMMARY

An overview is given of major fundamental aspects of sludge dewatering. First the presence of water in sludge and filter cakes is treated schematically. Subsequently attention is paid to the changes taking place during filtration and expression, the major changes being those in local porosity inside flocs and between the flocs. For the description of the flow through a bed of permeable flocs a new, simple approximating model is proposed : the **"DUAL FLOW model"**. This allows the estimation of the effective permeability of a bed with permeable particles; from simulation follows that for compressed beds the contribution of the flow through the flocs can be much higher than that of the flow in the small open spaces between the flocs. Simulations of the expression of a floc indicate that during filtration and expression the local floc porosity will very fast reach equilibrium with the local compression pressure. From these considerations it follows that major focussing points for further study are :

- a more detailed insight into the relations between porosity, permeability and pressure distribution during filtration and expression
- theoretical and experimental insight in the size, strength and permeability of flocs, in dependence of structure parameters and pretreatment.

1. INTRODUCTION

1.1. General

In the beginning of 1990 we started a study of some fundamental aspects of sludge dewatering in our laboratory, within the larger Dutch national project "Municipal Waste Water Treatment 2000 (RWZI-2000)". A team of three full-time project members (ir. Arend J.M. Herwijn, ir. Erik J. La Heij, ing. Paul M.H. Janssen) took up the study in cooperation with dr.ir. W. Jan Coumans and myself. Basic aim of the study is the understanding of physical and physico-chemical phenomena occurring on microscale, and how these phenomena manifest themselves on the macrosopic scale. As desired results we see the links between various characterisation methods and physical parameters, and the development of theoretical and simulation models predicting filtration and expression behaviour. These type of models could then be used in the optimisation of dewatering processes and equipment parameters. One of the methods used in our study is the drying of filter cakes or parts, which gives fundamental insights in water transport and binding, but also gives information from which drying processes for sludge can be modelled and optimised.

As will become clear in the following an important element will be the colloid-chemical aspects of floc formation, strength and structure, and the consequences for the engineering aspects.

I.2. The presence of water

In Fig.1 we present schematically the way that water may be present in sludge and sludge cakes. In a suspension or in a filter cake we may distinguish a water phase and a floc phase. The porosity or volume fraction of the water phase is denoted by ε_b, and so the volume fraction of the flocs is equal to $(1 - \varepsilon_b)$.

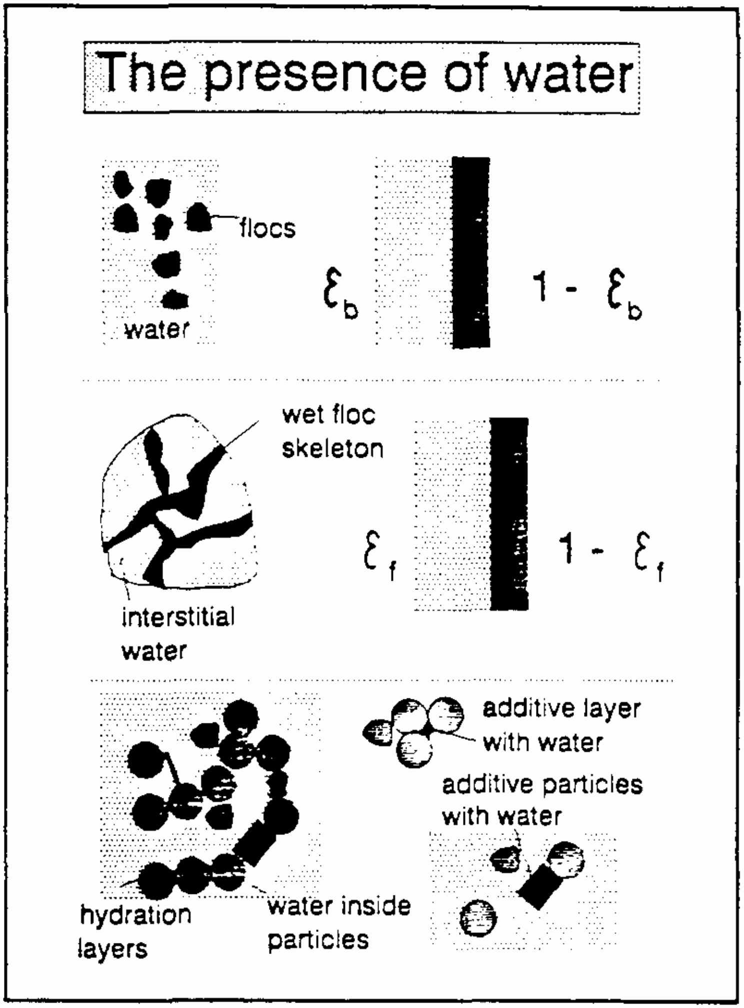


Fig. 1. Schematic representation of water in sludge

We also use the *void ratio* e_b, the volume of water phase per unit of volume of floc phase :

$$e_b = \frac{\varepsilon_b}{(1 - \varepsilon_b)} \tag{1}$$

The flocs in turn consist of a wet skeleton, between which interstitial liquid is present in the floc pores. We have then the *internal floc porosity* ε_f, which is the fraction of the floc volume, taken up by the liquid. The volume fraction of the wet skeleton is thus $(1 - \varepsilon_f)$. The floc void ratio e_f is then given by :

$$e_f = \frac{\varepsilon_f}{(1 - \varepsilon_f)} \tag{2}$$

Looking in more detail into the flocs we have a collection of *elementary particles*, made up of *basic sludge particles* and *additives*. The basic sludge particles stem from the waste water plant itself, and thus will vary with place, time of year, and other conditions. The additives are dependent on the specific sludge treatment given before solid-liquid separations. Now we have water inside the basic particles, like microbial cells, or pieces of wood,etc. Also we may have spots and/or particles of additives, which may contain water. Further we will have hydratation layers at the particle surfaces. For a quantitative account of the amount of water in the system, one should theoretically make a detailed mass balance of all substances in a sludge or filter cake. Although this is useful for scientific purposes, and also for a possible drying step, for the processes of filtration and expression we may reasonably approximate the water content of a system by :

$$X_{w,cake} = \frac{X_{w,i} + X_{add,1}\, X_{w,add} + \rho_l\, e_f\, (1 + e_b) / \rho_{sk}}{(1 + X_{add,1})} \quad (3)$$

In this equation :

$X_{w,cake}$	= water content of cake	[kg/kg dry solids]
$X_{w,i}$	= internal water content of particles	[kg/kg dry solids]
$X_{add,1}$	= amount of dry additives/kg dry sludge, remaining in the sludge flocs	[kg/kg ds]
$X_{w,add}$	= water content of additive particles or layers	[kg/kg ds]
ρ_l	= density of water phase	[kg/m^3]
e_f	= floc void ratio	[m^3 liquid/m^3 ds]
e_b	= bed void ratio	[m^3 bed liquid/m^3 flocs]
ρ_{sk}	= density of (wet) floc skeleton	[kg/m^3]

I.3. Factors influencing dewatering steps

From the above picture we can derive several important factors, all influencing the dewatering process.

Floc formation

The flocs are formed from the basic sludge particles, in many cases with the addition of additives. Thus the nature of the basic sludge particles is one of the prime factors. This will depend on the treatment plant, the waste water stream fed, the conditions changing over time. The amount and nature of additives, combined with the way the process is carried out, is for a given set of basic sludge particles, at a given concentration determining for the floc formation process. This implies the way the floc skeleton is built up, the floc size, the initial floc porosity, the chain length of aggregates, the number of links per volume of floc, and the bond strength between the elementary floc particles.
Thus the floc formation process determines the ***initial floc porosity, size and strength.***
Examples of floc size distributions as measured in our lab by means of a Malvern Mastersizer are given in Figs. 2 and 3, with $FeCl_3/Ca(OH)_2$ and polyelectrolyte as additives. We see that with increasing addition of $FeCl_3$ up to a certain point the distribution tends to go to increasing size; for the polyelectrolyte this effect is much more markedly. Also interesting is the amounts of both additives needed to obtain an effect; for polyelectrolytes this is roughly 1/100 of that for the iron compound. It is clear that other mechanisms will be operating in both cases, and also that in the case of iron addition the iron hydroxide will form a considerable part of the sludge flocs.

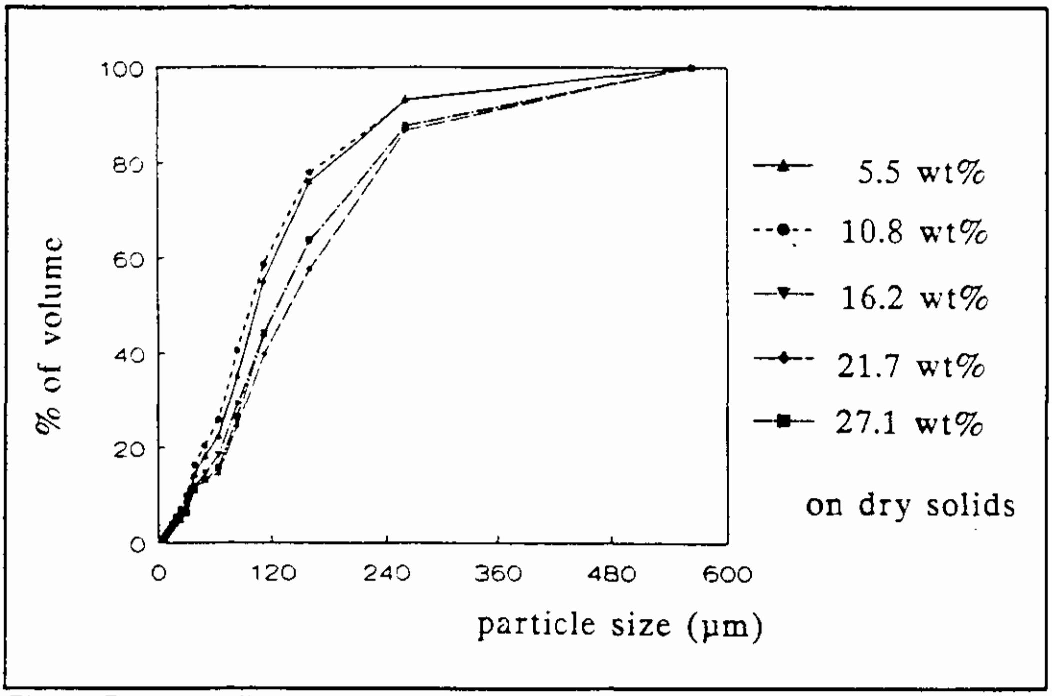


Fig. 2. Floc size distribution by flocculation with $FeCl_3$

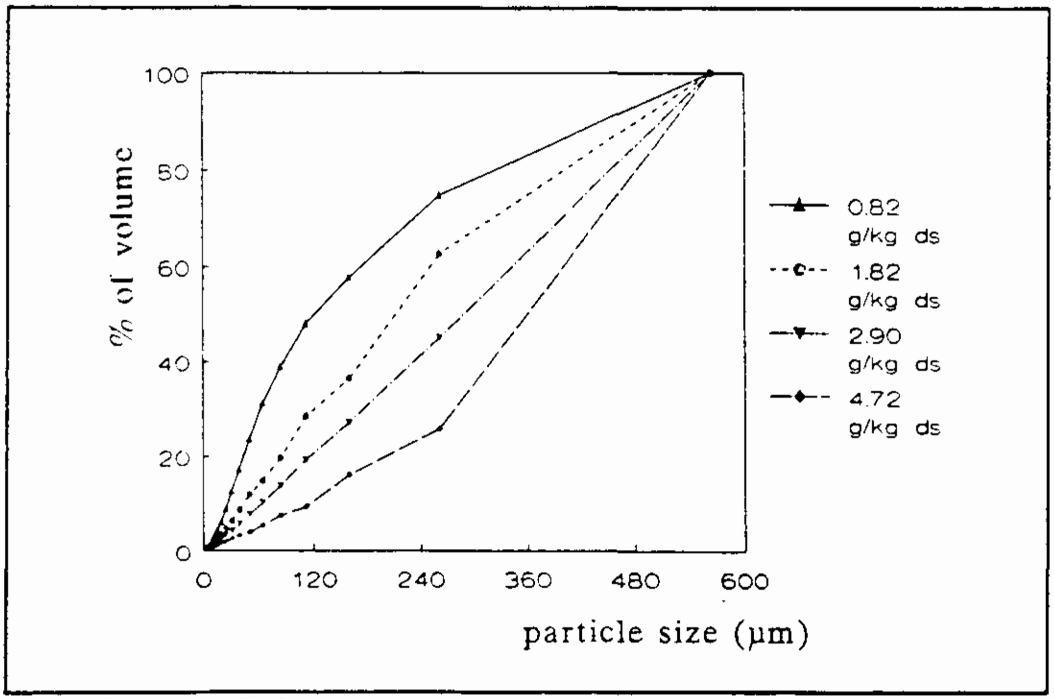


Fig. 3. Floc size distibution in case of polyelectrolyte addition

Dewatering

In general after a sedimentation step dewatering is performed by means of filtration, expression and in many cases drying. In filtration and expression the water phase moves relative to the solids under the influence of a gradient in liquid pressure. This can be represented by the following equations :

$$u_l = - \frac{K}{\eta} \frac{\partial p_l}{\partial z} = - \frac{1}{\eta \; R} \frac{\partial p_l}{\partial (z/L)} \qquad (4)$$

in which :

u_l = superficial liquid velocity [m/s]
K = permeability [m^2]
η = dynamic viscosity [Pa.s]
p_l = liquid pressure [Pa]
z = distance coordinate [m]
R = fitration resistance of cake (= L/K) [m^{-1}]
L = cake thickness [m]

Often is also used the specific filtration resistance α :

$$\alpha = R \, / \, w \qquad (5)$$

with α = specific filtration resistance [m/kg]
w = mass of cake per m^2 area [kg/m^2]

We may write the relation between α and K by :

$$\alpha = \frac{1}{\rho_S \, (1 - \varepsilon) \, K} \qquad (6)$$

with ρ_s = density of "bed particles" [kg/m^3]
ε = porosity [-]

For the permeability we may write the Blake - Kozeny equation :

$$K = \frac{\varepsilon^3 \, d_p{}^2}{150 \, (1 - \varepsilon)^2} \qquad (7)$$

with d_p = effective particle diameter [m]

The conditions chosen in practice with regard to additives and filter aids reflect the search for a compromise : the material should filter well, which would mean a low degree of deformation of flocs and beds, but on the other hand the final water content should be as low as possible, which means a considerable lowering of the porosity upon expression. This can be seen from Eq.(3), in which for a low deformable filter cake the main term in the numerator is that containing the void ratios: the major part of the water is present in the pores of the bed and in the interstitial pores inside the flocs.

During filtration there is a build-up of the cake, causing a conversion of liquid pressure into solid pressure, which tends to compress the cake. This compression can be viewed as a combination of simultaneous phenomena :

- expression and volume change of flocs
- bed compaction associated with floc deformation
- bed compaction associated with shear induced relative floc displacement

Next to the material properties of the flocs, also the choice of process conditions is determining the way the pressure profiles are built up. A higher filtration rate tends to increase the pressure gradients, thereby increasing the solid pressure on the particles, which in turn leads to a decrease in porosity and thus permeability K, which increases the pressure drop even more. At constant pressure filtration or expression, the resistance of the filter medium determines the initial flow rate and thereby also the steepness of pressure gradients in the initial phase of the process.

It is important to realise that it is the **combined effect of floc properties and process conditions**, which determines the filtration rate and/or pressure, and the final water content.

Although the description of filtration and expression requires a more detailed analysis, interesting observations for practice can be obtained from laboratory tests, such as filtration of a sample under constant pressure. By fitting the curve of filtrate volume over time, we find an average value of the specific cake resistance α_{av}. In Figs. 4 and 5 we see the influence of additives on this average resistance, again for the addition of $FeCl_3$, and for polyelectrolyte (Rohm KF-945), both at a filtration pressure of 2 bar.

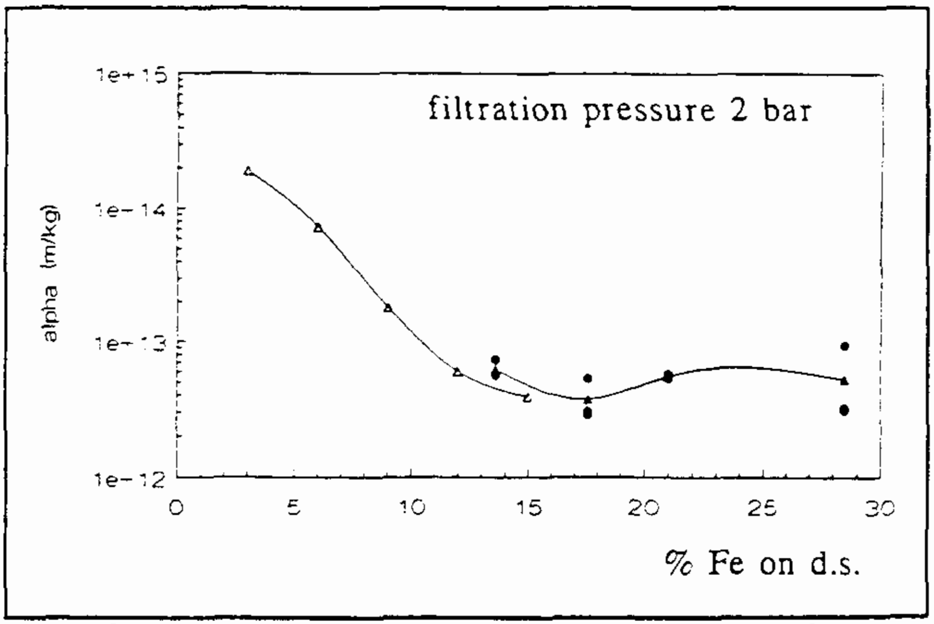


Fig. 4. Specific cake resistance α_{av} in case of addition of $FeCl_3$

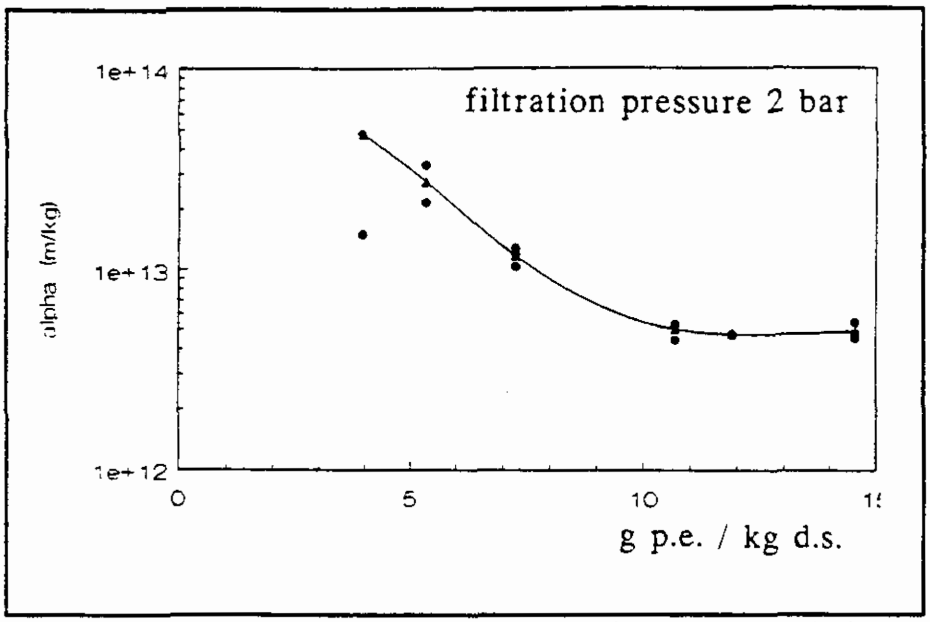


Fig. 5. Specific cake resistance α_{av} in case of addition of polyelectrolyte

It is clear that the resistance decreases upon addition up to a certain amount. The decrease is quite strong; comparison with the particle size data shows that there is no quantitative correspondence between the average particle size and the resistance. For $FeCl_3$ the effect is larger than expected from Eqs. (5) to (7), for polyelectrolyte it is lower. Apparently the deformation of the flocs and the bed under shear are also determining factors.

In Fig. 6 we have plotted the average specific cake resistance in dependence on the filtration pressure, which shows a typical power dependence with a power 0.8.

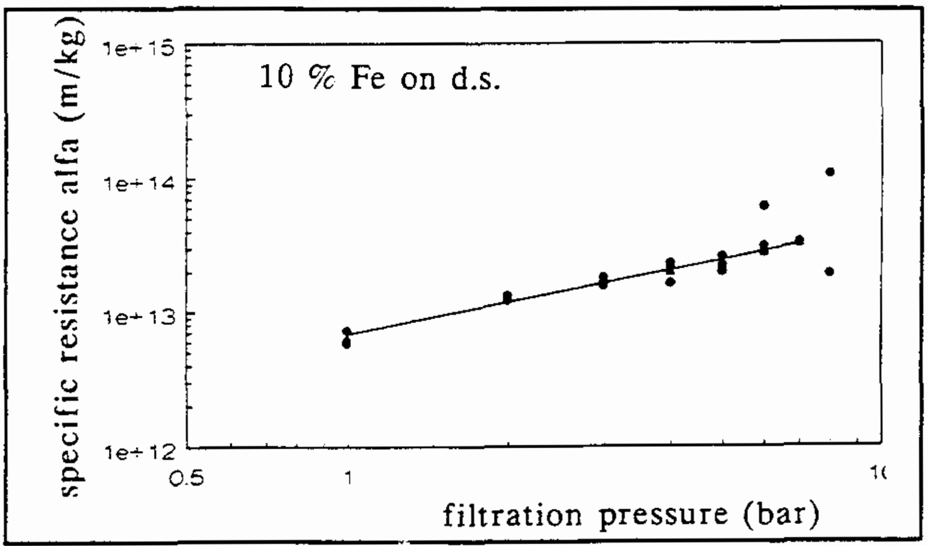


Fig. 6. Dependence of the specific cake resistance on the filtration pressure, for addition of 10 % Fe on d.s.

Although this and analogous tests give a practical insight into filtration and expression resistances, for treatment of more difficult situations, development of new equipment and optimization, more detailed knowledge is required of the distribution of pressure and permeability over the material and of the development in the course of time.

2. SOME HYDRODYNAMIC CONSIDERATIONS

2.1. Flow through a bed of permeable flocs : the DUAL - FLOW model

In considering the modelling of the filtration process we were faced with the problem that the Blake - Kozeny equation for a bed was in principle derived for particles with a closed surface, and is based on the concept of the hydraulic radius of the pores in a bed, connected with the "wetted particle surface" [1]. Now it is known that flocs may have a quite open structure, and so an extension was sought of the theory. The basic assumption is that water flows both through the pores between flocs and through the flocs themselves. This has two effects for a given liquid pressure gradient :

- ***the liquid between the flocs is less decelerated at the floc surface***
- ***liquid flows through the flocs themselves***

From a simple approximation starting with laminar flow through a pore of which the walls are formed by porous flocs we can derive :

$$\begin{aligned} u_l &= u_{l,bed} + u_{l,floc} \\ &= \frac{1}{\eta}\left(-\frac{\partial p_l}{\partial z}\right)\left[\left(K_{b,sol} + \varepsilon_b K_f\right) + (1-\varepsilon_b) K_f\right] \\ &= \frac{1}{\eta}\left(-\frac{\partial p_l}{\partial z}\right)\left[K_{b,sol} + K_f\right] \end{aligned} \tag{8}$$

Herein $K_{b,sol}$ is the permeability of a bed consisting of particles of the same size as the flocs, but with a closed surface, and K_f is the permeability of the flocs. The K's are given by :

$$K_{b,sol} = \frac{\varepsilon_b^3\, d_f^2}{150\,(1-\varepsilon_b)^2} \qquad K_f = \frac{\varepsilon_f^3\, d_e^2}{150\,(1-\varepsilon_f)^2} \tag{9}$$

with d_f = floc diameter [m]
d_e = effective diameter of elementary floc particles [m]

For the effective permeability of the floc bed in relation to that of a similar bed consisting of closed particles we obtain :

$$\frac{K_{eff}}{K_{b,sol}} = 1 + \frac{\varepsilon_f^3}{(1 - \varepsilon_f)^2} \frac{(1 - \varepsilon_b)^2}{\varepsilon_b^3} \frac{d_e^2}{d_f^2} \tag{10}$$

In order to estimate the importance of this effect we plotted in Fig. 7 this ratio vs. the bed porosity, for a floc porosity of 0.80, with the ratio of floc size to elementary particle size as parameter. We can see that for very high bed porosity there is hardly any influence, the bed behaves the same as for closed particles. For lower bed porosity, especially for $\varepsilon_b < 0.1$ we see that the flow through the flocs causes a large increase of the bed permeability, especially at decreasing floc size.

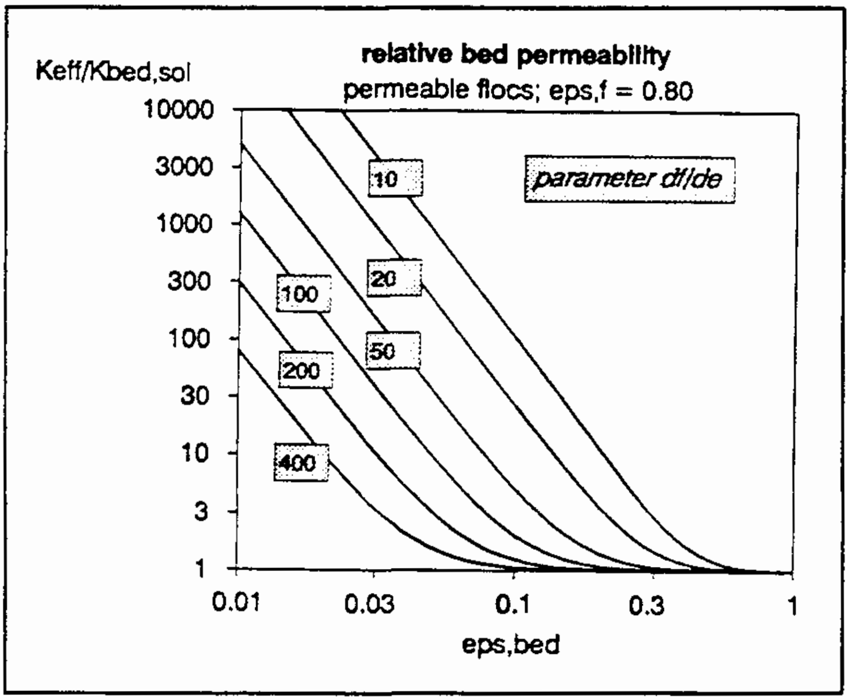


Fig. 7. Relative permeability of porous floc bed compared to closed particles, as function of bed porosity with relative floc size as parameter

Suppose that we can approximate the equilibrium void ratio of the flocs by :

$$e_f^* = e_{f0} \, (1 + a \, p_s)^{-\gamma_f (1 + e_{f0})} \tag{11}$$

with e_f = initial floc void ratio [-]
p_s = solid pressure [Pa]
a = coefficient [Pa^{-1}]
γ_f = floc compression coefficient [-]

In this hypothetical relationship is reflected that a more open floc will be weaker. Now for a bed we may assume that a small deformation and displacement of flocs causes a much larger change in bed porosity than in floc porosity :

$$\frac{e_b}{e_{b,0}} = \left(\frac{e_f}{e_{f,0}}\right)^{\beta} \qquad (12)$$

with β = bed compression exponent (>1) [-]

In Fig.8 the simulated effects of the compression pressure on the porosity of the flocs and of the bed is shown.

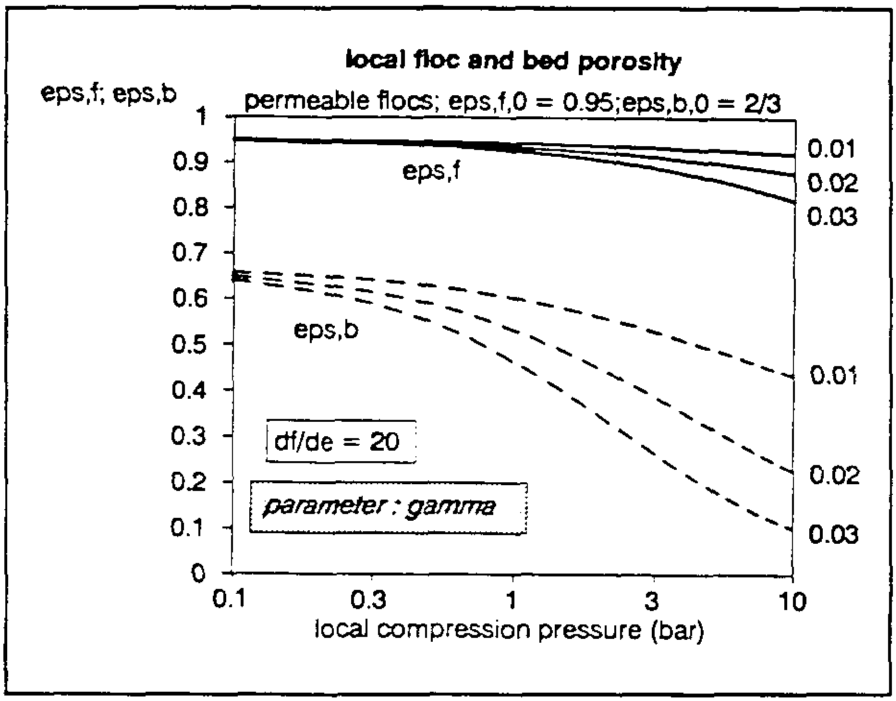


Fig. 8. Simulated effect of local solid pressure on porosity of flocs and of bed.

In Fig.9 the permeability of the bed with porous flocs is now compared with the initial permeability, for different local (uniform) solid pressures. The dotted lines represent the change in permeability of the spaces between the flocs, the drawn lines show the permeability through the whole bed. It is again clear that for th small flocs considered in this simulation a considerable amount of the total flow goes through the flocs. Also a considerable decrease is seen of the bed permeability with increasing pressure.

In Fig.10 again the relative permeability is plotted vs. the compression pressure, but now for larger flocs. We now see that the difference between a bed of flocs and a bed of closed particles is considerably less; the contribution of floc flow is here relatively low. This leads however to a much stronger decrease in overall permeability than in the case of smaller flocs.

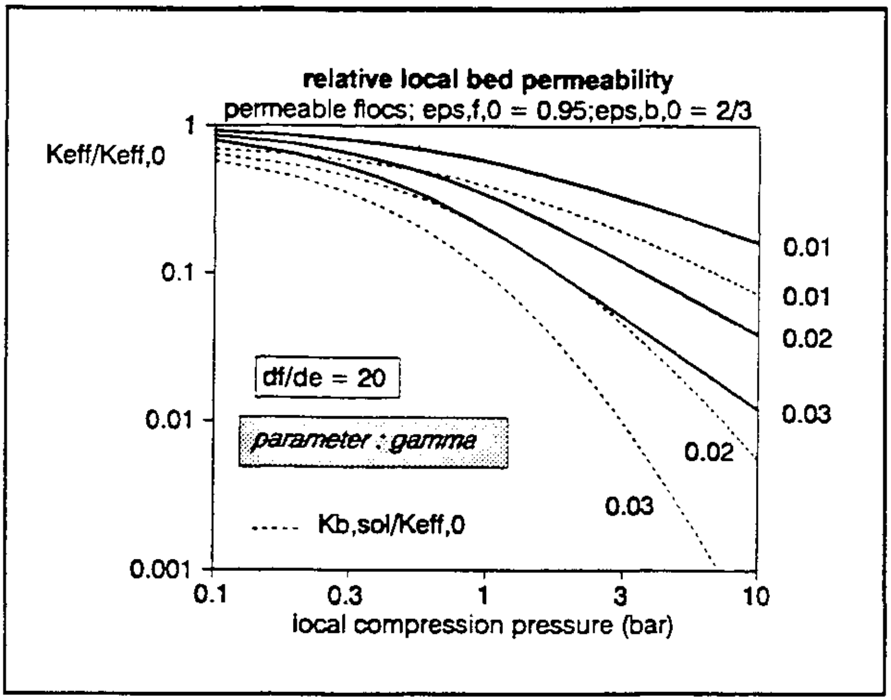


Fig. 9. Simulated effect of solid pressure on permeability of a bed of porous flocs. Small floc size.

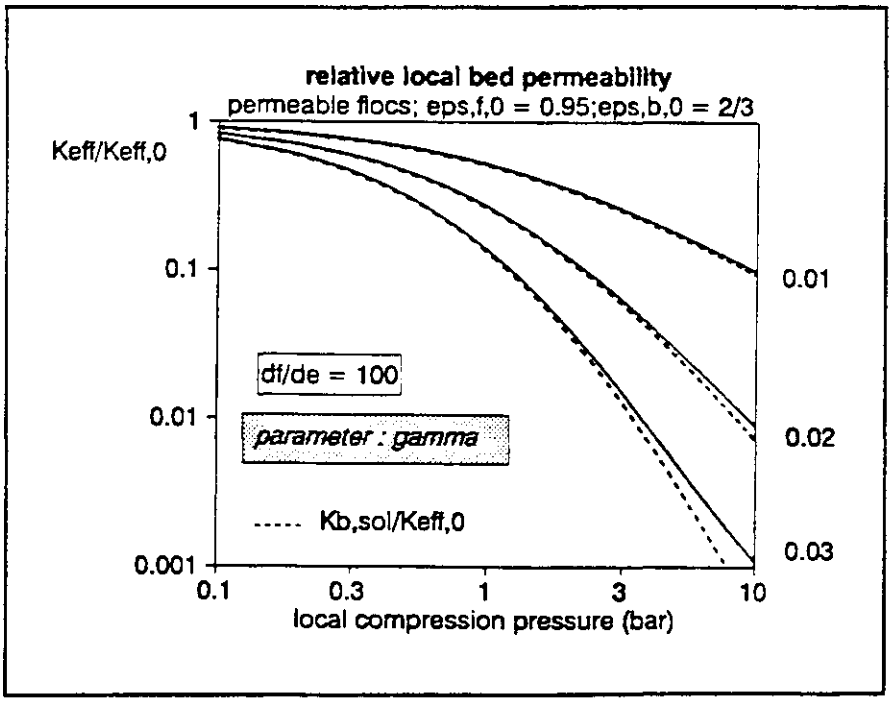


Fig. 10. Influence of the solid pressure on permeability of a floc bed. Larger flocs.

In conclusion we may state that this model is a good base for further refinement and experimental study. It will be especially of interest to investigate the separate deformations and porosity changes of flocs and beds, and the changes this brings in overall permeability.

2.2. Dynamic flow through compressible beds; application to floc expression

2.2.1. Description of theory

The theory of dynamic changes in pressure and porosity profiles during filtration and expression has been reported by several authors [2-4]. The basic equation for the porosity change in one-dimensional flow reads :

$$\frac{\partial \varepsilon}{\partial t} = - \frac{\partial}{\partial r}(u_l) \tag{13}$$

Herein r is the distance coordinate with respect to a fixed coordinate system.
We assume a modified version of D'Arcy 's law :

$$v_l - v_s = - \frac{K_f}{\eta \; \varepsilon} \left(\frac{\partial p_l}{\partial r} \right) \tag{14}$$

with v_l = *linear* liquid velocity [m/s]
v_s = *linear* velocity of solids [m/s]

The relation between the linear and superficial velocities reads :

$$v_l = \frac{u_l}{\varepsilon} \qquad v_s = \frac{u_s}{(1 - \varepsilon)} \tag{15}$$

Assuming no net volume production we have :

$$u_l + u_s = u_t \neq f(r) \tag{16}$$

Herein u_t is the total net convective flow per m^2, which for filtration is equal to the filtrate flow per m^2, and for expression is equal to 0.
Neglecting gravity terms we have for the force balance :

$$p_l + p_s = p_t \qquad \frac{\partial p_l}{\partial r} = - \frac{\partial p_s}{\partial r} \tag{17}$$

Combination of Eqs.(13) - (17) leads to the differential equation :

$$\frac{\partial \varepsilon}{\partial t} = \frac{\partial}{\partial r}\left(- u_t\, \varepsilon - (1 - \varepsilon)\, \frac{K_f}{\eta}\, \frac{\partial \varepsilon}{\partial r} \Big/ \frac{\partial \varepsilon}{\partial p_s} \right) \tag{18}$$

in which it is assumed that there is a unique relation between ε and p_s.

The boundary conditions depend on the type of operation. For filtration we have :

$$\begin{aligned} r &= 0 & - u_l &= R_m\,(p_{l,r=0} - p_0) \\ r &= R(t) & \varepsilon &= \varepsilon_0 \end{aligned} \tag{19}$$

with R_m = resistance of filter medium [m /Pa/s]
$r = 0$ place of filter medium
$R(t)$ = coordinate of cake front [m]

For the case of expression we have :

$$\begin{aligned} r &= 0 & \frac{\partial \varepsilon}{\partial r} &= 0 \\ r &= R(t) & \varepsilon &= \varepsilon^* \end{aligned} \tag{20}$$

with $r = 0$ coordinate of impermeable surface or of symmetry plane
$R(t)$ = coordinate of expression front [m]
ε^* = porosity in equilibrium with applied solid pressure [-]

The latter boundary conditions apply when the resistance of the medium with which pressure is applied is very low. In case there is a considerable resistance, a condition similar to Eq.(19) should be applied.

Solution of the differential equation can only be done numerically. Before doing that however a change of coordinates is necessary, taking the solids volume or mass as measure.

2.2.2. Application to expression of a sludge floc

As follows from the considerations of the DUAL FLOW model, for a theoretical description of a filtration or expression process of a bed one should know the changes both in bed and in floc porosity. Therefore it was thought interesting to model the expression of a floc. The system considered is that of a flat floc, which has a thickness 2 R_f, and is expressed symmetrically in the r-direction. The equations of 2.2.1. were transformed to solids volume coordinates, and a modified Crank-Nicholson finite difference scheme was used to solve the equations. For the effect of solid pressure on the equilibrium void ratio Eq.(11) was used. In Fig.11 the calculated porosity profiles for a floc with an initial thickness of 200 µm are given. Initially there is a very steep gradient at the expression surface, but in progress of time the porosity profile penetrates into the heart of the floc.

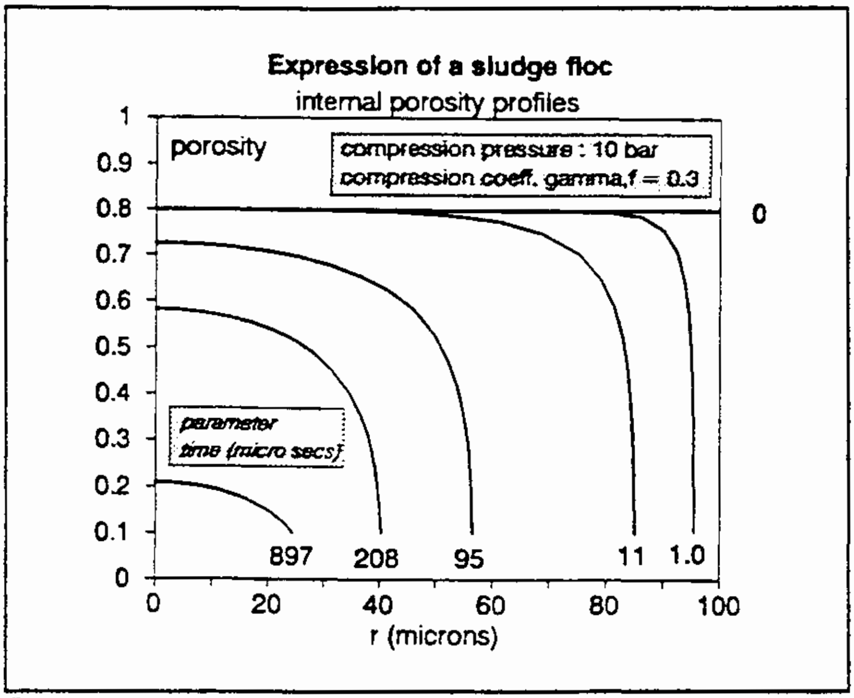


Fig. 11. Porosity profiles inside a floc during expression

In Fig. 12 we see the profiles of the solid pressure as this penetrates into the floc. we see that the pressure profile remains very steep near the floc surface, and only relaxes in the last phase of the expression.

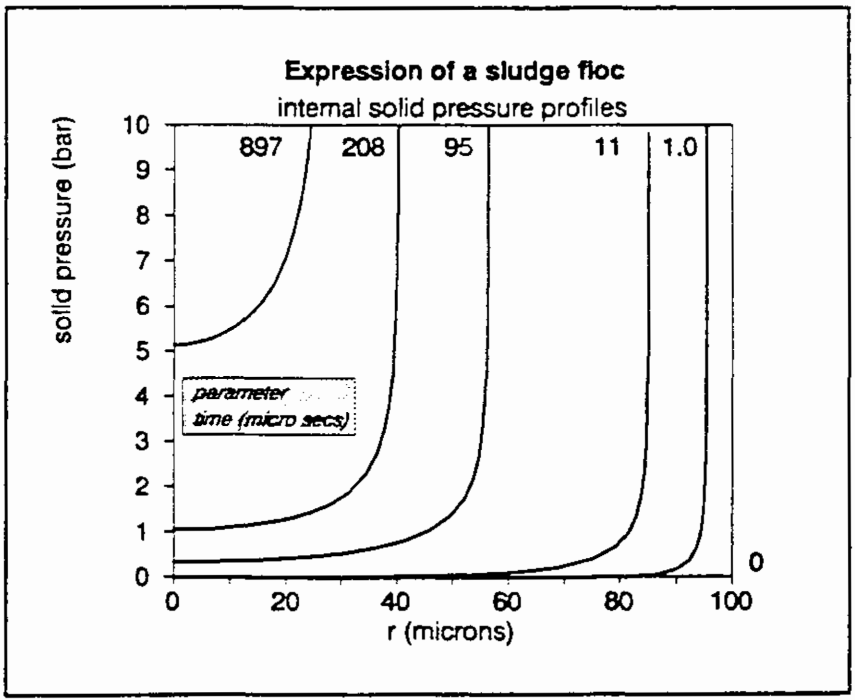


Fig. 12 Profiles of solid pressure inside a floc during expression.

In Fig.13 the average porosity of the floc is given vs the expression time. We see that for this case the largest part of the expression takes place within 1 ms. From this, and other simulations we conclude that within a larger scale filtration or expression the local floc porosity may be considered to reach equilibrium with the local compression pressure within a very short time.

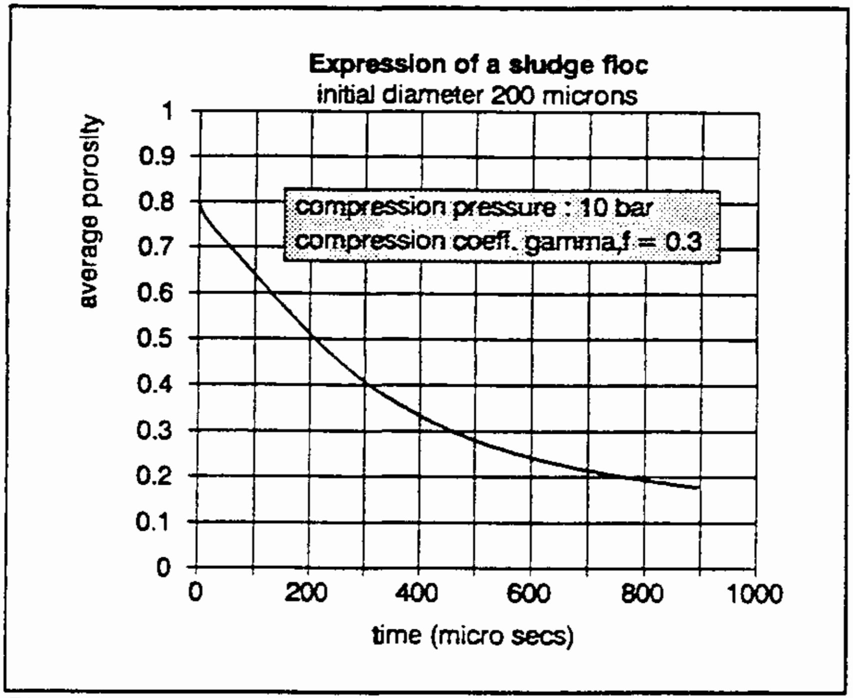


Fig. 13. Average porosity of a floc during expression in the course of time.

3. ELEMENTS OF FURTHER RESEARCH

3.1. Experimental

The general target for further experimental work may be formulated as :

obtaining insight into the relation between composition, treatment and physical properties of flocs and cakes.

As follows from the various considerations given above we may see as important physical properties :

1. Particle size distribution and particle morphology

These represent the initial conditions of the flocs as they enter the dewatering process. Next to laser-diffraction equipment also analysis of optical images and electron microscopy are important to get impressions of irregularities in shape, and of structural aspects.

2. Physico-chemical aspects

It is of great importance to lay the relation between the various factors influencing floc formation and the floc size, strength and structure. This means that measurements should include zeta-potential measurements for various additives, the measurements under 1. and rheological measurements. Important is also to study the properties in relation to different floc preparation methods. Maybe special methods should be devised to measure floc strength directly.

3. Water binding and transport

Since water may be present in various ways, it is of interest to determine the amounts. This may for a part be done by indirect methods. Experiments on freezing curves and the determination of water vapour sorption isotherms may provide information on the amounts of free and bound water. Drying experiments of filter cakes may be analysed in terms of the effective diffusion coefficient, which is related to the pore size distribution [5]. An example of some typical drying curves is given in Fig.14. The upper curve is for the more open part of a cake, the lower one for a sample near the filter. It is clear that the drying rate is influenced strongly by the differences in compression. Drying experiments thus offer a possibility of obtaining insight into the pore sizes and the liquid flow for cakes with different histories. Of course the information i also needed to design and optimize drying equipment.

4. Macroscopic water transport and filtration properties

A systematic investigation is needed into porosity, permeability and deformation of flocs and beds, as influenced by the initial floc properties and the process variables. One aspect is to carry out standard tests, such as the Capillary Suction Time, the (Modified) Filtration test. However for better understanding special experimental methods have to be devised to measure properties of cakes under homogeneous pressure conditions. Also it should be verified whether indeed the rate-controlling factor for cake and floc deformation is the time needed for water displacement, or whether purely mechanical properties may also play a role. Experimental investigation of the DUAL FLOW model could be done with model systems.

5. Filtration and expression rate

In order to verify more fundamental models, of course filtration and expression experiments under various process conditions should be performed, both in dedicated laboratory equipment and in well-instrumented practical scale equipment.

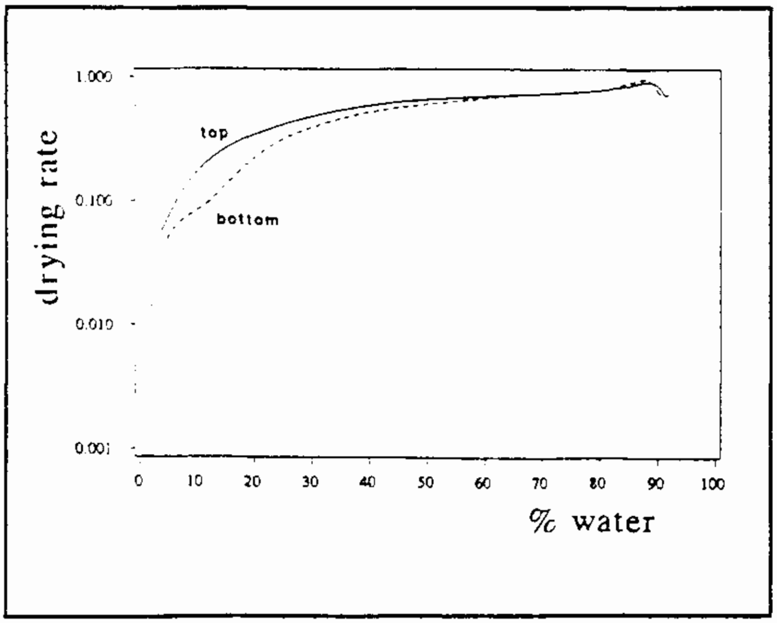


Fig. 14. Drying curves of different parts of a filter cake

3.2. Theoretical modelling

- The DUAL FLOW model must be tested and possibly refined.
- For the description of filtration and expression dynamics also the transport through the flocs should be accounted for.

□ As in reality filtration is not a one-dimensional process, a model should be formulated for the transport and deformation in 2- and 3-dimensional situations.
□ Models for the strength of flocs as dependent on structure may be set up, as related to efforts in food and polymer technology.
□ Models for the analysis of drying experiments will provide insight into the liquid motion in small pores.
□ For the various standard tests more detailed models must be made in order to relate the macroscopic outcome to local properties and dynamics.
□ Models for the binding of water must be compared with the outcome of freezing and sorption isotherm experiments.
□ Equipment models must finally be made for the description of large scale dewatering. These models can then be used for analysis of existing situations and as a simulation tool for the definition of process conditions or the design of new equipment.

4. FINAL REMARKS

Although considerable work has been carried out in the past, it is clear that on the fundamental side still a lot is to be done. As some new elements in this paper the DUAL-FLOW model was introduced, and a consideration of the expression of a floc.
In the framework as sketched here I think it is possible to make a link between all kinds of tests, fundamental properties, and theoretical descriptions for practical dewatering processes. This will lead to a much larger predictability of these processes, and combined with the insight from the physico-chemical effects to practical solutions in difficult situations.

References

1. Bird, R.B., Stewart,W.E. & Lightfoot, E.N., '*Transport Phenomena*', Wiley, 1960
2. Tiller,F.M. & Shirato,M., 'The Role of Porosity in Filtration: VI. New Definition of Filtration Resistance', AIChEJ, **10** (1), 61 - 67, 1964
3. Shirato, M., Sambuichi,M., Kato, H. & Aragaki, T., 'Internal Flow Mechanism in Filter Cakes, AIChEJ, **15** (3), 405 - 409, 1969
4. Wakeman, R.J., 'A numerical integration of the differential equations describing the formation of and the flow in compressible filter cakes', TransIChemE, **56**, 258 - 265, 1978
5. Krischer, O. & Kast, W., '*Die wissenschaftlichen Grundlagen der Trocknungstechnik*', 3d ed., Springer, 1978

SLUDGE MANAGEMENT IN JAPAN

M. Ibuki
Environmental Systems & Equipment Division, NGK-Insulators Ltd.

TABLE OF CONTENT

1. INTRODUCTION

In Japan, the sewer system and its treatment systems have been rapidly installed in the last 30 years. In 1990, the population equiped with the sewer sytem reached 40 % of total population. Central government plans the sewered population will be 54 % in 1995 and 70 % in 2000, respectively.

The construction of sewer piping and municipal wastewater treatment plant results in the increase of sewage sludge. Statistical data shows that annual sludge generation amounted to 240 million tons (0.5 % solid basis) in 1988, and increased by 7 % a year.

Treatment and disposal of municipal sewage sludge in a safe manner are very important to protect our environment. Currently, most local municipalities dispose sludges as a form of either dewatered cake or incinerated ash. For local municipalities, landfill disposal of sludge becomes much difficult in finding sites and in controlling secondary pollutions. Incineration, which enables to reduce the volume of sludge and to stabilize the organic compounds, has widely spreaded in Japan. Currently, approximately 54 % of total sludge is treated by incineration. This number is higher than European countries and USA. Incineration is suitable in highly populated area and complies with stringent regulations for aquous environment. Though incineration requires costs for construction and operation, the Japanese social situation will make incineration more popular in the future.

This paper describes the overview of sludge handling, and focuses on the incineration. Three types of incinerators are explained, Multiple Hearth Furnace, Fluidized Bed Incinerator, and Melting Furnace. From the environmental view point, concerns arise for the control of gaseous emissions and leaching from residues. Drying process prior to incineration is recommended to improve the energy efficiency, and to enhance the capacity.

2. SLUDGE TREATMENT OVERVIEW

2-1. SLUDGE TREATMENT SYSTEM

The sludge treatment system consists of several units processes, such as Thickening, Digestion, Conditioning, Dewatering, Drying, Composting, and Incineration. Among these processes, most POTWs have Thickening, Conditioning, and Dewatering processes. On the other hand, the selection of Digestion, Drying, or Incineration depends on each specific case. Table-1 shows the breakdown of sludge treatment system and its number of POTWs in Japan. 14 % of POTWs have Incineration, while 54 % of generated sludge is incinerated. This means Incineration is mainly applied into large scale POTWs. Drying is adapted as a pre-treatmant of composting or incineration. Composting has only 4 % share, due to marketing problems. Figure-1 summarizes sludge treatment system.

2-2. THICKENING

Gravity thickening is still popular in Japan as shown in Table-2. Recently, mechanical thickening has been applied mainly in waste activated sludge, which is difficult to separate its moisture. Dr.Shimizu has calculated the energy consumption of total sludge treatment system, and emphasized the importance of thickening process. The selection of mechanical thickening brings distinct energy saving to the total sludge treatment system.

2-3. DIGESTION

Effects of the conventional anaerobic digestion were disinfection, recovery of methane gas, and reduction of solid amount. On the contrary, the heat energy is necessary to enhance sludge temperature, and the dewatering performance is often deteriorated. On energy view point, the amount of methan gas is generally not enough to maintain the temperature of digestion tank. Several alternatives are proposed and investigated such as,

(1) Two phase digestions (aerobic and anaerobic digestions)
(2) Addition of chmicals or enzymes to improve gasfication ratio
(3) Mechanical thickening to decrease the energy to heat up

2-4. CONDITIONING AND DEWATERING

The conventional inorganic chemical dozing has been changed into the organic polymer dozing, together with the selection of dewatering machine. Since considerable amount of inorganic chemicals (Ca, Fe) remains in the dewatered cake and the incinerated ash, the inorganic dozing causes the increase in the disposed volume. As shown in Table-3, the vacuume filter with inorganic dozing has been replaced by the belt filter press with organic dozing.

3. CHARACTERISTICS OF SEWAGE SLUDGE

The sewage sludge consists of water and dry solid. Dry solid is devided into organic (combustible) compounds and ash compounds. Here, the sludge means the mixed sludge of primary sludge and waste activated sludge. The amount of the primary sludge and that of waste activated sludge was formerly equal, but recently the ratio of waste activated sludge is increasing. Therefore, the ratio of organic compounds in dry solid is also rising, and its value is now 70-80 %. Figure-2 shows the yearly change in the generating ratio and the composition of dry solid at the sewage treatment plant in the Tokyo Metropolitan District. This rising trend of organic compounds has brought two phenomena which have affected sludge treatment practice. One is the difficulty of solid/liquid separation (at thickening and dewatering), and the other is a rise in the sludge calorific value (at incineration).

Murakami,PhD has investigated the thermal characteristics of dewatered cake at sewage treatment plants. Various types of sludges were gathered according to the sewer collection system (separated or combined), digestion, conditioning (inorganic or polymer), and dewatering machine. Consequently, one interesting classification was found with conditioning process. Figure-3 shows the relation between the Volatile Solids and the Higher Calorific Value (HCV) of dry solid. HCV of polymer dozing cake is larger than that of lime dozing cake. Following equations are proposed to explain the relation.

Polymer Cake ; $HCV = 58.3 \times VS - 193$

Lime Cake ; $HCV = 56.4 \times VS - 513$

where, HCV ; Higher Calorific Value of Dry Sloid (Kcal/Kg-D.S.)

VS ; Volatile compounds inDry Solid (%)

At the incineration process, the sludge calorific value is the factor to determine the auxiliary fuel consumption. The Lower Calorific Value of dewatered sludge is calculated by the following equation,

$$LCV = SOL. \times (HCV - HYD. \times 18/2 \times LAT.) - (100 - SOL.) \times LAT.$$

where, LCV ; Lower Calorific Value of Deawtered Wet Cake (Kcal/Kg-Wet)

SOL.; Solid Content in Dewatered Wet Cake (%)
HYD.; Hydrogen Content in Dry Solid (%)
LAT.; Latent Heat of Moisture (appro. 600) (Kcal/Kg)

Table-5 summarizes the sludge characteristics in Japan according to the Conditioning and Dewatering Machine. If LCV equals zero (0), it means that the sludge has the calorie to evaporate its moisture at 0° C. Usually, autogeneous combustion point of incinerator itself is the LCV of 600 to 900 Kcal/Kg-Wet, where exhaust gas temperature is 300 to 500 ° C.

4. SLUDGE INCINERATION

4-1. ADVANTAGES OF INCINERATION

Advantages of incineration are the reduction of volume and the stabilization of organic compounds. Moisture is evaporated, and most organic compounds are decomposed and turned into stable gaseous forms. Figure-4 explains the volume reduction effect through incineration, and the dewatered sludge cake is reduced into 6 % by weight and 10 % by volume, respectively. This volume reduction effect means advantages in reducing (1) disposal site cost, (2) transportation cost to disposal site. Furthermore, if the melting furnace is employed, more margin of volume reduction effect can be obtained.

In spite of these advantages, careful considerations should be taken for planning the incinerator. One is the economical evaluation compared with other options. In Japan, the disposal cost is generally high compared to the burden of construction and operation cost. The other is secondary pollution caused by incineration, such as the gaseous emissions and the leaching of toxic materials from incinerated ash.

4-2. TYPES OF INCINERATOR

There are two types of incinerator commonly used in Japan, Multiple Hearth Furnace (MHF) and Fluidized Bed Furnace (FBF).

The first incinerator of MHF was installed in 1965, and MHF was dominating the market in the 1960s to the 1970s. That is, for the incineration of deawtered cake which has high moisture content and low calorific value, MHF was most suitable as far as structure and function. MHF has also been modified and improved to meet the time-to-time demands, such as the energy-saving and the prevention of heavy metal leaching. Some of these improvements are Pyrolysis, Starved Air mode operations, and Re-Circulation of exhaust gas.

In the 1980s, the conditioning process and the dewatering machine were changed. The Previous Lime-dozed vacuume filter was replaced by a Polymer-dozed belt filter press. This change resulted in the increase of sludge calorific value. In some cases, the calorific value of sludge exceeded the self-sustained combustion point, and caused the rise in furnace outlet temperature. It was diffcult to incinerate the high calorie cake with MHF.

In the 1960s to the 1970s, FBF was applied only in small scale applications, in which the intermittent operation was required. FBF had also the disadvantage that it needs more auxiliary fuel than MHF. In the 1980s, to incinerate the high calory cake smoothly, FBF had taken the place of MHF. The scale-up of the Furnace, and the pre-drying to save the energy had been accomplished. Nowadays, most of the newly installed incinerator are FBFs.

Melting Furnace is the newest type of incinerator, where the furnace temperature exceeds the melting point of ash. Melting Furnace can realize the maximum volume reduction ratio. Many furnaces have been developed, and some of them are currently being operated.

4-3. REGULATORY FRAME

The exhaust gas from the incinerator and the leaching of heavy metals from ash are of concern. Table-5 & 6 are regulations set forth in the exhaust gas of sludge incinerator and the leachate from ash.

To comply with emission regulations , gas treatment devices are applied together with improvements of incinerating conditions. For acid gases (SOx, HCl) and dust, most incinerators in Japan are already combined with the alkali scrubber and the electro static precipitator. Odor problems are serious in case of MHF incinerating the polymer dozed cake. After-burning of exhaust gas (exit gas of scrubber) or catalytic combustion is sometimes applied. Though the mechanism of NOx formation is not completely clear yet, data shows the lime dozed cake generats higher NOx than the polymer dozed cake in FBF. Some incinerators equip a catalytic converter.

Table-7 is the limit of leaching, when sludge is disposed on a landfill site. If the sludge does not meet this regulation, sludge shall be disposed in the engineered barrier (usually concrete structure) with treatment devices for leaching water. In Japan, some POTWs, which receive industrial wastewater, are concerned with chromium leaching.

5. MULTIPLE HEARTH FURNACE

5-1. STRUCTURE AND FUNCTION

The MHF is a cylindrically rigid multi-hearth type supported by a column as shown in Figure-5. Inside the steel shell is lined with heat-insulating bricks and heat-resisting bricks. Furnace is devided into a number of hearths by the floors of fire bricks. The center shaft, which is double piped structure cooled by air, are provided from the bottom to the top of furnace. Two or four sets of rabble arms are attached in each hearth to stir and feed sludge. The exit of gas and the inlet of sludge cake are provided on the top of furnace. The hot air, together with combustion gas of auxiliary fuel, is introduced from the main burner.

The sludge is fed at the periphery of the top hearth, scattered over the furnace floor by the rotation of the arm, and makes a spiral movement toward the drop hole at the center. On the second hearth, sludge is moved outward to the holes at the periphery of the hearth. (see Figure-6) The alternating location of the drop holes and the alternating spiral direction of sludge movement ensures a long retention time of sludge.

The furnace consists of four zones (shown in Figure-7). The top of the furnace is the first zone of drying , where most moisture in sludge is evaporated. The second zone is the combustion zone, where the remainig moisture is completely evaporated and most organic compounds are combusted. Usually, the temperature is the highest in this zone. The third zone is the carbon combustion zone to combust the remaining carbon in sludge. The incineration itself is finished in this zone, and the sludge is turned into innert ash. The last zone is the ash cooling zone, and the ash is cooled by the combustion air.

The features of MHF are as follows,

(1) The retention time of sludge is approximately one (1) hour, and this long retention time enables the flexible operation for the fluctuation of cake feed rate.

(2) Relatively low dust content in the exhaust gas.

(3) Less horse power is required compared with FBF.

(4) Less fuel oil consumption, unless the odor is of concern.

5-2. STARVED AIR COMBUSTION OF MHF

In early days, MHF was operated at the air-ratio of 2.5 (2.5 times of theoretically required amount). This conventional air-ratio was selected in order to increase the capacity of the furnace and to prevent the clinkers caused by local heating. But, such operation of high air-ratio had demerits of,

(1) exhaust-gas volume increase.

(2) large gas treatment devices were required.

(3) extra auxiliary fuel was necessary to heat up the surplus air.

Since the oil shock of 1972, strong demand has risen to reduce the auxiliary fuel consumption at incinerators. The starved air combustion has been applied into many plants to reply to this demand and to solve the demerits of the conventional operation mentioned above.

The starved air combustion is the operation under the low air-ratio of 1.5 to 1.8. The existing MHF was modified for air tightness. The simple reduction of the combustion air caused these difficulties asfollows,

(1) Clinker trouble

The temperature of the combustion zone is increased by reducing the excess air amount. If the temperature of the combustion zone exceeds 950° C, some ash compounds start to melt.

(2) Decrease in drying performance

In the drying zone, dewatered cake contacts with the combustion gas. The evaporating rate is represented by the equation of,

$$Rd = A \times G^{0.8} \times Tm$$

Where, Rd ; Evaporating Rate (Kg-water/m2 Hr)
A ; Rate constant
G ; Gas Weight Velocity (Kg-gas/m2 Hr)
Tm ; Averaged temperature difference between gas & sludge

If the air-ratio is reduced from 2.5 to 1.5, the gas weight velocity is reduced by 40 %, and this results in the decrease of the evaporating rate by 35 %.

To solve these conditions, the exhaust gas circulation system was employed.

The exit gas of either the furnace or the scrubber was re-circulated into the bottom of the furnace. The average gas temperature in the combustion zone is decreased by the diluting effect of the ciculated gas, and the sufficient gas amount is provided. Table-8 explains the effect of the exhaust gas circulation, concerning the temperature of the combustion zone and the evaporation rate in the drying zone. The starved air combustion was establised, and many existing MHFs were modified. Figure-8 shows the flowsheet of the starved air combustion.

The starved air combustion can reduce the amount of exhaust gas and the amount of auxiliary fuel. Figure-9 explains these efects compared with the conventional operation.

5-3. PYROLYSIS

Most Japanese sewer systems receive industrial wastewater. In the 1970s, the tanning and metal-finishing industries caused the problem of chrome compounds in sewage sludge. When the sludge containing chrome compounds (usually as a form of tri-chromium) was incinerated, chromium was oxidized into toxic hexavalent chromium and remained in the ash as such. The concern was the possibility of leaching of hexavalent chromium from ash at the disposal site. The solution for this problem was urgently needed. A number of methods have been proposed, and the pyrolysis system is the one most frequently used in actual applications.

Generally, the word of 'Pyrolysis' is used for the thermal decomposition of organic materials under the absense of oxygen. In this paper, the 'Pyrolysis' means the thermal decomposition of sludge under the reduction atmosphere. And sludge is turned into the pyrolysis gas, the tar, and the fixed carbon.

5-3-1. Laboratory Test Results

Laboratory tests have been carried out to prove the effectiveness of pyrolysis. Figure-10 shows the test results. The experiments on thermal decomposition of sludge under various atmospheres revealed the following facts on the behavior of chrome.

(1) In case of oxygen free atmosphere, no hexavalent chromium is formed.

(2) In case of partial combustion, in which the oxygen/nitrogen ratio is less than normal air and the part of pyrolysis gas is inflamed, no hexavalent chromium is formed during the period, in which a reducing atmosphere is

maintained by the remaining pyrolysis gas. The formation of hexavalent chromium dose starts when pyrolysis gas disappears.

(3) Hexavalent chromium is formed under incineration atmosphere.

Also, the formation of hexavalent chromium is substantially facilitated by the existance of calcium, which is usually used in the tanning industries and is often added to sludge at the conditioning process.

It has been confirmed that the oxidation of tri-chromium into hexavalent-chromium does not occur in the oxygen-free atmosphere or the partial combustion atmosphere, as long as the reactor is maintained in non-oxidizing (reducing) condition.

5-3-2. Pilot Plant Test

The pilot plant was installed to confirm the performance of pyrolysis. Prior to MHF, dewatered cake was dryed by the indirect steam dryer in order to decrease the moisture content to 30 - 40%. MHF had the capacity of 2 tons/day, and was designed to enable to change the air ratio for sludge combustibles. Figure-11 shows the chromium behavior according to various air ratio. Under the reducing atmosphere (air ratio < 1), the formation of the hexavalent chromium is prevented. Through the pilot plant test, the following design criteria of MHF is obtainted,

* Air ratio for sludge combustibles	0.4 to 0.8
* Capacity of MHF	25 Kg-Dry Solid/m2 Hr
* Maximum temperature	900 ° C
* Moisture content of inlet sludge	30 to 40 %

Another important point in the test is the gasification ratio of sludge combustible. To use the heat energy of sludge effectively, this ratio should be high. Combustible compounds consist of the volatile matters and the fixed carbon. The gasification of the fixed carbon react mainly by equations of,

$$C + H_2O \rightarrow CO + H_2$$

$$C + CO_2 \rightarrow 2CO$$

In case of sludge pyrolysis, Ca and Fe compounds in sludge cake catarytically facilitate those reactions. Consequently, the sludge pyrolysis can achieve similar decomposition rate to the conventional incineration.

5-3-3. Drying-Pyrolysis System

The Drying-Pyrolysis System has been established and applied into several plants in Japan. Figure-12 shows the flowsheet of the system. First, the dewatered sludge cake is fed to the dryer, then the dryed cake with the moisture content of 30 to 40 % is fed to the pyrolysis furnace which is usuallly a four hearth MHF. The indirect steam heating dryer has been adapted for this process. The advantages of using the dryer are,

(a) Most of sludge moisture is evaporated at the dryer, the following pyrolysis furnace can be compact.

(b) The temperature of the evaporated moisture at the dryer is much lower than that at pyrolysis furnace, and the large energy saving can be expected.

The exhaust gas from the dryer, which has high humidity, is water-scrubbed and induced into the furnace as the partial combustion air.

In the pyrolysis furnace, the combustibles in sludge are gasified and 40 to 60 % of generated gas is combusted in the furnace. Here, the ash compounds are kept under the reduction atmosphere, and the formation of toxic hexavalent chromium is prevented. The exhaust gas from the pyrolysis furnace still contains the combustible gas compounds, and it is completely burned out at the after-burner with the air-ratio of 1.2 - 1.3 at the temperature of 900 ℃. The combusted gas is then introduced into the waste heat boiler to recover the steam which is used at the dryer and the heat exchanger. The exhaust gas from the boiler is cleaned up through the gas treatment devices.

The Drying-Pyrolysis System can solve the chromium problem in some areas of Japan, and also reduce the auxiliary fuel amounts compared with the conventional incineration. The fuel oil amounts of the Drying-Pyrolysis System and the conventional incineration are calculated and shown in Figure-13.

6. FLUIDIZED BED FURNACE

6-1. STRUCTURE AND FUNCTION

The FBF consists of a rigid cylindrical main body which is covered with a steel shell. Inside the shell, the wall is lined with the fire-resisting bricks and the heat insulating bricks. The furnace is devided into two sections, the vacant free board on the top and the sand layer on the bottom. In the middle of the sand layer, the air feeding pipes are arranged to distribute the combustion air. The sand layer above the pipe is called the sand bed, which achieves the heat transfer and mixing media during the operation. The lower part of the layer is the fixed bed (dead space) in ususal operation. The feeder of sludge and the auxiliary fuel burner (and/or oil gun) are installed on the side wall.

The combustion air is usually pre-heated at the heat exchanger, in which the exhaust gas of FBF is induced. Fed sludge is radically crushed and stirred in the bubbling sand bed. The moisture of sludge is evaporated and the combustibles are gasified in a short time. Most of the gas is combusted in the sand bed and remaining is in the free board. The incinerated ash is in the form of very fine particles and transfered with the exhaust gas flow. The auxiliary fuel oil is used to control the sand bed temperature between 700 - 800 ° C.

Main design criteria of FBF is listed below.

* Air Ratio for sludge	1.3
* Free Board Temperature	800 - 850° C
* Combustion Air Temperature	400 - 600° C
* Free Board Gas Velocity	0.8 - 1.0 m/sec.
* Sand Bed Height	appro. 1.0 m (at stoppage)
* Capacity	170 - 200 Kg-moisture evaporation/m2 Hr.

These values are available for the sewage sludge combustion, which moisture range is 65 % or more.

The features of FBF are as follows,

(1) Higher Incineration Performance

Since the sewage sludge and high temperature sand are stirred and mixed rapidly, the drying and the combustion of sludge can be finished quickly and more completely.

(2) Clean Exhaust Gas

The combustion gas is retained for several seconds under 800 - 900° C. The volatile compounds in exhaust gas are completely burned out. Odor of exhaust gas is extremely low.

(3) Easy Intermittent Operation

Since the thermal capacity of the sand bed is large, the temperature drop after stopping the operation is slow. The intermittent operation (day time operation) can be achieved easily.

6-2. FBF INCINERATION SYSTEM

Two different incineration systems has been used for FBF. One is the Direct FBF system and the other is the Drying FBF system. Figure-15 shows the flowsheet of these two systems. The furnace temperature and the air-ratio are essentially the same between the two systems. The selection of the system depends on each specific case, considering the capacity of the furnace and the operation schedules,etc..

In the Direct FBF system, the dewatered sludge cake is directly fed into the furnace. The exhaust gas from the furnace is heat-exchanged at the primary and secondary heat-exchangers to increase the temperature of the combustion air and the air for the mist prevention (plume control), respectively. The exhaust gas is introduced into the gas treatment devices, which include the Cyclone, the Scrubber, the Alkali Absorption Tower, and the Electro Static Precipitator. Finally, the treated gas and the hot air (from the secondary heat exchanger) are mixed and discharged from the stack.

In the Drying FBF system, prior to the furnace, part of the dewatered cake is dryed by the indirect steam dryer, and mixed with remaining part. The feeding rate and the dryed cake moisture are controlled to maintain the furnace in the self sustained combustion range. The exhaust gas from the furnace is led to the heat exchanger, where the combustion air is heated up to 400 ° C. Then, the exhaust gas is induced into the waste heat recovery boiler, which generates the steam used for the indirect steam dryer, etc.. The exit gas of the boiler is treated through the same gas treatment devices as the Direct FBF system. Finally, the pre-heated air from the steam air heater is mixed with the exhaust gas to prevent mist, and discharged into atmoshere.

The selection of the system depends on each specific case, such as the plant scale, the sludge characteristics, the economical conditions (including fuel oil price), and the operation schedule. The environmental impacts, such as the ash characteristics and the exhaust gas quality, are the same between the two systems. Figure-16 shows the auxiliary fuel oil consumption of FBF systems, according to various moistures and calorific values of the dewatered cake. In the Drying FBF System, the moisture evaporated in the dryer is discharged at the temperature of 100° C, and the remaining moisture evaporated at the furnace and discharged at 250 ° C after the boiler. On the other hand, the moisture in the Direct FBF System is dischaged at 450 ° C. This temperature difference realizes the substantial fuel oil saving in the Drying FBF System.

Though the Drying FBF system can save fuel oil as shown in Figure-16, it exhibits higher initial cost due to the cost of the dryer and boiler. Therefore, the Drying FBF system is economical in large scale applications and for relatively low calory sludge.

6-3. LARGE SCALE APPLICATION

6-3-1 Background

As mentioned above, FBF option was not been popular in the 1970s. In those days, large cities selected mainly MHF, in which there were already a lot of large scale applications and MHF required less fuel comsumption with lower furnace exit gas temperature. On the other hand, FBF was applied in small scale plants and had excess fuel consumption.

In the 1980s, the sludge conditioning process had been changed from the inorganic dozing into the polymer dozing and the Belt Filter Press replacing the conventional Vacuume Filter. As a result of these tendencies, the lower calorific value of the dewatered cake had increased drastically.

In MHF, the furnace temperature rose and this caused problems of,

(a) Clinker formation in the combustion zone

(b) Evaporation of organic compounds in the drying zone

Though MHF was improved in a manner of the starved air combustion and the extraction of exhaust gas from the combustion zone, the operation of MHF became difficult and complicated.

In FBF, the fuel oil consumption has been reduced by the increase of sludge LCV and the progress of heat recovery equipment. The remaining drawback to FBF was scale.

6-3-2 Large Scale Drying FBF Sytem

The largest fluidized bed incinerator for sludge was constructed at the Shingashi plant of the Tokyo Metropolitan District in 1986. Two units of the indirect steam dryer are equipped prior to the fluidized bed furnace. A part of dewatered cake is fed into dryers to reduce its moisture to 40 - 50 %. The dryed cake is mixed with remaining dewatered cake, and the moisture of mixture is adjusted within the range of 70 - 75 %. The system is designed to treat the dewatered cake of 250 Tons/Day. The specifications of the main parts and the design conditions are shown in Table-9 & 10. The fluidized bed furnace has a diameter of 6200 mm at the sand bed. Though, in a small plant, sludge is fed into the furnace by a screw feeder, the cake spreader (Figure-17) was developed in this plant to distribute sludge on bed equally.

Table-11 shows the actual operation data of this plant, and the data is summarized in Figure-18. In this figure, the actual fuel consumption is lower than the design value. From the actual data, LCV for autogenous combustion is calculated as 320 Kcal/Kg-wet. That value is substantially lower than the conventional MHF of 620 Kcal/Kg and the Direct FBF System of 900 Kcal/Kg. The bounds of this autogenous combustion points are shown in Figure-18.

6-4. ECONOMICAL ASPECTS FOR ENERGY RECOVERY

Recently, the improvement of the heat recovery equipment and the increase in the sludge calorific value may suggest the future direction of FBF system, which will be not only the treatment facility but the energy recovery system to supply heat or electricity. Kasakura,PhD has engaged in the economic evaluation to verify the merit of the energy recovery system. He has compared four (4) systems. Two (2) of them are planned to recover the electricity from the steam which is generated at boiler, and the other two (2) systems are designed only for treatment use as shown in Table-9. The economical analysis has been carried out, considering the initial cost, its annual amortizing cost, the operation and maintenance cost, and the income of energy supply. When the local municipality constructs the incineration system, a certain part of the initial cost is supported by the central government of Japan. The calculation result shows that the the systems with electrical generation has the economical advantage, when the local municipality can receive large subsidy. But, systems without electric generation are low cost in case of the lower subsidy.

7. MELTING FURNACE

The melting furnace, which inside temperature exceeds 1350 to 1400° C, melts the ash in sludge into the liquid form. In last 10 years, the melting furnace of sludge has been the object of many discussion. Central government and local municipalities have conducted a number of reseach projects together with private sectors. Several commercial plants are currently operated in Japan. Advantages of the Melting Furnace are,

(1) The maximum volume reduction can be obtained.

(2) Slag is stable, and the leaching of heavy metals can be completely stopped.

(3) Slag can be used as construction materials.

The Melting Furnace has been interested by major cities, who are troubled with the shortage of disposal site. Though the melting furnace can solve the sludge disposal problems, such high temperature operation is still the obstacle for decreasing the fuel consumption, and establishing the O & M manners.

7-1. MELTING PROFILE OF ASH COMPOUNDS

Table-13 shows composition, basicity, and melting point of ash compounds in sewage sludge. The ash composition of sludge is generally affected by the life style of inhabitants, the sewer system, the soil condition, and the conditioning process in sludge treatment. Among them, the conditioning process is a factor to affect it most significantly. For inorganic dozing, Ca and Fe are added at the conditioning process and remains in ash. Basicity is given by ($CaO/SiO2$) . The melting point is the temperature at which a circular cone of ash halves its height.

To evaluate the melting characteristics practically, the Flow-Out Test (shown in Figure-20) is proposed. Considering an actual operation of the melting furnace, the temperature is kept at constant point. The viscosity of melted slag at this temperature is a key to successful operation. The highest flow-out ratio is obtained at the basicity of 0.8 to 1.0. Usually, the inorganic dozed sludge has higer basicity than that of suitable operation range, and the polymer dozed sludge has lower basicity. Therefore, the melting additive is necessary to adjust the sludge basicity. Pumice (silica based glassy sand) is used for the inorganic sludge, and lime is used for the polymer sludge, respectively.

7-2. TYPES OF MELTING FURNACE

There are several melting furnaces currently developed as in Table-14. For all furnaces, the melting of either the dryed cake or the incinerated ash is applicable. Since there are few commercial plants and relatively short term of operation, it is difficult to discuss the comparative superiority of each type.

The principle and feature of furnaces are described below.

7-2-1. Cyclone Melting Furnace

The cyclone furnace was originally developed several decades ago, and used for the coal fired boiler. The cyclone furnace is usually devided into three portions, primary, secondary, and tertiary chambers. The configulations of each chamber are different among manufacturers.

Dewatered sludge cake shall be dryed to 10% moisture content, and crushed. The cake and the combustion air is injected tangentially into the primary combustion chamber (so-called cyclone) to make a spiral flow. The sludge combustible is burned immediately and ash is melted by the combustion heat. The furnace is kept at the temperature of 1400 to 1500° C, and the melted slag is captured on the inner wall and flowed down. In the secondary chamber, the slag from the primary chamber is discharged from the gate. The combustion gas collides against the wall, where the slag, in mist form, is also captured and flows down to the gate. In the tertiary chamber, the combustion gas is mixed with a part of dryer exit gas to decompose its odor compounds. The upper part of the chamber is equipped with a heat recovery boiler to generate steam for the drying equipment.

The cyclone melting furnace is compact furnace and utilizes sludge calorie effectively. On the other hand, the pre-treatment process (drying and crushing) become larger than other types.

7-2-2. Film Surface Melting Furnace

The film surface furnace consists of the primary & secondary combustion chambers. Feeding sludge cake shall be dryrd to approximately 20% moisture content. The primary combustion chamber has a double cyclindrical structure of vertical type. The outer cylinder and the bottom bed rotate at 1 to 4 revolutions per hour. Fed material (dryed cake or ash) is filled in the vacant space between the outer and inner cylinders, and conveyed towards the center by the rotation of the outer cylinder. As cake is exposed to high temperature in the center, moisture and volatiles are gasified. Ash compounds are melted and flowed down into the secondary combustion chamber. The secondary combustion chamber is located below the primary chamber, and its function is to complete combustion and melting. The slag is water quenched and discharged by the slag conveyor.

The film surface furnace features the flexibility to treat wide range of sludge property and feedrate. However, the structure of the furnace is relatively complicated than others.

7-2-3. Coke Bed Melting Furnace

The coke bed furnace is widely used as the cupola furnace in steel industry, and applied into sludge treatment field. The furnace is a vertical cylinder. Sludge with 50 % moisture and coke are fed to the furnace one after the other. The primary combustion air is supplied to the bottom of the furnace. The sludge is heated by gas from the bottom, and finally melted at high temperature of coke combustion. The upper portion of the bed is called as free board, in which the secondary air is injected to complete the combustion.

The moisture content of the fed cake can be higher than that of other furnaces. So, drying equipment prior to the furnace shall be smaller than others. The furnace requires at least a fixed amount of coke to maintain a coke bed. Usually, the furnace generates more heat than required for dryer or other devices.

7-2-4. Arc Melting Furnace

The arc furnace is also widely used in the steel industry. Though the furnace can treat dryed cake in principle, the drying and the gas treatment equipment become larger and expensive. Usually, the furnace is recommended for the treatment of incinerated ash.

When electricity is loaded to graphite electrodes, arc discharge occurs across electrodes and base metal. Ash is heated and melted at selected temperature.

The arc furnace has advantages of higher temperature and lower exhaust gas. However, the energy cost is expensive compared with other furnaces.

REFERENCE

1. Sludge treatment technology
   (1) Hiraoka, kasakura
   (2) Handbook of wastewater treatment and sludge handling, pp 632-654
   (3) 1984, Environmental Research Intitute

2. Heat characteristics of dewatered sludge cake
   (1) Murakami
   (2) Journal of JSWA, 23(265) pp38-47, 23(266) pp27-36
   (3) 1986, Japan Sewage Works Association (JSWA)

3. Recent progress of sludge treatment and disposal
   (1) Hiraoka
   (2) annual report of studies on sludge tretment
   (3) 1988, ERI

4. Co-disposal of sewage sludge and municipal solid waste
   (1) Kyosai
   (2) Jounal of water and waste, 28(4) pp376-382
   (3) 1986, The Industrial Water Institute

5. Sludge treatment technology in Japan
   (1) Kasakura
   (2) published at CHISA'90

6. Sludge incineration practices in Japan
   (1) Murakami
   (2) Proceedings 4th WPCF/JSWA Joint Technical Seminar, pp179-195
   (3) May, 1990

7. Municipal Wastewater Sludge Combustion Technology
   (1) United States EPA
   (2) EPA/625/4-85/015
   (3) 1985

8. Future Trend of Sludge Management
   (1) Ohshima
   (2) Jounal of Sewerage, Monthly, 13(7) pp2-6
   (3) July 1990

9. An Economic Evaluation of Indirect Sludge Drying Prior to Incineration Process
   (1) Schwartz,
   (2) Research Journal WPCF, 62(3) pp275-280
   (3) 1990, WPCF

10. Study on Sludge Melting Process and Effective Utilization of Slag Generated
   (1) Yuhki
   (2) Third German/Japanese Workshop on Wastewater and sludge Treatment
   (3) 1989

11. Study on Characteristics of Melting System for Sewage Sludge
   (1) Murakami
   (2) Forth German/Japanese Workshop on Wastewater and Sludge Treatment
   (3) 1990

12. Melting Treatment for Incineration Residue of Municipal Waste
   (1) Fujimoto, Tanaka, Taniguchi
   (2) Pacific Basin Conference on Hazardous Waste
   (3) April 1989

13. Environmental Technology News
   (2) Technical Journal of NGK-Insulators,LTD.
   (3) 1980 to 1990

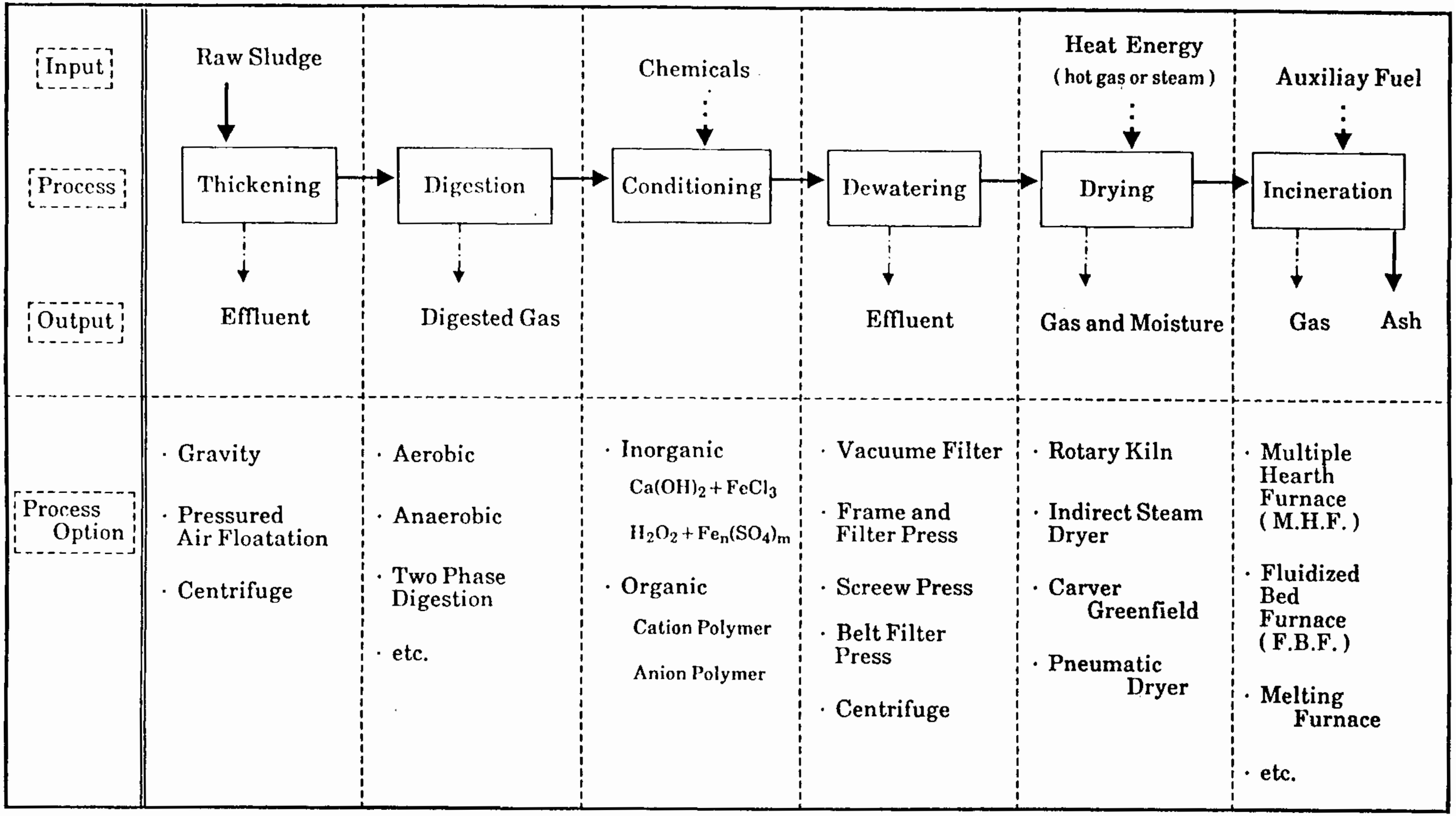


Figure - 1 SLUDGE TREATMENT SYSTEM

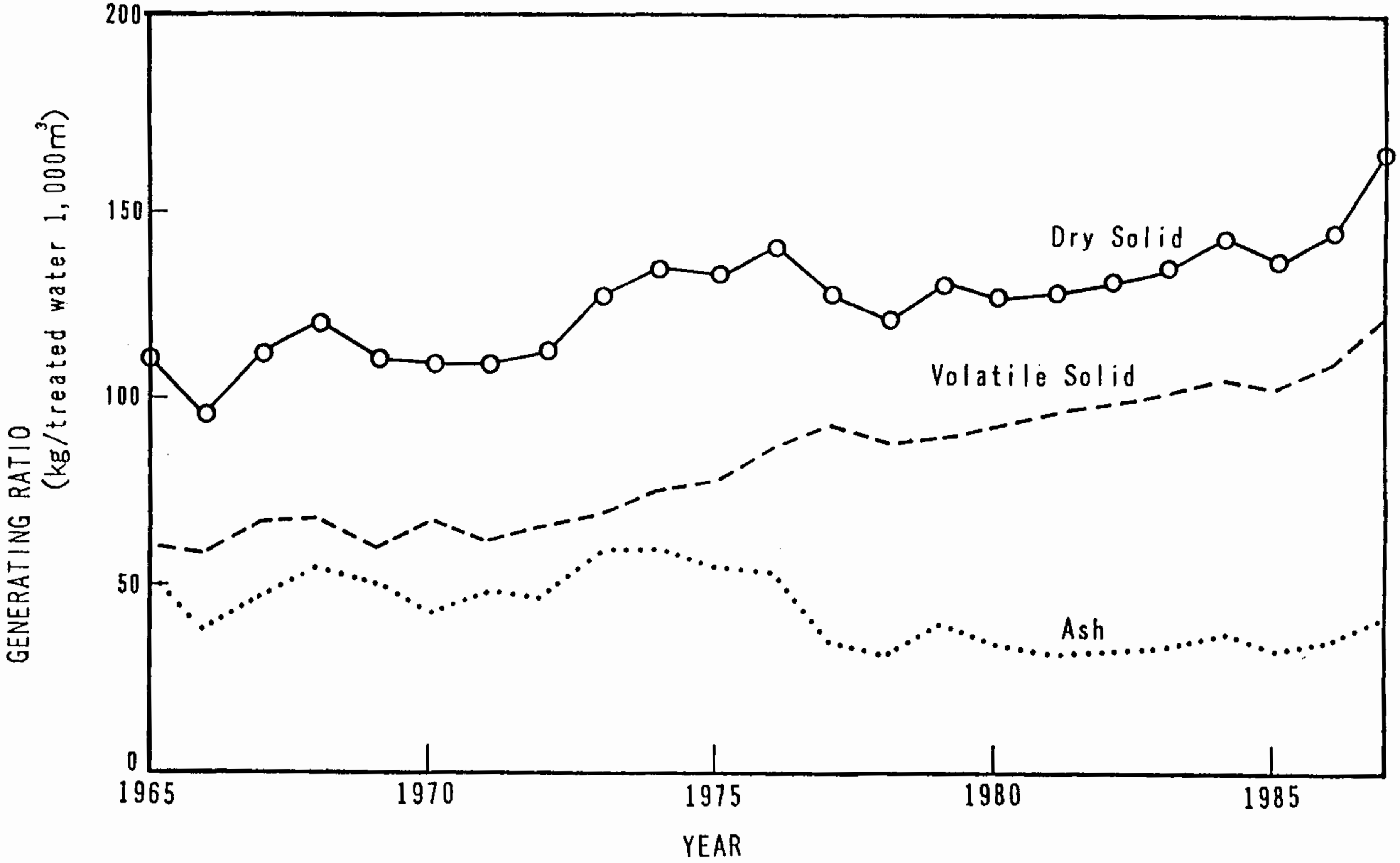


Figure 2 YEARLY CHANGE IN GENERATING RATIO AND COMPOSITION OF DRY SOLID

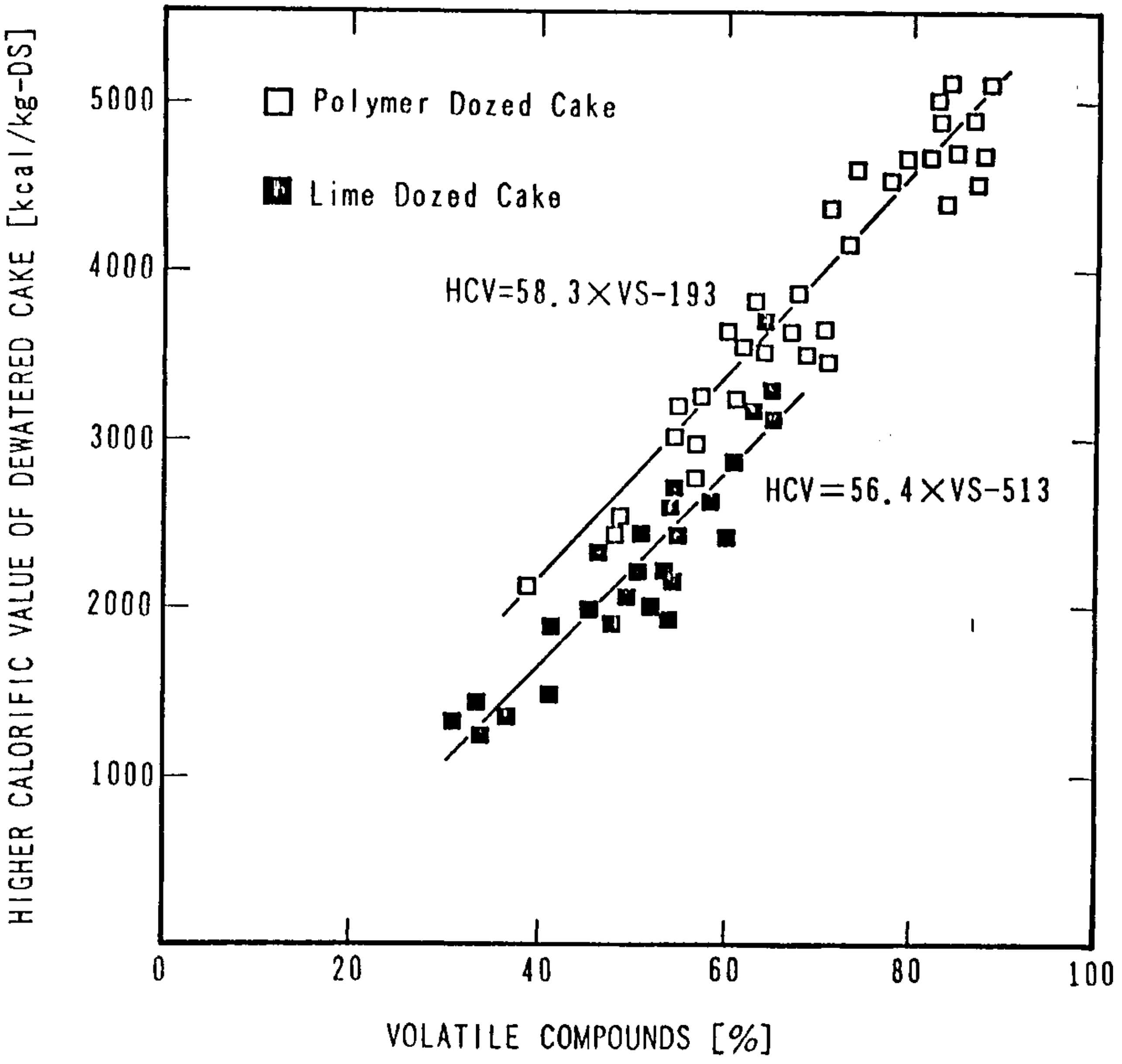


Figure 3 HIGHER CALORIFIC VALUE AND VOLATILE COMPOUNDS

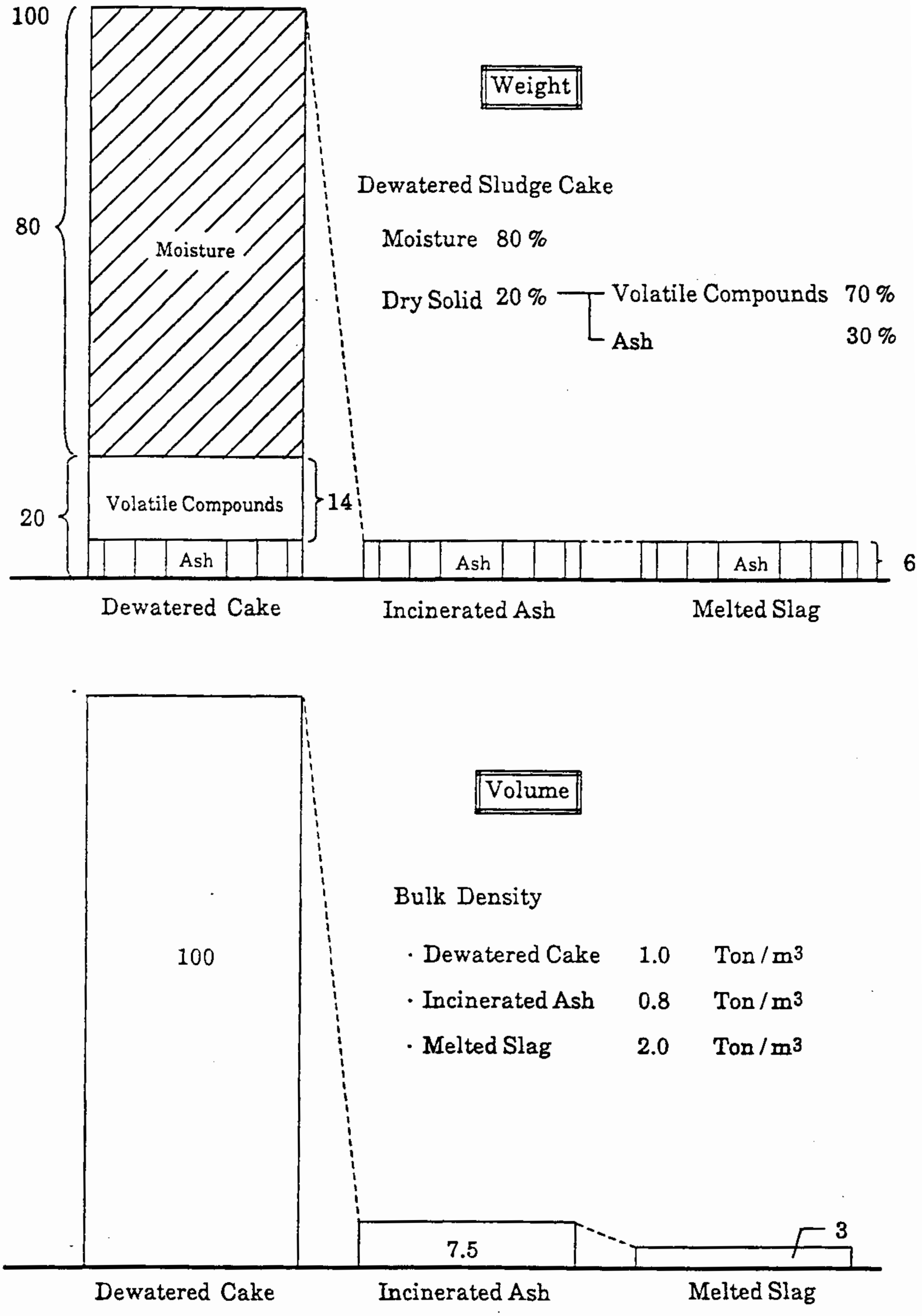


Figure - 4 THE VOLUME REDUCTION EFFECT

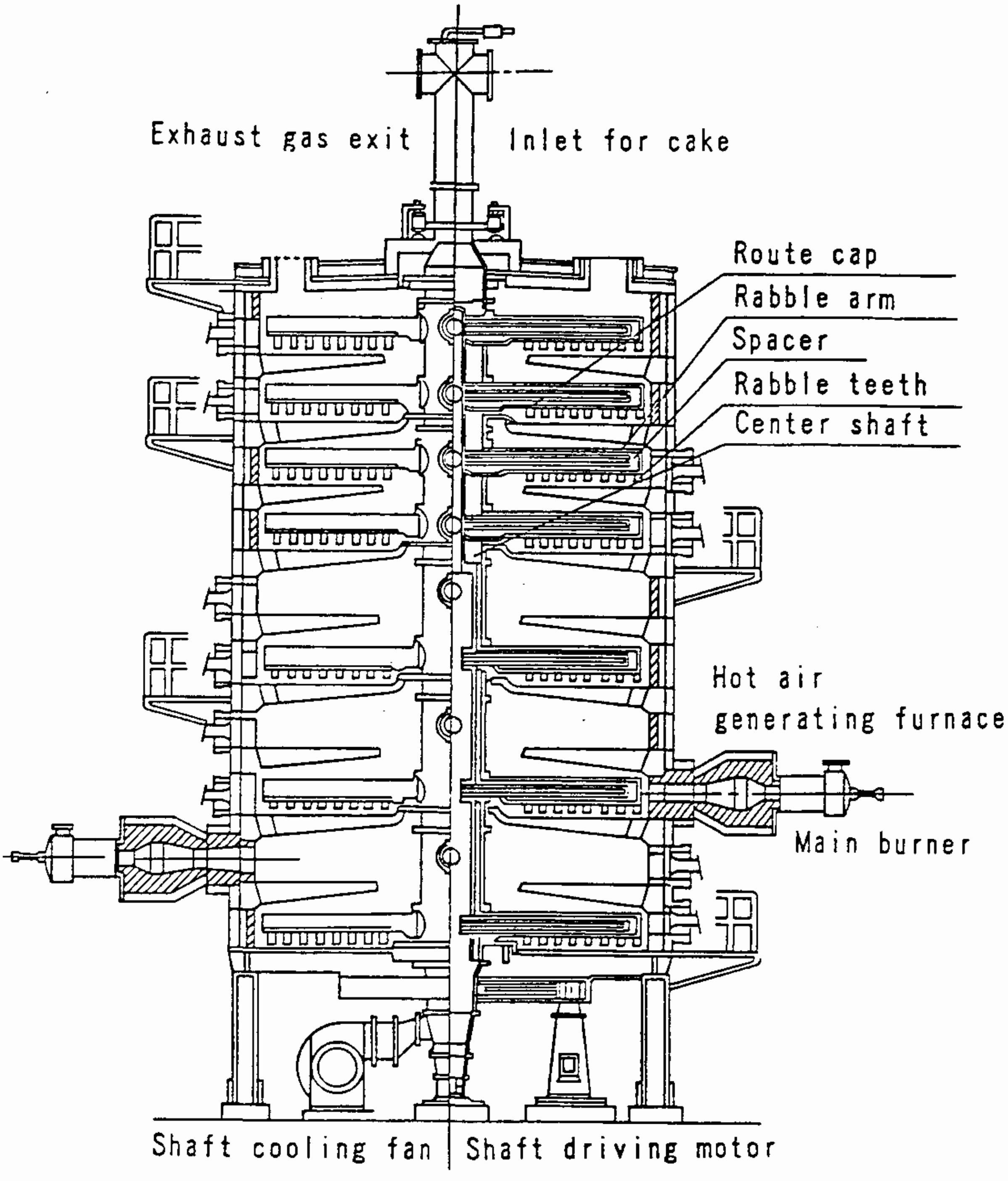


Figure 5 CROSS SECTION OF MULTIPLE HEARTH FURNACE

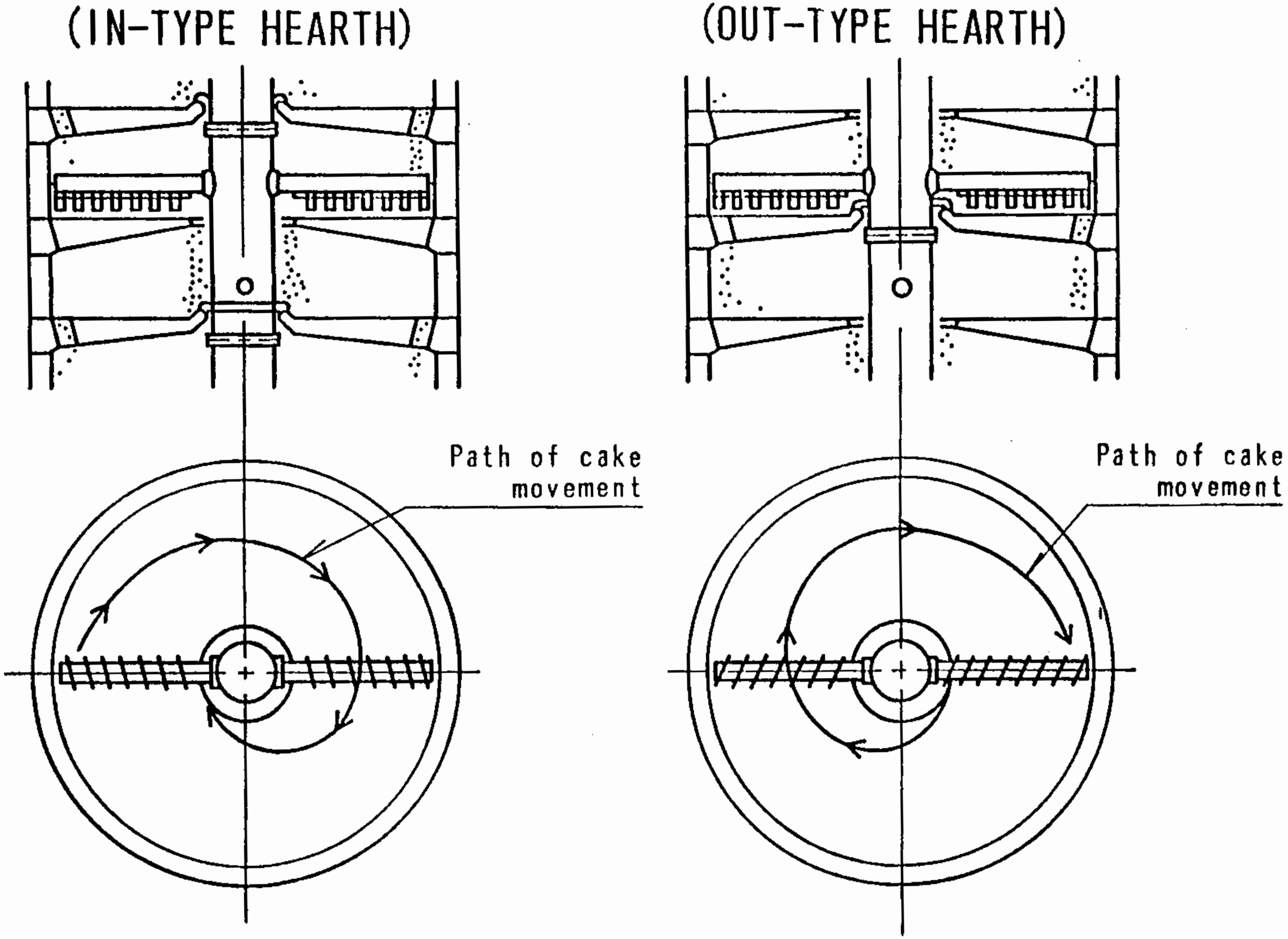


Figure 6 MOVEMENT OF CAKE

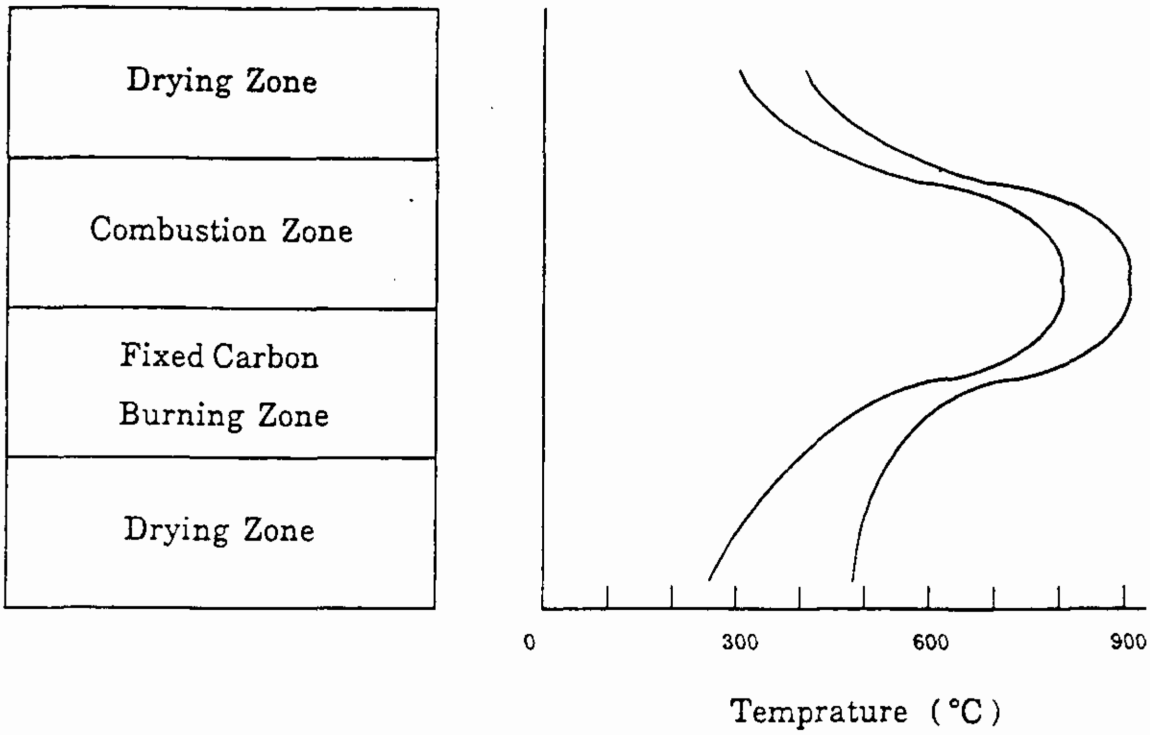


Figure-7 ZONE IN A MULTIPLE-HEARTH FURNACE

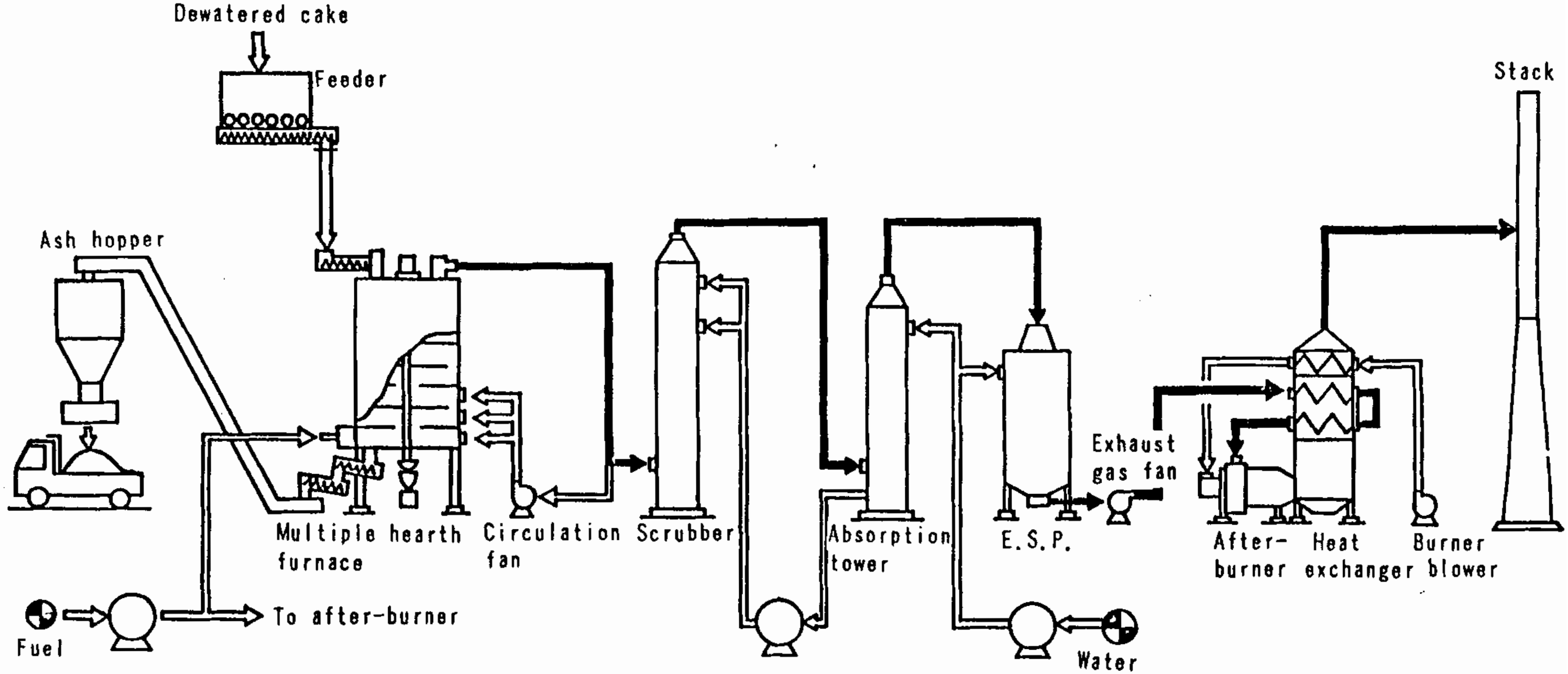
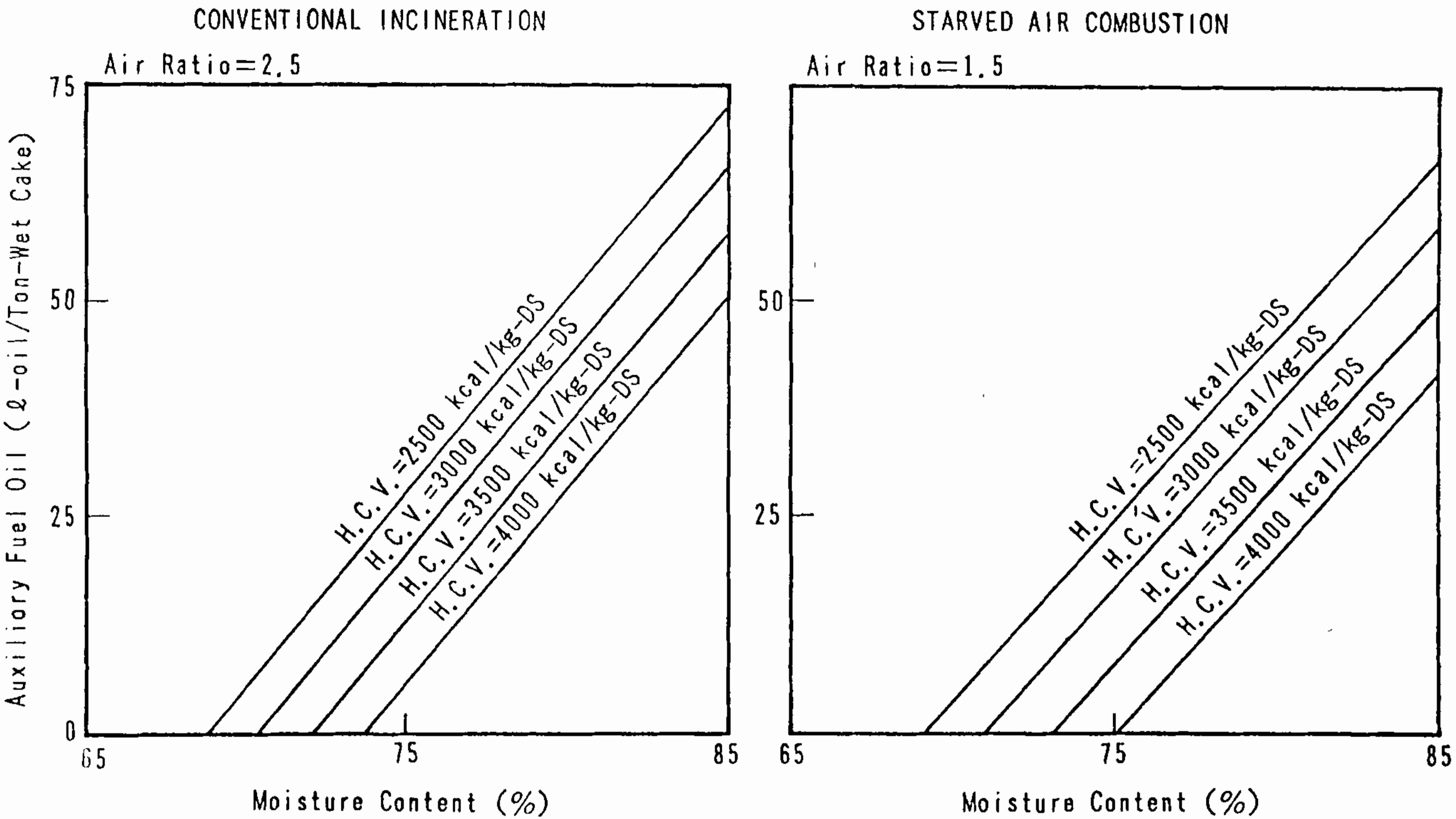


Figure 8 FLOW SHEET OF THE STARVED AIR COMBUSTION SYSTEM



CONVENTIONAL INCINERATION

STARVED AIR COMBUSTION

Figure 9 FUEL OIL COMSUMPTION OF THE STARVED AIR COMBUSTION

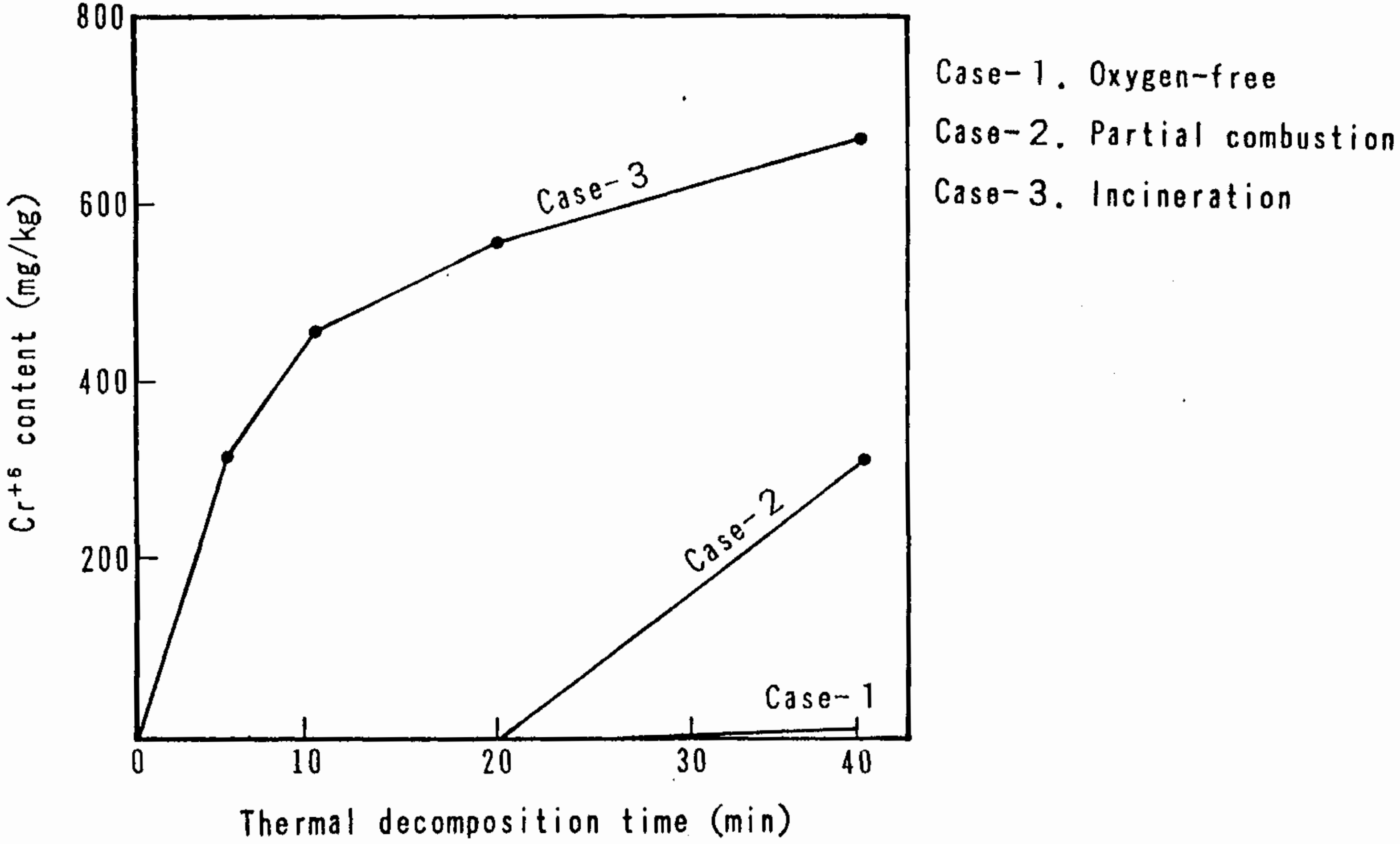


Figure10 BEHAVIOR OF Cr^{+6} IN THE RESIDUE
FOR VARIOUS OPERATIONS

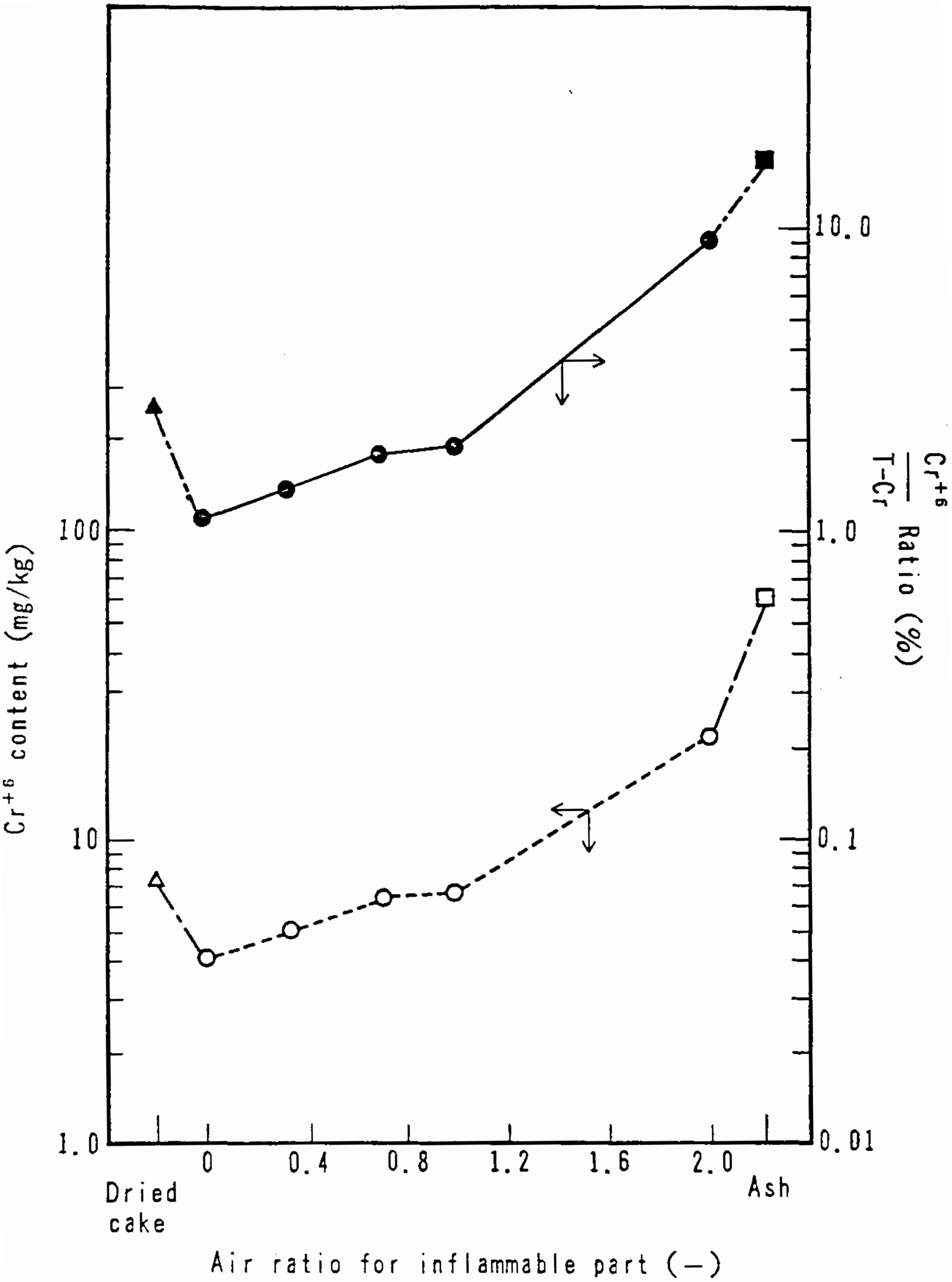
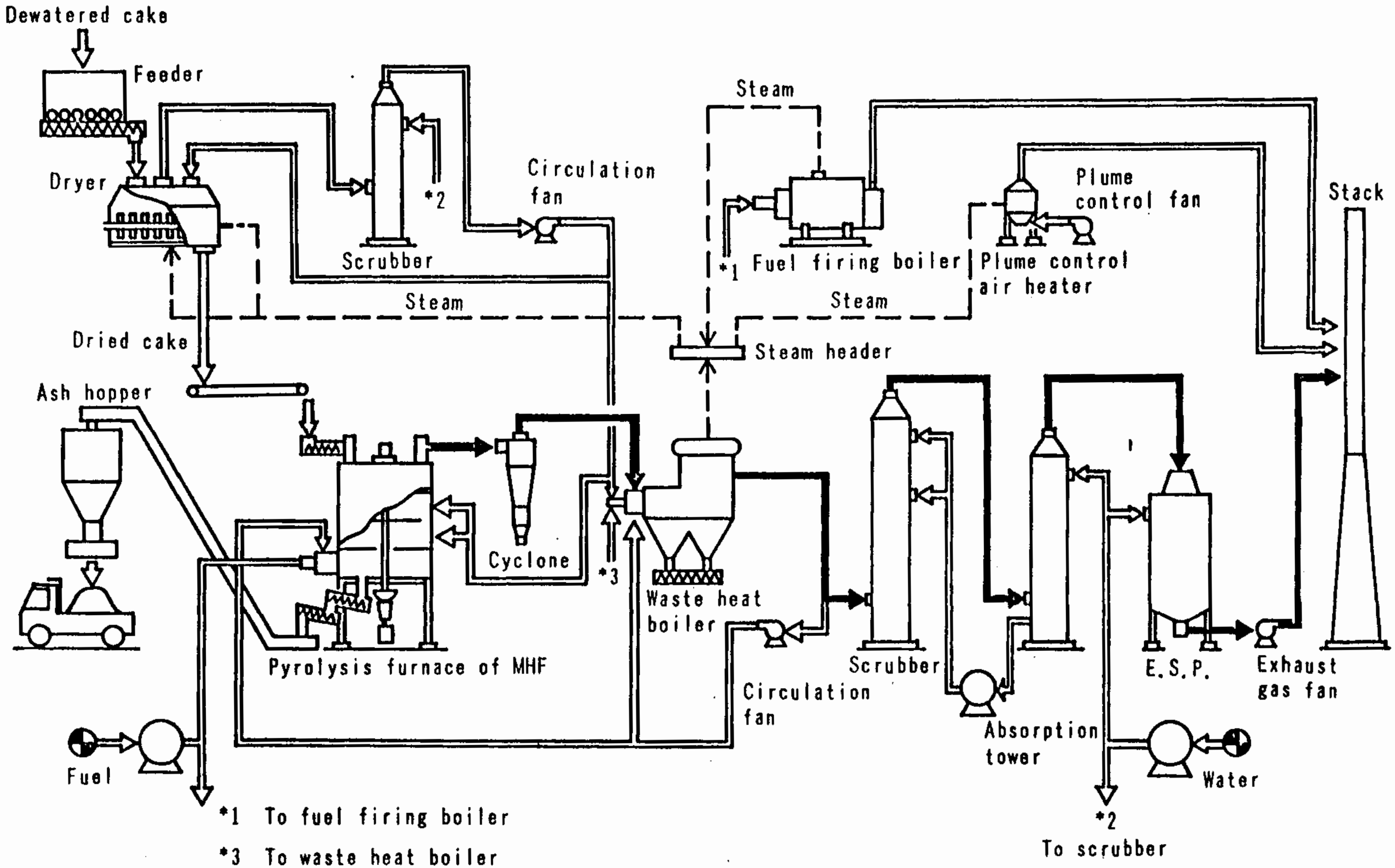


Figure11 CHROME BEHAVIOR IN PILOT PLANT STUDY



*1 To fuel firing boiler

*3 To waste heat boiler

Figure12 DRYING-PYROLYSIS SYSTEM

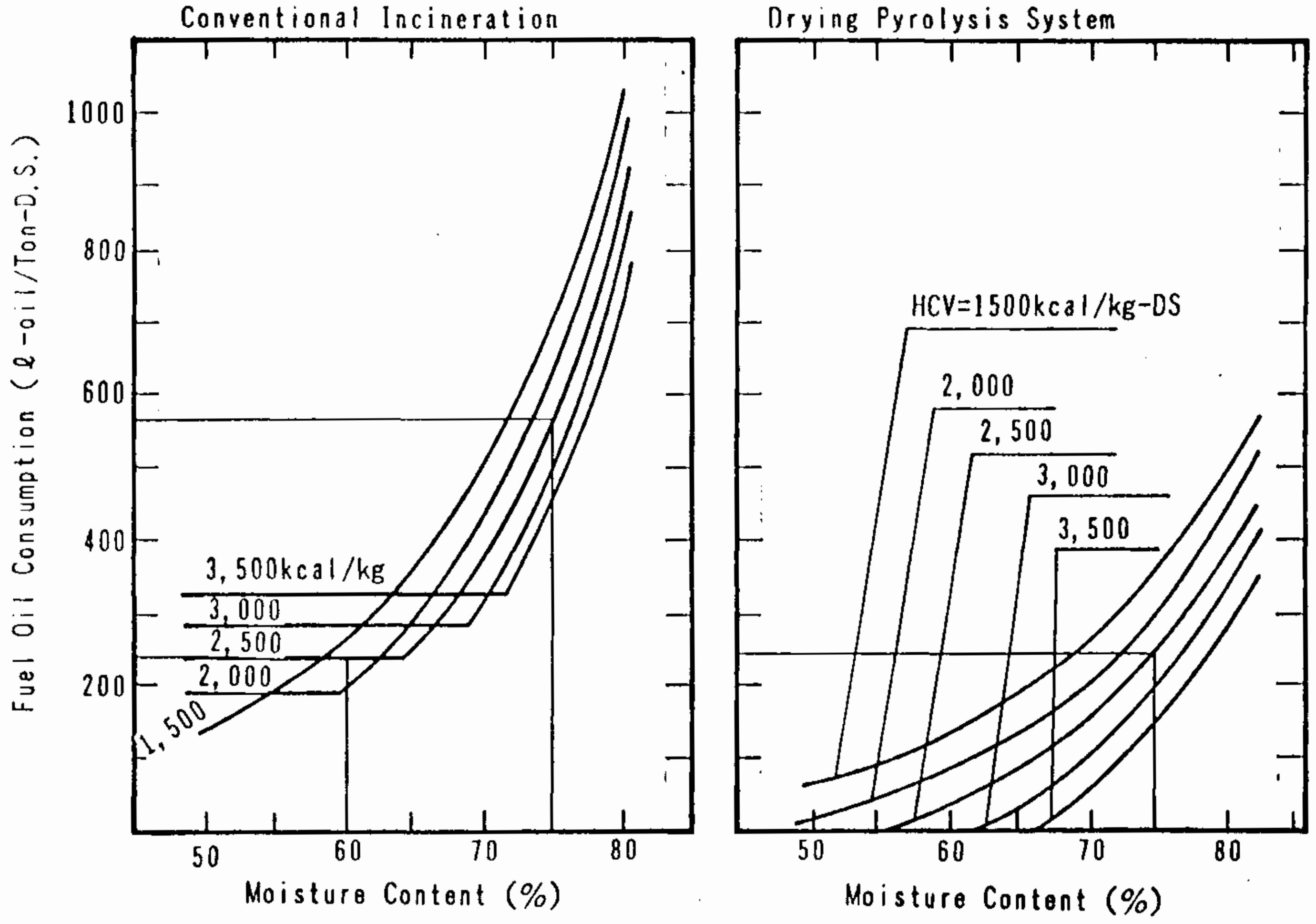


Figure13 FUEL OIL CONSUMPTION OF THE DRYING PYROLYSIS SYSTEM

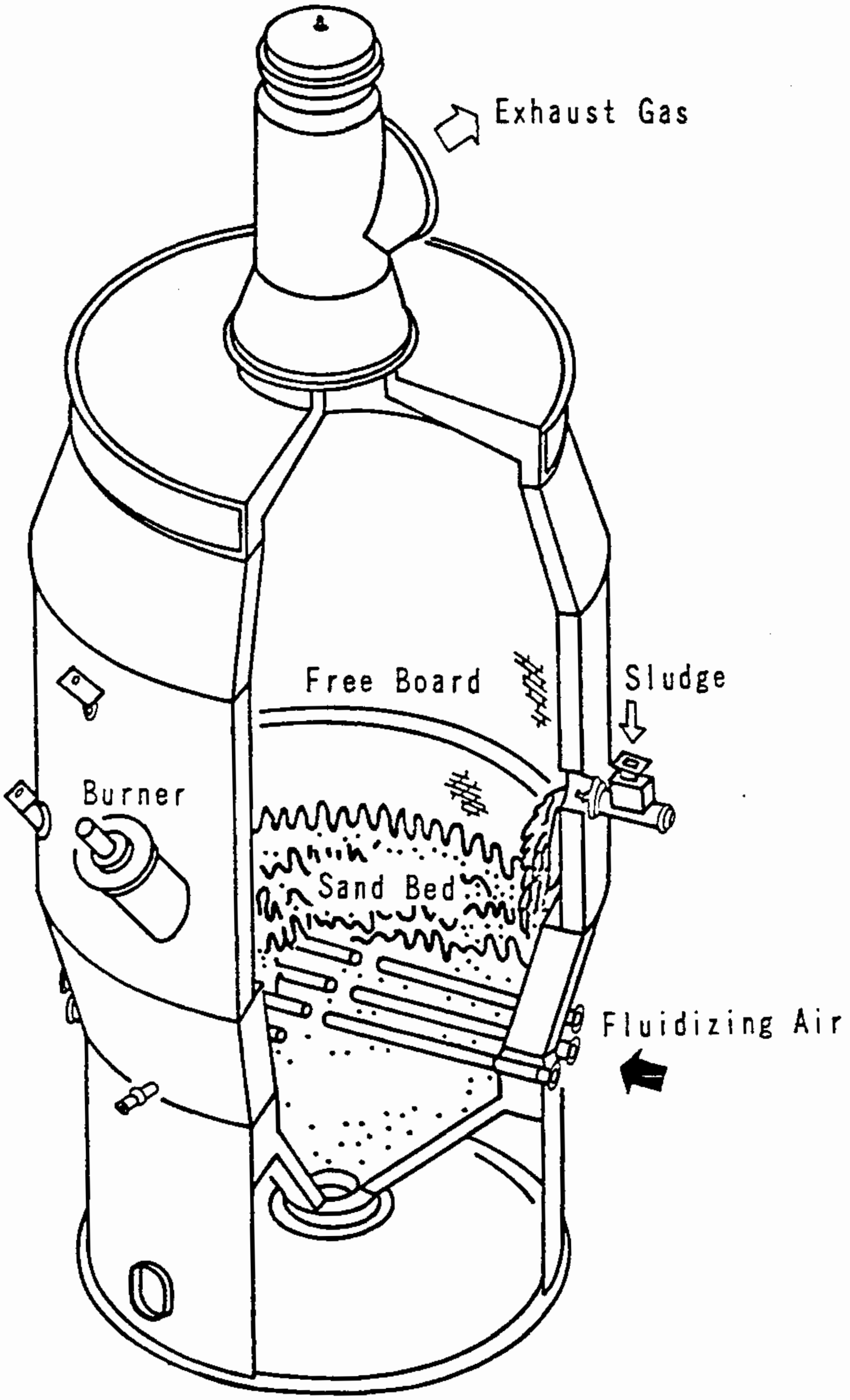


Figure14 STRUCTURE OF FLUIDIZED BED FURNACE

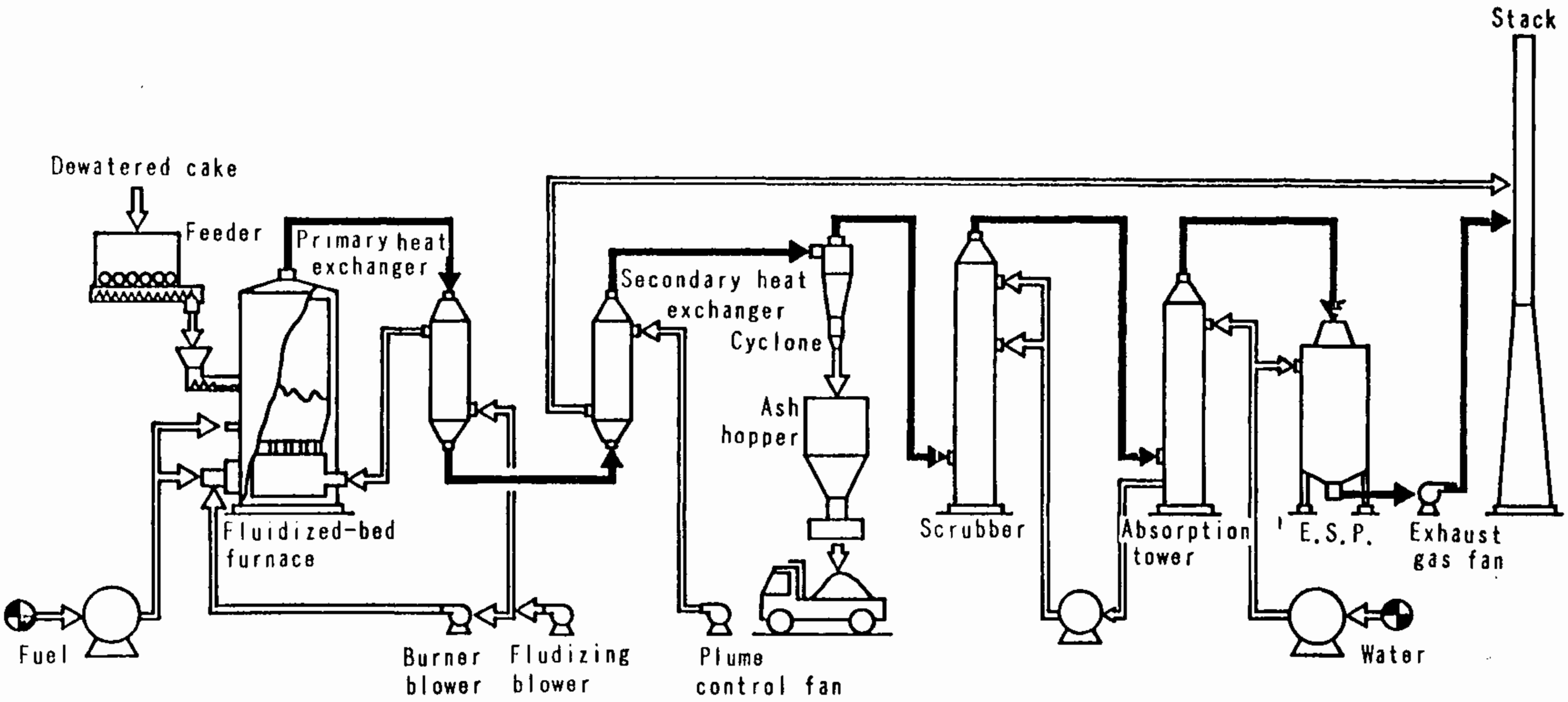


Figure15-(a) DIRECT FBF SYSTEM

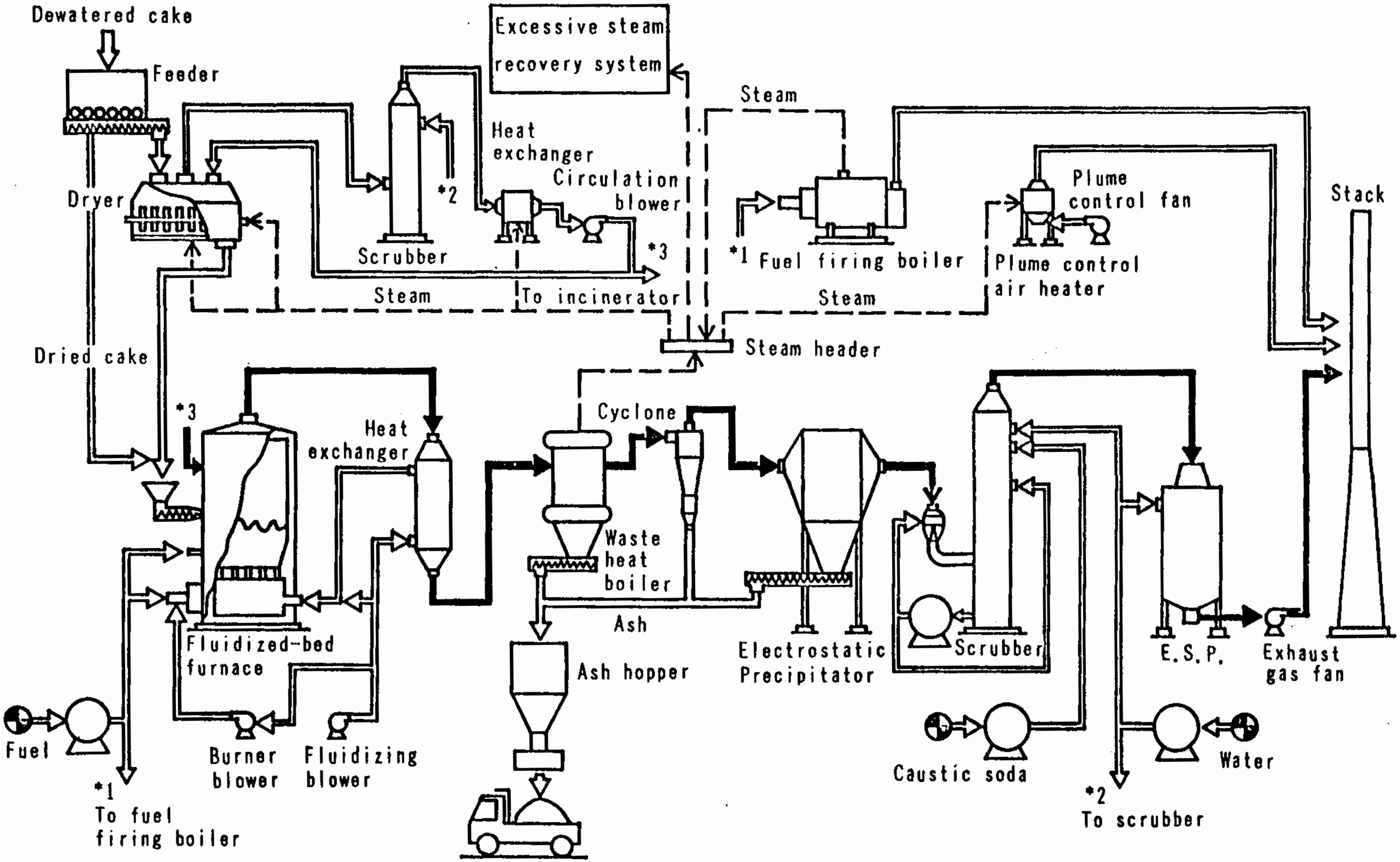


Figure15-(b) DRYING FBF SYSTEM

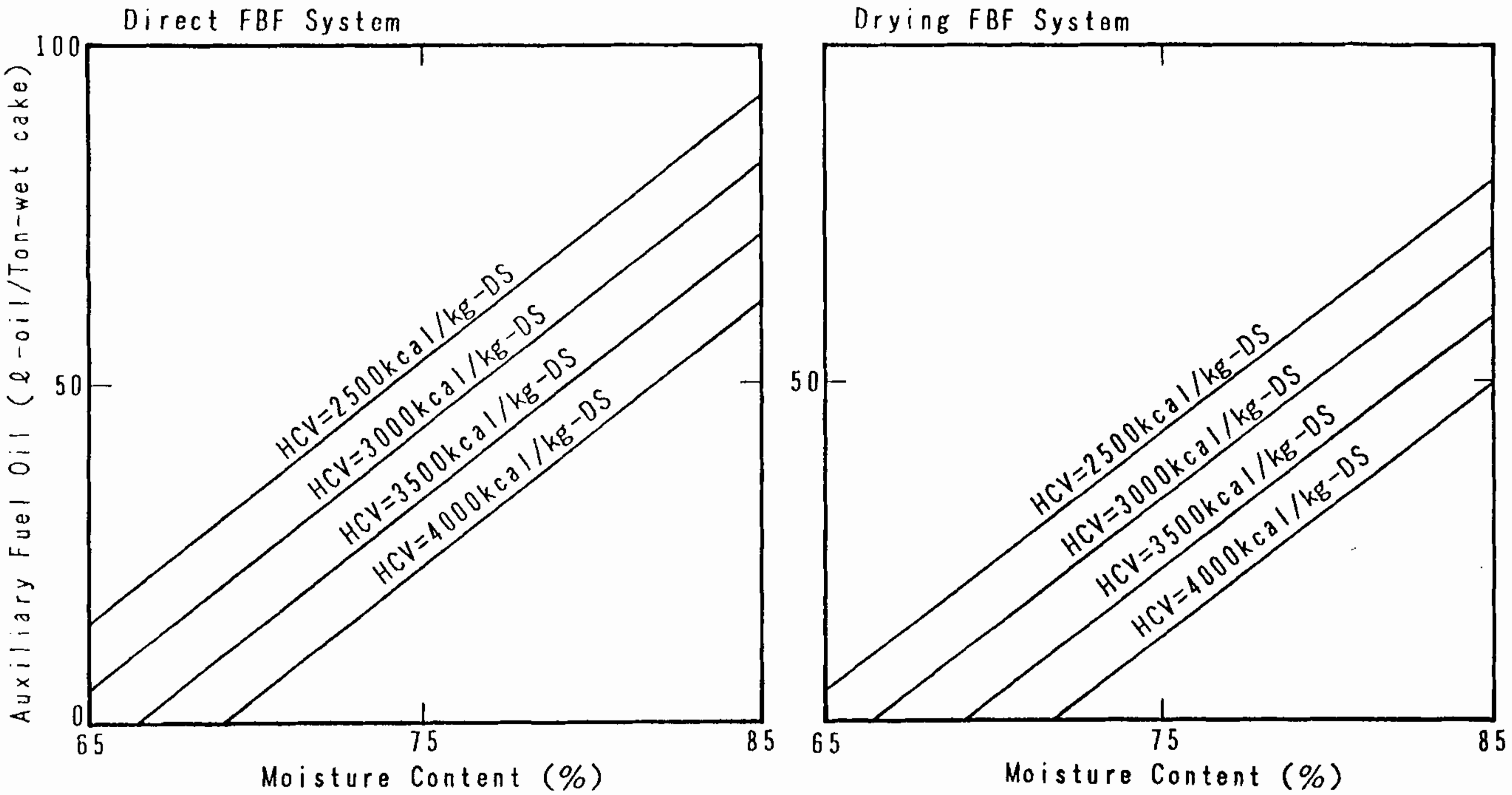


Figure 16 FUEL OIL CONSUMPTION OF FBF SYSTEM

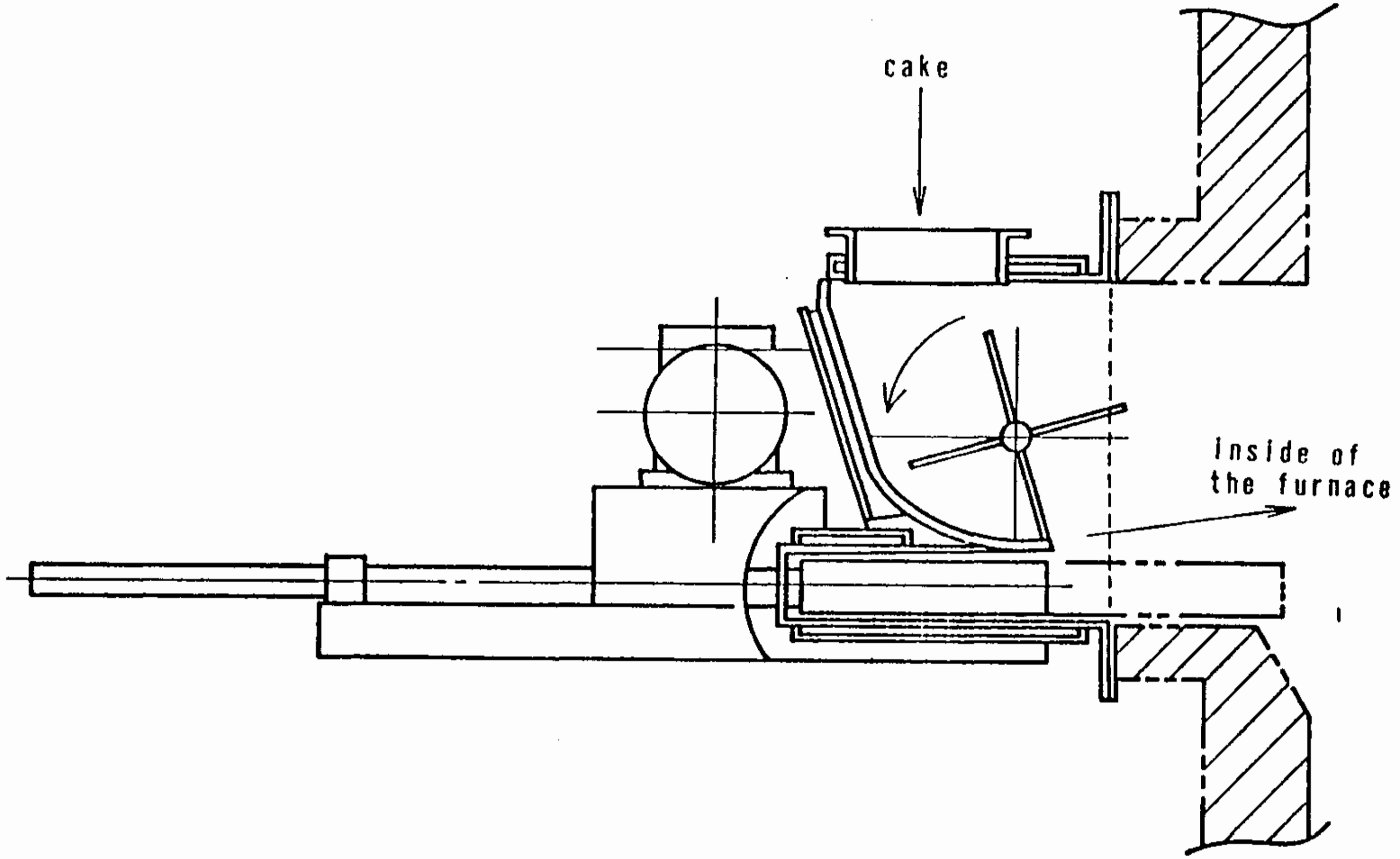


Figure 17 STRUCTURE OF CAKE SPREADER

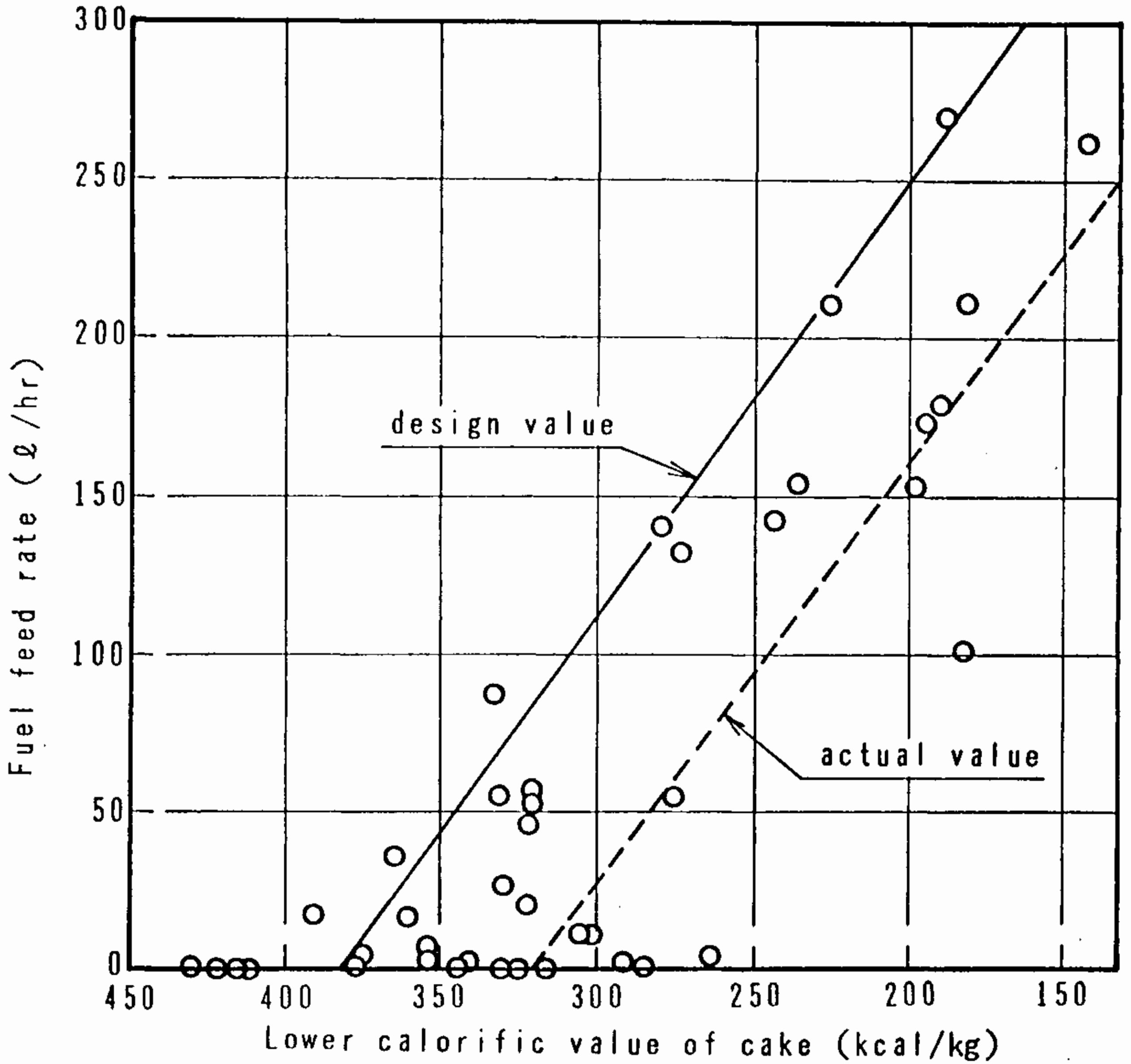


Figure 18 CALORIFIC VALUE VS. FUEL CONSUMPTION

- Measuring conditions for flow-out ratio

Melting temperature	1400℃
Melting time	20min.
Crucible	Alumina B-1(20φ×30min.)
Sample weight	20 g

$$\text{Flow-out ratio} = \frac{\text{Weight slag flowed out of crucible}}{\text{Weight of sample charged in crucible}} \times 100\%$$

- Measuring procedure for flow-out ratio

20g of incinerated ash charged in crucible ⇒ Temp. maintained at 1400℃ for 20 minutes ⇒ Incline the crucible to discharge slag ⇒ Measure the Weight of slag overflow

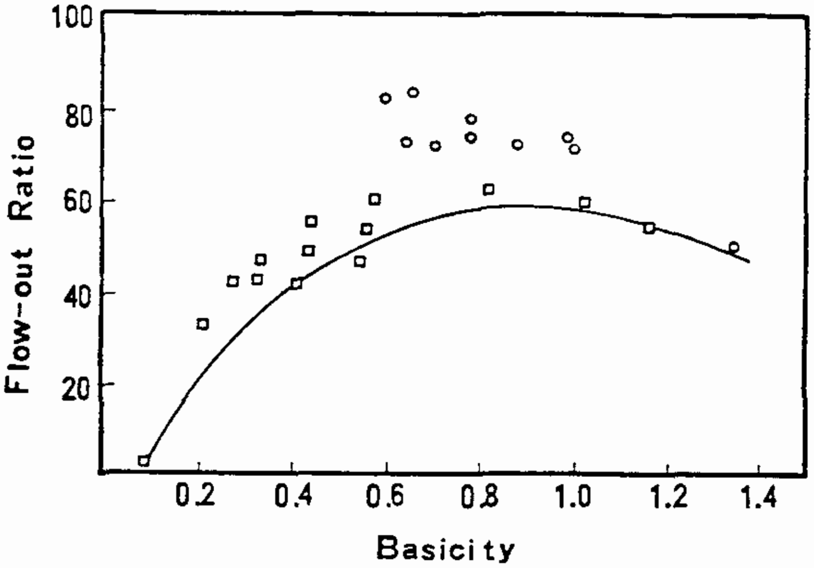


Figure 20 BASICITY V.S. FLOW-OUT RATIO

Table - 1 SLUDGE TREATMENT SYSTEM IN JAPASN

Treatment System	Number of POTWs	Percentage in Total (%)
1) Thickening	34	4.6
2) Thickening - Dewatering	218	29.6
3) Thickening - Dewatering - Incineration	63	8.5
4) Thickening - Dewatering - Drying	8	1.1
5) Thickening - Dewatering - Composting	11	1.5
6) Thickening - Dewatering -- Incineration or Composting	2	0.3
7) Thickening - Digestion	9	1.2
8) Thickening - Digestion - Dewatering - Drying	17	2.3
9) Thickening - Digestion - Dewatering	197	26.8
10) Thickening - Digestion - Dewatering - Incinerating	29	3.9
11) Thickening - Digestion - Dewatering - Composting	13	1.8
12) Thickening - Digestion - Dewatering - Incineration or Composting	2	0.3
13) Thickening - Drying (Sand bed)	4	0.5
14) Thickening - Heat Treatment - Dewatering	3	0.4
15) Thickening - Heat Treatment - Dewatering - Incineration	8	1.1
16) Digestion	3	0.4
17) Digestion - Dewatering	16	1.1
18) Digestion - Dewatering - Composting	2	0.4
19) Dewatering	11	2.2
20) Drying (sand bed)	6	0.3
21) Others	80	10.9
Total	736	100.0

	Number of POTWs	Percentage in Total (%)
System Including Thickening	618	84.0
System Including Digestion	288	39.1
System Including Dewatering	600	81.5
System Including Composting	30	4.1
System including Incineration	104	14.1

Table-2 THICKENING PROCESS IN JAPAN

Year Method	1981	1982	1983	1984	1985	1986	1987	1988
(1) Gravity	402	420	434	461	491	518	520	535
(2) Pressured Air Floatation	2	4	5	8	9	15	10	8
(3) Centrifuge	1	1	1	2	2	5	3	5
Two Steps or Parallel								
(1) + (2)	17	20	24	29	31	37	40	42
(1) + (3)	6	8	10	16	21	23	30	41
(2) + (3)	-	-	-	1	-	-	-	-
(1) + (2) + (3)	-	-	-	-	2	2	2	2
Others	1	1	1	1	-	-		-
Without Thickener	65	68	73	69	75	62	98	103
Total	494	522	548	587	631	662	703	736

(Numbers of POTW)

Table - 3 DEWATERING UNIT IN JAPAN

Year / Method	1982	1983	1984	1985	1986	1987	1988
Vacuume Filter	560	552	541	517	486	472	452
Frame and Filter Press	262	284	294	325	333	324	326
Centrifuge	320	315	306	311	324	332	342
Belt Filter Press	245	289	334	378	428	484	561
Screw Press	10	13	13	17	20	23	24
Others	18	10	1	0	20	3	8
Total	1,415	1,463	1,489	1,548	1,611	1,638	1,713

(Number of Unit)

Table - 4 INCINERATION IN JAPAN

Year / Type	Numbers of Incineration System							Persentage in Total (%)
	1982	1983	1984	1985	1986	1987	1988	
Multiple Hearth Furnace	54	55	55	57	61	71	72	41.6
Fluidized Bed Furnace	27	27	36	36	44	52	53	30.0
Rotary Kiln	14	15	15	16	16	20	20	11.6
Pyrolysis	2	3	3	3	2	2	1	0.6
Rotary Bed	3	3	3	3	0	0	1	0.6
Melting Furnace	1	1	1	1	2	2	3	1.7
Wet Oxidation	1	1	1	1	2	2	1	0.6
Dryer , and Others	11	11	11	11	16	17	22	12.7
Total	112	116	125	128	143	166	173	100.0

Table - 5 CALORIFIC VALUE OF SLUDGE

Dewatering Machine	Conditioning		Moisture of Dewatered Cake (%)	HCV of Dry Solid (kcal/kg-DS)	LCV of Dewatered Cake (kcal/kg-Wet)	Amount of Dewatered Cake $\left(\frac{\text{kg-Dewatered Cake}}{\text{kg-Original Solid}}\right)$
	Chemical Dozing	Dozing Ratio (%)				
Vacuume Filter	Inorganic	$FeCl_3$: 10 % $Ca(OH)_2$: 50 %	75 ~ 80	2,780	180 ~ 390	6.3 ~ 8.0
Centrifuge	Organic Polymer	Anion : — Cation : 1.0 %	79 ~ 84	4,400	150 ~ 380	4.8 ~ 6.3
Belt Filter Press	Organic Polymer	Anion : — Cation : 0.8 %	77 ~ 82	4,400	240 ~ 470	4.3 ~ 5.6
Frame & Filter Press	Inorganic	$FeCl_3$: 10 % $Ca(OH)_2$: 35 %	63 ~ 68	3,070	722 ~ 930	3.9 ~ 4.5
Screw Press	Organic Polymer	Anion : 0.4 % Cation : 0.8 %	68 ~ 73	4,400	660 ~ 890	3.1 ~ 3.7

Table - 6 REGURATION FOR EXHAUST GAS IN JAPAN

Items	Tokyo M.D.	Osaka City	Nagoya City
Dust at $O_2 = 12$ %	$Q < 40{,}000$ Nm^3/h 0.15 g/Nm^3 $Q \geqq 40{,}000$ Nm^3/h 0.08 g/Nm^3	same as the left	same as the left
SOx	$q = K \times 10^{-3} He^2$ $K = 1.17$ He : modified stack hight	same as the left	same as the left
NOx	150 p.p.m.	90 p.p.m.	60 p.p.m.
HCl	700 mg / Nm^3	same as the left	same as the left
Odor	※ 300 ~ 1,000 times	1,000 times	1,000 times

※ Dilution ratio to vanish away the odor

Table - 7 REGULATION FOR LAND DISPOSAL

R - Hg	N.D.
T - Hg	0.005 mg/ℓ
Cd	0.3 〃
Pb	3 〃
Org - P	1 〃
Cv (VI)	1.5 〃
As	1.5 〃
CN	1 〃
PCB	0.003 〃

The values in table show the regulatory limits of the leaching test.
In leaching test, sludge is dilluted by 10 times water and stirred for 8 Hours.
Then the soluble concentration of each compound is measured.

Table - 8 EFFECT OF THE EXHAUST GAS CIRCULATION

System		Conventional System	Starved Air Incineration System		
Operational Condition	Air Ratio (-)	2.5	1.5		
	Gas Circulation (%)	0	0	50	100
Temperature at the Combustion Zone (°C)		817	996	722	665
Rd / Rc ※		1.00	0.79	0.95	1.18

※ Rd : Drying Velocity at Each Condition

Rc : Drying Velocity at the Conventional System

Moisture : 78 %

HCV : 3,500 kcal / kg-DS

Rd : $Rd = A \times G^{0.8} \times \Delta tm$

Table 9 SPECIFICATION OF MAIN PART

Name	Type	Capacity
dryer (2)	indirect steam heating type	vaporized water 1.35t/hr
dehumidifing tower	vertical cylinder type	1,600㎜φ×8,000㎜H
furnace	fluidized bed type	200t (wet cake)/d
		250t (dried cake)/d
air preheater	radiation type	exchanged heat 1.61×10^6kcal/hr
waste heat boiler	water tube type	recovered steam 4.9t/hr
cyclone (2)	2 towes connected in series	22,150N㎥/hr (at 250℃)
dry type E.S.P.	horizontal flow type	27,160N㎥/hr (at 250℃)
absorber	vertical cylinder type	3,000㎜φ×18,000㎜H
wet type E.S.P.	down flow type	19,330N㎥/hr (at 40℃)
induced blower	turbo blower	450㎥/min×1,300㎜Aq
ash hopper (2)	twin type elvated tight tank	50㎥
exclusive boiler	flue and smoke tube type	generated steam 3.0t/hr

Table 10 DESIGN CONDITIONS

1. dewatered cake: cake dewatered with centrifugal sparator
   chemicals: organic polymer
   moisture content: standard 78% (range 75～82%)
   composition of solid: combustibles 70%, ash 30%
   high calorific value of solid: 4,000kcal/kg (range 3,500～4,500kcal/kg)
2. auxiliary fuel
   for the furnace: low sulfer A oil
   for the exclusive boiler: kerosene
3. incinerating capacity
   in the case of steam drying process: 250t/d
   in the case of direct cake feed process: 200t/d
4. operating condition of the furnace
   air ratio: m=1.3
   gas out let temperature: 800℃

Table 11 OPERATION DATA

date	cake feed rate (t/d)	cake moisture content (%)	dryed cake moisture content (%)	furnace fed cake moisture content (%)	H.C.V. of solid (kcal/kg)	L.C.V. of cake (kcal/kg)	L.C.V. of furnace fed cake (kcal/kg)	fuel feed rate (ℓ/hr)
4·20	205	79.3	45.7	73.9	4160	322	564	56
4·21	236	79.3	42.9	73.7	4160	322	571	52
4·22	251	77.7	41.6	71.8	4020	365	621	35
4·23	248	77.3	43.4	71.3	3980	374	631	4
4·24	260	79.3	44.6	73.9	4160	322	562	21
5·18	257	78.9	41.6	73.2	4120	332	584	55
5·19	257	80.3	43.0	74.8	4170	280	523	140
5·20	257	81.7	43.0	76.4	4010	190	419	178
5·21	252	82.5	44.8	77.8	4180	183	395	211
5·22	256	82.8	40.0	78.3	4010	143	338	262
5·23	247	83.1	44.1	78.5	4380	188	402	270
5·24	245	82.5	44.8	77.6	4450	227	457	210
5·25	266	82.1	42.8	77.2	4400	237	466	153
5·26	260	82.1	45.3	77.0	4440	244	482	143
5·27	262	81.4	43.5	76.1	4420	274	520	132
5·28	263	81.0	48.0	75.9	4360	282	517	101
5·29	266	80.5	45.5	74.7	4200	276	537	54
5·30	270	80.1	48.1	74.2	4260	305	573	12
5·31	256	81.3	42.8	75.9	4340	264	515	3
6·1	260	78.4	47.4	72.8	4300	391	648	17
6·2	255	80.3	45.3	74.5	4460	333	607	87
6·3	262	83.1	43.4	77.2	4450	199	475	153
6·4	251	83.4	52.0	78.3	4530	197	440	174
6·5	261	81.1	46.6	75.5	4610	321	595	47
7·1	244	79.0	46.3	73.2	4130	330	588	27
7·2	232	78.1	49.0	71.8	4050	354	626	6
7·3	243	78.1	48.7	71.9	4050	354	624	3
7·4	248	75.1	45.6	69.7	3780	422	642	0
7·5	209	75.6	47.2	68.8	3830	413	695	0
7·6	250	74.8	47.0	69.4	3760	430	650	0
7·7	243	78.1	46.1	72.0	4050	354	620	5
7·8	247	77.8	48.2	72.0	4020	360	613	16
7·9	238	79.5	47.6	73.7	4180	317	578	0
7·10	243	78.5	47.2	72.2	4090	344	620	0
7·11	210	79.2	46.0	71.1	4150	325	685	0
7·12	195	79.5	47.9	71.4	4180	317	679	0
7·13	215	80.6	47.9	73.0	4280	286	634	0
7·14	212	80.4	49.5	73.0	4260	292	627	3
7·15	250	80.0	48.4	73.8	4220	302	580	11
7·16	243	75.4	46.2	68.8	3810	416	687	0
7·17	235	77.1	45.3	70.2	3960	378	671	0
7·18	248	78.6	47.3	73.0	4100	342	588	2
7·19	254	79.0	48.3	72.4	4130	330	621	0

Table - 12 FBF SYSTEMS FOR ECONOMIC ANALYSIS

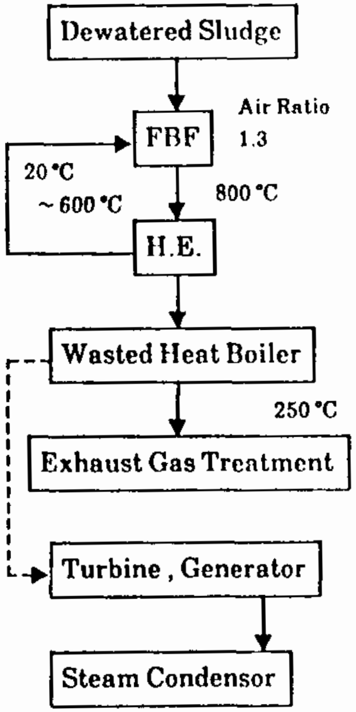
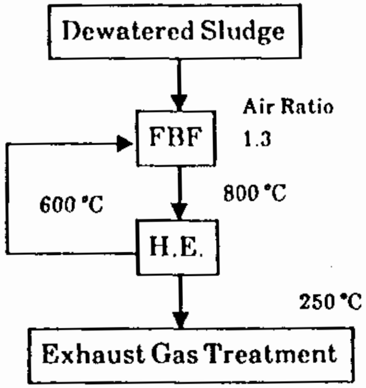
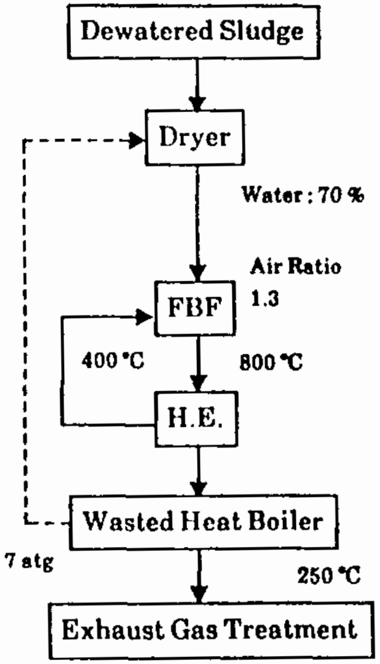
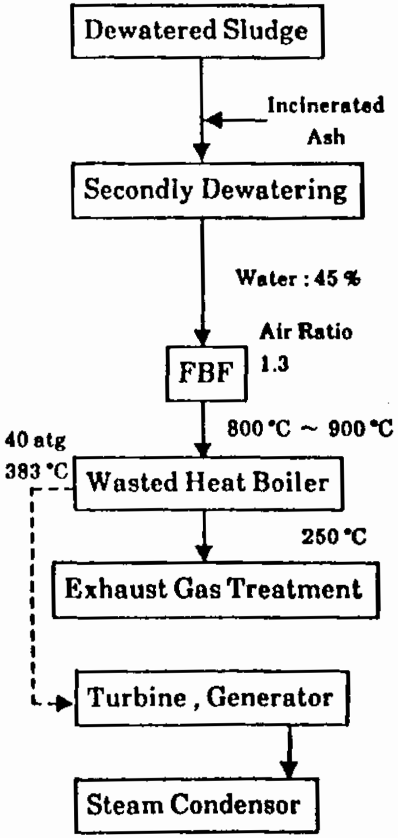
	1	2	3	4
System Name	Direct FBF + Generator	Direct FBF	Drying FBF	High Pressure Dewatering
System Merit	Electricity Recovery	Simple	Energy Saving	Electricity Recovery
Block Flow				

Table - 13 ASH COMPOUNDS IN SLUDGE

Chemical Dozing	Ferric Chloride , Lime Dozing	Organic Polymer
SiO_2	20 ~ 25	38 ~ 48
A_2O_3	7 ~ 10	10 ~ 20
Fe_2O_3	14 ~ 17	10 ~ 15
CaO	27 ~ 38	4.0 ~ 10
MgO	3 ~ 4	1.7 ~ 1.9
Na_2O	0.5 ~ 1.0	0.34 ~ 0.37
K_2O	0.5 ~ 0.8	1.6 ~ 1.8
P_2O_5	5 ~ 6	4.0 ~ 5.5
Melting Point	1100 °C	1100 °C
Basicity	1.0 ~ 1.7	0.08 ~ 0.1

Table14-1 TYPES OF THE MELTING FURNACE

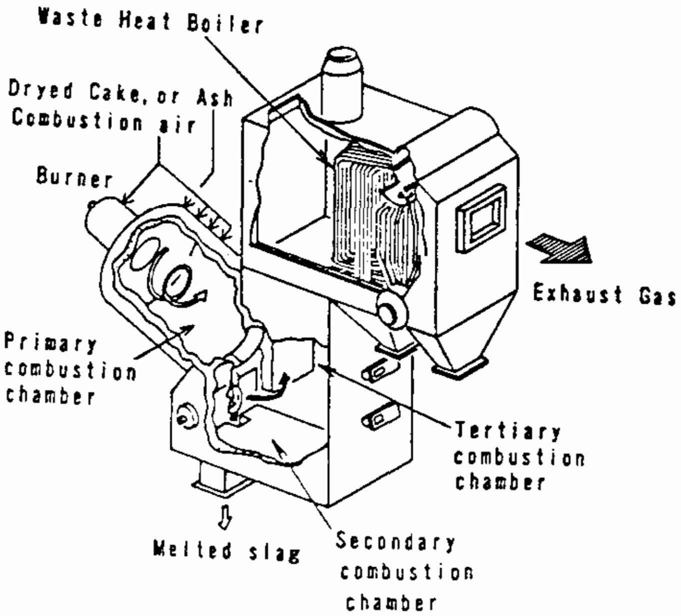
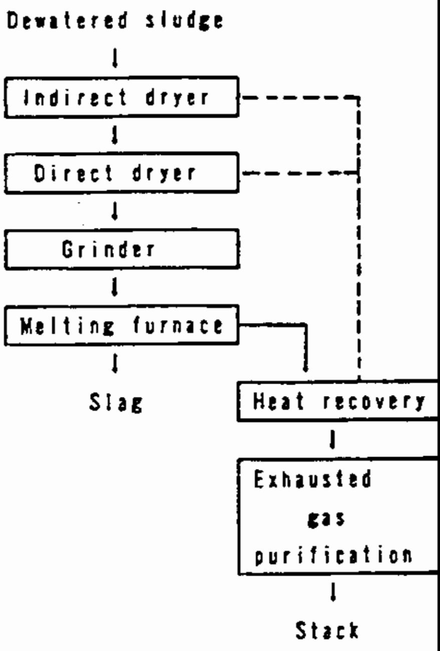
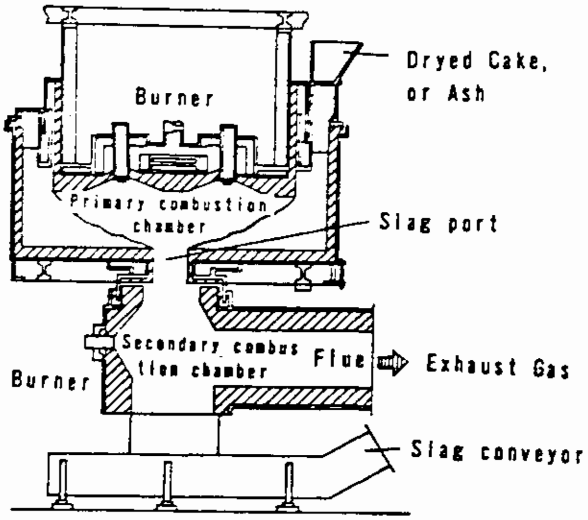
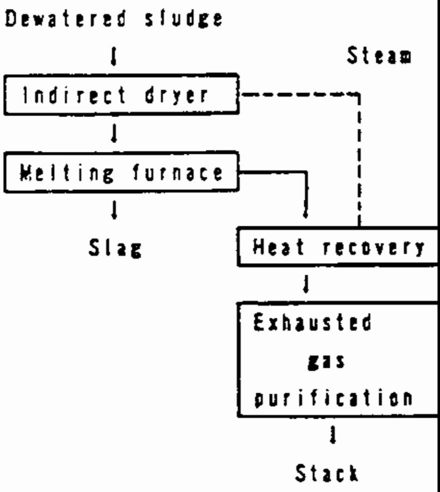
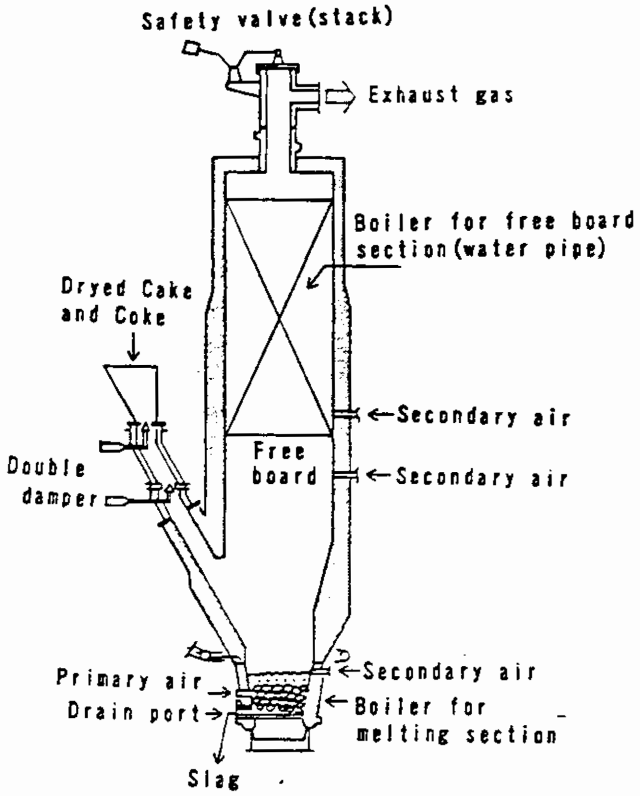
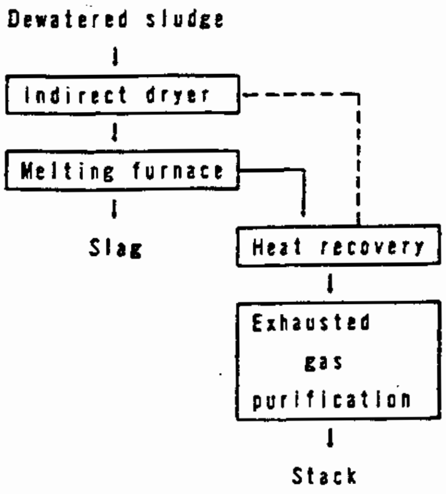
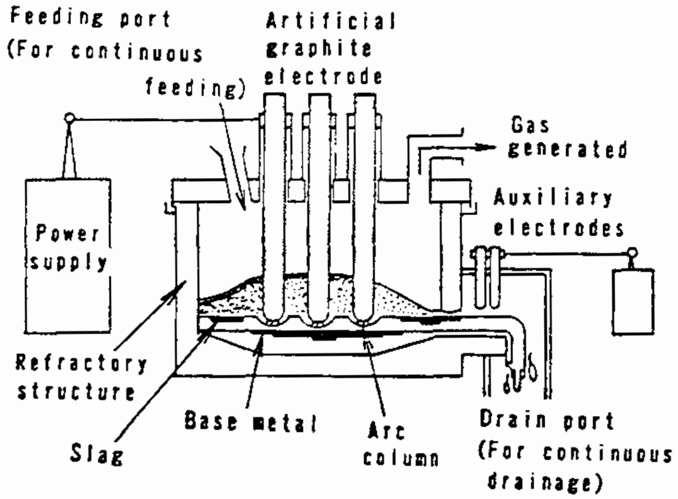
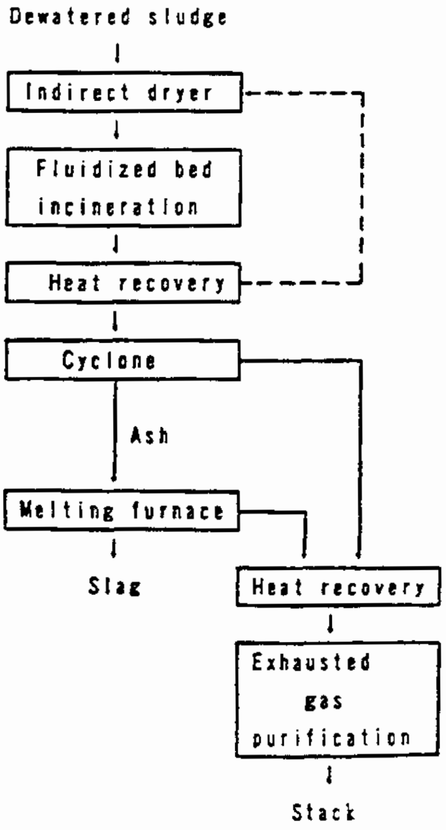
Type	Energy Source	Furnace Structure	System for Dewatered sludge
Cyclone Furnace	Gas Oil Powdered Coal		
Film Surface Furnace	Gas Oil PowderedCoal		

Table14-2 TYPES OF THE MELTING FURNACE

Type	Energy Source	Furnace Structure	System for Dewatered sludge
Coke Bed Furnace	Coke		
rc urnace	Electricity		

NEW TECHNIQUES OF SLUDGE MANAGEMENT IN THE NETHERLANDS

W.G. Werumeus Buning
Water Authority "Hollandse Eilanden en Waarden"

1. Introduction

In the coming ten years a lot will change in the treatment and disposal of sewage sludge. This is influenced by two main factors. On one hand the amount of sewage sludge produced will increase, while on the other hand the regulations for sludge disposal will be tightened.

In 1988 the amount of domestic sewage sludge produced in the Netherlands was 280,000 tons on a dry matter basis. Phosphate removal from effluents can increase the amount of domestic sludge by 100,000 tons per year. In the year 2000 the amount of domestic sewage sludge produced is expected to be 400,000 tons on a dry matter basis.

Table 1 indicates the various ways in which domestic sewage sludge was disposed of in the period 1981-1988.

Table 1: Production and disposal of sewage sludge in the Netherlands (1000 tons dry matter).

	Production	Disposal				
		Agriculture	Compost/ Soil production	Controlled tipping	Incineration	Remainder
1981	175	67 (38%)	37 (21%)	62 (36%)	5 (3%)	4 (2%)
1982	195	77 (39%)	43 (22%)	63 (32%)	6 (3%)	7 (3%)
1983	199	72 (36%)	53 (27%)	61 (30%)	6 (3%)	8 (4%)
1984	202	65 (32%)	62 (31%)	66 (33%)	6 (3%)	3 (1%)
1985	217	69 (32%)	72 (33%)	68 (31%)	5 (2%)	2 (1%)
1986	237	82 (35%)	53 (22%)	92 (39%)	9 (4%)	0 (0%)
1987	252	70 (28%)	53 (21%)	119 (47%)	8 (3%)	1 (1%)
1988	282	66 (23%)	55 (20%)	151 (54%)	8 (3%)	1 (0%)

In recent years the controlled tipping of sludge (after dewatering) has become relatively intensive. The agricultural use of sludge in the Netherlands is subject to the strict Guidelines of the Water Authorities Association. These Guidelines set limit values for the heavy metal content of the sludge and for the maximum quantity which may be applied per hectare per year: 2 tons dry matter per hectare per year for crops and 1 ton dry matter per hectare per year for grassland. The new Soil Protection Act places more stringent limits on heavy metal concentrations, extends the limit values for organic micropollutants, and requires an analysis of farmland soil composition.

Table 2 gives present and future limit values for heavy metals and arsenic in sewage sludge for agricultural use.

Table 2: Heavy metal limits for sewage sludge for agricultural use (mg/kg dry matter)

	Limit values prior to January 1, 1991	Limit values 1991 to 1995	Limit values proposed 1995 to 2000
Zinc	2,000	1,400	300
Copper	900	425	75
Lead	500	300	100
Chromium	500	350	75
Nickel	100	70	30
Cadmium	5	3.5	1.25
Mercury	5	3.5	0.75
Arsenic	25	25	10

The objective is to create a situation in the year 2000 in which the use of fertilizers will introduce no more pollutants into the soil than is removed by the crops.

The quality of the sewage sludge has improved in recent years owing to a reduction in the discharge of heavy metals by industry. Nevertheless, a tightening of the regulations for the agricultural use of sewage sludge will result in a significant reduction in the quantity of sludge which is reused. Other means of disposal will have to be found. The same situation will occur with the reuse of sludge in compost and soil production. New legislation will also bring an end to this disposal method within a few years. This means that sewage sludge will change from a 'fertilizer' into a waste product. A minimization of the volume of the sewage sludge is thus becoming increasingly important.

For the short term the purchase of dewatering equipment and dumping of the dewatered sludge seems to be the only solution. Over the long term this will create a tendency towards an improvement in the mechanical dewatering of sludge and to a reduction of the quantity of the remaining solids by drying, composting or incineration. In all cases the use of dumping locations is inescapable. Environmental impact assessment procedures make the development of new controlled tipping sites a time-consuming activity. Better technical solutions, other than those mentioned above, will have to be developed.

In the following sections attention will be paid to the state of the art of sludge dewatering, sludge drying, sludge composting, sludge incineration and some promising new techniques.

2. Sludge dewatering

At this moment, in the Netherlands, the sewage sludge from approximately 100 wastewater treatment plants (WTPs) is mechanically dewatered. In most cases belt filter presses are installed. Filter presses and centrifuges are used less frequently. The use of centrifuges is on the increase.

Table 3 presents results obtained from a 1989 study of 28 belt filter presses.

Table 3: Dry solid content of sewage sludge dewatered by belt filter presses (number of installations : 28)

Oxidation ditch sludge	18% (range 16-21%)
digested sludge	21% (range 18-22%)

The dosage of polyelectrolyte is ± 4 kg/ton dry matter for both oxidation ditch sludge and digested sludge.

2.1 Recent developments

In 1989 and 1990 some full scale tests were performed using belt filter presses and centrifuges to process different types of sewage sludge. It was found that, because the dewatering of sewage sludge is becoming increasingly important, it is both necessary and possible to optimize both belt filter presses and centrifuges.

Table 4 presents a synopsis of the test results.

Table 4: Full scale dewatering test results: dry mater contents

	Range	Number of tests
oxidation ditch	24-34%	3
digested sludge	29-40%	9
mixed sludge	30-32%	3
aerobically stabilized sludge	30%	1

The dry matter concentrations obtained in the full scale tests are 5 to 10% higher than the results achieved in current practice. Centrifuges achieve the highest dry matter concentrations, although polyelectrolytes have to be used at a 50-100% higher dosage as compared with belt filter presses in order that centrifuges can achieve these results.

The full scale tests also indicated that a constant sludge composition is of great importance for the optimum performance of both belt filter presses and centrifuges. In the design of a sludge dewatering facility one factor which thus has to be taken into account is a sludge buffer to even out the sludge composition, coupled with a 24 hour per day operation of the dewatering unit.

A second factor which is also becoming more important due to the need to improve the operation of WTPs is the pollution generated by the sludge treatment. In particular, the mechanical dewatering of digested sludge reintroduces a non-negligible amount of nitrogen into the water line of the WTP.

3. Sludge drying

One of the methods by which the volume of the sludge can be reduced is by thermical drying after mechanical dewatering. In principle there are two drying methods. The oldest method is direct drying, in which there is a direct contact between the hot air or gas and the sludge. The second method is indirect drying. In this method there is no direct contact between the hot air or flue gas and the sludge.

A disadvantage of the direct drying method is that the drying medium (air or flue gas) is contaminated with odourous compounds, which means that, before the medium can be discharged into the open air, it is necessary to treat the flowing gas. It is exactly this odour aspect which militates against the use of a direct drying system. There is thus an increasing tendency towards the use of indirect drying systems.

Generally speaking the use of an indirect drying system is not economic for drying sludge to a dry solids concentration of more than 50%. However, new indirect drying systems have been developed which can reach dry solids concentrations of up to 95% in an economical manner. One such system, called the Exergy system, uses pressurized superheated transport steam to feed in the dewatered sludge. The sludge is dried in a multi-stage process.

As part of the WTP 2000 Research Programm a desk study has been undertaken to examine the possibilities of the U.S.-developed Carver-Greenfield process under Dutch circumstances. This process, just like the Exergy system, is a multi-stage drying process. The characteristic of the Carver-Greenfield process, however, is that a high boiling water insoluble carrier liquid is used (most frequently oil). This makes it possible to keep the sludge liquid during the drying process.

The process exists in both a light and a heavy oil variant. The system using light oil as carrier liquid produces a dried sludge with less than 1% oil. The use of heavy oil means that the concentration of the oil in the dried sludge can rise to 35%. The dried sludge from this process has to be used as a fuel.

In the WTP 2000 study most attention was paid to the light oil system, which provides a dried sludge which can be dumped. The light oil system, however, is not a proven technology, whereas some experience has been gained with the heavy oil system, which is in operation in Japan, for example.

Preliminary indications show a price per ton of dry solid which lies in the same range as other sludge handling techniques. For that reason the suggestion was made that, before introducing the process into the Netherlands, more information should be gained about operational experience in the U.S.A. and Japan.

4. Sludge composting

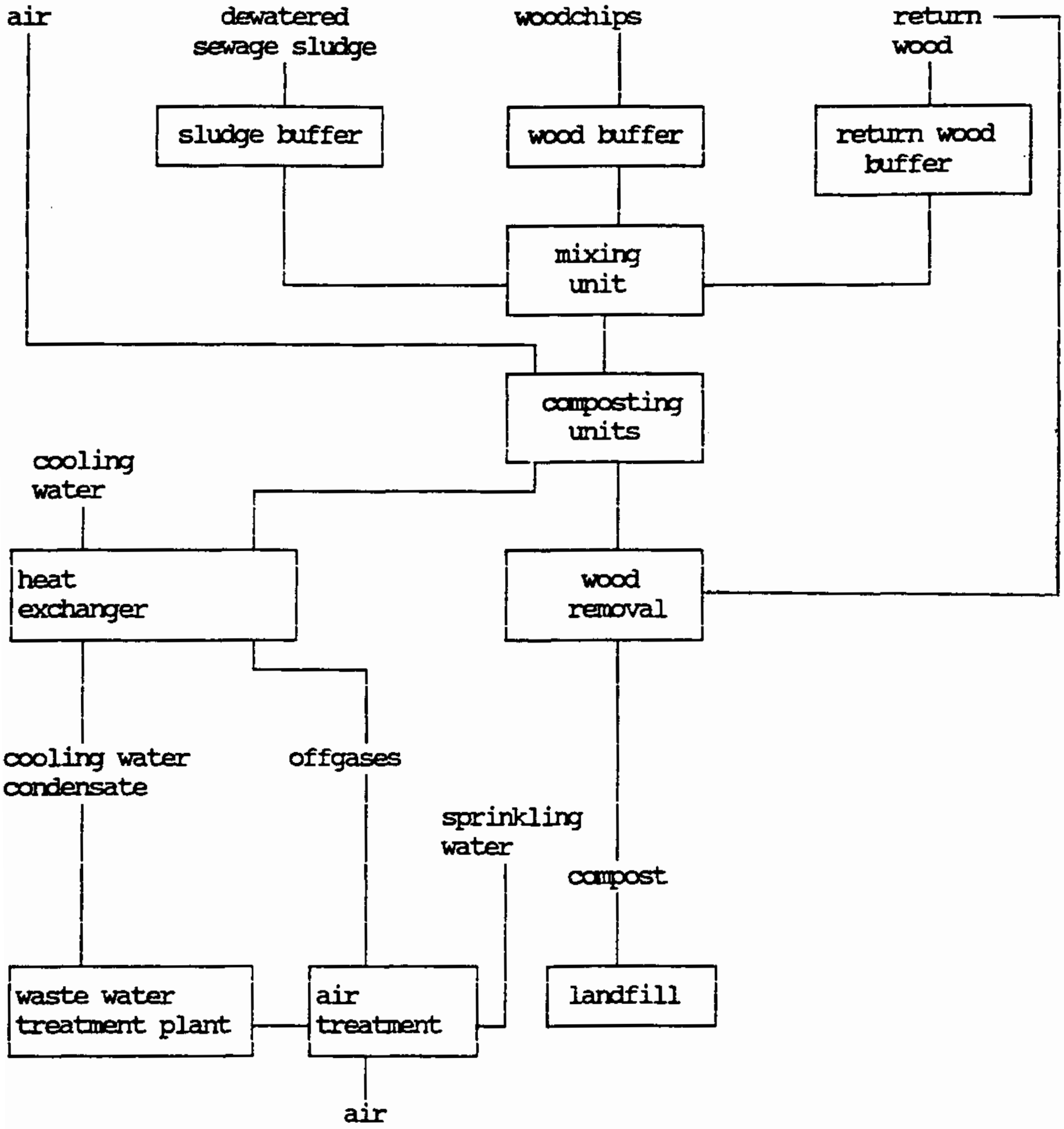
Sludge composting is mostly used to produce compost as fertilizer or for use in the soil production. Because of the increasingly strict regulations on the beneficial use of composted sludge, the use of compost as a fertilizer is no longer interesting. Composting, however, can also be used to reduce the volume of the sludge.

Composting is an aerobic microbiological process during which part of the organic matter present in the sludge becomes oxidized. A certain amount of heat is also produced. This results in a temperature of 50-70° C in the composting heap. The reduction in the volume is mostly achieved by the evaporation of water.

The province of Utrecht is to build a composting installation, due to come into operation in 1992, the capacity of which will be 15,000 tons dry matter per year. For the composting process the sludge will be mixed with wood chips in a wood : sludge ratio of 3:1. The installation will be a completely closed system through which the sludge will be transported by a 'walking floor'. Since the installation is closed it is necessary to use forced ventilation. The total amount of offgas is reduced by using air recirculation. The offgases will be treated in compost filters. Before the sludge is transported to a landfill the wood chips will be recovered.

Figure 1 shows a schematic of the process used in the compost installation.

Figure 1: Scheme of the process used in the compost installation of the Province of Utrecht



5. Sludge incineration

A maximum reduction of the volume of sludge which is to be dumped is achieved by incineration.

After mechanical dewatering with belt filter presses or centrifuges the sludge has a dry matter content of 16-22% (see Section 2). This means that it is not possible to burn the sludge without the addition of extra fuel. The introduction of a new generation dewatering unit might make it possible to incinerate the sludge autothermally in the future.

One barrier to the creation of an autothermal incineration process is formed by the new restrictions placed on the offgases before they can be discharged. Cooling of the flue gases followed by reheating for the purposes of treatment implies a large loss of heat. This means that it is necessary to dewater the sludge to more than 30-35% dry matter before an autothermal incineration becomes possible. The introduction of simultanous chemical phosphate removal demands an even higher dry matter concentration for autothermal incineration.

Present circumstances and the (near) future nevertheless constrain the designer of an incineration plant to start on the basis of sewage sludge containing approximately 18-20% dry matter, which means that external energy will be needed for the incineration process. It therefore pays to build an installation with an optimum energy budget.

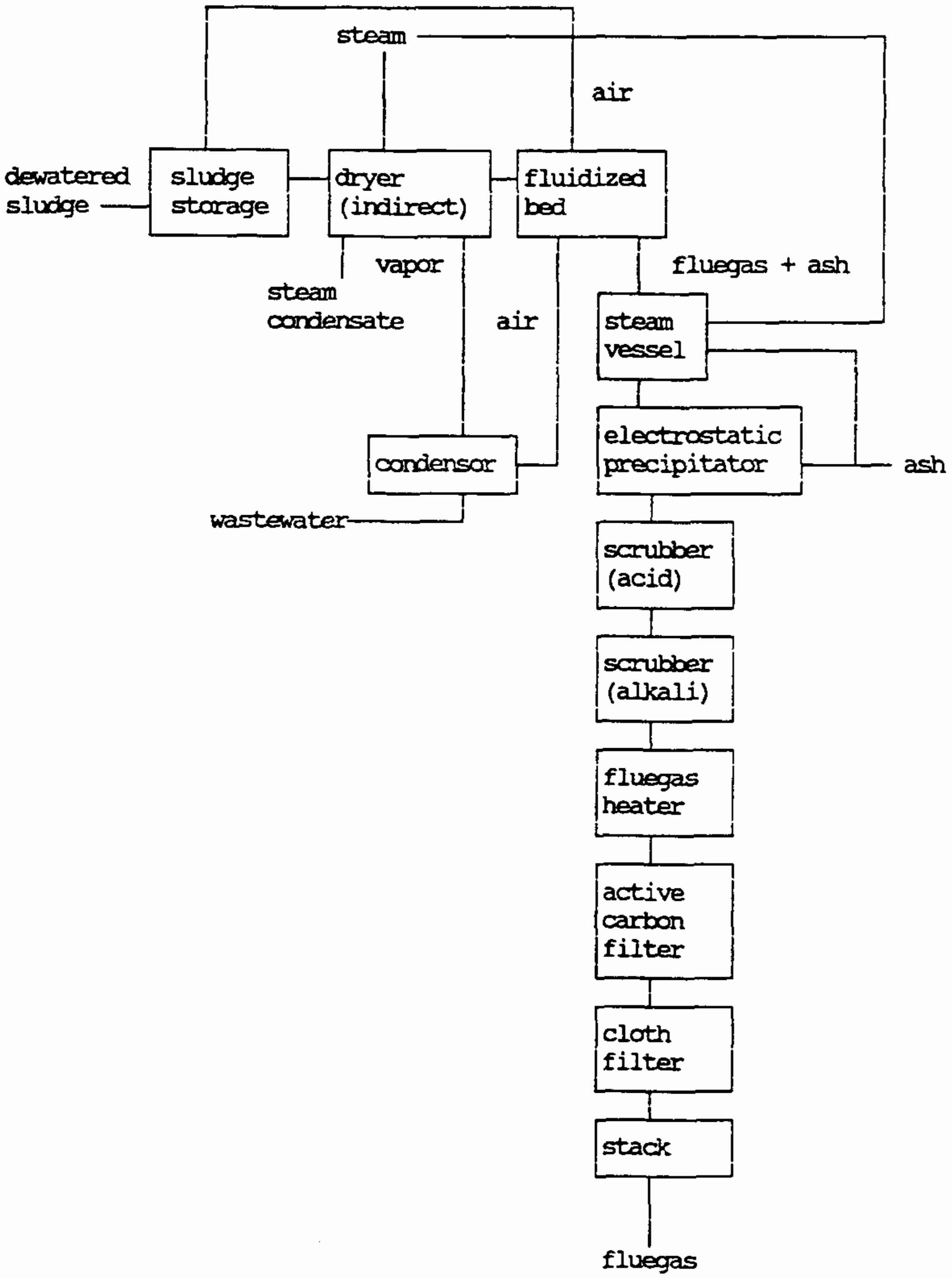
In the Netherlands there are at present three sludge incineration plants in operation. Yearly about 10,000 tons of dry solids is incinerated. One example of a modern sludge incineration plant in the Netherlands is the plant which is to be built in Dordrecht. The design criteria for this plant are presented in Table 5.

Table 5: Design criteria for the Dordrecht sludge incineration plant

- design capacity	45,000 ton dry matter/year
- number of furnaces	3
- capacity per furnace	2.25 ton dry matter/h
- operation hours	6,667 h/year
- dry matter concentration dewatered sludge	19%
- range dry matter concentration dewatered sludge	16-24%
- organic matter concentration (including chemical phosphate removal)	50%
- amount of sludge	238,850 ton/year
- reserve capacity	20%

Figure 2 shows the process flowsheet of the plant.

Figure 2: Process flowsheet of Dordrecht sludge incineration plant



The process can be describes as follows.

The mechanically dewatered sludge, having a dry matter content of about 19%, will be dried in an indirect sludge dryer to 40% dry matter. The medium used for drying the sludge is steam which is partly derived from a garbage incineration plant and partly from heat recovery from the hot flue gases. After drying the sludge will be incinerated in a fluidized bed. This is at present the only type of incineration system which can achieve the very strict emission standards, after treatment of the flue gases.

The treatment of the flue gas is a very important part of the incineration plant. The emission standards for flue gases in the Netherlands (Guideline on Incineration, August 1989) and the German TA Luft are given in Table 6.

Table 6: Emission standard for flue gases (in mg/Nm^3)

	Guideline on Incineration, August 1989	TA Luft 1986
- total solids	5	30
- HCl	10	50
- HF	1	3
- CO	50	100
- organic compounds (as C)	10	20
- SO_x	40	100
- NO_x	70	500
- heavy metals:		
Sb+Pb+Cr+Cu+Mn+V+Sn+As+Ni+Se+Te	1.0	5.0
Cd	0.05	0.1
Hg	0.05	0.1
- PCDDs and PCDFs	0.1 ng TEQ/Nm^3	

It will be noticed that the Dutch emission standards for flue gases are among the most strict in the world. The measures required for the achievement of this standard are discussed below for some specific components.

- Total solids. Normally it is possible to obtain a concentration of total solids of 5 mg/Nm^3 with the use of an electrostatic precipitator and a two-stage wet chemical scrubber. Because an active carbon filter is necessary (see below) a cloth filter is required downstream in order to eliminate small active carbon particles.
- HCl, HF, SO_x. These components are eliminated in the two-stage wet

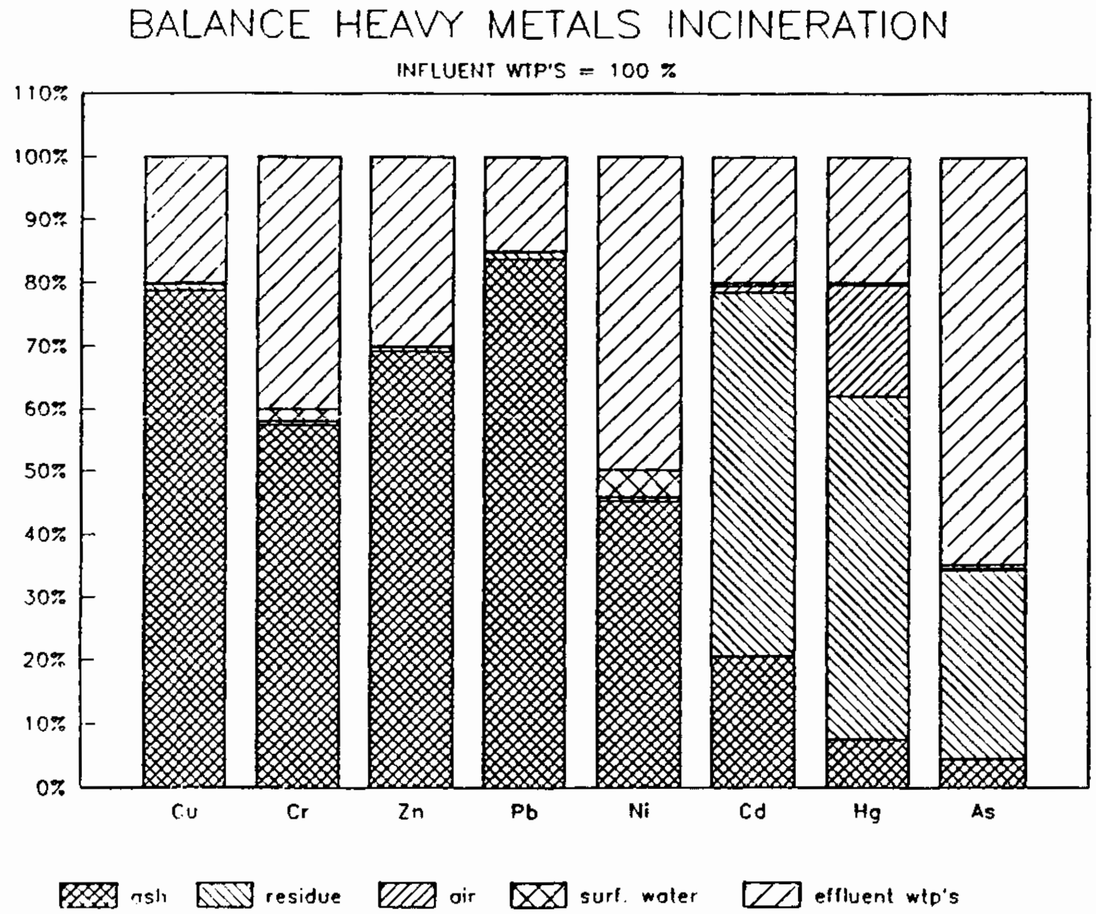
chemical scrubber. In order to diminish the SO_x emission from the fluidized bed a lime injector will be installed in the fluidized bed.

- CO, C_xH_y. Normally the emission of these components from a fluidized bed is no problem. An afterburner chamber will be incorporated in the installation. In this chamber the flue gas will attain a temperature of more than 850°C for a minimum of 2 seconds.
- Heavy metals. The heavy metals are mostly attached to the solids (fly ash). Thus the removal of the total solids also removes most of the heavy metals.
  Mercury (Hg) and Cadmium (Cd) have a certain volatility in the temperature range of the flue gas. These components are mostly eliminated in the acid scrubber. There is some doubt about the stability of the removal of mercury in these scrubber units. Since the emission standard for mercury is very strict an active carbon filter will be installed for reasons of safety.
- PCDDs and PCFDs. There is a lower production of PCDDs and PCDFs when sludge is incinerated than there is with the incineration of garbage since sludge contains no synthetic polymeric organochlorine materials. The shape of the furnace selected provides an extra safety factor. The circumstances under which the incineration is conducted, too, in combination with the afterburner chamber, provides a minimum risk of PCDD and PCDF production. In combination with the active carbon filter this allows the strict standard for PCDDs and PCDFs to be attained.
- NO_x. The incineration conditions used allow the production of NO_x to be avoided to a large degree. It is very difficult to predict the NO_x concentration in the flue gas in advance. The approach is first to take the incineration plant into operation and to measure the NO_x concentration in the flue gas. If, unexpectedly, the NO_x emission standard is not achieved, then extra measures will have to be taken.

An important factor in the discussion of any system for handling sludge is the environmental impact of the system. In the incineration of sludge a very important factor is the amount of pollution introduced by the stack into the environment. Sometimes these emissions of pollutants in the flue gas form the reason for a decision in favour of sludge drying, for example.

Figure 3 illustrates the balance of some heavy metals over wastewater treatment and sludge handling processes. The 100% level represents the total amount of heavy metals coming into the wastewater treatment plant with the influent. It will be noticed that only for mercury is the total amount which is rediffused into the environment non-negligible.

Figure 3: Heavy metal balance, sludge wastewater treatment and sludge incineration.



The cost of the incineration plant in Dordrecht is calculated at this moment to be D.Fl. 660.00/ton dry solids. Table 7 presents the division of this cost over several areas.

Table 7: Estimated cost of the Dordrecht sludge incineration plant

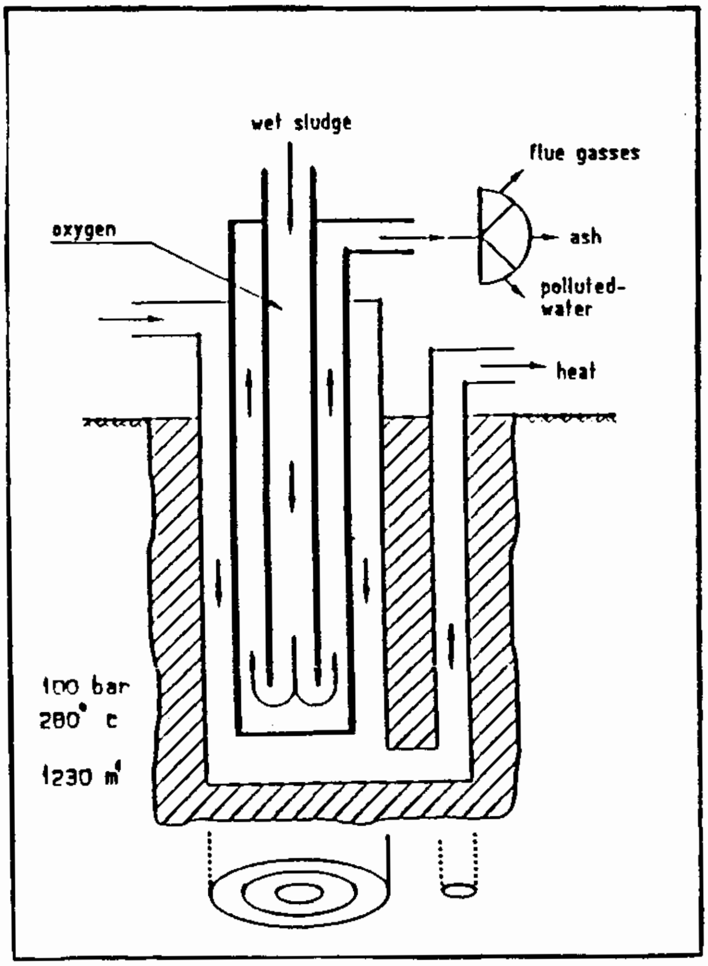
	f/ton dry solid
- capital cost	342
- maintenance	71
- personnel	20
- steam/electricity	54
- chemicals	60
- transport sludge to incineration plant	40
- transport and disposal residual roducts	56
- sundries	18
total	660

6. New techniques

A new sludge handling technique which will come into operation in 1991 in the Netherlands is the Vertech wet oxidation process. The Water Authority of the Veluwe has opted for this process.

The technology of the process the use of the priciple of aqueous-phase oxidation (APO) commonly called wet air oxidation has been developed in the U.S.A. The first full-scale installation to investigate this process was built in Longmont (U.S.A.). This installation has a capacity of 23,000 tons dry matter per year. Figure 4 gives a schematic impression of the installation.

Figure 4: Vertech Reactor



Under normal conditions organic compounds like sewage sludge will not react significantly with oxygen under aqueous conditions. The special process conditions used in the APO process makes direct oxidation possible. These conditions are:

- temperature higher than 200° C;
- pressure higher than the corresponding vapor pressure of water;
- exposure time longer than 5 minutes.

Essentially, the Vertech reactor consists of two concentric pipes (downcomer and upcomer) having a length of 1200 m. The sludge, having a dry matter content of about 5%, enters the deep well reactor through the central pipe (downcomer). At a certain depth pure oxygen is added. Initially, the process mixture is heated by an external heater via a heat exchange system. After the process has started heat is exchanged between the effluent leaving the reaction zone and the incoming sludge. The oxidation process starts at a temperature of approximately 175° C. As soon as the oxidation takes place, the temperature of the process mixure will increase as a result of exothermic combustion reactions, which take place in the lower part of the reactor.

Temperature and pressure increase as the process mixure flows down through the reactor. Near the bottom the weight of the liquid column creates a pressure of about 100 bar. This high pressure is achieved without incurring high pumping costs. The feed pump needs to compensate for the small pressure differential between influent and effluent. The temperature near the bottom is approximately 275° C.

The oxidized liquid flows back to the surface through the upcomer of the reactor system. The effluent from the reaction vessel flows directly to the gas/liquid/solids separator. The gas is stripped off, the separated solids (ash) is mechanically dewatered in a filter press and the remaining effluent is treated in a wastewater treatment plant.

The costs of the process are approximately D.Fl. 680.00/ton dry matter.

7. Dumping

As a result of the very strict Government regulations it will be very difficult in the near future to use sludge as a fertilizer in the Netherlands. This means that, regardless of the system used for sludge handling, the end products will have to be dumped. There is a standard in the Netherlands which stipulates that the dumping sites have to be constructed and operated according to certain rules. Thus the dumping sites have to be isolated by means of PVC or a clay layer. The whole dumping site has to be controlled so that if, for example, there is a leakage of percolation water it should be monitored.

We have no experience in the Netherlands with the dumping of large quantities of dried sludge which contains a large percentage of organic matter (> 50%). When organic matter is dumped it starts to ferment, depending on the ambient humidity. One possible risk is that the high dry matter concentration will result in a relatively long hydrolysis and acid production phase. This is undesirable. The dumping of ash from incineration plants does seem to be feasible.

8. Conclusion

New more stringent legislation for the reuse of sewage sludge will bring an end to the disposal of this material as fertilizer. Sewage sludge will change from a fertilizer into a waste product. For the short term mechanical dewatering and dumping seems to be the only solution. Over the long term there will be a tendency towards an improvement in the mechanical dewatering of sludge and to a reduction of the quantity of the remaining solids by drying, composting and incineration. In all cases the endproduct will be controlled tipped.

9. Literature (in dutch)

- NVA Programmagroep 3, Symposiumbundel zuiveringsslib, een probleem van de eerste orde? d.d. 8 november 1990 te Amsterdam;

- Dijkema-Bolkestein mr. J.A., ir. P.J. Tessel, H.E.J. Rademaker VerTech-natte oxydatie; techniek en privatisering, Waterschapsbelangen 1991, nr. 2, pag. 37 t/m 42.

TWO METHODS OF NITROGEN REMOVAL IN JAPAN

Y. Hamamoto[*], Y. Okubo[*] and H. Nonaka[**]
[*] Nishihara Environmental Sanitation Research Corp. Ltd.
[**] Nishihara OM-Tech. Corp. Ltd.

TWO METHODS OF NITROGEN REMOVAL IN JAPAN

Y.Hamamoto*, Y.Okubo* and H.Nonaka**

* Nishihara Environmental Sanitation Research Corp.,Ltd.
3-6-18 Shibaura, Minato-ku, Tokyo, 108, Japan

** Nishihara OM-Tech. Corp., Ltd.
3-6-18 Shibaura, Minato-ku, Tokyo, 108, Japan

1 INTRODUCTION

There has been a serious problem in Japan that eutrophication occurs in enclosed water bodies such as Kasumiga-Ura, Tokyo Bay, Seto Inland Sea, and Lake Biwa. In the conventional wastewater treatment the main purpose is the removal of organic substances. But in order to prevent eutrophication, it is important to reduce nitrogen and phosphorus in wastewater.

One of the nitrogen removal methods is the physical and/or chemical methods. Another is the biological methods. But the physical and/or chemical methods cost more and make more sludge. So recently the biological methods are more favorable.

Biological nitrogen removal can be generally classified into the following two systems. The first is the multi-tank system, which usually uses two or more tanks. For example,the A^2/O process and the Bardenpho process are included in this system. In the multi-tank system, nitrification and denitrification occur in different tanks. The role of the tanks is separated by space.

The second system is the single tank system. This contains batch systems and intermittent aeration systems. In the single tank system, nitrification and denitrification occur in the same tank. The role of the tank is separated by time. The single tank system is simple in construction, easy to control and does not need recycling of mixed liquor. This paper introduces the operational results of two methods of the single tank system in Japan.

2 SIMULTANEOUS NITROGEN AND PHOSPHORUS REMOVAL BY THE INTERMITTENT CYCLIC PROCESS

2.1 INTRODUCTION

We have developed the Intermittent Cyclic process to remove nitrogen and phosphorus simultaneously.(Hayakawa el al., 1986) This process is one of the batch activated sludge treatment processes, and it consists of four processes: mixing, aeration, settling and drawing in one cycle. We conducted both 20-liters laboratory studies and 18-m^3 pilot plant study.

2.2 OUTLINE OF INTERMITTENT CYCLIC PROCESS

Figure 1 shows a schematic diagram of the Intermittent Cyclic process. The reactors are operated under time control, in one cycle consisting of four processes: mixing, aeration, settling and drawing.

Two reactors are used and operated alternately. While reactor A is in mixing and aeration, reactor B is in settling and drawing, and influent wastewater is fed to reactor A. After predetermined time is passed, reactor A turns to settling and drawing, reactor B turns to mixing and aeration, and feeding of influent wastewater is switched to reactor B. In this manner, two reactors are operated alternately, so that continuously fed wastewater can be treated.

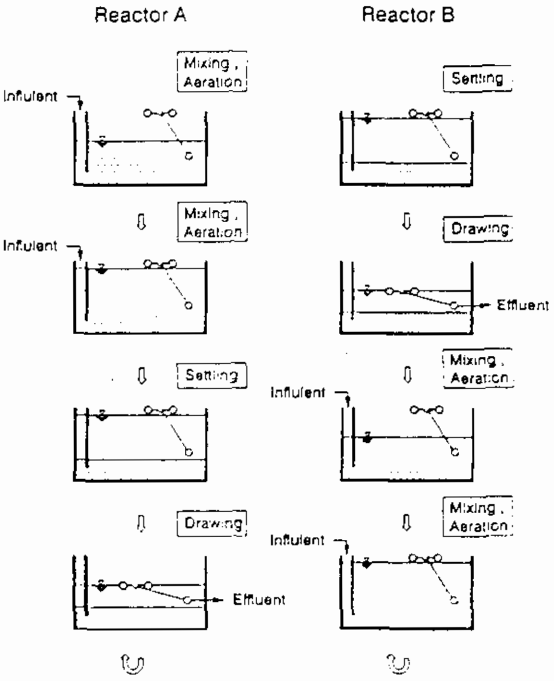


Fig.1 Schematic diagram of the Intermittent Cyclic process

The mixing and aeration are repeated 3 times in one cycle to make anaerobic and aerobic conditions for simultaneous removal of organic substances, nitrogen and phosphorus. In the settling, solid-liquid separation occurs. In the drawing, the decanter lowers slowly and draws out the clear effluent from the top quietly. When the decanter lowers at the predetermined low water level, the decanter automatically goes up to the standby position. This completes one cycle. Several cycles of operation are performed in one day.

2.3 MATERIALS AND METHODS

Laboratory apparatus

Figure 2 shows a schematic diagram of laboratory scale reactors. The volume of each reactor was 20 liters at high water level, 15 liters at low water level. The reactors were stirred by the mixers during mixing and aeration, and had electric solenoid valves for drawing the supernatant except Run 1. A solid separator with a pad to separate the supernatant from the activated sludge was used only in Run 1 for continuous aeration process. The microtube pumps fed the influent to the reactors. All apparatus were controlled by times.

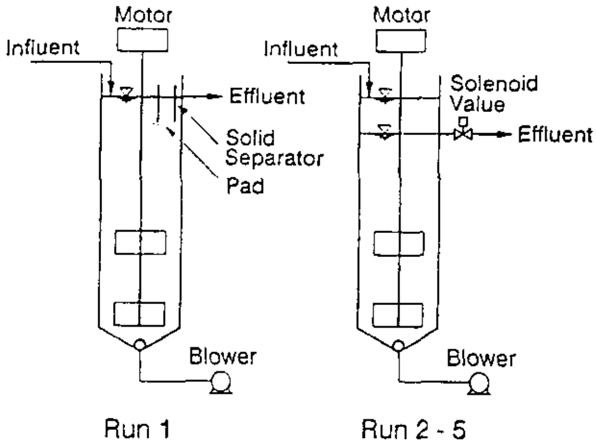


Fig.2 Schematic diagram of the laboratory apparatus

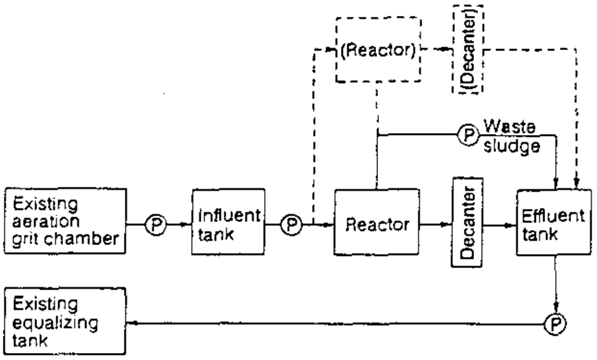


Fig.3 Flow diagram of the experimental pilot plant

Pilot plant

A flow diagram of the experimental pilot plant is shown in Figure 3. As illustrated in Figure 1, this process originally uses two reactors. But the experiment used only one reactor on the assumption that two reactors are under the same conditions.

The reactor was a steel plate tank, 1.5 m wide × 5.6 m long × 2.5 m high. The volume of the reactor was 18 m^3 at high water level, 12 m^3 at low water level. The reactor had a baffle on the wastewater inlet side (see Figure 1). The baffle was open at the bottom to allow wastewater intake. The reactor had a decanter consisting of a weir to draw out the supernatant, pipes and scum stopper floats. The decanter was driven by a hydraulic cylinder, and moved up and down between the predetermined levels at a constant speed. It quietly drew out the supernatant. Excess sludge was withdrawn by a submersible sludge pump.

Experimental conditions

The experimental conditions are shown in Table 1 and the operating conditions are shown in Figure 4. The water temperature was 20 ℃ in laboratory studies and 11.2 - 24.0 ℃ in the pilot plant study. The number of cycles was 4 cycles per day. The operating conditions were continuous aeration (Run 1), 3 hours of aeration (Run 2), 3 times of 15 minutes of mixing and 45 minutes of aeration (Run 3), 3 times of 30 minutes of mixing and 30 minutes of aeration (Run 4), 3 times of 45 minutes of mixing and 15 minutes of aeration (Run 5 and the pilot plant study). Settling was 1 hour and drawing was 2 hours. The aeration time ratio can be defined by the following equation.

$$\text{aeration time ratio}(-) = \frac{\text{aeration time in one cycle(hours/cycle)}}{\text{one cycle time(hours/cycle)}}$$

Thus the aeration time ratio was 1 as Run 1, 4/8 as Run 2, 3/8 as Run 3, 2/8 as Run 4, 1/8 as Run 5 and the pilot plant study. Influent was fed continuous (Run 1), in mixing and aeration (Run 2 - 5 and the pilot plant study). The composition of synthetic wastewater which was used in laboratory studies is shown in Table 2. Domestic wastewater was fed in the pilot plant study.

TABLE 1 Experimental Conditions

Run number	1	2	3	4	5	pilot
Water temperature(^{o}C)	20	20	20	20	20	11-24
Number of cycles (cycles/day)	-	4	4	4	4	4
Flow rate (l/day)	20	20	20	20	20	18m^3
Volume of reactor(l at HWL)	20	20	20	20	20	18m^3
(l at LWL)	-	15	15	15	15	12m^3
Aeration time ratio (-)	1	4/8	3/8	2/8	1/8	1/8
BOD-SS load (gBOD/gSS/day)	0.096	0.090	0.085	0.083	0.083	0.059

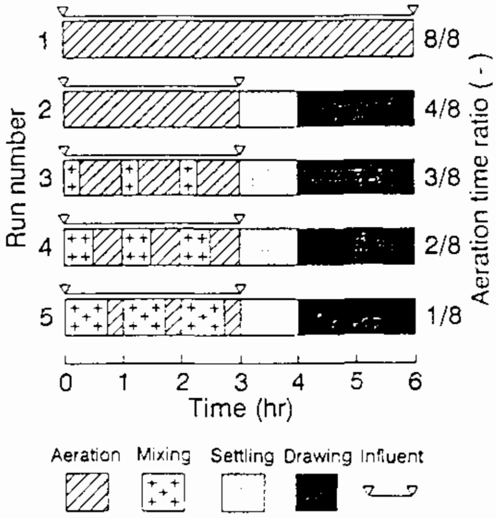


Fig.4 Operating conditions

TABLE 2 The Composition of Synthetic Wastewater (Laboratory Studies)

Composition	Concentration(mg/l)
Peptone	180
Meat extract	120
$Na_2HPO_4 \cdot 12H_2O$	25.2
$CaCl_2 \cdot 2H_2O$	5.56
$MgSO_4 \cdot 7H_2O$	6.2
KCl	4.2
NaCl	90
NaHCO3	250

- Measured values(mg/l) : BOD = 169, T-N = 35.8, T-P = 4.5

Analytical methods

All analysis was done in accordance with JIS K0102 or The Analytical Methods for Sewage (in Japan).

2.4 RESULTS AND DISCUSSION

Aeration time ratio and nitrogen, phosphorus removal

In laboratory studies, several operating conditions were compared. Run 1 was the conventional continuous aeration process. Run 2 was the batch process that had only aeration. Run 3 - 5 had repeated mixing and aeration for simultaneous nitrogen and phosphorus removal.

Figure 5 shows the relationship between the aeration time ratio and the nitrogen and phosphorus removal rate. Figure 6 shows the relationship between the aeration time ratio and the T-P content of sludge and the yield of sludge. The TKN removal rate was constantly high even under the condition that the aeration time ratio was 1/8. The T-N removal rate decreased largely as the aeration time ratio increased, and was lowered about 10 % in the condition that the aeration time ratio was 1, that is, in the continuous aeration process (Run 1).

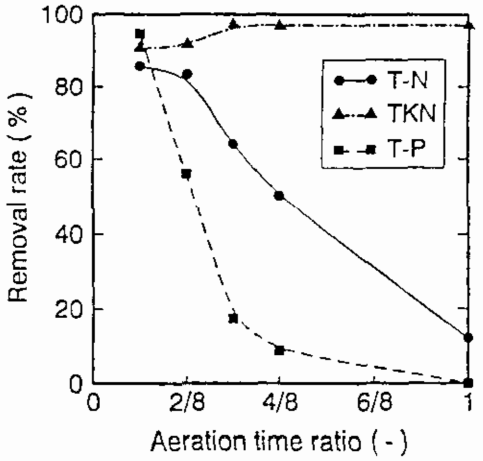


Fig.5 Relationship between aeration time ratio and nitrogen and phosphorus removal rate

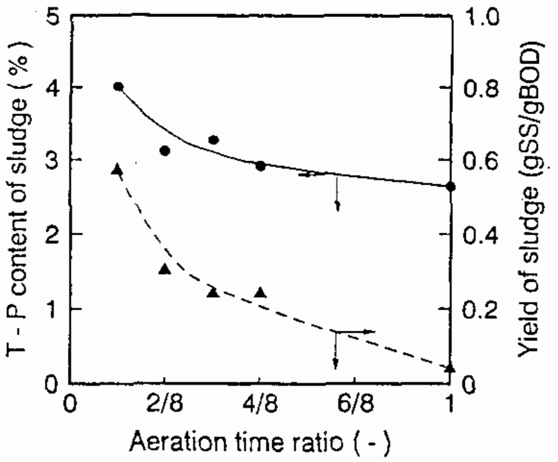


Fig.6 Relationship between aeration time ratio and T-P content of sludge and yield of sludge

Similarly the phosphorus removal rate decreased rapidly as the aeration time rate increased, and no phosphorus removal occurred in the condition that the aeration time ratio was 1. It is due to NOx-N (= NO_2-N + NO_3-N) not being denitrified enough in mixing when the aeration time ratio increased and the remaining NOx-N preventing release of phosphorus. This also proved that T-P content of sludge was 4 % in Run 5, although 2.5 % in Run 1 from Figure 6. But the yield of sludge increased as the aeration time ratio decreased.

From laboratory studies, Run 5 showed the highest removal rate of nitrogen and phosphorus, so the operating condition of Run 5 was adopted in the pilot plant study.

Results of pilot plant operation

Figure 7 shows variations in the water temperature and influent and effluent BOD for eight months. The influent showed a little variation. The figure is represented by monthly means. The reactor water temperature in the experiment was between 11.2 ^{o}C and 24.0 ^{o}C. The results of the removal of organic substances were good. The influent BOD values were in the range of 170 - 290 mg/l, the effluent BOD values were below 20 mg/l, its average was 8.1 mg/l.

Figure 8 shows variations in influent and effluent T-N. The influent T-N values were in the range of 30 - 42 mg/l, the effluent T-N values were below 10 mg/l with some dispersion, its average was 5.0 mg/l. The mean of the nitrogen removal rate was 86 %. It was the same value as in Run 5 of laboratory studies.

Figure 9 shows variations in influent and effluent T-P. The influent T-P values were in the range of 4 - 5 mg/l, the effluent T-P values were almost below 1.5 mg/l with some dispersion, its average was 0.79 mg/l. The mean phosphorus removal rate was as high as 82 %. Simultaneous nitrogen and phosphorus removal was also achieved in the pilot plant study.

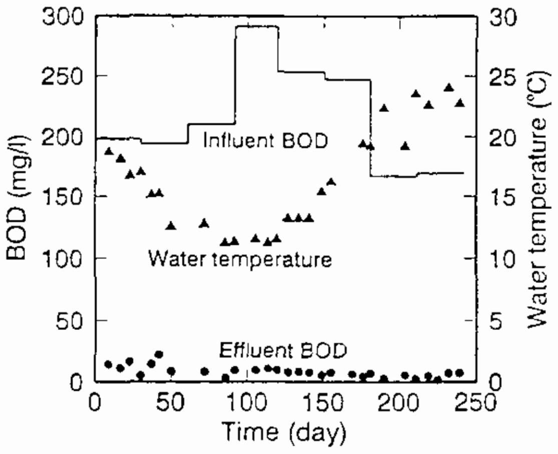


Fig.7 Variations in water temperature and influent and effluent BOD

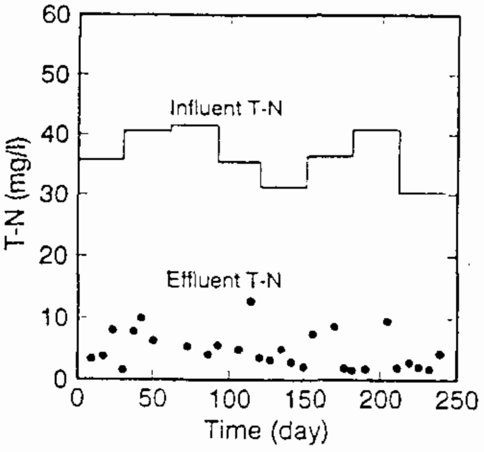


Fig.8 Variations in influent and effluent T-N

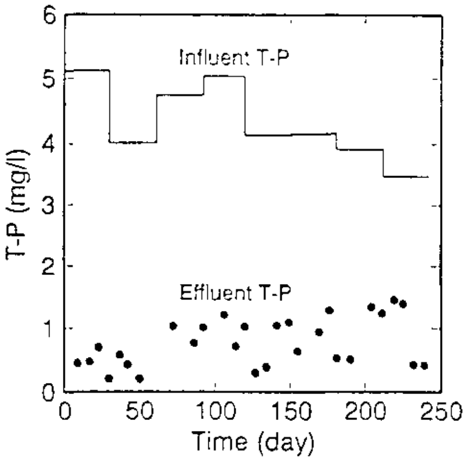


Fig.9 Variations in influent and effluent T-P

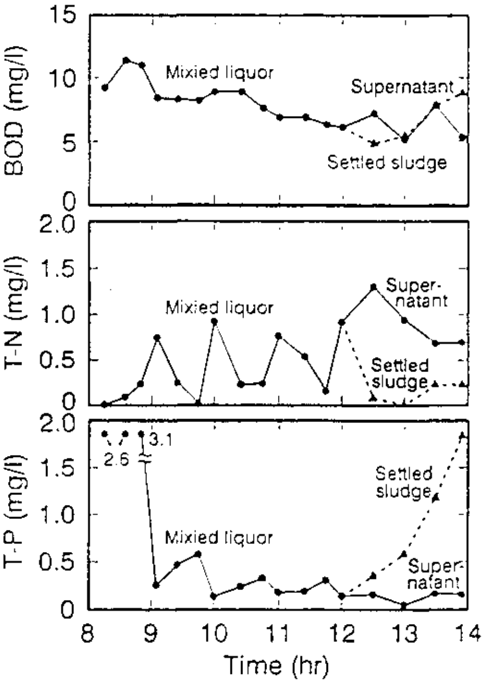


Fig.10 BOD, NOx-N and T-P variations in one cycle

Variations in one cycle

Figure 10 shows BOD, NOx-N and T-P variations in one cycle. The samples, obtained from the reactor mixed liquor, were filtered through filter paper No. 5 C and subjected to an analysis of soluble components. First, BOD concentration decreased linearly, when aerobic or anaerobic conditions were conducted.

NOx-N declined like zero-order reaction caused by denitrification in mixing, and rose like zero-order reaction by nitrification in aeration. In the repetition of alternated aerobic/anaerobic conditions during mixing and aeration, NOx-N rose and declined concurrently.

T-P changed in reverse compared with that of NOx-N. In other words, T-P was raised by the release of phosphorus from cells in the anaerobic condition of mixing, while T-P was lowered through the rapid uptake of phosphorus caused by biological luxury uptake in the aerobic condition. Similarly to NOx-N , T-P declined and rose in the repeated alternation of aerobic and anaerobic conditions with its variation diminishing hourly.

During the settling and drawing, phosphorus was released in the sludge zone in the bottom of the reactor, and T-P of the supernatant was maintained low. It proves that phosphorus released in the sludge zone would not enter the supernatant at least during settling and drawing.

2.5 CONCLUSIONS

Our study conducted 20 liters laboratory studies and 18 m^3 pilot plant study. We used the Intermittent Cyclic process designed for wastewater treatment. The process included alternating aeration and mixing. The results were the following:

(1) In laboratory studies, the aeration time ratio from 1 to 1/8 were compared. Nitrogen and phosphorus removals increased as the aeration time ratio decreased.

(2) Nitrogen and phosphorus removal rates were most heightened in the operation condition, Run 5, that is, 3 times of 45 minutes of mixing and 15 minutes of aeration in laboratory studies.

(3) Eight months of operation in the pilot plant recorded high nitrogen removal rate: mean 86% and maximum 96%, phosphorus removal rate: mean 82% and maximum 93% about same as laboratory studies.

(4) In one cycle, NOx-N of the reactor declined in the anaerobic condition and rose in the aerobic condition. The T-P concentration showed, in reverse, an increase during mixing and a decrease during aeration.

3 NITROGEN REMOVAL BY AUTOMATIC CONTROL OF AERATION TIME IN THE INTERMITTENT AERATION ACTIVATED SLUDGE PROCESS

3.1 INTRODUCTION

Recently, the activated sludge process using the intermittent aeration has attracted general attention (for example Iwai _et al_., 1981). With the intermittent aeration, the process can achieve not only high nitrogen removal but also reductions in the electric power costs of the blowers. In previous intermittent aeration operations, timers have been used to switch on and off at preset times, however these system could not adjust to actual variations in flow rate, influent quality and the activated sludge concentrations (MLSS) frequently seen in wastewater treatment plants.

To make the treatment process appropriate for variations in flow rate, influent quality and MLSS, we have developed the system in which we measure and calculate the respiration rate of the microorganisms within every brief segment of the aeration period,and by calculating the integral of the respiration rate, obtain the oxygen consumption amount during the aeration period of the present cycle.(Hamamoto _et al_., 1990)

This allows us to calculate the necessary aeration time for the next cycle and to control the aeration time with an on-off control of the aeration equipment. We applied this system to nine existing wastewater treatment plants to control the aeration times automatically, and achieved high nitrogen removal.

3.2 CONTROL THEORY

The dissolved oxygen (DO) mass balance in a complete mixing aeration tank is shown in Figure 11. Variations in DO concentration can be expressed as in equation (1) from the DO mass balance.

$$V\,dC/dt = 1/24\ Q(1+r)C_o - 1/24\ Q(1+r)C + K_La(C_s-C)V - R_rV \quad (1)$$

where Q : flow rate (m^3/day)
V : volume of the aeration tank (m^3)
r : sludge recycle ratio (-)
C : DO concentration in the aeration tank (mg/l)
C_o : DO concentration in the liquor transported to the aeration tank (mg/l)
C_s : saturated DO concentration (mg/l)
R_r : respiration rate (mg/l-hour)
dC/dt: variations in DO concentration in the aeration tank (mg/l-hour)
K_La: overall oxygen transfer coefficient (1/hour)

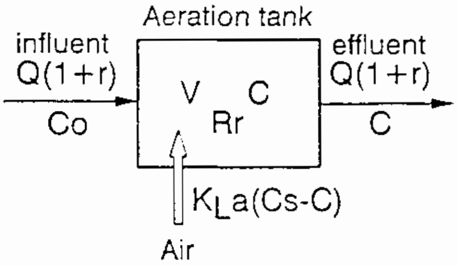


Fig.11 Dissolved oxygen mass balance in a complete mixing aeration tank

The respiration rate (R_r)of the activated sludge can be expressed as in equation (2) from equation (1).

$$R_r = 1/24\ Q(1+r)/V\ (C_o-C) + K_La(C_s-C) - dC/dt \quad (2)$$

Here attention should be paid to the magnitude of each coefficient. For example, in the wastewater treatment plant of the extended aeration process under the supposition that the terms are usually given in the following ranges:

$Q = 0.2Q_o - 1.0Q_o$
$r = 1 - 3$
$V = Q_o$
$K_La = 1.5 - 5$

Q_o : design flow rate (m^3/day)

The coefficient of the first term of the equation (2) is $1/24\ Q(1+r)/V = 0.017 - 0.17$ and the coefficient of the second term is $K_La = 1.5 - 5$.

Because the coefficient associated with the first term is only 0.003 - 0.11 as large as the coefficient of the second term, the first term can usually be ignored. Then equation (2) can then be expressed as in equation (3).

$$R_r = K_La(C_s-C) - dC/dt \tag{3}$$

The oxygen consumption amount (O_2) completed by the activated sludge during one aeration cycle can be expressed as in equation (4).

$$O_2 = V \int_0^{To} Rrdt\ 10^{-3} \tag{4}$$

O_2 : oxygen consumption amount within one cycle (kgO_2/cycle)
To : aeration time within one cycle (hours/cycle)

The oxygen transferred to the aeration tank (OC) can be expressed as in equation (5).

$$OC = K_La\ C_s\ V\ 10^{-3} \tag{5}$$

OC : oxygen transferred to the aeration tank (kgO_2/hour)

The necessary aeration time within one cycle (T_a) can be expressed as in equation (6).

$$T_a = O_2/OC\ S_f \tag{6}$$

T_a : necessary aeration time within one cycle (hour/cycle)
S_f : surplus factor (-)

K_La can be expressed as in the equation (7) after correction is made for the water temperature and wastewater (Eckenfelder and O'Connor,1961).

$$K_La = \alpha\ \theta^{(T-20)}\ K_La(20) \tag{7}$$

α : (K_La in waste water)/(K_La in tap water) (-)
θ : temperature coefficient of K_La (-)
T : temperature in the aeration tank (℃)
$K_La(20)$: K_La at 20 ℃ in tap water (1/hour)

Saturated DO (C_s) in the aeration tank can be expressed as in equation (8) after correcting for the water temperature, air-diffused submergence depth and wastewater (Eckenfelder and O'Connor,1961).

$$C_s = \beta\ (1+H/20.66)(14.16-0.3943T+0.007714T^2-0.0000646T^3) \tag{8}$$

β : (C_s in waste water)/(C_s in tap water) (-)
H : air-diffused submergence depth (m)

Using equations (3) through (8), the aeration tank volume (V) and air-diffused submergence depth (H) can be specified for a particular plant. When the K_La of the plant is determined through measurements, and the DO (C) and the water temperature (T) in the aeration tank are evaluated, the necessary aeration time can be calculated. This aeration time can be used for the next aeration cycle.

This concept can be explained by a simple figure, Figure 12. Figure 12 means the following:

(1) Supposing that there is an aeration tank filled with tap water without DO and the aeration is started, the DO concentration of the tank will increase according to the theoretical ascending DO curve like 1.

(2) When the activated sludge exists in the tank and influent wastewater enters into the tank, the DO will increase according to the curve like 2.

(3) At any brief segment the difference between curve 1 and curve 2 corresponds to the respiration rate of the activated sludge.

(4) Area 3 which is surrounded by curve 1 and curve 2 means the total oxygen consumption in the first aeration cycle.

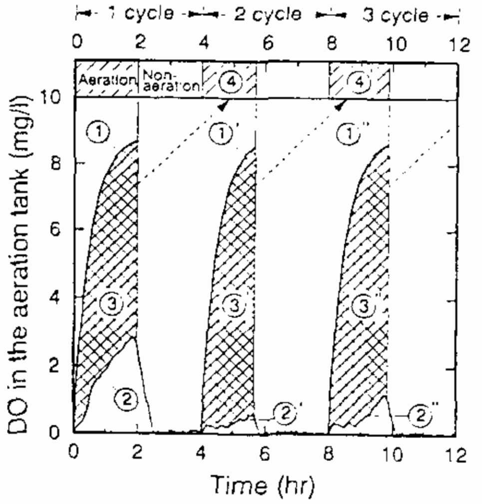
(5) The oxygen transferred to the tank can be calculated by using K_La. The aeration time 4 in the second cycle is calculated to divide the total oxygen consumption of the first cycle by the oxygen transferred to the tank.

(6) After the second aeration cycle, the same processes from (1) to (5) are repeated, so the aeration time is decided one by one.

In Figure 12, when the DO is high like in the first cycle, the total oxygen consumption is small, so the next aeration time is shortened. When the DO is low like in the second cycle, the total oxygen consumption is large, so the next aeration time is prolonged.

This system assumes that the influent load of the next cycle roughly equals the load of the present cycle. But the influent load of a plant sometimes increases or decreases suddenly. In these cases if we use only calculated aeration times, aeration is not enough or is in excess.

To correct these defects, we have a backup system in this process. If the DO of the tank is lower than the predetermined minimum DO level at the time which aeration stops, the aeration time is prolonged. If the DO is higher than the predetermined maximum DO level, the aeration time is shortened.



① Theoretical ascending DO curve in tap water
② Real ascending DO curve in an aeration tank
③ The total oxygen consumption of this cycle
④ The aeration time of the next cycle

Fig.12 Simple figure of this concept

3.3 MATERIALS AND METHODS

Experimental plants

An outline of the experimental plants is shown in Table 3. There were seven experimental plants of the extended aeration process with a design flow rate from 54 to 590 m^3/day, and two plants of the conventional activated sludge process with a design flow rate from 2,565 to 3,100 m^3/day. The seven plants treated wastewater from houses, the two from schools, the one from stores and public halls.

An example of the flow diagram of the experimental plants is shown in Figure 13. The DO and water temperature sensors were located at the last compartment of the aeration tanks in all plants. The measured values of the DO and water temperature were sent to the microcomputer. The microcomputer calculated the values by equations from (3) to (8), and sent control signals to blowers.

TABLE 3 Outline of the Experimental Plants

Plant name	KO	SG	K	I	C	S	N	SI	Y
Treatment system	conv.*	conv.	exten.*	exten.	exten.	exten.	exten.	exten.	exten.
Facilities	houses	houses	stores	houses	school	houses	houses	school	houses
Design flow rate(m^3/day)	2,565	3,100	590	538	390	313	200	107	54
Volume of aeration tank(m3)	943	1,034	702	542	343	321	215	75	57

- conv. = the conventional activated sludge process
  exten.= the extended aeration activated sludge process

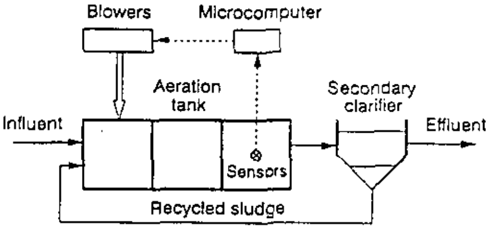


Fig.13 Example of flow diagram of the experimental plants

Experimental conditions

The experimental conditions are shown in Table 4. The operating cycles were set at 12 cycles/day in the conventional activated sludge plants, and at 6 cycles/day in the extended aeration plants.

TABLE 4 Experimental Conditions

Plant name	KO	SG	K	I	C	S	N	SI	Y
Number of cycles (cycles/day)	12	12	6	6	6	6	6	6	6
Recycled sludge operation	continous*	aeration*	aeration	aeration	aeration	aeration	aeration	aeration	aeration
KLa(20) (1/hour)	3.23	2.00	2.44	1.80	1.70	2.84	2.73	2.80	2.41

* continuous = sludge recycling occurred continuously.
aeration = sludge recycling occurred only when aeration was performed.

The pumps for recycled sludge were air-lift pumps except for in KO plant. Some of this air was also sent by the same blowers for aeration. Thus, sludge recycling occurred only when aeration was performed. Only KO plant had motored pumps for recycled sludge, thus in KO plant sludge recycling occurred continuously. The values of $K_La(20)$ used by automatic control in each plant are shown in Table 4.

The coefficients used in this experiment were as follows:

$$\alpha = 0.9, \ \beta = 0.95, \ \theta = 1.024, \ S_f = 1.2$$

Analytical methods

All analysis was done in accordance with JIS K0102 or The Analytical Methods for Sewage (in Japan).

3.4 RESULTS AND DISCUSSION

K_La measurement

K_La in all plants was derived under the condition in which both influent and sludge recycling were stopped, and the activated sludge was present. It was calculated using the triple division -overall addition method (Iwamoto, 1982) after obtaining the ascending DO concentration curve under unsteady-state. The ascending DO concentration curve used for calculating K_La in K plant is shown in Figure 14. From this analysis, $K_La(20)$ in K plant was found to be 2.44 1/hour.

$K_La(20)$ values measured by the same method in all plants were shown in Table 4. The values of $K_La(20)$ of each plant were entered in the microcomputer and were used in automatic aeration time control.

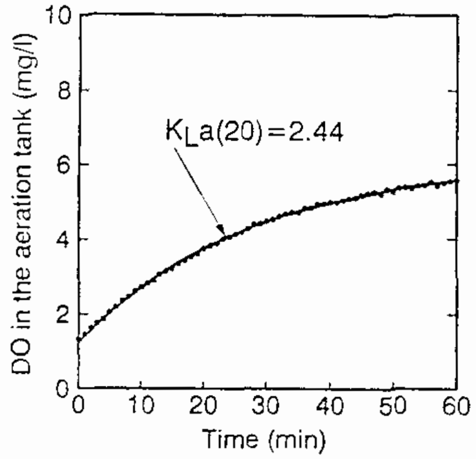


Fig.14 Ascending DO curve from which K_La is calculated

Variations in respiration rate during the aeration cycle

Figure 15 shows variations in the DO concentration in the aeration tank during one cycle in KO plant, and Figure 16 shows variations in respiration rate derived from equation (3) based on the above DO concentration. We measured the respiration rate according to The Analytical Methods for Sewage (in Japan) in the same cycle. It is shown in Figure 16.

The respiration rates derived by calculation varied slightly, but they were integrated before they were used so that their variations, when moderate, have no ill effects when used for automatic control. The respiration rates by equation (3) were almost the same as the respiration rates by The Analytical Methods for Sewage (in Japan).

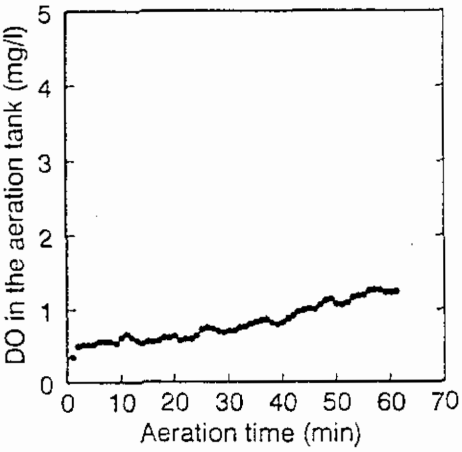


Fig.15 Variations in DO in the aeration tank during one cycle

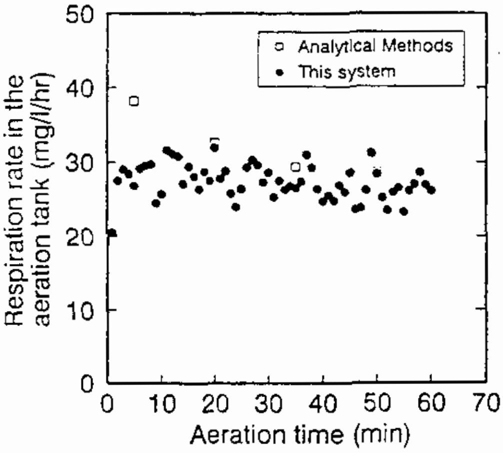


Fig.16 Variations in respiration rate in the aeration tank during one cycle

Automatic control of aeration time

Figure 17, 18, 19 and 20 show examples of automatic control of aeration time during 1 week in KO, I, C and K plants, respectively. KO and I plants were facilities for treating wastewater from houses. In these cases aeration times were controlled by daily variations in the flow rate. C plant was a facility for treating wastewater from a college. In this case the influent load had daytime peaks on weekdays, thus aeration times were controlled according to these peaks. K plant was a facility for treating wastewater from stores and public halls. In this case the influent load was large on weekends (Saturdays and Sundays). Thus aeration times were controlled by weekly variations in the influent load. In all the experimental plants aeration times were controlled by the influent load which was characteristic to each plant.

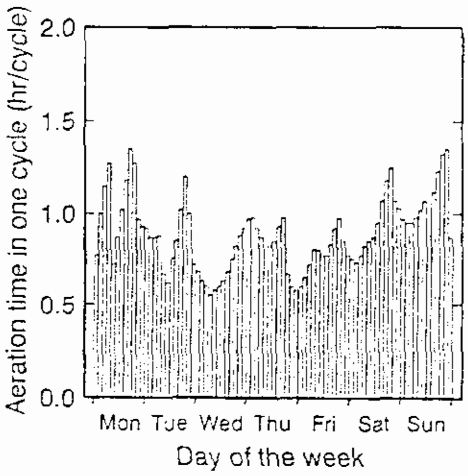


Fig.17 Aeration time control of one week (KO plant)

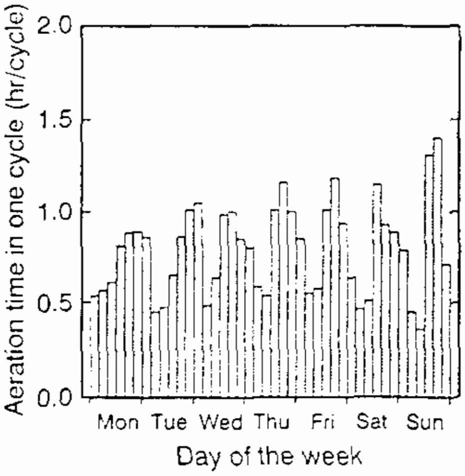


Fig.18 Aeration time control of one week (I plant)

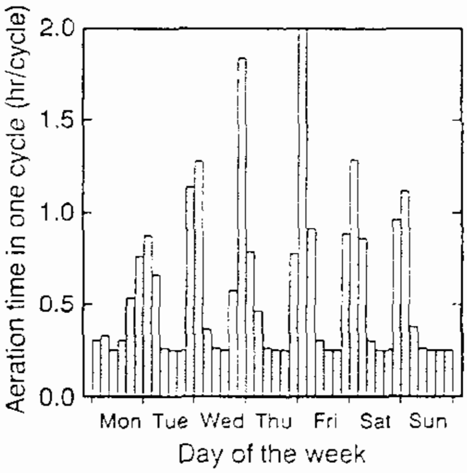


Fig.19 Aeration time control of one week (C plant)

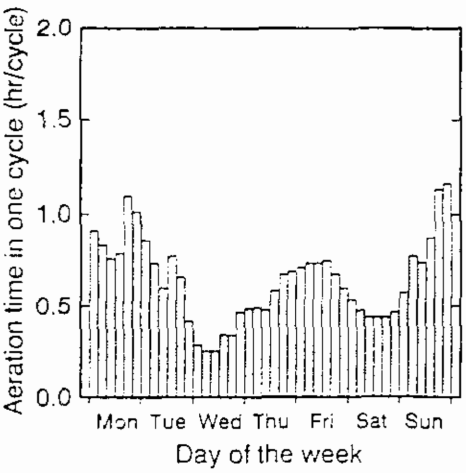


Fig.20 Aeration time control of one week (K plant)

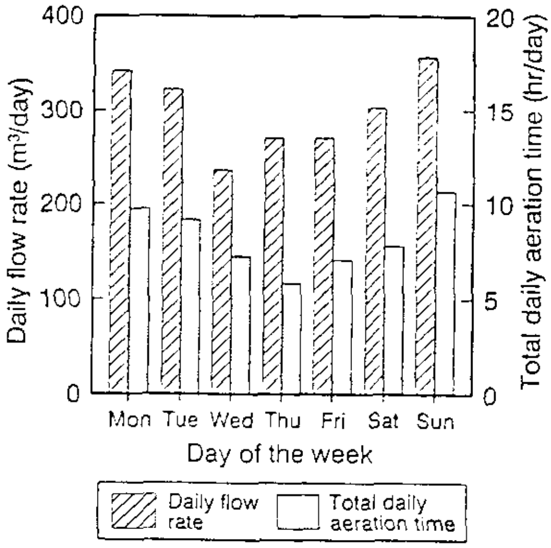


Fig.21 Daily flow rate and total daily aeration time by day of the week (K plant)

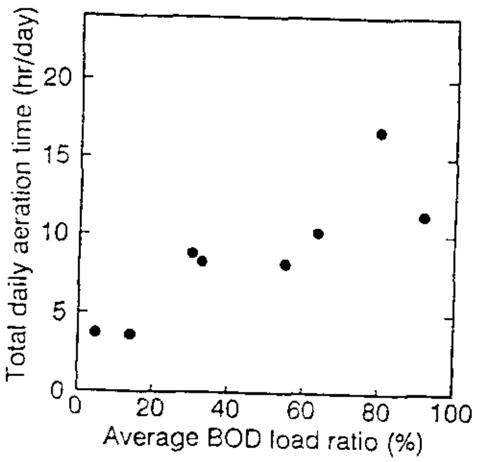


Fig.22 Relationship between average BOD load ratio and total daily aeration time

In K plant, the influent load varied according to the day of the week. Thus, the flow rates and total daily aeration times of every day of the week are plotted in Figure 21.

Figure 21 shows that the flow rate was highest on Sundays, and lowest on Wednesdays. The total daily aeration time was shortest on Thursdays. This is due to the hydraulic detention time of wastewater in the aeration tank which was about one day including recycled sludge; thus variations in the aeration time that followed the changes in the flow rate was delayed in proportion to the hydraulic detention time.

From this analysis it becomes evident that the aeration time was effectively adjusted by the automatic control system to meet such large weekly variations in the influent load as seen in K plant.

Figure 22 shows the relationship between the average BOD load ratio and the total daily aeration time in each plant. Here the average BOD load ratio means the following equation.

$$\text{average BOD load ratio} = \frac{\text{average flow rate} * \text{average influent BOD}}{\text{design flow rare} * \text{design influent BOD}}$$

In Figure 22, when the average BOD load ratio was larger, the total daily aeration time was longer. From this analysis, the total daily aeration time was controlled by the average BOD load ratio. This means that in any plant aeration time is controlled properly from the early times of the plant start-up to the period in which the flow rate is gradually increased as the time proceeds.

Treatment results

Average treatment results in each plant are shown in Table 5. Figure 23 and 24 show variations in effluent BOD and T-N in I and N plant, respectively. And Figure 25 and 26 show variations in influent and effluent organic substances and nitrogen in KO plant, respectively.

In all plants, effluent organic substances were stable and low. The average effluent BOD values were in the range of 2.3 - 7.0 mg/l, average COD-Mn(100 ℃, $KMnO_4$) values were in the range of 6.0 - 10.3 mg/l, average SS values were in the range of 2.7 - 7.2 mg/l. The removal of organic substances was quite satisfactory.

The average effluent T-N values in each plant were in the range of 1.6 - 6.1 mg/l. From Figure 26, the nitrogen removal rate in KO plant was 29% in conventional operations, and 72% in automatic control. The nitrogen removal rate increased over 40%.

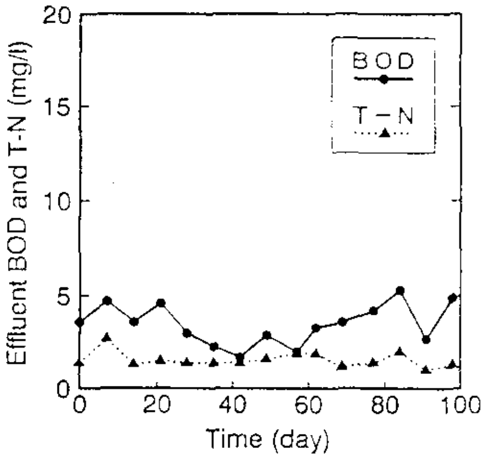


Fig.23 Variations in effluent BOD and T-N (I plant)

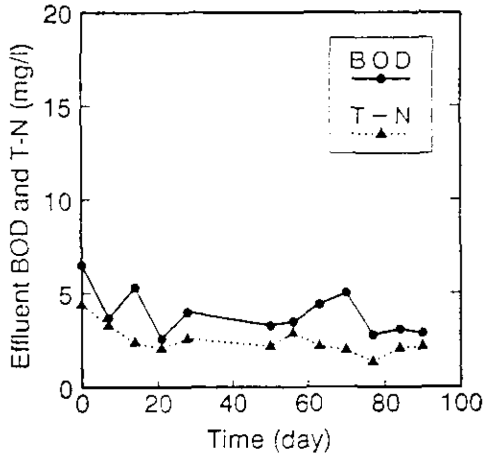


Fig.24 Variations in effluent BOD and T-N (N plant)

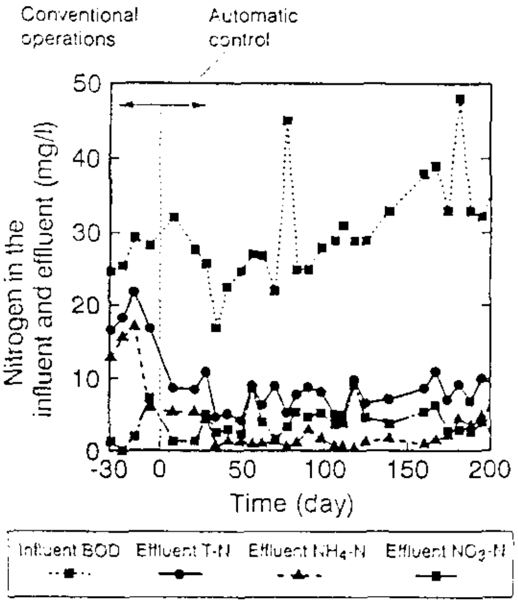


Fig.26 Variations in influent and effluent nitrogen (KO plant)

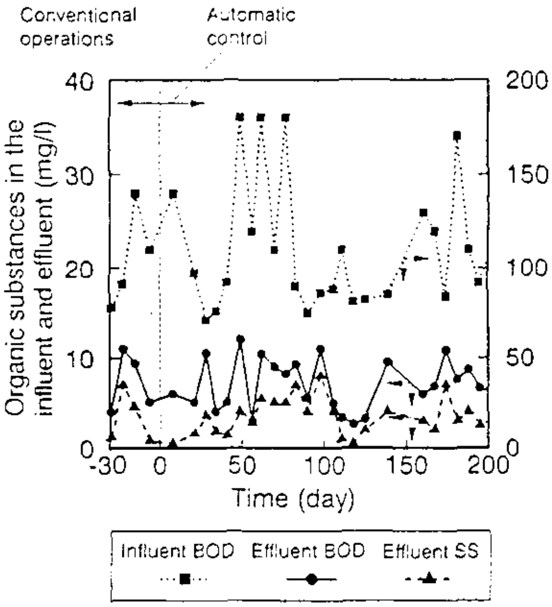


Fig.25 Variations in influent and effluent organic substances (KO plant)

TABLE 5 Average Treatment Results

Plant name	KO	SG	K	I	C	S	N	SI	Y
Design flow rate(m^3/day)	2,565	3,100	590	538	390	313	200	107	54
Flow rate (m^3/day)	2,240	2,280	274	278	134	176	134	31	32
Total daily aeration time(hours/day)	11.3	16.7	10.2	8.8	3.6	10.5	8.2	8.3	8.6
MLSS(mg/l)	2,210	2,060	5,930	7,370	6,350	3,500	4,780	5,450	5,810
influent(mg/l)	110	217	341	117	81	-	164	227	-
BOD effluent(mg/l)	7.0	4.7	2.6	3.5	2.3	4.2	3.9	7.0	5.7
removal (%)	93.7	97.8	99.2	97.0	97.2	-	97.6	96.9	-
influent(mg/l)	67	205	190	197	114	-	195	126	-
S S effluent(mg/l)	3.4	3.8	4.8	4.1	3.5	3.1	2.7	6.8	7.2
removal (%)	94.8	98.2	97.5	97.9	96.9	-	98.6	94.6	-
influent(mg/l)	30.4	40.1	34.7	33.0	41.8	-	20.3	40.3	-
T-N effluent(mg/l)	7.7	8.1	3.4	1.6	3.8	3.7	2.5	4.4	4.7
removal (%)	74.6	79.8	90.2	95.2	90.9	-	87.7	89.1	-

The relationship between the influent C/N ratio and the effluent T-N in each plant is shown in Figure 27. In conventional operations the effluent T-N increased as the influent C/N ratio was lowered. In automatic control, the effluent T-N was low and stable under these experimental conditions with the influent C/N ratio between 2 and 10.

From these results, if you apply this system to an existing wastewater treatment plant, the nitrogen removal rate will be greatly heightened without reconstructing the plant on a large scale.

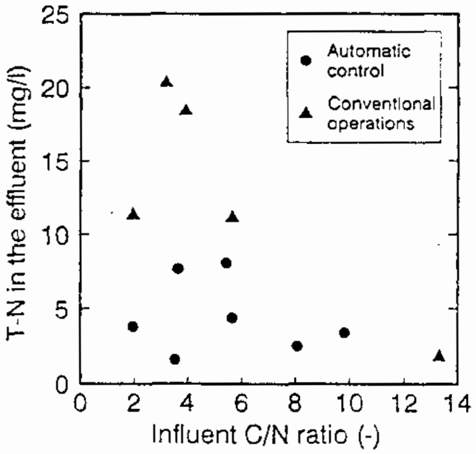


Fig.27 Relationship between influent C/N ratio and effluent T-N

Reductions in electric power consumption

The reductions in electric power consumption by automatic control are shown in Table 6. Electric power consumption was reduced by 1,700 - 22,900 kWh/month in each plant.

TABLE 6 Reductions of Electric Power Consumption

Plant name	KO	SG	K	I	C	S	N	SI	Y
Design flow rate(m^3/day)	2,565	3,100	590	538	390	313	200	107	54
Flow rate (m^3/day)	2,240	2,280	274	278	134	176	134	31	32
Total daily aeration time(hours/day)	11.3	16.7	10.2	8.8	3.6	10.5	8.2	8.3	8.6
Electric power of blowers(kW)	60	69	30	26	22.5	15	11	3.7	3.7
Reductions of electric power consumption (kWh/month)	22,900	15,000	12,400	11,800	13,800	6,100	5,200	1,700	1,700

This system requires only a minimum amount of aeration, which helps to reduce electric power consumption. The reduction in power consumption achieved by the system will increase with the size of the plant and the lowering of the influent load.

From these results, if you apply this system to a new wastewater treatment plant, electric power consumption will be reduced largely in the early times after the plant start-up, and will be reduced according to the influent load afterward.

Operation and maintenances

In order to achieve satisfactory treatment in wastewater treatment plants like K plant for which the weekly and annual variations in the influent load were large, it is necessary for the plant operators to depend on 24-hour timers so that they can frequently adjust the operation time of the blowers.

The other plants have their own characteristic influent conditions. If operators want to achieve satisfactory treatment, they must find characteristic influent conditions and adjust operations frequently. This system, however, automatically controls the operation of the blowers, thus simplifying plant operation. Farther, it will be helpful with the lack of expert operators in the future.

The DO sensor and water temperature sensor require maintenances in the present system. However, the water temperature sensor remains practically unaffected in its sensitivity even when it is covered with dust or slime.

TABLE 7 The Results of DO Sensor Calibrations in K Plant

Day from starting to use (day)	Interval between calibrations (day)	The values before calibration	
		zero (mg/l)	span* (mg/l)
487	27	0.02	10.33
515	28	0.00	10.58
571	56	0.10	10.68
620	49	-0.09	9.63
657	37	-0.04	9.12
704	47	-0.02	8.95
727	23	0.00	11.33
795	68	0.00	9.10
901	106	0.16	12.25
950	49	-0.09	9.65
1013	63	-0.02	8.86
1061	48	0.03	9.60
1169	108	0.00	10.78
1357	188	0.06	0.06

• Span was calibrated to 10.00 mg/l in the air.

The results of the DO sensor calibrations in K plant are shown in Table 7. We have been using the same DO sensor for the last three and a half years, including the experimental period. The sensor was made for industrial use and had no automatic cleaning units. The sensor was cleaned and calibrated about once per month during the initial one and a half years. Since no problems were seen during that period, the cleaning interval was gradually prolonged until it became two to six months long.

The result showed that the sensor became defective owing to adhering slime only when the interval was prolonged to six months. In the other plants, no problems were seen in cleaning and calibrating about once per a month or two months. From this we concluded that to use this system in existing plants, would require cleaning and calibrating the DO sensor about once a month ensure sufficient safety.

3.5 CONCLUSIONS

We measured and calculated the respiration rate of microorganisms. Based on this calculation, we developed a system to automatically control the aeration time during intermittent aeration so that enough oxygen would be supplied to the microorganisms. We applied the system in nine existing wastewater treatment plants with a design flow rate from 54 to 3,100m^3/day. We found the following results:

(1) The system could automatically control the aeration time in accordance with characteristic variations of the influent load in each plant.

(2) Total daily aeration time was controlled according to the BOD load ratio in each plant.

(3) In automatic control the average effluent BOD values in each plant were in the range of 2 - 7 mg/l, COD-Mn values were in the range of 6 - 10 mg/l and SS values were in the range of 2 - 7 mg/l. The removals of organic substances were quite satisfactory.

(4) The average effluent T-N values in each plant were in the range of 1 - 8 mg/l, thus indicating an excellent removal of nitrogens. The nitrogen removal rate was greatly heightened as compared with that in conventional operations.

(5) If you apply this system to an existing wastewater treatment plant, the nitrogen removal rate will be greatly heightened without reconstructing the plant on a large scale.

(6) This system requires only a minimum amount of aeration, thus electric power consumption of blowers in each plant was reduced by 1,700 - 22,900 kWh/month.

(7) This system automatically controls operations of blowers, thus can simplify plant operations. It will be helpful with the lack of expert operators in the future.

(8) Maintenance of this system requires only cleaning and calibrating the DO sensor about once a month.

REFERENCES

Hayakawa, N., Tsuji, J. and Hamamoto, Y. (1986). Simultaneous Nitrogen and Phosphate Removal by the Intermittent Cyclic Process, Water Science Technology, 18, 319-326, IAWPRC's First Asian Conference, Tokyo, Japan, 1-3 October, 1985

Iwai, S., Oomori, H. and Yamamoto, K. (1981). Comparison of Nitrogen Removal between Continuous Aeration and Intermittent Aeration Applied to the Extended Aeration Process. J. Water Treatment Techniques, 22(8),665-672

Hamamoto, Y., Ideshita, Y. and Nonaka, H. (1990). Application of a System with Automatic Control of Aeration Time to the Intermittent-Aeration Activated Sludge Process in an Existing Wastewater Treatment Plant, Instrumentation, Control and Automation of Water and Wastewater Treatment and Transport System, Proceedings of the 5th IAWPRC Workshop, Yokohama and Kyoto, Japan, 26 July - 3 August, 103 - 112.

Eckenfelder,W.W.Jr and O›Connor,D.J. (1961). Theory and practice of aeration. In: Biological Waste Treatment, Pergamon Press, New York.

Iwamoto, Y.(1982). An Easier Method for Calculating K_La - The Batch Aeration System 2. Nishihara Technical Report, No.24, 145-150

N-REMOVAL: RESEARCH AND STATE OF THE ARTS IN THE NETHERLANDS

J.H.J.M. van der Graaf
Witteveen & Bos Consulting Engineers

1. INTRODUCTION

Reducing nitrogen emissions from wastewater treatment plants is not a new issue. More than fifteen years ago, Dijkstra (Ref. 1) presented a paper on the "state of the art" of the removal of nitrogen compounds from wastewater, which is still relevant today. Yet, given the tremendous developments in surface-water quality management since that time, it might have been expected that much of this technology would have been incorporated into the design of wastewater treatment plants. However, this is not the case.

Attention was focused mainly on removal of oxygen-consuming substances, and some attention was given to the conversion of ammonium, because of its oxygen demand. Between 1970 and 1990, the overall removal efficiency of wastewater treatment plants increased by more than 90 % for oxidizing substances, but for nitrogen it has lagged behind and is now about 45 %.

Wastewater treatment aims at removing nitrates where possible with a view to saving energy. However nitrogen removal by denitrification can adversely effect the ammonium conversion with the result that denitrification is often not carried out optimally. Moreover, the technical design of wastewater treatment plants in the Netherlands has not been oriented towards denitrification because there were no emission guidelines for nitrate and total nitrogen. Until now only a nitrogen standard of 20 mg/l Kjeldahl N in the summer period is applied for discharge into the surface waters.

In order to counteract eutrophication of the surface water, the phosphate path rather than the nitrogen path was used. This is firstly because phosphate is the limiting nutrient in most inland waterways, and the situation could be improved by reducing the phosphate loading (Ref. 2). Secondly, the reduction of nitrogen compounds will produce less results because processes such as nitrogen fixation by blue algae must also be taken into account, and because the nitrogen loading mainly originates from diffuse sources. Until recently, Dutch policy on counteracting eutrophication was based on this theory; see the symposium on phosphates organized by the NVA (Dutch Association for Wastewater Treatment and Water Quality Management; Ref. 3). A change became apparent with the publication of the third CUWVO survey on eutrophication in May 1988 (Ref. 4). It was observed in investigations that reducing the nitrogen concentration in the surface waters, supplemental to the phosphate reduction, will reduce eutrophication significantly.

Shortly afterwards, the results of the North Sea eutrophication surveys (Ref. 5) showed that the nitrogen concentration diminished at a certain distance from the coast, due to denitrification, thus making nitrogen the growth-limiting nutrient. This led to the opinion that high priority should be given to reducing nitrogen emissions as well as phosphorous into the surface water.

This somewhat sudden change in policy on eutrophication created problems. Plans already made for building or renovating wastewater treatment plants had to be expensively revised, and it meant that recently-built plants were likely to become obsolete within a short period. Moreover, ways still had to be found of extensively reducing phosphate levels and suitable depot locations had to be found for the waste sludge produced.

2. PRESENT SITUATION

At present there are 480 wastewater treatment plants in operation in the Netherlands. Data until the end of 1988 available from the CBS survey (Ref. 6) are summarized in tables 2.1 and 2.2.

Table 2.1: number and capacity of municipal wastewater treatment plants in the Netherlands, 31 December 1988

	Number	Capacity (p.e.) (* 1000)
primary treatment	23	303
trickling filters	51	1 743
aeration tanks	67	10 494
oxidation tanks	54	1 114
oxidation ditches	121	988
carrousels	85	4 511
two-stage treatment	36	3 722
various	44	645
total	481	23 520

Table 2.2: average effluent composition of wastewater treatment in the Netherlands

	wastewater treatment plants		flow	pH	COD	BOD	Kjeldahl N	tot. N	tot. P
	number	capacity							
		1,000 p.e.	1,000 m_3/d	pH	mg/l				
the Netherlands									
1985	504	22.352	3.913	7.5	102	22	18.3	25.1	8.6
1986	492	22.905	4.100	7.5	102	22	18.7	26.4	8.7
1987	491	23.088	4.494	7.5	98	20	17.0	24.2	8.0
1988	486	23.786	4.962	7.5	90	19	14.7	22.2	5.7
primary treatment									
mecanical units	26	1.704	353	7.5	310	121	39.6	39.6	8.6
trickling filters	52	1.759	379	7.6	117	26	24.1	31.7	6.5
aeration tanks	66	9.389	1,871	7.5	78	13	18.3	27.5	5.5
oxidation tanks	54	1.114	258	7.5	58	7	7.8	17.0	5.1
oxidation ditches (continuous)	121	960	220	7.5	66	9	9.5	17.7	5.5
carrousels	86	4.509	1,016	7.5	67	10	8.4	14.3	4.9
oxidation ditches (discontinuous)	32	43	14	7.6	115	24	18.6	22.6	5.4
parallell units	8	564	90	7.5	75	10	11.7	20.7	6.6
two stage treatment	36	3.722	756	7.4	70	11	16.4	27.3	6.6
various units	5	22	6	7.5	118	25	17.5	29.6	4.6

Table 2.3: total nitrogen concentration in effluent from various wastewater treatment systems in the Netherlands, 1985

total nitrogen (mg/l)	< 10	10-15	15-20	> 20
activated sludge	1	4	17	51
oxidation ditches	29	66	69	78
trickling filters	-	1	2	48
two-stage treatment	-	-	1	17

The average annual total nitrogen removal from various types of wastewater treatment systems is substantially better with oxidation ditches than with other systems. Some oxidation ditches achieve an average annual total nitrogen concentration in the effluent of < 15 mg/l. Small oxidation ditches usually produce total nitrogen concentrations of > 20 mg/l. Because of the low sludge loading, denitrification is good, and extensive reduction in total nitrogen is due to anoxic zones in the oxidation ditches.

A few activated sludge plants reach an average annual total N of less than 20 mg/l. There are still plants with higher sludge loadings (> 0.15 kg BOD/kg d.s.d.) but these are not designed for full nitrification. Only a small number of the two-stage nitrification installations achieve satisfactory reduction of total N.

Extensive nitrogen reduction cannot be achieved with trickling filters. Even where nitrification occurs in low-loaded plants, conditions cannot be created for denitrification.

In the few two-stage plants in operation in the Netherlands (trickling filter/activated sludge and trickling filter/oxidation ditch) the BOD level is insufficient in the second stage to produce satisfactory denitrification. These two-stage installations therefore give high total nitrogen levels.

3. NITROGEN REMOVAL TECHNIQUES

3.1 Nitrogen removal in present systems

In the Netherlands extensive nitrogen reduction is achieved in oxidation ditches and low-loaded activated sludge plants. The principle used is biological nitrification and denitrification. Oxidation ditches and related plants, whether by accident or design, have between their aerobic zones anoxic zones in which denitrification occurs. Denitrification also occurs in the anoxic centre of the sludge particles if the oxygen concentration is sufficiently low. In activated sludge plants there are a number of possible configurations whereby the nitrate-containing water from the nitrification zone is brought into contact with BOD in an anoxic area.

The following techniques are used in the Netherlands to provide denitrification:

- Pre-denitrification in a pre-denitrification tank
  Effluent with a high nitrate content flows back to the first stage of the process, where the nitrate is converted into nitrogen gas while removing BOD. Examples are: Schreiber, Rotoflow, activated sludge systems with pre-denitrification tanks such as in the wastewater treatment plants at Alkmaar and Emmen. With sufficiently large nitrification and denitrification zones, the supply of BOD, and sufficient recirculation, total nitrogen values in the order of 10 mg/l can be obtained (design criteria, Barger Es plant).

- Simultaneous denitrification
  By alternating aerated and non-aerated zones on both sides of the aerators, both nitrification and denitrification can occur in a low-loaded aeration circuit. Under favourable conditions, total nitrogen can be reduced < 10 mg/l. The process can be adjusted by controlling aeration and by locating the influent feed in an anoxic zone.

3.2 Nitrogen removal in new systems

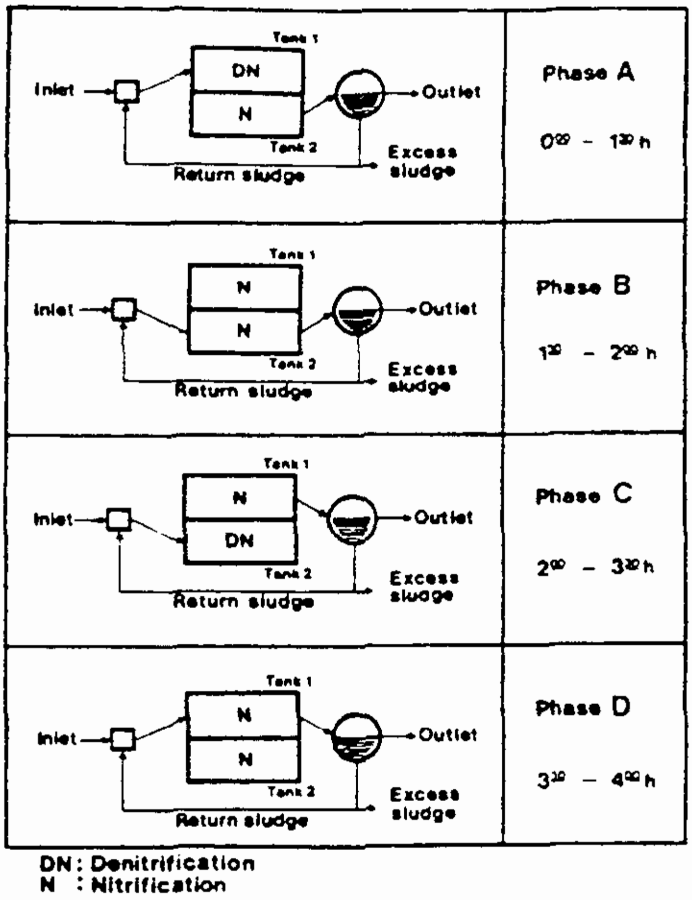
New types of systems for extensive nitrogen reduction are being tested in existing wastewater treatment plants or are still at the pilot stage. The most important developments are summarized below.

Activated sludge plants with alternating aeration
By the use of a number of aeration tanks which are alternately aerobic and anoxic, nitrification and denitrification can occur consecutively in the same tank. The wastewater passes through the aeration tanks in different phases of the process so that both the direction of flow and the time of aeration in the tanks are alternated.

An example of this is the Bio-Denitro process (see Figure 3.1). In Denmark this process has produced total nitrogen effluent concentrations of 3.4 mg/l in existing installations and 3.5-6.8 mg/l in experiments. A plant of this type, with a projected capacity of 330,000 p.e. is now being constructed in Aalborg. The plant's projected total nitrogen concentration is 8 mg/l. A number of succesful pilot plant experiments have been carried out in the Netherlands (Ede wastewater treatment plant, for example).

The Tricycle-Verfahren process is also based on alternately aerating the aeration tanks and rendering them anoxic. The system consists of three tanks which are alternately fed (no aeration, denitrification) and aerated (nitrification). The flow is regulated by on-line measurement of the ammonium and nitrate concentrations. Test data are not yet available, but it is expected that the total nitrogen effluent concentration will be only a few milligrams per litre.

Figure 3.1: Bio-Denitro production process



Integrated biofilm systems

Extensive reduction of BOD and total nitrogen can be achieved in systems with two submerged filters. The first filter is anoxic, with a downward flow. The second filter is aerated, with an upward flow. Part of the effluent is recirculated. A total nitrogen effluent concentration of 9-15 mg/l can be attained, depending on the recirculation factor used. An experimental plant of this type (Sulzer method) with a capacity of 63 m^3/h has been in use in Geneva since 1988. The nitrification efficiency is 85-98%, with an ammonium effluent concentration of < 3 mg/l. The denitrification efficiency is 50-60%, and the total nitrogen reduction efficiency is 42-60%.

The Biocarbone process is another type of biofilm reactor for nitrogen reduction. It consists of a single upward-flow filter, the lower part of which is anoxic and the upper part aerobic. Denitrification occurs as the effluent is recirculated. Experiments with presettled household wastewater have shown an average total nitrogen reduction efficiency of about 60 %, corresponding to about 15 mg/l total nitrogen.

Separate biofilm systems

In addition to the processes described above, which integrate biological nitrification and denitrification in one system, extensive total nitrogen reduction can also be obtained by separating the nitrification and denitrification steps. For the nitrification step, a common low-loaded secondary purification method (activated sludge plants, oxidation ditches, trickling filters, two-stage systems) can be used, or high-loaded secondary purification followed by tertiary purification. A number of options for tertiary nitrification are given in table 3.1.

Table 3.1: options for tertiary nitrification

type	loading kg NH_4-N/m3d	NH_4-inf mg/l	NH_4-eff mg/l	efficiency %	lit.
fluidized bed	1.0	25	4.5	82	12
biocarbone	0.46-0.73	28-39	0- 4	86-100	20
biorotors			< 5	>90	21
biofor	0.41	14-32	2	90	22
biofor	0.24	4-12	0-11	82	22

In all these methods Kjeldahl N is extensively nitrified, resulting in high concentrations of nitrate. To remove these in an additional denitrification step, a carbon source must be added; generally methanol is used. The disadvantage of adding carbon is the high costs involved, while the advantage of integrated systems is that BOD from the influent is used as a carbon source. A further advantage is the fact that the BOD is partly broken down during denitrification, which saves on the energy required for aeration.

Many different systems can be used for the extra denitrification step with the addition of carbon, including anoxic filters, fluidized bed systems, and biorotors.

The major advantage of reducing nitrogen separately is that extensive total nitrogen reduction can be achieved in a reactor of limited size. A disadvantage is the need for an external carbon source of carbon for denitrification.

3.3 Physiochemical nitrogen removal

There are in principle three methods of physiochemical nitrogen removal:
- the use of ion exchange columns;
- the stripping of ammonium;
- the chemical precipitation of ammonium.

Ion exchange columns are not commonly used for nitrogen reduction in wastewater treatment. The costs are high and the columns require frequent cleaning. Furthermore, the removed nitrogen has to be processed in a regeneration phase. Clinoptilolith is an anorganic ion exchange material which binds the ammonia, which can then be retrieved by backwashing. Ion exchange to fix nitrate is used in the treatment of drinking water. Regeneration is carried out by biological denitrification with the application of methanol.

For the stripping of ammonium, the pH of the wastewater must be increased to 10-11. The amount of air required is 100 to 1000 times higher than the amount needed for biological denitrification in an activated sludge plant. The air must then be washed with acid in order to claim the ammonia: it is environmentally undesirable to release it into the atmosphere.

Chemical precipitation of ammonium is possible through the formation of the poorly-soluble magnesium ammonium phosphate ($MgNH4PO4.6H2O$) (23). This precipitate is deposited in crystalline form which does not include pollutants such as BOD or heavy metals.

The concentration ratio for NH_4:MG:PO_4 with an ammonium concentration of 42 mg/l must be 42:73:96. To attain these concentrations in wastewater, both phosphoric acid and magnesium-oxide should be added. This method is expensive, but costs can be reduced if the crystalline precipitate can be sold to the fertilizer industry. The consequences of the addition of phosphate to the phosphate concentration of the effluent are difficult to predict, and depend on the controllability of the precipitation process. The stringent requirements for phosphate concentration in the effluent limits the practical applicability of this method.

Thus for the time being, there are insufficient prospects for the physiochemical techniques for nitrogen reduction in wastewater to be used efficiently and economically.

3.4 Evaluation of technically feasible techniques

The following conclusions can be drawn about the feasibility of achieving effluent levels of 5, 10, or 15 total N/l (see Table 3.2):

- 15 mg N/l is attainable through limited adaption of activated sludge and oxidation ditch systems. At least one extra purification step is required in trickling filter systems.
- 10 mg N/l requires pre-denitrification in activated sludge systems; adaptation of aeration and influent feed is sufficient in oxidation ditch systems.
- 5 mg N/l is possible under favourable circumstances in oxidation ditch systems with proper control of aerobic and anoxic zones and the addition of BOD in the anoxic phase. Activated sludge systems require provisions for extensive pre-denitrification and return flow.
- An effluent level of 5 mg N/l or less also seems feasible in new systems, but few data are available at present.

Table 3.2: feasibility of achieving effluent levels 5, 10 and 15 mg N/l

type of unit	20 mg/l	15 mg/l	10 mg/l
activated sludge	- low loading - controllable and low oxygen concentration (simultaneous denitrification) - sufficient recirculation (through return sludge or recycling system)	- low loading - pre-denitrification - recirculation of nitrate containing activated sludge to pre-denitrification	- very low loading - large pre-denitrification - high recirculation ratio
oxydation ditches	+	- aeration adapted to nitrification and denitrification zones	- influent input in dentrification zones - control of aeration on (de) nitrification, based on O_2, NH_4 and/or NO_3 measurement
trickling filters	-	-	-
two stage	+	- low loaded second stage with nitrification and denitrification zones - large bypass to second stage	-
alternate systems:			
oxydenitro process 1)	+	+	+
biodenitro process 2)	+	+	+/-
tricycle-Verfahren 3)	+	+	+
Bio-denitro process	+	+	+
Sulzer process 1)	+	+/ -	-
Biocarbone 1)	+	+/ -	-
Biofor 1)	+	+/-	+/-
two-stage combinations of current systems 4)	+	- often only with methanol dosage in denitrification stage	- often only with methanol dosage in denitrification stage

+	average effluent concentration attainable
●	average effluent concentration at or around this value
-	average effluent concentration not attainable
1)	results based on experimental plant
2)	various practical results
3)	anticipated results
4)	systems such as fluid bed, (an)aerobic filters, biorotors, etc.

4. RESEARCH

The research on nitrogen reduction in the effluent from wastewater treatment plants is currently being undertaken in the Netherlands within the framework of the PNS-1992 investigation programme sponsored by Stora. This work includes:

- collecting practical experience;
- semitechnical pre-denitrification investigations;
- compact systems for nitrification/denitrification;
- measurement and control systems.

4.1 Practical experience

Experience has been gained in the reduction of phosphate and nitrogen. Since phosphate reduction has to be introduced shortly and nitrogen reductions must also be adopted in the near future, it is important that experience and new developments are documentated. Moreover, the potential of various systems (for example, extensive nitrogen reduction in circuits) must be investigated further and incorporated into a practical manual. These investigations need to be coordinated to ensure efficient expenditure of resources and wide circulation of information. To this end, a users' forum of technologists has been set up.

The forum coordinates and evaluates research, acts as a sounding board for users, evaluates available information, and disseminates knowledge and experience. A technical secretariat initiates activities, collects data, coordinates research, and prepares progress reports on practical experience and research results.

Collective experience, investigation data, and guidelines on plant size will be gathered together into two handbooks, one on phosphate reduction and the other on nitrogen reduction, which will form the basis for the implementation of such measures in about 1995. The work group meets three to six times a year.

The forum addresses the following issues on nitrogen deduction:

a. Collected data on nitrogen removal in the Netherlands
a.1 Primary process conditions and results
  - design of system
  - composition of influent
  - effect of process variables (pH, alkalinity, temperature)
  - dry weather flow/maximum hydraulic capacity
a.2 Secondary technological effects
  - effect of phosphate removal (prior and simultaneous precipitation)
  - effect on effluent BOD, NH4, O2
a.3 Management factors
  - monitoring of ammonium, nitrate, oxygen
  - regulation of aeration, recirculation

b. Investigation on nitrogen removal
Performed by users in cooperation with the technical secretariat; coordination, reporting, and evaluation by technical secretariat.
b.1 Investigation on extensive nitrogen reduction at specific plants
  - nitrogen reduction systems under consideration
    # pre-denitrification (Alkmaar, Barger Es, etc.)
    # modified carrousels and oxidation ditches
    # systems with a central denitrification zone
  - survey programme: about one year
  - evaluation, reports, recommendations for design

b.2 Influence on effluent quality of peak loads and internal process flows (including sludge water);

b.3 Effect of phosphate reduction on nitrogen reduction in existing installations
  - identification of plants with problems with nitrogen in relation to phosphate reduction
  - on-site investigation (process data, supplementary analyses, measurement of nitrification and denitrification capacity)
  - variation of process factors
  - evaluation of research data and recommendations

c. <u>Preparation of nitrogen removal handbook</u>
  - collection of data on the practical application of nitrogen reduction
  - comparison of systems (circuits, pre-denitrification, alternately aerated systems etc.) and guidelines for dimensioning related to effluent requirements
  - technical factors; initial activities finished after about six months; later adjustment based on new results

d. <u>Practical research on nitrogen removal</u>
  - research on problems in practice and requiring investigation
  - a few possible experiments
    # further investigation on nitrogen reduction kinetics in pre-denitrification and circuits
    # adaptation of existing plants for improved nitrogen reduction
    # establishment of test procedure for nitrification and denitrification capacity of sludge

4.2 Semitechnical pre-denitrification investigation

Modifying activated sludge plants to systems for extensive nitrogen reduction usually involves the addition of a pre-denitrification tank. Guidelines are partly based on research carried out in West Germany, but there is insufficient information on the applicability of these guidelines to the Dutch situation and the effect of local variations in wastewater composition. Research is being done on the potential and limitations of activated sludge systems in combination with denitrification under Dutch conditions so that design guidelines of such systems can be prepared. The work is being undertaken on a semitechnical scale in order to reduce time and costs. The results and practical aspects of full-scale pre-denitrification fall under the practical research of the forum on nitrogen reduction (see Section 4.1).

The investigation is on a scale of at least several hundred p.e. (flow 1-5 m^3/h, aeration volume 10-20 m^3) in order to give a reasonable approximation of actual circumstances. The installation is set up at a wastewater treatment plant and fed with (presettled) wastewater. As a parallel unit, an activated sludge system is being used as a control.

The following aspects are being investigated:
primary process parameters (sludge loading, nitrogen loading of both nitrification and denitrification zones, pH)
process variables (recirculation, oxygen supply)
effect of wastewater composition (BOD/N, temperature)

The experiments are being carried out at 3 to 5 sites in order to assess the effect of different types of wastewater and purification methods. The effects of the primary process parameters (check on design guidelines) are being investigated in a pilot plant at one location.

4.3 Nitrification and denitrification in a compact system

Nitrification can be improved in relatively high-loaded activated sludge systems by reducing the sludge loading. This requires a substantial increase in the size of the aeration tanks, which is costly. Moreover, many plants have insufficient space to accommodate these tanks. This unfavourable combination of factors applies particularly to large plants in urban areas, which are responsible for a substantial amount of the total nitrogen emission. Nitrifying sludge on-carrier systems yield a considerably higher biomass per volume unit than do activated sludge systems. Also, no extra sedimentation phase is needed because there is very little sludge growth. This makes nitrifying sludge on-carrier systems suitable as an additional phase to upgrade a purification system with an inadequate nitrification capacity. Fluid bed systems produce the greatest activity per volume unit (about ten times as high as activated sludge).

Denitrification can be improved in nitrifying systems such as activated sludge systems by installing a pre-denitrification tank. The aeration tanks will require about 30-50 % more space and the costs are considerable. Moreover, many plants do not have enough room for this extension. Denitrifying sludge on-carrier systems with an extra source of BOD (e.g. methanol) can achieve a substantially higher biomass per volume unit than can pre-denitrification.

Combined nitrification and denitrification systems in the form of packed bed or fluid bed systems are still in the development stage and are not yet in use in the Netherlands. The favourable prospects for these systems, particularly for plants with limited space, make a demonstration project on a small practical scale desirable.

This investigation will comprise three phases.

Phase 1 (preliminary phase)

First, preliminary investigations will be carried out into the available technology. Systems will be investigated on site, and their use evaluated. An evaluation report will be drawn up and a test design made for a practical investigation.

Phase 2 (practical investigation)

Nitrification
A pilot unit based on an existing process (minimum volume 10 m^3) will be set up in a wastewater treatment plant and fed with poor to moderate nitrified effluent. The nitrogen loading of the reactor can be adjusted.

Denitrification
A pilot unit based on an existing process with one or more reactor systems (volume 1-5 m^3 each; about 5,000-20,000 p.e.) will be built and fed with the nitrified effluent. A fluid bed or a packed bed reactor will be used depending on the results of phase 1.

The testing will last for at least a year, during which time the following aspects will be investigated:
- operation subject to hydraulic and biological loading;
- effect of temperature, recirculation, and other production factors;
- practical aspects of the fluid bed (process control, sludge growth, behaviour of the bed, washing etc.).

Tests may be carried out at several locations.

Phase 3 (evaluation)

The test results will be evaluated. The potential, limitations, and costs of adaptation to the practical situation in the Netherlands will be determined, and guidelines on design and installation drawn up.

4.4 Measurement and control systems

A first condition for low effluent concentrations is an appropriate system and size of installation. Even then, the great variation in flow and load makes it difficult to achieve a consistently good performance. Consequently, optimal steering and control is essential.

Steering here means that an action causes a systematic reaction. There is no check on the result of the action. Control refers to the maintenance of a situation by taking action, if measurements show a variation from an established value. The investigation of measurement and control systems is directed particularly at practical applicability, and involves the following aspects:

testing measuring equipment under practical conditions (ammonium, nitrate, turbidity); at first purely as a measuring device;

testing control systems, with or without the use of new measuring equipment; this is clearly related to system set-up and process control; this will be tested particularly in practical situations.

4.5 General review

The nitrogen investigation will be guided by a steering committee, PN 1992, which confers regularly on progress and initiates and approves parts of the project. Stora will pay out a total of about fl. 3,500,000 for research in the period 1990-1992. Since part-contributions are involved in some projects the total research input for nitrogen reduction is estimated to be about fl. 5,000,000 to fl. 6,000,000.

5. REFERENCES

1 Dijkstra, F. 1974. The removal of nitrogen compounds from wastewater, H2O (7) 1974, no. 22.

2 Unie van Waterschappen (Dutch Water Boards Association)
Regional investigation on eutrophication 1976-1981. Report of the working group on eutrophication, February 1983

3 Nederlandse Vereniging voor Afvalwaterbehandeling en Waterkwaliteitsbeheer (Dutch Association for Wastewater Treatment and Water Quality Management), special issue on phosphates, H2O (210) 1988, no. 9.

4 CUWVO Work group VI, May 1988
Comparative investigation into eutrophication in Dutch lakes and ponds. Results of the third eutrophication survey

5 Rijkswaterstaat (Netherlands Department of Transport and Public Works). Policy analysis of reduced emission of nutrients and eutrophication in the North Sea, Tidal Water Service, note GWWS - 89.504.

6 CBS survey, 1988

7 Scenarios for nitrogen removal in wastewater treatment plants, CUWVO, April 1990

8 STORA
Research programme P+N+sludge removal 1992, Rijswijk 1989