United States
Environmental Protection
Agency

Water Engineering Research
Laboratory
Cincinnati OH 45268

EPA/600/9-88/010
April 1988

Research and Development

EPA

Proceedings:

Eleventh United States/ Japan Conference on Sewage Treatment Technology

October 12-14, 1987
Tokyo, Japan

EPA/600/9-88/010
April 1988

PROCEEDINGS

ELEVENTH UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

OCTOBER 12 - 14, 1987
TOKYO, JAPAN

OFFICE OF INTERNATIONAL ACTIVITIES
OFFICE OF WATER
WASHINGTON, D.C. 20460

OFFICE OF RESEARCH AND DEVELOPMENT
WASHINGTON, D.C. 20460
CINCINNATI, OHIO 45268

WATER ENGINEERING RESEARCH LABORATORY
U.S. ENVIRONMENTAL PROTECTION AGENCY
OFFICE OF RESEARCH AND DEVELOPMENT
CINCINNATI, OHIO 45268

U.S. Environmental Protection Agency
Region 5, Library (5PL-16)
230 S. Dearborn Street, Room 1670
Chicago, IL 60604

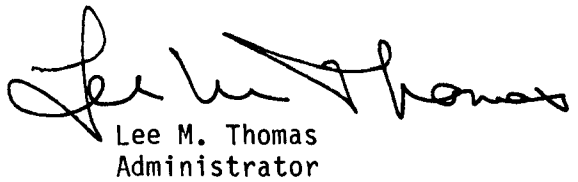
DISCLAIMER

These Proceedings have been reviewed by the U.S. Environmental Protection Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the U.S. Envvironmental Protection Agency, no does mention of trade names or commercial products constitute endorsement or recommendation for use.

FOREWORD

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

The participants in the United States-Japan cooperative project on sewage treatment technology have completed their Eleventh Conference. These conferences, held at 24-month intervals, give the scientists and engineers of the cooperating agencies an opportunity to study and compare the latest practices and developments in the United States and Japan. These Proceedings of the Eleventh Conference comprise a useful body of knowledge on sewage treatment, which will be available not only to Japan and the United States but also to all nations of the world who desire it.



Lee M. Thomas
Administrator

Washington, D.C.

TABLE OF CONTENTS

Foreword..... iii

List of Japanese Delegates and Presentation Topics..... vi

List of United States Delegates and Presentation Topics..... viii

Joint Communique..... 1

Japanese Papers..... 3

United States Papers..... 5

JAPANESE DELEGATION

Delegates	Presentation Topics
TSUNEKAZU FUKUI - Team Leader Director, Department of Sewerage and Sewage Purification, Ministry of Construction	° Current Status of Sewerage Project in Japan
SHIGERU ANDO Director, River-Basin Sewerage Division, Department of Sewerage and Sewage Purification, Ministry of Construction	° Progress in the Research on Application of Biotechnology to Wastewater Treatment
KAZUNORI HOSOKAWA Director, Construction Division Sewage Works Bureau, Yokohama City	° Characterization of the Various Diffusers and its Application
KATSUHIKO KITAI Director, Sewerage Division Osaka Prefectural Government	° Flood Control of Urban Runoff in the East Area of Osaka Prefecture (Neya River Basin)
TAIGO MATSUI Director, Research and Technology Development Division Japan Sewage Works Agency Ministry of Construction	° Evaluation of Energy Saving Using Heat Balance Model for Melting Process
YOSHIO OSHIMA Chief, Ultimate Disposal Section Water Quality Control Division Public Works Research Institute	° Beneficial Utilization of Sewage Sludge
HIROSHI ONIKI Director, Planning Division Sewage Works Bureau, Fukuoka City	° Reuse of Treated Wastewater in Fukuoka City
KENICHI OSAKO Director, Construction Section The Second Improvement and Constructive Office, Sewerage Bureau Tokyo Metropolitan Government	° Autogenous Incineration of Sewage Sludge by Recovering the Heat of Exhaust Gas

JAPANESE DELEGATION (continued)

MITSUYOSHI OZAWA
Director, Water Quality Management
Division, Water Quality Bureau
Environment Agency
Government of Japan

- Lake Water Quality Management in Japan

KAZUAKI SATO
Chief, Advanced Waste Treatment
Section, Water Quality Control Division
Public Works Research Institute
Ministry of Construction

- Current Practices of Biological Nitrogen Removal Processes in Japan

TSUTOMU TAMAKI
Executive Director,
Japan Sewage Works Agency

- Areawide Sewage Sludge Treatment and Disposal Project -- Ace Plan --

NAOHIRO TANIGUCHI
Senior Researcher, Research and
Technology Development Division
Japan Sewage Works Agency

- Full Scale Evaluation of Phosphorus Balance in Biological Phosphorus Removal

---

(Other Delegates)

TAKESHI KUBO
Senior Advisor,
Japan Sewage Works Agency

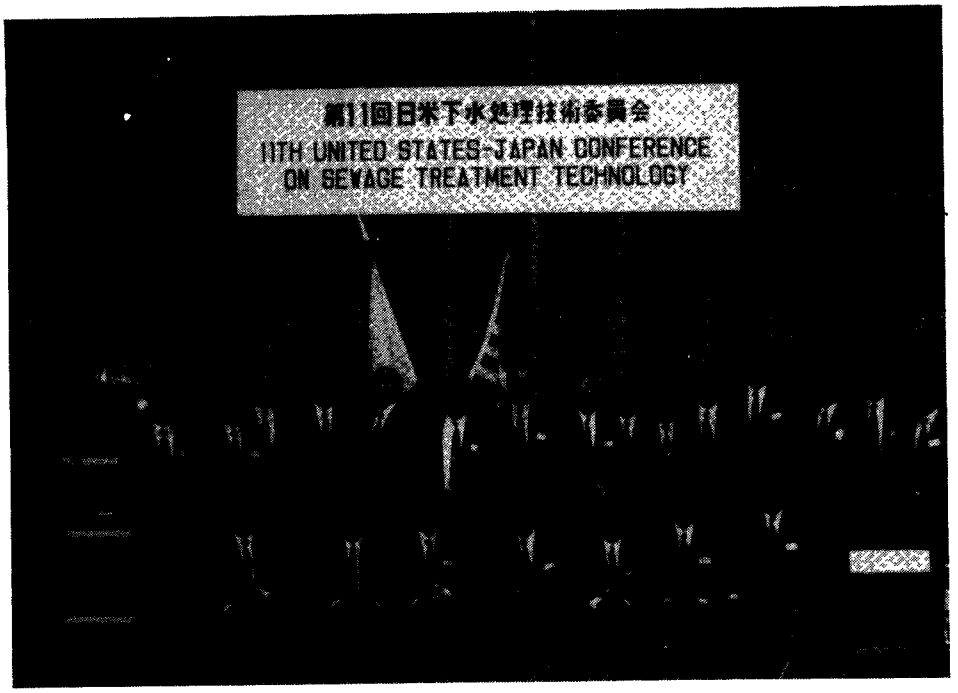
KEN MURAKAMI
Director, Water Quality Control Division
Public Works Research Institute
Ministry of Construction

KENJIRO SAITOH
Director, Public Sewerage Division
Department of Sewerage and
Sewage Purification
Ministry of Construction

TAKEOMI YASUDA
Director, Sewerage Planning Division
Department of Sewerage and
Sewage Purification
Ministry of Construction

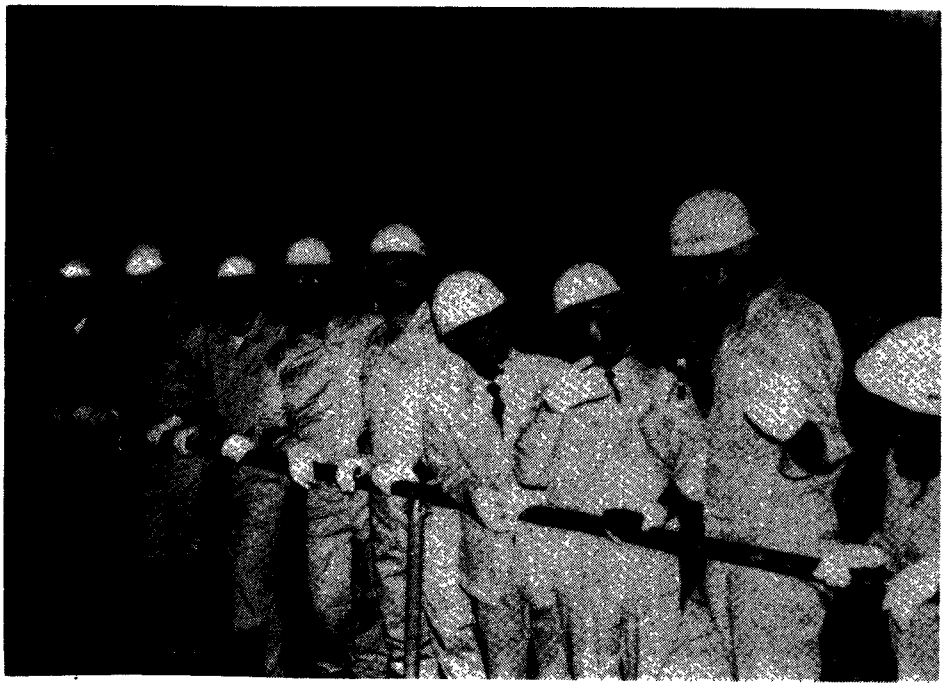
UNITED STATES DELEGATION

Delegates	Presentation Topics
JOHN J. CONVERY - Delegation Leader Director, Wastewater Research Division Water Engineering Research Laboratory U.S. Environmental Protection Agency Cincinnati, Ohio 45268	° Status of the Potable Water Reuse Demonstration Project at Denver ° Selected Innovative and Alternative Technology Developments
ROBERT J. BLANCO (WH-595) Director, Municipal Facilities Division Office of Water U.S. Environmental Protection Agency Washington, D.C. 20460	° The Nation's Water Quality: Into the 21st Century
RICHARD C. BRENNER Environmental Engineer, Innovative & Alternative Technology Staff Systems & Engineering Evaluation Branch Wastewater Research Division Water Engineering Research Laboratory U.S. Environmental Protection Agency Cincinnati, Ohio 45268	° Status of Fine Pore Aeration in the United States ° Summary of Chemical and Biological Phosphorus Removal Experience in North America
DR. CARL A. BRUNNER Chief, Systems & Engineering Evaluation Branch Wastewater Research Division Water Engineering Research Laboratory U.S. Environmental Protection Agency Cincinnati, Ohio 45268	° Selected Municipal Sludge Topics
JAMES F. STAHL Assistant Chief Engineer and Assistant General Manager County Sanitation Districts of Los Angeles County 1955 Workman Mill Road Whittier, California 90607	° Sludge Management and Energy Production at Los Angeles County Sanitation Districts (LACSD)
PAUL D. ZUGGER Chief, Surface Water Quality Division Michigan Department of Natural Resources Lansing, Michigan 48909	° Michigan's Process for Regulating Toxic Substances in Surface Water Permits ° The Use of Biomonitoring in Michigan's Surface Water Quality Program

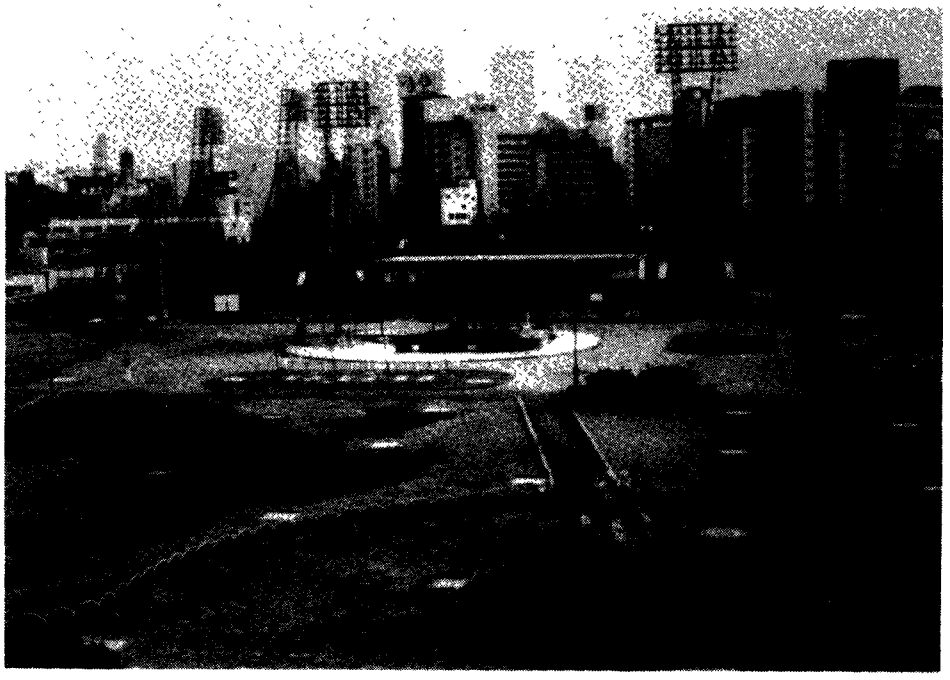


UNITED STATES AND JAPAN DELEGATES TO THE ELEVENTH CONFERENCE, HELD AT THE MINISTRY OF FOREIGN AFFAIRS, TOKYO, JAPAN

MR. TSUNEKAZU FUKUI, TEAM LEADER OF THE JAPANESE DELEGATION EXTENDS WELCOME TO MR. JOHN J. CONVERY, HEAD OF THE UNITED STATES DELEGATION AT OPENING OF ELEVENTH CONFERENCE, TOKYO, JAPAN



CONFERENCE DELEGATES VIEWING NEW CONSTRUCTION OF STORM SEWER
CITY OF OSAKA



PARK BUILT OVER OCHIAI SEWAGE TREATMENT PLANT - TOKYO

CONFERENCE DELEGATES READYING FOR AERIAL TOUR OF SEWAGE TREATMENT PLANTS IN YOKOHAMA CITY

CONFERENCE DELEGATES ARRIVING HOKUBU SEWAGE SLUDGE TREATMENT CENTER OF YOKOHAMA CITY TO OBSERVE EGG-SHAPED DIGESTERS

JOINT COMMUNIQUE

Eleventh United States/Japan Conference
on
Sewage Treatment Technology

Wednesday, October 14, 1987

1. The Eleventh United States/Japan Conference on Sewage Treatment Technology was held in Tokyo, Japan from October 12 to 14, 1987.

2. The United States Delegation headed by Mr. John J. Convery, Director, Wastewater Research Division, Water Engineering Research Laboratory, U.S. Environmental Protection Agency (USEPA), Cincinnati, Ohio was composed of four representatives from the USEPA, one representative from the State of Michigan and one representative from a local authority (Los Angeles County) in California.

3. Mr. T. Fukui, Director, Department of Sewerage and Sewage, Ministry of Construction, was the Head of the Japanese Delegation, which consisted of eight national government representatives, four Japan Sewage Works Agency representatives, and four local government officials (Tokyo, Osaka, Yokohama and Fukuoka).

4. During the Conference papers relating to the joint research projects on sludge treatment/disposal, energy conservation, diffused aeration, phosphorus removal and wastewater reuse were presented by both sides. A Progress Report on United States/Japan Joint Research Projects (Japanese Side) was presented. Data and findings on the joint research were useful to the development of improved control technology practices for each country. A decision was made to expand the scope of the joint research projects conducted by the two countries to include disinfection.

5. Principal topics of the Conference were current water pollution control programs in each country including innovative and alternative technology, regional approaches of toxics control, nutrients removal and application of biotechnology.

6. In addition to the Conference, a discussion meeting is to be held at the Public Works Research Institute, Ministry of Construction in Tsukuba, and about 50 engineers from the Ministry of Construction, local governments, Japan Sewage Works Agency and private firms are registered to attend the discussion meeting. During the meeting, topics of small flow wastewater facilities, application of advanced wastewater treatment, sewer corrosion, and toxics control in municipal wastewater treatment plants are to be presented and discussed.

7. Field visits in the Tsukuba, Fukuoka, Osaka, Tokyo and Yokohama areas are planned to inspect wastewater treatment facilities and related facilities in these areas.

8. Recent engineer exchanges between the two countries include a two-week visit in 1986 to Japan by Dr. A. E. Eralp, Wastewater Research Division, Water Engineering Research Laboratory, U.S. Environmental Protection Agency, and an eighteen-month visit to the United States by Mr. Hitoshi Nakazawa, Japan Sewage Works Agency from 1987 to 1988. A twelve-month visit from 1987 to 1988 to the United States by Mr. Takehiko Kawai of the Japan Sewage Works Agency was agreed to by both parties.

9. It was proposed by the United States side that the Twelfth Conference shall be held in the United States about October, 1989.

10. A proceedings of the conference will be printed in English and Japanese.

JAPANESE PAPERS

CURRENT STATUS OF SEWERAGE PROJECT IN JAPAN..... 7
Tsunekazu Fukui, Director, Department of Sewerage and Sewage Purification, City Bureau, Ministry of Construction

LAKE WATER QUALITY MANAGEMENT IN JAPAN..... 29
Mitsuyoshi Ozawa, Director, Water Quality Management Division, Water Quality Bureau, Environment Agency, Government of Japan

AREAWIDE SEWAGE SLUDGE TREATMENT AND DISPOSAL PROJECT -- ACE PLAN --... 51
Tsutomu Tamaki, Executive Director, Japan Sewage Works Agency

AUTOGENOUS INCINERATION OF SEWAGE SLUDGE BY RECOVERING THE HEAT OF EXHAUST GAS..... 75
Kenichi Osako and Yorimitsu Yabu, Sewerage Bureau, Tokyo Metropolitan Government, Japan

BENEFICIAL UTILIZATION OF SEWAGE SLUDGE..... 93
Yoshio Oshima, Chief, Ultimate Disposal Section, Water Quality Control Division, Public Works Research Institute, Ministry of Construction

EVALUATION OF ENERGY SAVING USING HEAT BALANCE MODEL FOR MELTING PROCESS.....121
Taigo Matsui, Director, and Dr. Tadahiro Murakami, Senior Researcher, Research and Technology Development Division, Japan Sewage Works Agency

REUSE OF TREATED WASTEWATER IN FUKUOKA CITY.....153
Hiroshi Oniki, Director, Planning Division, Sewage Works Bureau, Fukuoka City

FLOOD CONTROL OF URBAN RUNOFF IN THE EAST AREA OF OSAKA PREFECTURE (NEYA RIVER BASIN).....207
Katsuhiko Kitai, Director, Sewerage Division, Osaka Prefectural Government

FULL SCALE EVALUATION OF PHOSPHORUS MASS BALANCE IN BIOLOGICAL PHOSPHORUS REMOVAL PROCESS.....235
Naohiro Taniguchi, Senior Researcher, and Shuzou Koike, Researcher, Research and Technology Development Division, Japan Sewage Works Agency and Takao Murakami, Senior Research Engineer, Ultimate Disposal Section, Public Works Institute, Ministry of Construction

JAPANESE PAPERS (continued)

CURRENT PRACTICES OF BIOLOGICAL NITROGEN REMOVAL PROCESSES IN JAPAN..... 265
Kazuaki Sato, Chief, Advanced Waste Treatment Section,
Water Quality Control Division, Public Works Research Institute,
Ministry of Construction

CHARACTERIZATION OF THE VARIOUS DIFFUSERS AND ITS APPLICATION..... 291
Kazunori Hosokawa, Director, Construction Division,
Sewage Works Bureau, Yokohama City

PROGRESS IN THE RESEARCH ON APPLICATION OF BIOTECHNOLOGY
ON WASTEWATER TREATMENT..... 345
Shigeru Ando, Director, River-Basin Sewerage Division,
Department of Sewerage and Sewage Purification,
Ministry of Contruction

UNITED STATES PAPERS

THE NATION'S WATER QUALITY: INTO THE 21ST CENTURY.....365
Robert J. Blanco, Director, Municipal Facilities Division, Office of Water, U.S. Environmental Protection Agency, Washington, D.C.

SLUDGE MANAGEMENT AND ENERGY PRODUCTION AT LOS ANGELES COUNTY SANITATION DISTRICTS (LACSD).....391
James F. Stahl, Assistant Chief Engineer and Assistant General Manager, County Sanitation Districts of Los Angeles County, Whittier, California

SELECTED MUNICIPAL SLUDGE TOPICS.....421
Dr. Carl A. Brunner, Chief, Systems & Engineering Evaluation Branch, Wastewater Research Division, Water Engineering Research Laboratory, U.S. Environmental Protection Agency, Cincinnati, Ohio

STATUS OF POTABLE WATER REUSE DEMONSTRAATION PROJECT AT DENVER.....443
William C. Lauer, Project Officer, Denver Reuse Plant, Denver, Colorado and John J. Convery, Director, Wastewater Research Division, Water Engineering Research Laboratory, U.S. Environmental Protection Agency, Cincinnati, Ohi

MICHIGAN'S PROCESS FOR REGULATING TOXIC SUBSTANCES IN SURFACE WATER PERMITS.....475
Paul D. Zugger, Chief, and James E. Grant, Surface Water Quality Division, Michigan Department of Natural Resources, Lansing, Michigan

SUMMARY OF CHEMICAL AND BIOLOGICAL PHOSPHORUS REMOVAL EXPERIENCE IN NORTH AMERICA.....603
Richard C. Brenner, Environmental Engineer, Wastewater Research Division, Water Engineering Research Laboratory, and Denis J. Lussier, Chief, Environmental Control Systems Staff, Center for Environmental Research Information, U.S. Environmental Protection Agency, Cincinnati, Ohio

STATUS OF FINE PORE AERATION IN THE UNITED STATES.....669
Richard C. Brenner, Environmental Engineer, Wastewater Research Division, Water Engineering Research Laboratory, U.S. Environmental Protection Agency, Cincinnati, Ohio and William C. Boyle, Ph.D., Professor, Civil and Environmental Engineering, University of Wisconsin, Madison, Wisconsin

UNITED STATES PAPERS (continued)

THE USE OF BIOMONITORING IN MICHIGAN'S SURFACE WATER QUALITY PROGRAM...703
Paul D. Zugger, Chief, Surface Water Quality Division,
Michigan Department of Natural Resources, Lansing, Michigan

SELECTED INNOVATIVE AND ALTERNATIVE TECHNOLOGY DEVELOPMENTS.....785
John J. Convery, James F. Kreissl, Dr. James A. Heidman,
Richard C. Brenner and Dr. Richard A. Dobbs, Wastewater
Research Division, Water Engineering Research Laboratory,
U.S. Environmental Protection Agency, Cincinnati, Ohio and
Richard Field, Land Pollution Control Division, Hazardous
Waste Engineering Research Laboratory, U.E. Environmental
Protection Agency, Edison, New Jersey

CURRENT STATUS OF SEWERAGE PROJECT IN JAPAN

by

Tsunekazu FUKUI

Director, Sewerage and Sewage Purification Department,
City Bureau, Ministry of Construction

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

Prepared for Presentation at:
11th United States/Japan Conference
on
Sewage Treatment Technology

October 1987
Tokyo, Japan

ABSTRACT

Sewage works in Japan are now being implemented under the Sixth Five-Year Program for Sewerage Construction that began in fiscal year 1986. Public investment in sewage works has been on the downtrend in the last several years owing to the financial difficulties faced by the Government.

This report introduces the existing state of Japan's sewerage construction schemes, with specific reference to the Sixth Five-Year Program for Sewerage Construction, model projects, technological development related to sewage works, etc., and touches on the problems in promoting the construction of sewage works.

CONTENTS

1. INTRODUCTION 10

2. SIXTH FIVE-YEAR PROGRAM FOR SEWERAGE CONSTRUCTION, AND SEWAGE WORKS PROMOTION POLICY 13
2-1. SIXTH FIVE-YEAR PROGRAM FOR SEWERAGE CONSTRUCTION 13
2-2. RECENT SEWERAGE CONSTRUCTION POLICY 16

3. MODEL PROJECTS 19
3-1. AQUATOPIA SCHEME 19
3-2. AMENITY SEWERAGE MODEL PROJECTS 21
3-3. WATER-FRONT VERDURE LANDSCAPING MODEL PROJECTS 22
3-4. MODEL SEWERAGE PROJECTS 22
3-5. UNTREATED MISCELLANEOUS WASTEWATER CONTROL MODEL PROJECTS ... 23
3-6. EFFLUENT RECYCLING MODEL PROJECT 25

4. RECENT SEWERAGE TECHNOLOGY DEVELOPMENT 25
4-1. THE MINISTRY OF CONSTRUCTION'S GENERAL TECHNICAL DEVELOPMENT PROJECT 25
4-2. TECHNICAL EVALUATION SYSTEMS FOR SEWERAGE CONSTRUCTION 26

5. CONCLUSION 28

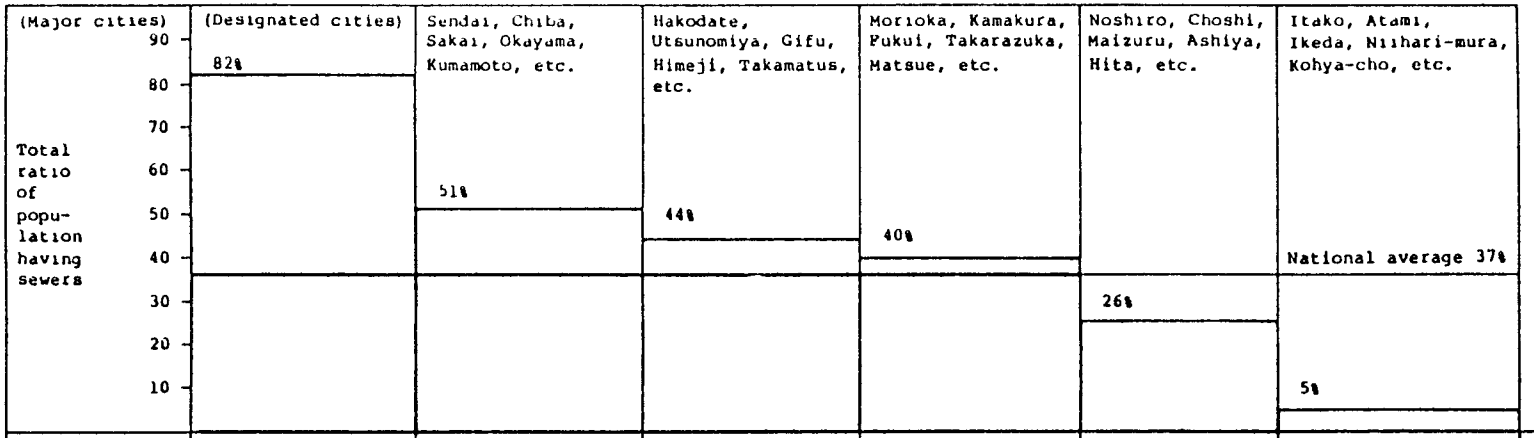
1. INTRODUCTION

The ratio of the population having sewers in Japan is now 37% on the national average. As seen in Table 1, the ratio stands at 80% in large cities but is very low in local cities having a small population. Cities with a population of more than 100,000 include some where no sewage works have yet been undertaken. Accordingly, systematic construction of sewage works needs to be promoted positively not just in large urban areas but in local cities having a small population.

In the 21st century, Japan is expected to face an aging of society going far beyond the levels in other countries. Before the turn of the century, the country must reinforce its social overhead capital including sewage works. In the "Long-Term National Land Construction Scheme" formulated by the Ministry of Construction to chart a new course for national land construction toward the 21st century, it is envisaged that the construction of sewage works will be promoted to cover about 90% of the total population. This long-term target is planned to be generally realized by 2000 in all urbanization promotion areas. The ratio of population having sewers in other areas is also planned to be raised to about 70%. In addition, stormwater drainage capabilities are planned to be enhanced to the extent that by 2000, all urban areas will be protected against inundation resulting from heavy rainfall occurring in a return period of 5 years and areas with concentrated population and urban functions will likewise be protected against inundation resulting from heavy rainfall occurring in a return period of 10 years.

Table 1 Level of Sewerage Project Implementation in Japan by Municipal Population

(As of end of 1986)



Population	1 million or more	500,000 - 1,000,000	300,000 - 500,000	100,000 - 300,000	50,000 - 100,000	Less than 50,000	Total
Total population	24,670,000	5,460,000	15,380,000	24,170,000	15,240,000	36,450,000	121,370,000
Population having sewers	20,310,000	2,780,000	6,710,000	9,690,000	3,980,000	1,950,000	45,410,000
Total number of municipalities	11	9	40	145	224	2,825	3,254
Number of project-implemented municipalities	11	9	40	143	196	611	1,010
Number of serviced municipalities	11	9	39	134	152	234	579

Notes:

1. Total number of municipalities (3,254) comprises 654 cities, 2,006 towns, and 594 villages (incl. Tokyo's ward section counted as a city).

2. Number of project-implemented municipalities is the sum total of municipalities where public sewerage construction has been implemented and those where special environmental protection sewerage system construction has been implemented.

Table 2 Major 2000 Targets of the Ministry of Construction Projects

	Project particulars	Existing state (1985 estimates)	2000 targets	Remarks
Road construction	Expressway/national highway construction	3,721 km	Near completion of 7,600 km	Construction along official route
	Footpath construction	83,000 km	Near completion of 236,000 km	Total length of roads to be built with footpaths
Housing construction	Amount of housing property	¥147 trillion*	¥283 trillion	* Net value as of the end of 1983 at 1980 prices, to be approximately doubled by 2000
	Housing standard	85.9 m^2/household*	To ensure that the induced housing standard is secured by 50% of the Japanese households. Example) Urban type induced housing standard (4-member household: 91 m^2) General type induced housing standard (4-member household: 123 m^2)	* Standard at the end of 1983
Sewerage construction	Total ratio of population having sewers	36%	Approx. 70%	To be generally attained in all urban areas.
City park construction	Per capita park area creation under city planning	4.9 m^2/person	10 m^2/person	To be doubled from the present level.
River improvement	Large rivers: Long-term target - Prevention of inundation due to floods occurring in a return period of 100 to 200 years. Short-term target - Prevention of inundation due to floods equivalent to the heaviest recorded after the war.	Short-term target of inundation prevention area - approx. 13,600 km^2	Short-term target of inundation prevention area - approx. 19,600 km^2	Estimated inundation prevention area - approx. 23,800 km^2
	Urban rivers: Long-term target - Prevention of inundation and avalanche/landslide damage due to hourly rainfall of 80 mm. Short-term target - Prevention of damage due to hourly rainfall of 50 mm.	Short-term targets of inundation and avalanche/landslide damage prevention areas - approx, 3,200 km^2 and approx. 110 km^2	Short-term targets of inundation and avalanche/landslide damage prevention areas - approx. 5,700 km^2 and approx. 560 km^2	Estimated inundation and avalanche/landslide damage prevention areas - approx. 6,700 km^2 and approx. 700 km^2

2. SIXTH FIVE-YEAR PROGRAM FOR SEWERAGE CONSTRUCTION, AND SEWAGE WORKS PROMOTION POLICY

2-1. SIXTH FIVE-YEAR PROGRAM FOR SEWERAGE CONSTRUCTION

In Japan, the systematic construction of sewerage systems was started under the Five-Year Program for Sewerage Construction and the Five-Year Program for Wastewater Treatment Plant Construction, both formulated in accordance with the Emergency Measures Law for Domestic Environment Institutions (1963). From 1967 on, it has been promoted under the Second to Sixth Programs for Sewerage Construction worked out by the enactment of the Emergency Measures Law for Sewerage Construction (1967). Sewerage construction carried out under these five-year programs has recently slowed down owing to the tight fiscal policy adopted by the Government under the financial reform program, with the result that the Fifth Five-Year Program registered a target attainment ratio of only about 70%.

Table 3 Target Attainment Ratios of Fifth Five-Year Program for Sewerage Construction

	Budget for Five-Year Program (¥100 million)	1981	1982	1983	1984	1985	Total for five years	Attainment ratio
Public sewerage system	83,910	14,609	13,937	13,009	12,946	13,889	68,390	81.5%
Regional sewerage system	22,300	2,947	2,483	2,406	2,411	2,662	12,910	57.9%
Urban storm drainage system	4,600	650	554	510	509	540	2,764	60.1%
Specific public sewerage system	270	32	26	25	25	25	133	49.4%
Specific environmental protection sewerage system	1,020	111	97	97	117	164	584	57.3%
(Subtotal)	112,100	18,349	17,097	16,047	16,007	17,280	84,781	75.6%
Adjustment budget	5,900	-	-	-	-	-	-	
(Total)	118,000	18,349	17,097	16,047	16,007	17,280	84,781	71.8%

Note: Budget for each fiscal year includes the adjustment budget for the National Land Comprehensive Development Project.

The Sixth Five-Year Program for Sewerage Construction, formulated with a total budgetary appropriation of ¥12,200 billion (¥2,220 billion of which is appropriated as adjustment budget) for implementation beginning in fiscal 1986, is aimed at raising the rate of population coverage and the rate of storm sewer coverage to 46% and 44%, respectively, by the end of fiscal 1990, from 36% and 35% recorded at the end of 1985.

Table 4 Breakdown of Budget for Five-Year Program for Sewerage Construction

(Unit: ¥1 million)

Classification	Sixth Five-Year Program (1986 to 1990)	Fifth Five-Year Program (1981 to 1985)
Public sewerage system	7,720,000	8,391,000
General public sewage works	4,607,000	5,161,500
Independent municipal sewage works	3,113,000	3,229,500
Regional sewerage system	1,760,000	2,230,000
General public sewage works	1,628,000	2,073,900
Independent municipal sewage works	132,000	156,100
Urban storm drainage system	300,000	460,000
Specific public sewerage system	20,000	27,000
General public sewage works	10,000	18,100
Independent municipal sewage works	10,000	8,900
Specific environmental protection sewerage system	180,000	102,000
General public sewage works	135,000	76,500
Independent municipal sewage works	45,000	25,500
Adjustment budget	2,220,000	590,000
Total	12,200,000	11,800,000
Total budget for general public sewage works	6,680,000	7,790,000
Total budget for independent municipal sewage works	3,300,000	3,420,000

Table 5 Estimated Level of Sewerage System Improvement under Five-Year Program for Sewerage Construction

Indicator	End of 1985	End of 1990
Total population	120,300,000	122,830,000
Population having sewers	43,330,000	54,390,000
Ratio of population having sewers (%)	36	44
Rate of storm sewer coverage of the population (%)	35	43

Note: Figures include those of independent sewage works run by municipalities.

The following are the priority targets in the Sixth Five-Year Program:

(1) Sewerage construction shall be promoted to provide the people with a better living environment and to meet the requirements for the regional environmental pollution control program, earlier accomplishment of environmental water quality standards, area-wide water pollution reduction program, lake water quality protection program, etc.

(2) The percentage of sewers in major cities shall be raised to the levels in advanced countries in Europe and North America, and sewerage construction in local cities shall be positively promoted.

(3) Sewage works shall be promoted to improve stormwater drainage in frequently flooded urban areas and to raise the general stormwater drainage capabilities in major cities.

(4) Sewage works shall be promoted to conserve the valuable natural environment and to improve the living environment in rural districts embracing mountains, farming and fishing villages. Small public sewage works shall also be constructed in areas with a population of less than 1,000, if required for a specific reason.

(5) Sewage works in newly urbanized areas shall be promoted positively as part of the housing and housing lot development program.

(6) Advanced treatment processes shall be introduced positively at wastewater treatment plants located in those areas needing such treatment for prevention of eutrophication of lakes and water quality conservation of rivers used as water supply sources.

(7) The Amenity Sewerage Project shall be promoted to create a more comfortable living environment through sewerage construction.

(8) Regional sludge disposal programs shall be promoted for stable sludge treatment and disposal in major city areas. For this purpose, Treasury investments and loans shall be introduced to the Japan Sewage Works Agency for execution of the Areawise or Resional Sewage Sludge Treatment and Disposal Project (ACE Plan).

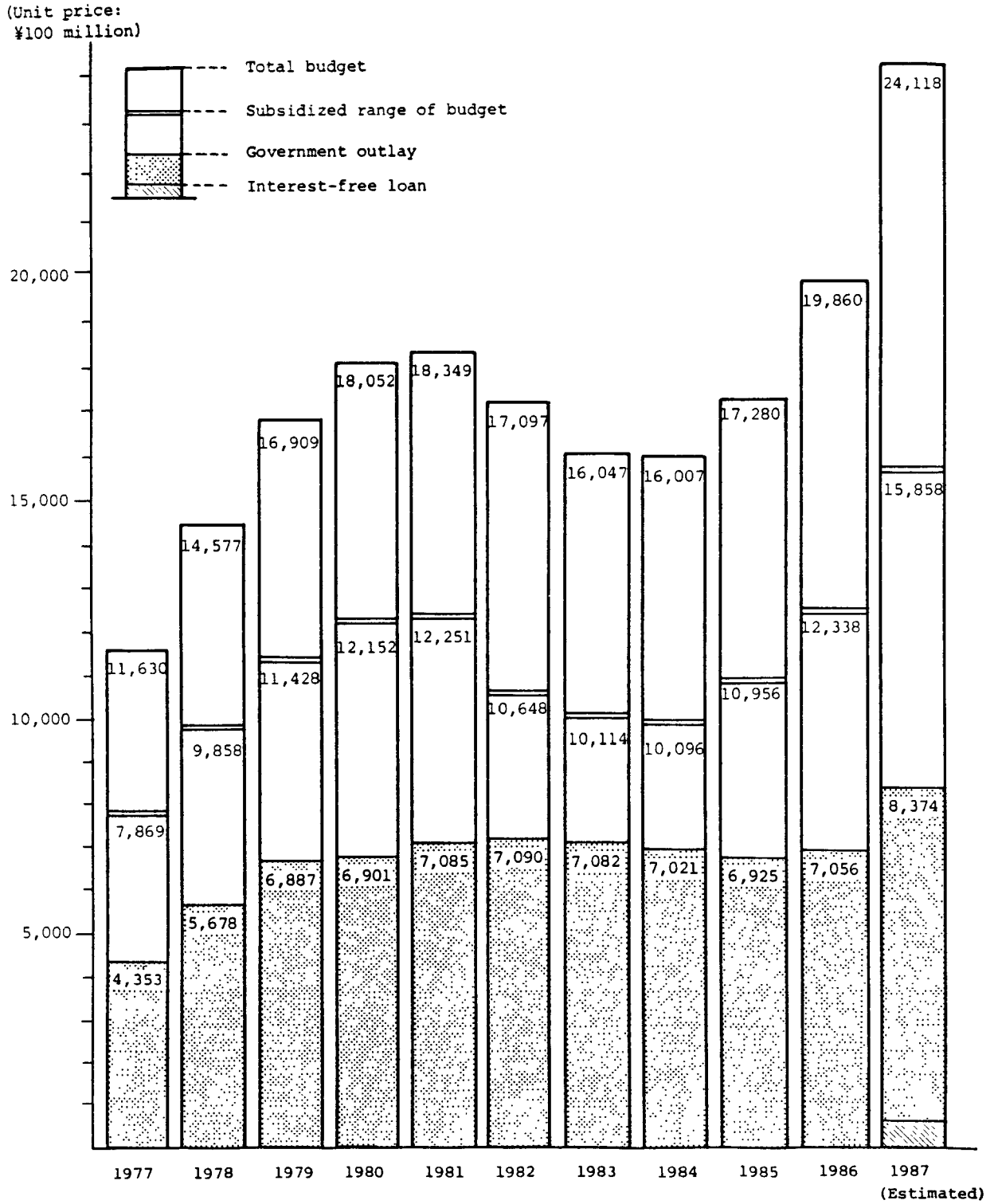
(9) To make the best of the resources value of sewage works, efforts shall be made for recycling of effluents from sewage treatment plants, effective use of sewage sludge, and multipurpose utilization of sewerage facilities.

(10) Resources- and energy-conservation programs shall be promoted, along with the development of new technologies such as biotechnology for wastewater treatment.

2-2. RECENT SEWERAGE CONSTRUCTION POLICY

Owing to the stringent fiscal policy adopted by the Government, the budget for sewage works kept dwindling until 1984 after reaching its peak in 1981. In subsequent years, the total amount of the budget was on an uptrend and registered a record high for fiscal 1986. However, the government outlay in the total budget dropped to ¥656.6 billion in 1987 from the 1981 peak of ¥709 billion, thus showing the same level as reached 8 to 9 years ago.

This means that the total budget was raised to the highest ever recorded without any increase in government outlay. Contributing factors to this bugetary expansion include the flexible application (increase) of special local bonds, increases in Treasury investments and loans, and reduction in the subsidy rate.



(Note) 1. The budgets in 1986 or earlier include the adjustment works expenses for overall national land development and other expenses.
2. The estimated budget in 1987 is a revised one.

Fig. 1 Annual Changes in Budget for Sewage Works (Initial Budget)

For fiscal 1987, the Government compiled an enlarged revised budget aimed at activating domestic demand under an emergency economic policy. Accounting for 31% of the initial budget, it amounts to a total of ¥386 billion, including ¥65.3 billion from the proceeds from the sale of NTT (Nippon (Japan) Telegraph and Telephone Corporation) shares to be used for credit granting without interest, and is intended for business invigoration in areas suffering from depression by investments in public sewerage sewers and regional sewerage construction.

Introduced below are some of the new measures enforced recently for promotion of sewerage construction.

(1) Construction of small public sewage works

Small public sewage works where there is a population of less than 1,000 are planned to be constructed in districts embracing stagnant water bodies such as lakes and pounds and in water source areas as part of the specific environmental protection sewerage system. Since these public sewerages are to be constructed in areas having a small population and low population density, it is necessary to adopt a sewerage system which is different from the urban sewerage system and is designed with full consideration to all important local characteristics of each individual area. For this reason, the Ministry of Construction has formulated a "Design Guideline for Small-scale Sewerage Systems (draft)" to promote this project.

(2) Extension of subsidized range

Sewerage construction in municipalities is prevented from making smooth progress because most municipalities have an extremely low ratio of population who have sewers and limited financial capabilities and also because the unit construction cost is made inevitably high by the small size of the required sewage works. For this reason, the subsidized range is extended for the construction of municipal public sewerage works and specific environmental protection sewerage systems.

(3) Extension of subsidized range for movable sludge treatment facilities

The subsidized range is extended to cover movable sludge treatment facilities (sludge treatment vehicles, sludge carrying cars, etc.) for sewage sludge treatment using multiple small-scale sewerage systems. To cope with the anticipated future expansion of small-scale sewerage construction projects in rural areas, these facilities will be used to provide itinerant sludge treatment service covering several treatment plants, or they will be installed together at a central sewage treatment plant or in other suitable place for treatment of sludge carried from other treatment plants by sludge carrying vehicles. This is because the movable facilities incur less sludge treatment cost than the fixed facilities if the sewerage system is small in scale.

(4) Implementation of Water-front Verdure Landscaping Model Project
This model project is being implemented with a positive national subsidy granted to create eye-pleasing, reposeful water-front environments within city areas by introducing particular structural designs of sewerage facilities (storm sewers) and by promoting the canal-side verdure landscaping and promenade construction. A detailed introduction of this project is given later.

(5) Implementation of Areas-Wide or Regional Sewage Sludge Treatment and Disposal Project (ACE Plan)
In large urban areas where sewerage construction is advanced, the increasing generation of sewage sludge is causing a heavy financial burden resulting from the rising treatment cost and at the same time making it increasingly difficult to secure a place for its treatment and disposal. This project is being implemented for integrated, systematic collection, treatment and disposal of sewage sludge generated at wastewater treatment plants operated under the management of several municipalities, with the Japan Sewage Works Agency serving as the principal executing organization.

3. MODEL PROJECTS

One of the salient features of the sewerage construction program in Japan is that a number of model projects are being implemented by establishing clear-cut objectives to improve the efficiency and image of sewerage construction projects as a whole.

3-1. AQUATOPIA SCHEME

This model project is being implemented to promote the efficient construction of sewage works in those cities which are selected for city planning and is aimed at creating an environment that will bring back aquatic animals that have disappeared, raise the water level to enable children to play in the water in the streets, and re-establish the disturbed harmony between the inhibitants and clear water. It was started in fiscal 1984, and has been implemented in a total 26 places so far, 17 in 1984, 3 in 1986, and 3 in 1987.

Table 6 New Sites Selected in 1987 for Aquatopia Model Project Implementation

Prefecture	Municipality	Main rivers	Project description/objective
Ishikawa	Kanazawa	Sai river, Asano river	Since older times, the citizens of Kanazawa have always felt close to the Sai river and the Asano river flowing through the city, regarding them as a symbol of the castle town of Kanawawa. The bonds uniting the citizens and the two rivers are still very strong, as evidenced by many traditional events such as the lighted lanterns floating on the water and the sweetfish liberation. The project is aimed at improving the water quality of the two rivers to bring back the flow of a limpid stream in both of them.
Kochi	Aki	Eno river, Aki river	Carp are liberated in the Eno river flowing through the central part of Aki City. The project is aimed at improving the water quality of the Eno river to bring back a clear stream suitable for carp to grow and propagate, and to create a comfortable water-front environment.
Miyazaki	Nobeoka	Ose river, Itsugase river	The Ose river and the Itsugase river flowing through Nobeoka City are well known for the sweet melt weir fishery. The citizens feel greatly attached to the rafs driving down the Itsugase and the fireworks display on the banks of the Ose river. The project is aimed at improving the water quality of the two rivers to bring the flow of a clear stream to both of them and to create a resposeful river-side environment for the citizens.

3-2. AMENITY SEWERAGE MODEL PROJECTS

This type of model project is implemented to make the citizens' daily lives more comfortable by making effective use of treated wastewater, and it is divided into two parts, "recovering the flow of a murmuring stream in open canals" and "quick snow removal in high snowfall areas." The former is intended to restore the flow of a murmuring stream by introducing treated wastewater into city canals (public sewerage storm sewers, urban storm drainage system) that are dried up or degraded in water quality. The latter is intended to introduce treated wastewater into public sewerage storm sewers and urban storm drainage systems in cities to melt the dumped snow for quick snow removal in cities. Since it was started in 1985, the project has been implemented in a total of 14 places, including six newly selected in 1987.

Table 7 New Sites Selected in 1987 for Amenity Sewerage Model Project

1. Recovering the murmuring flow in open canals

Kanagawa Prefecture	Yokohama City	Terao Storm Trunk Sewer	Kanagawa Sewage Treatment Plant of Yokahama City Public Sewerage System
Aichi Prefecture	Nagoya City	Arako River Storm Trunk Sewer	Uchide Treatment Plant of Nagoya City Public Sewerage System

2. Quick snow removal in high snowfall areas

Hokkaido	Sapporo City	Yasuharu Storm Trunk Sewer	Sosei River Treatment Plant of Sapporo City Public Sewerage System
Niigata Prefecture	Yuzawa-machi	Nunoba No. 1 Storm Trunk Sewer	Yuzawa Clarification Center of Yuzawa town Public Sewerage System
Ishikawa Prefecture	Kaga City	Joganji Storm Trunk Sewer	Katayamatsu Wastewater Treatment Plant of Kaga City Public Sewerage System
Fukui Prefecture	Tsuruga City	Maisaki-Shimizu Storm Trunk Sewer	Tento Clarification Center of Tsuruga City Public Sewerage System

3-3. WATER-FRONT VERDURE LANDSCAPING MODEL PROJECTS

Public sewerage storm sewers and urban storm drainage systems in city areas include many that can add a feeling of latitude to bustling urban life and are suitable for creating a reposeful water-front environment. Against the background of strong demand of urban dwellers for creating such water-front environment and enhancing the structural design of sewerage facilities to access to water, this project was started in 1987 to promote the construction of sewerage facilities (storm sewers) as well as canal-side verdure landscaping and promenade construction with a positive national subsidy. In 1987, the project is being carried out in four places including Asahikawa City, Hokkaido.

Table 8 Sites of Water-front Verdure Landscaping Model Project

Prefecture	Municipality	Site	Project outline
Hokkaido	Asahikawa	Hiyama No. 2 river No. 1 upstream storm trunk sewer	Planting and promenade construction over a distance of 2,800 m
Tochigi	Kanuma	Sende No. 1 storm trunk sewer	Planting and promenade construction over a distance of 285 m
Osaka	Toyonaka	No. 2 Hono southern storm trunk sewer	Planting and promenade construction over a distance of 1,640 m
Ehime	Saijo	Kannonsui storm trunk sewer	Planting and promenade construction over a distance of 1,440 m

3-4. MODEL SEWERAGE PROJECTS

3-4-1. Appeal Sewerage Project

This project is aimed at early completion of those sewage works that can strengthen the bonds uniting the citizens' daily lives and the sewerage systems, exhibit the effect of the sewerage construction project in a short time, and make a popular appeal to local residents and people at large. It was initiated in 1982, and has been implemented in a total of 52 places including five newly selected in 1987.

3-4-2. Idea Sewerage Project

This project is aimed at promoting those sewage works which are constructed by positively applying new, advanced technologies that are effective in cutting down the construction cost or maintenance cost. It was initiated in 1982, and has now been implemented in a total 25 places including one new place selected in 1987.

Table 9 New Sites Selected in 1987 for Model Sewerage Projects

1. Appeal Sewerage Project

Prefecture	Municipality	
Fukushima	Kitakata City	Snowless drainage system construction
Fukushima	Inawashiro Town	Snowless drainage system construction
Shizuoka	Shizuoka City	New sewerage construction to create a habitat for carp
Kochi	Ino Town	Rainwater storage strategy
Nagasaki	Fukue City	Spring water canal construction to create a firefly breeding environment

2. Idea Sewerage Project

Prefecture	Municipality	
Fukuoka	Fukuoka	Construction of a hybrid pump station combining cultural facilities

3-5. UNTREATED MISCELLANEOUS WASTEWATER CONTROL MODEL PROJECTS

3-5-1. Urban Water Channel Clean Up Model Project

This model project is intended to improve the water quality of urban storm drainage systems (channel) by constructing simplified treatment facilities in and outside those channels presenting the problem of water pollution due to domestic miscellaneous wastewater discharge in areas excluded from the public sewerage construction project. It was initiated in 1983, and has been implemented in a total of 13 places including three newly selected in 1987.

3-5-2. Miscellaneous Wastewater Control Project for Lake Basin

Water quality conservation of stagnant water bodies such as lakes is a matter of urgent necessity. This model project was implemented because the water quality conservation of such water bodies calls not only for the promotion of sewerage construction but also for the implementation of untreated miscellaneous domestic wastewater control measures that produce a quick water quality improvement effect. The project is aimed at reducing the inflow of pollution loads into lakes and ponds by allowing untreated domestic wastewater discharged into canals to flow into the nearby sewerage sewers in areas where sewerage construction is not undertaken. It was initiated in 1984, and has been implemented in a total of 10 places including four newly selected in 1987.

Table 10 New Sites Selected in 1987 for Untreated Miscellaneous Wastewater Control Model Projects

1. Urban Water Channel Clean Up Model Project

Prefecture	Municipality	Urban drainage system
Yamagata	Yome Town	Yome Urban Drainage System
Saitama	Yashio City	Taisho No. 1 Trunk Sewer Urban Drainage System
Shiga	Nagahama City	Sagami River Urban Drainage System

2. Miscellaneous Wastewater Control Project for Lake Basin

Prefecture	Sewerage system	Service area	Remarks
Chiba	Kashiwa City Public Sewerage System	Natogaya area, Kashiwa City	Related to the Teganuma Basin Sewerage Project
Chiba	Kashiwa City Public Sewerage System	Wakashiba and Toyoni areas, Kashiwa City	Related to the Teganuma Basin Sewerage Project
Chiba	Abiko City Public Sewerage System	Shimogato and Okahatto areas, Abiko City	Related to the Teganuma Basin Sewerage Project
Chiba	Kamagaya City Public Sewerage System	Satsuma area, Kamagaya City	Related to the Teganuma Basin Sewerage Project

3-6. EFFLUENT RECYCLING MODEL PROJECT

Treated wastewater (effluent from the plant) available in abundance in urban areas is a promising source of water supply. This project is aimed at purifying treated wastewater to supply it to office buildings for various non-drinking purposes such as flushing toilets, etc. in city areas where the water supply is falling short of demand. It has been implemented in Fukuoka, Tokyo, and other areas.

Tokyo and its vicinities have suffered an unusual drought from around June this year, and this has caused the people to direct their keen attention to the recycling of treated wastewater for effective utilization of water resources. The Ministry of Construction plans to make further efforts to promote its recycling scheme including this project.

Table 11 Outline of Effluent Recycling Model Project

Prefecture	Chiba	Tokyo	Hyogo	Okayama	Kagawa	Fukuoka
Sewerage system	Inbanuma Basin Sewerage System	Tokyo Metropolis Public Sewerage System	Kobe City Public Sewerage System	Yamate-mura Public Sewerage System	Takamatsu City Public Sewerage System	Fukuoka City Public Sewerage System
Time of project implementation	1986	1981	1986	1986	1987	Phase 1: 1979 Phase 2: 1987
Time of service commencement		October 1984				June 1980
Service area	Makuhari expanded new city center area	Shinjuku subcenter area	Rokko island area	Yamate-mura city center area	Fukuoka and Asahi No. 1 and 2 areas	Tenjin-Watanabe Street and Chigyo Momochi areas
Maximum design water recycling volume	4,500 m^3/day	4,000 m^3/day	4,200 m^3/day	100 m^3/day	1,000 m^3/day	3,000 m^3/day

4. RECENT SEWERAGE TECHNOLOGY DEVELOPMENT

4-1. THE MINISTRY OF CONSTRUCTION'S GENERAL TECHNICAL DEVELOPMENT PROJECT

The Ministry of Construction's General Technical Development Project is a system for integrated, organized research activities that are planned and promoted by relevant administrative departments in close cooperation with universities and private sector research institutions by selecting those construction technology research and development themes which need to be given high priority and cover diverse fields. Instituted in 1973, the system covers two programs related to sewerage construction, i.e., "Program for Waste Utilization Technology Development for Construction Works (1981 to 1985)" and "Program for Application of Biotechnology to Wastewater Treatment System. (from 1985)."

The former program is aimed at developing waste recycling technologies that can be applied to reduction of the huge materials requirements for construction works to meet the dual purpose of solving the growing difficulty in securing sites for waste treatment and disposal and ensuring effective utilization of resources. In the sewerage construction areas, a technique for processing sewage sludge to use it as construction materials has been developed under the program.

The latter program, aimed at reducing the construction and maintenance costs of sewerage systems and on-site wastewater treatment plants and improving their processing capacity, has been implemented to develop a new wastewater treatment system applying the rapidly advancing biotechnological know-how such as the microorganism immobilization technique, bioreactor technique, cell hybridization technique, gene recombination technique, etc., to realize energy- an cost-saving wastewater treatment, recover useful resources from wastewater, and reduce the size of treatment plants.

4-2. TECHNICAL EVALUATION SYSTEMS FOR SEWERAGE CONSTRUCTION

Since sewage works are undertaken as public works, no error or failure is allowed in their execution. This makes it inevitable to pay attention to risks involved in newly developed techniques and makes it difficult to introduce advanced techniques despite the rapid progress of technological development in recent years. Hence, it is imperative to establish a system for evaluating new techniques in some way for their accelerated introduction.

4-2-1. Ministry of Construction's Technical Evaluation System for Construction

The Ministry of Construction's Technical Evaluation System for Construction was established in 1978 to make unbiased evaluation of the functional advantages, performance, economic efficiency, etc. of private sector research results relating to construction technologies so as to accelerate the utilization of new technologies and promote private sector research and development activities by making such research results widely known to the public. Under the system, the Ministry of Construction announces 4 to 5 development themes each year together with the development objectives, and invites private sector research institutions to take part in the research of selected themes. These themes include those related to sewerage construction almost every year, and a microwave melting hearth, aeration equipment, belt-press dewatering machine, DO meter, etc. have so far been evaluated under the system.

Table 12 List of New Development Themes Evaluated under the Ministry of Construction's Technical Evaluation System for Construction

Year	1979	1980	1981	1982	1983
Theme	Development of new microwave melting hearth for sewage sludge processing	Development of high strength, large-diameter, oval hard PVC pipe	Development of deep shaft process for sewage treatment	Development of energy-saving, diffused aeration system	Development of mechanical aeration equipment for oxidation ditch process
Year	1984	1985	1986	1987	
Theme	Development of sewage solid-liquid separation method using screen process	Development of high-efficiency, belt-press type sewage sludge dehydrating machine	Development of automatic dissolved oxygen meter for aeration tank control	Development of monitoring equipment for sewer pipe inside inspection	

4-2-2. Japan Sewage Works Agency's Technical Evaluation System

A technical evaluation committee was organized within the Japan Sewage Works Agency in 1974 to pass impartial judgement on the conditions and suitability of introducing for practical purposes newly developed technologies relating to sewerage construction. Under the system, the committee evaluates the work method, treatment process, and other aspects relating to the principle of each newly developed technique, and its reports are made public both in Japan and abroad and reflected in the Agency's services.

Table 13 List of Development Themes Evaluated under Japan Sewage Works Agency's Technology Evaluation System

1	2	3	4	5
Automated control of sewage treatment plant (July 1973)	Pure Oxygen activated sludge process (July 1973)	Existing sewage sludge incinerators (July 1975)	Rotary Kiln (August 1977)	Carver Greenfield Process (August 1977)
6	7	8	9	10
Rotating biological contactor process (August 1977)	Sewage Sludge Compositing facilities (June 1981)	Oxidation ditch process (December 1982)	Biological nitrogen and/or phosphorus removal processes (November 1984)	Sequencing batch activated sludge process (August 1985)

4-2-3. Evaluation System for Private Sector Techniques

This is the third technical evaluation system instituted this year to accelerate the application of newly developed private sector techniques to keep pace with the rapid progress of technological renovation in various industrial and business sectors. Implemented by the Japan Sewage Works Agency, the system has two outstanding features not found in the previously initiated systems, i.e., the development themes are selected by private enterprises themselves and evaluation is completed in a shorter time.

It is hoped that the operation of these systems will open up the way to introducing new techniques without any apprehension about the possible risks involved and at the same time accelerate the pace of technological development.

5. CONCLUSION

In the foregoing, the current status of Japan's sewage works has been briefly introduced. There are many problems to be solved in the future to improve the level of Japan's sewerage system, such as increasing the ratio of the population having sewers, and technological development for efficient implementation of sewerage construction projects. To increase the ratio of the population having sewers, it is necessary to realize a sharp increase in the budget for sewage works that has been curbed for some years past and to expand sewage works execution organization. Technological development needs to be pushed forward to carry out sewerage construction efficiently within the limits of meager budgetary appropriations. Fortunately, the budget has shown an uptrend in the last few years partly because of the general recognition of the importance of the sewerage system and the delay in its construction. The Ministry of Construction and all other parties concerned are intent on redoubling their efforts to promote the construction of sewage works in Japan.

LAKE WATER QUALITY MANAGEMENT IN JAPAN

by

Mitsuyoshi OZAWA

Director

Water Quality Management Division
Water Quality Bureau, Environment Agency
Government of Japan

Kasumigaseki, Chiyodaku, Tokyo, Japan, 100

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

Prepared for Presentation at:
11th United States/Japan Conference
on
Sewage Treatment Technology

October 1987
Kasumigaseki, Tokyo

LAKE WATER QUALITY MANAGEMENT IN JAPAN

by: Mitsuyoshi OZAWA
Director
Water Quality Management Division
Water Quality Bureau,
Environment Agency
Government of Japan
100

ABSTRACT

The factors responsible for the water pollution in lakes and reservoirs are not confined to those of factories and other business establishments but come in a broad range including those associated with livelihood, agriculture, animal husbandry and fisheries, and due to the fact that the controls exercised under the existing Water Pollution Control Law alone were not adequate, the Law concerning Special Measures for Preservation of Lake Water Quality was enacted in July 1984 and enforced in March 1985. This law is designed to preserve water quality of lakes and reservoirs by designating the lakes and reservoirs where the maintenance of EQS is urgently required. Lake water preservation plan is formulated for each designated lake. The plan aims to carry out various measures in comprehensive and systematic manner, including the development of sewer systems, effluent controls over various sources of pollution, and preservation of surrounding areas of the lakes and reservoirs.

As regards the prevention of eutrophication in lakes and reservoirs, the effluent standards for nitrogen and phosphorus were set at 120 mg/ℓ (daily average 60 mg/ℓ) for nitrogen and 16 mg/ℓ (daily average 8 mg/ℓ) for phosphorous under the Water Pollution Control Law. The number of lakes and reserviors to which the effluent standards were applied is 1,022 for phosphorus and 45 for nitrogen on July 15, 1985.

CONTENTS

ABSTRACT

1. INTRODUCTION 32

2. CLEAN LAKES LAW 38

(1) Basic Policy for the Preservation of Lake Water Quality 38

(2) Designated Lakes and Designated Areas 41

(3) Plan for the Preservation of Lake Water Quality 42

(4) Promotion of Preservation Projects 43

(5) Regulation for Reducing Pollutant Load 43

(6) Measures Other than Controls 46

3. EFFLUENT CONTROL ENFORCED TO CURB NITROGEN AND PHOSPHORUS IN LAKES 47

(1) Effluent Control on Nitrogen and Phosphorus 47

(2) Effluent Standards 47

(3) Water Areas Subject to Standards 48

1. INTRODUCTION

Lakes and rivers in Japan have played an important role as resources for municipal water supply, recreation, fresh water fisheries, and other purposes. However, the country's high economic growth, particularly from the late 1950's to the late 1970's, significantly changed both the social and economic environment surrounding these water bodies, resulting in rapid deterioration of their quality nationwide starting in the late 1960's.

To cope with this situation, the government enacted the Water Pollution Control Law in 1970 to establish control measures to prevent contamination of public water. And the national government established the Environmental Water Quality Standards for rivers, lakes, and sea area in 1971.

These efforts to improve the water quality of public waters through regulatory actions resulted in a remarkable improvement in both river and sea water quality. However, the Environmental Water Quality Standards achievement rate, which is defined as the ratio of number of water bodies meeting the standards to the total number of water bodies, was extremely low for lakes and other enclosed waters; moreover their quality has remained at the same level for several years (Fig. 1).

Fresh water red tide has appeared even in Lake Biwa, which is the largest lake in Japan, and is considered to have the strongest resistance to eutrophication among all of Japan's lakes. Eutrophication has brought about a variety of harmful effects, such as scenic damage to lakes and reservoirs due to decrease in the transparency or changes in the color of their water, filter trouble with water service and unpleasant odor or taste of tap water, and changes in the kinds of fish caught by fishing operators.

In lakes which are more susceptible to eutrophication, such as Lake Kasumigaura, water bloom develops abnormally every year, creating impediments to lake use in addition to disturbing the scenic beauty. Lake Biwa and other lakes have accordingly become an object of concern amongst surrounding and downstream residents. Thus the matter of improving lake water is of concern to the entire nation, and not just to the people living near and being closely related to any particular lake.

Under these circumstances, in December 1982, the Environment Agency established the Environmental Water Quality Standards on nitrogen and phosphorus as a first step to control lake eutrophication. The Law Concerning Special Measures for Preservation of Lake Water Quality - Clean Lakes Law - was enacted and promulgated on July 27, 1984. The law is intended to lay down a basic policy for the preservation of lake water quality and opens way for the enforcement of special measures for those lakes seriously needing action to meet the Environmental Water Quality Standards. The Clean Lake Law went into force in March, 1985 by Cabinet Order. And in July 1985, the Water Pollution Control Law laid down uniform National Effluent Standards on nitrogen and phosphorus in order to control the abundant organic production of phytoplankton — entrophication.

The Environmental Water Quality Standards are shown in Table 1 and the Effluent Standards are shown in Table 2.

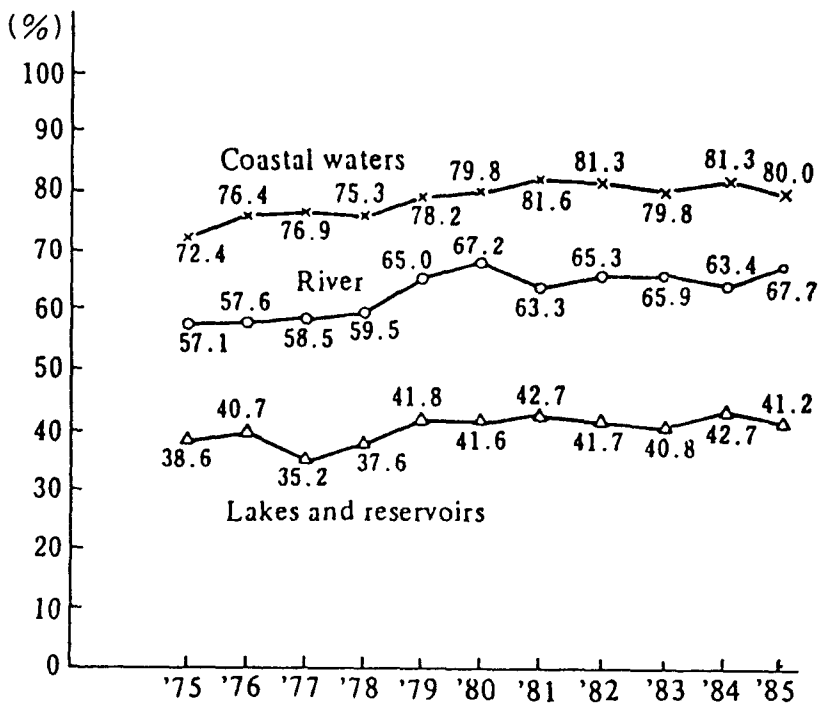


Fig. 1. Rate of Achievement of Water Quality Standards (BOD or COD) (Number of water areas meeting the standards over the total number of water areas under nation-wide monitoring program)

Table 1. The Environmental Water Quality Standards

(1) Water quality standards related to the protection of human health:

Item	Standard Values*
Cadmium	0.01 ppm or less
Cyanide	Not detectable
Organic phosphorus**	Not detectable
Lead	0.1 ppm or less
Chromium (VI)	0.05 ppm or less
Arsenic	0.05 ppm or less
Total mercury	0.0005 ppm or less
Alkyl mercury	Not detectable
PCBs	Not detectable

Notes: * Maximum values. But with regard to total mercury, standard value is based on the yearly average value. (1974 -)

** Organic phosphorus includes parathion, methyl parathion, methyl dimenton and E. P. N.

*** JIS K 0102: Japan Industrial Standard for testing and measurement of waste water.

Table 1. (continued)

(2) Water quality standards related to the preservation of the living environment for lakes (natural lakes, reservoirs, marshes and artificial lakes with more than 10 million cubic meters of water)

Category	Item / Purpose of Water Use	Standard Values: pH	Chemical Oxygen Demand (COD)	Suspended* Solids (SS)	Dissolved Oxygen (DO)	Number of Coliform Groups
AA	Water supply, class 1; fishery, class 1; conservation of natural environment, and uses listed in A-C	6.5 – 8.5	1 ppm or less	1 ppm or less	7.5 ppm or more	50 MPN/100 ml or less
A	Water supply, class 2 and 3; fishery, class 2; bathing, and uses listed in B - C	6.5 – 8.5	3 ppm or less	5 ppm or less	7.5 ppm or more	1,000 MPN/100 ml or less
B	Fishery, class 3; industrial water, class 1, agricultural water, and uses listed in C	6.5 – 8.5	5 ppm or less	15 ppm or less	5 ppm or more	
C	Industrial water, class 2; conservation of the environment	6.0 – 8.5	8 ppm or less	Floating matter such as garbage shall not be observed	2 ppm or more	

* With regard to fishery, classes 1,2 and 3, the standard value for suspended solids shall not be applied for the time being.

Notes: 1. Conservation of natural environment: Conservation of scenic spots and other natural resources.

2. Water supply, class 1: Water that requires treatment by simple cleaning operation, such as filtration.
Water supply, class 2: Water that requires treatment by normal cleaning operation, such as sedimentation and filtration.
Water supply, class 3: Water that requires treatment by highly advanced cleaning operation including pretreatment.

3. 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 silver carp inhabiting β-mesosaprobic water.

4. Industrial water, class 1: Water given normal cleaning treatment such as sedimentation.
Industrial water, class 2: Water given advanced treatment by chemicals.
Industrial water, class 3: Water given special cleaning treatment.

5. Conservation of the environment: Up to the limits at which no unpleasantness is caused to people in their daily life (including walking by the riverside, etc.).

Table 1. (Continued)

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

Note: 1. The Standards are measured in terms of annual averages.
2. The Standards for total nitrogen are applicable to lakes and reservoirs where nitrogen is judged to be the causal factor the growth of phytoplankton.
3. The Standards for total phosphorus are not applicable to agricultural water use.

Table 2. National Effluent Standards

(1) Substances related to the Protection of Human Health[1)]

Toxic substances	Permissible limits
Cadmium and its compounds	0.1 mg/ℓ
Cyanide compounds	1 mg/ℓ
Organic phosphorus compounds	1 mg/ℓ
(parathion, methyl paration, methyl demeton and EPN only)	1 mg/ℓ
Lead and its compounds	1 mg/ℓ
Hexavalent chrome compounds	0.5 mg/ℓ
Arsenic and its compounds	0.5 mg/ℓ
Total mercury	0.005 mg/ℓ
Alkyl mercury compounds	Not detectable[2)]
PCBs	0.003 mg/ℓ

Notes: 1. Prefectures may, by decree, set more stringent standards.
2. By the term "not detectable" is meant that the substance is below the level detectable by the method designated by the Director-General of the Environment Agency.

Table 2. (continued)

(2) Items related to the Protection of Living Environment[1),2)]

Item	Permissible limits
pH	5.8 ~ 8.6 for effluent discharged into public water bodies other than coastal waters 5.0 ~ 9.0 for effluent discharged into coastal waters
BOD, COD[3)]	160 mg/ℓ (daily average 120 mg/ℓ)
SS	200 mg/ℓ (daily average 150 mg/ℓ)
N-hexane extracts	5 mg/ℓ (mineral oil) 30 mg/ℓ (animal fat and vegetable oil)
Phenols	5 mg/ℓ
Copper	3 mg/ℓ
Zinc	5 mg/ℓ
Dissolved iron	10 mg/ℓ
Dissolved manganese	10 mg/ℓ
Chrome	2 mg/ℓ
Fluorine	15 mg/ℓ
Number of coliform groups (per cc)	3,000 (daily average)
Nitrogen[4)]	120 mg/ℓ (daily average 60 mg/ℓ)
Phosphorus[4)]	16 mg/ℓ (daily average 8 mg/ℓ)

Notes:
1. Prefectures may, by decree, set more stringent standards.
2. The standard values in this table are applied to the effluents from industrial plants and other places of business whose volume of effluents per day is not less than 50 m^3.
3. The standard value for BOD are applied to public waters other than coastal waters and lakes, while standard value for COD is applied only to effluents discharged into coastal waters and lakes.
4. Standard values for nitrogen and phosphorus are applied to lakes and reservoirs in which problems due to eutrophication may occur.
   The phosphorus standards are applicable to lakes and reservoirs where water stays for 4 days or longer (excluding those with a chlorine ion content of more than 9,000 mg/ℓ, and those where special dam operations are conducted) out of lakes and reservoirs with a drainage area of more than 1 Km^2 and a total surface area of more than 0.1 Km^2 (this limitation does not apply to lakes and reservoirs used as source of tap water). Also covered are rivers and other water bodies designated as "public water areas" flowing into the above lakes and reservoirs.
   The nitrogen standards are applicable to lakes and reservoirs where the figure obtained by dividing the nitrogen content of water by the phosphorus content is less than 20 and the phosphorus content of water exceeds 0.02 mg/ℓ, out of the lakes and reservoirs subject to the phosphorus regulation. Also covered are rivers and other water bodies designated as public water areas flowing into those lakes and reservoirs.

2. CLEAN LAKES LAW

The aims of the Clean Lakes Law can be summarized into two points.

Firstly, the law introduced well-defined regulations and other measures to cope with those pollution sources for which the conventional effluent controls do not work because of the special nature of lake environments. Secondly, the law provides for 1) drafting, for each of the designated lakes, lake preservation plans stipulating such environmental preservation projects as sewage construction and regulatory actions for the reduction of pollutant load, and 2) implementation of various preservation measures which are to be coordinated or promoted under the above plans.

The outline of the Clean Lakes Law is shown in Figure 2.

(1) Basic Policy for the Preservation of Lake Water Quality

The law requires the national government to lay down a basic policy for the preservation of lake water quality. The policy is to consist of: 1) basic concepts on the preservation of lake water quality; 2) fundamental issues concerning the setting up the lake preservation plan and other countermeasures for the preservation of water quality in designated lakes; and 3) other important issues concerning lake water quality (see note 1).

Note 1:

The Basic Policy for the Conservation of Lake Water Quality was decided at a Cabinet meeting on December 7, 1984.

Abstract of the Basic Policy

1. Basic Concepts on the Conservation of Lake Water Quality

Lakes have been playing an important role in everyday life and human activities since ancient times. To name some of their benefits, they provide stable water resources for drinking and for other purposes, fishery grounds, and outdoor recreation grounds in combination with the natural surrounding environment. They fulfill flood control and other functions, and have scientific values, such as forming a peculiar ecosystem.

These various benefits derive from the fact that lakes serve as receptacles on land for storing water. On the other hand, because of the hydrologic characteristics of water stagnancy, pollutants which flow in are apt to deposit, and thus they are vulnerable to pollution. In recent years, the situation has been deteriorating as a result of the increasing pollutant load on lakes due to the expansion of social and economic activities.

These circumstances call for action to ensure that future generations will be able to continue enjoying the diverse benefits of lakes. In order to conserve the quality of lake water, proper measures must be taken according to the characteristics of respective lakes. It is also important to promote various measures in a comprehensive manner responding to diverse pollutant sources.

The basic direction of lake water quality conservation measures to be taken are as follows:

(a) Taking measures according to characteristics of respective lakes such as water quality and water use (prevention of pollution and promotion of priority measures).

(b) Taking countermeasures to cope with pollution by organic matter and eutrophication.

(c) Making a balanced approach to deal with the diverse sources of pollution.

On the basis of this Basic Policy, the conservation measures must be strenuously promoted, with broad agreement and cooperation among the related organizations and those concerned.

2. Fundamental Issues concerning the Setting Up the Lake Conservation Plan and Other Countermeasures for the Conservation of Water Quality in Designated Lakes

(1) Fundamental issues concerning the setting up the lake conservation plan

The duration of conservation plans should in principle be five years. The following procedures should be taken to set up them.

(a) Determining the present levels of water quality in lakes and of the pollutant load to lakes and estimating future levels.

(b) Comprehensive deliberations on countermeasures which can be carried out during the duration of the plan.

(c) Setting targets and putting together countermeasures to be taken.

(2) Fundamental issues concerning countermeasures for the conservation of water quality in designated lakes

Basic matters concerning the following points should be decided.

(a) Construction of sewerages, night soil disposal plants and the like.

(b) Countermeasures for effluent from factories and other business establishments.

(c) Measures for domestic effluent.

(d) Measures for stockbreeeding.

(e) Measures for fish farming.

(f) Measures for non-point sources.

(g) Dredging and other cleanup measures for lakes.

(h) Protection of the natural environment on lake shores, such as the conservation of greenery areas.

3. Other Important Issues Concerning Lake Water Quality

(1) Water quality conservation measures for non-designated lakes

In the way of the countermeasures for designated lakes, necessary steps according to the characteristics of respective lakes and their causes of pollution should be taken.

(2) Water quality monitoring

(3) Promotion of surveys, research and technical development

(4) Dissemination of knowledge and uplift of consciousness

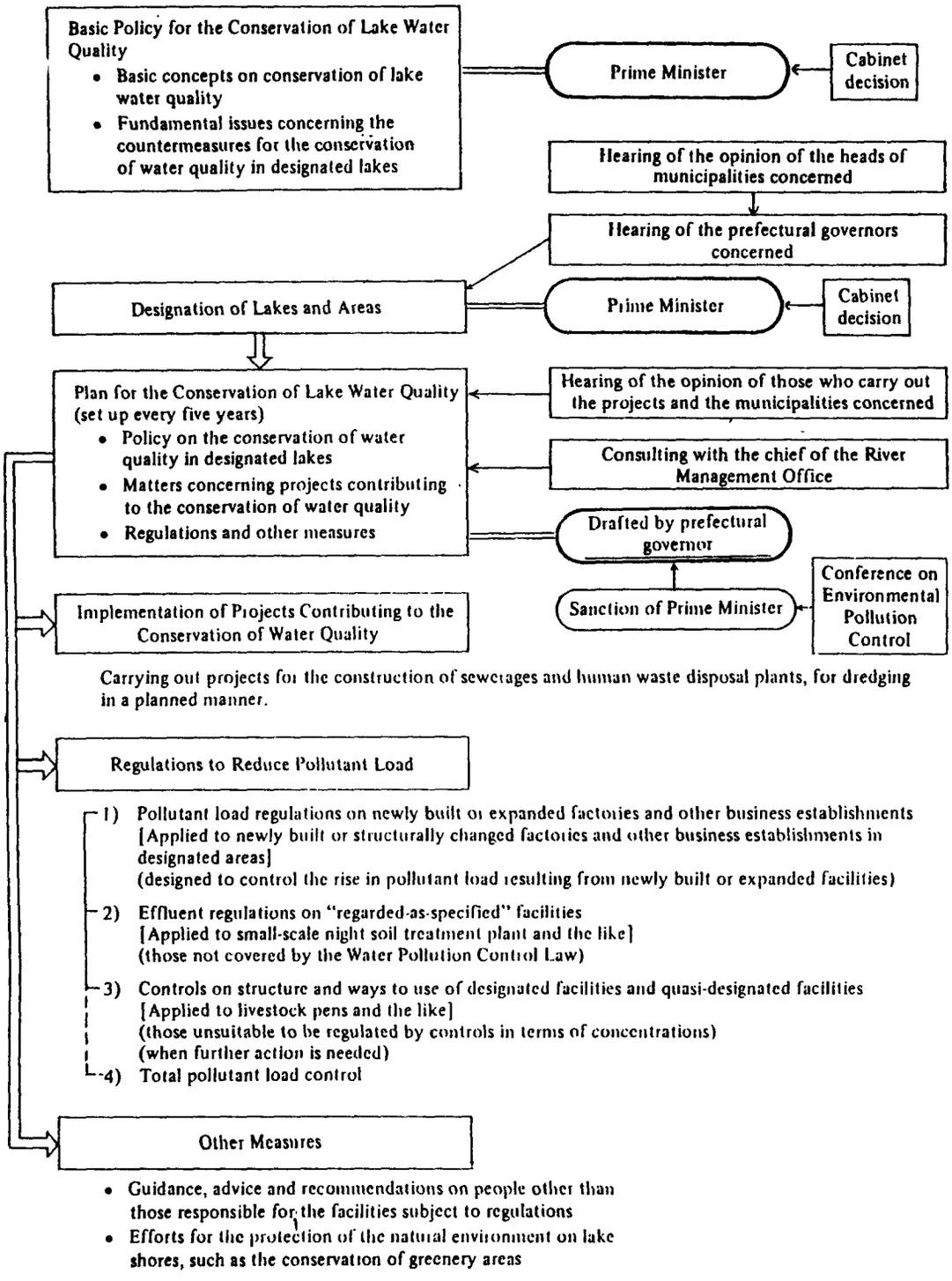


Fig. 2 Outline of Clean Lakes Law

(2) Designated Lakes and Designated Areas

Controls and other special measures under the Clean Lakes Law will be introduced for the "designated lake" and the "designated area" related to such lakes. Based on proposals from the prefectural governors, the Prime Minister will, identify the "designated lakes" for which the comprehensive measures for the preservation of water quality are needed, and will identify the "designated areas" effected by the water pollution in the designated lakes (see note 2).

Note 2:

(1) Lakes and Areas Designated according to Clean Lakes Law

- o Dec. 1985; the designation of five lakes: Lake Kasumigaura, Inbanuma, Teganuma, Lake Biwa and Lake Kojima
- o Oct. 1986; the designation of one lake: Lake Suwa.

(2) Data on the designated lakes are shown in Table 3.

Table 3. Water Quality and Environmental Problems in Six Designated Lakes

Name of the Designated Lakes	Prefecture	Water Quality (COD)		Problems (1972-)
		Water Quality Standards	Water Quality (1985)	
		(mg/ℓ)	(mg/ℓ)	
Lake Biwa	Shiga	(north) 1 (south) 1	2.4 3.7	Offensive oder or bad taste of drinking water, filtration troubles in water purifying plants, closing of bathing resorts
Lake Kasumigaura	Ibaragi	3	10	Offensive odor or bad taste of drinking water, damage to carp in piscicultural facilities, closing of bathing resorts
Inbanuma	Chiba	3	12	Offensive odor or bad taste of drinking water, deteriorated environment for sight-seeing and fishing
Teganuma	Chiba	5	29	Deteriorated environment for damage to carp in piscicultural facilities, deteriorated environment for sight-seeing, fishing and damage to fish
Lake Kojima	Okayama	5	10	Damage to rice crops
Lake Suwa	Nagano	3	5.2	Deteriorated environment for damage to carp in piscicultural facilities, deteriorated environment for sight-seeing, fishing and damage to fish

(3) Plan for the Preservation of Lake Water Quality

Every five years the prefectural governors will draw up plans, based on the basic policy, for preservation of water quality in the designated lakes. Plans are to include: 1) policy on the preservation of water quality in the lakes; 2) sewerage construction, dredging and other projects contributing to the preservation of water quality; 3) controls and other measures for preservation; and 4) other necessary measures to preservation quality of the lake water (see note 3).

Note 3:

Plan for the preservation of Lake Water Quality

Jan. ∿ March 1987 : Each plan for preservation of Lake Water Quality on five lakes has been formulated (1986-1990) (See Tables 4 and 5)

Table 4. Present and Predicted Water Quality in Five Designated Lakes

Name of the Designated Lakes	1985	1990 (interim target)	Environment Water Quality Standards in terms of COD (mg/ℓ)
Kasumigaura			
(Nishiura)	10	8.9	3
(Kitaura)	10	7.4	3
(Hitachitone River)	10	7.2	3
Inbanuma	12	10	3
Teganuma	29	18	5
Biwa			
(north)	2.4	2.2	1
(south)	3.7	3.4	1
Kojima	10	8.8	5

note: COD concentration (75% value) in mg/ℓ

Table 5. Breakdown of Total Pollutant Load in Terms of COD in Five Designated Lakes

Name of the Designated Lakes	Source (%)		
	Domestic	Industrial	Others
Kasumigaura	43	5	52
Inbanuma	63	4	33
Teganuma	77	5	18
Biwa	31	20	49
Kojima	68	11	21

(4) Promotion of Preservation Projects

The promotion of various projects as public works is needed to preserve lake water quality. These projects could be divided into two categories: 1) projects contributing to the reduction of the domestic pollution load, such as construction of sewage and night soil treatment plants; and 2) projects for cleaning up lakes, such as dredging, aeration and flushing. Viewing these projects as crucial to the preservation plans, the law provides for their promotion with cooperation between national and local governments.

(5) Regulation for Reducing Pollutant Load

In addition to the current regulatory standards the following regulations will be introduced in the designated areas. These deal with concentration of pollutants in effluent from factories and other business establishments under the Water Pollution Control Law (see Fig. 3).

1) Pollutant load controls on new or expanded factories and other businesses

The Clean Lakes Law prescribes additional pollutant load controls for newly built of structurally changed factories and other business establishments in designated areas, which have a scale above a certain level (at the level of facilities discharging effluent at the average rate of more than 50 cubic meters a day). These measures are designed to minimize the rise in the pollutant load resulting from the construction or expansion of factories and other businesses. The control standards, to be set as the permissible levels for the daily discharged pollutant load, will be specified by the prefectural governors on the basis of the formula laid down in the ordinance of the Prime Minister's Office. Prefectural governors are empowered to order changes in construction or expansion plans or to order improvement in facilities so as to secure compliance with such controls.

2) Effluent controls on "regarded-as-specified" facilities

Some facilities could be considered as pollution sources for lakes, although they are not covered by the controls under the Water Pollution Control Law. Under the provisions of the Clean Lakes Law, these facilities

will be regarded as "specified facilities" and brought under the effluent controls of the Water Pollution Control Law.

3) Controls on structure of designated and quasi-designated facilities and on their use

Standards in terms of the concentration or pollutant load do not suit the regulation of some pollution sources. Under the Clean Lakes Law, they will be treated as "designated facilities" and "quasi-designated facilities", and control will be applied on their structure and the ways they are used.

4) Total pollutant loads control

Controls in terms of the total pollutant load can be introduced for designated lakes, where it is deemed difficult to attain the Environmental Water Quality Standards by means of the current effluent controls under the Water Pollution Control Law and by the other above-mentioned measures, because of the high concentration of population and industry.

Fig. 3 Comparison of Water Pollution Control Law and Law concerning Special Measures for Preservation of Lake Water Quality

Water Pollution Control Law		Law concerning Special Measures for Conservation of Lake Water Quality
Conservation of the water quality of public waters	Purpose	Conservation of the water quality of lakes and reservoirs (special measures for designated lakes and reservoirs)
———	Conservation program	Formulation of conservation program for the lake water ↓ Systematic enforcement of measures such as projects and controls for the water quality conservation
Factories and business establishments (specified business establishments) — Effluent controls (controls on concentration) National uniform standards Prefectural standards stricter than national ones (wherever necessary) Areawide total pollutant load control	Control measures	• Factories, business establishments (specified business establishments) — Pollution load controls on new and additional facilities (specified business establishments) which discharge effluent into a lake basin Areawide total pollutant load control (if necessary) • Septic tanks over certain scales (Quasi-specified facilities) — Effluent control • Cattle sheds and facilities for aquaculture (designated facilities) — Regulation on structure and use
Water quality monitoring of public waters	Others	Guidance, advice and recommendations to persons not under the control of the law Protection of the peripheral natural environment of lakes and reservoirs

(6) Measures Other than Controls

In addition to those legal controls, the prefectural governors can extend guidance, and make advice or recommendations concerning pollution sources in order to carry out lake preservation plans. Small factories and other businesses will conceivably be the first targets of such measures. The national and local governments are required to make efforts for the preservation of greenery areas and the natural environment on the lake shores in order to contribute to the preservation of water quality in the designated lakes.

3. EFFLUENT CONTROL ENFORCED TO CURB NITROGEN AND PHOSPHORUS IN LAKES

(1) Effluent Control on Nitrogen and Phosphorus

The Environment Agency added new environmental quality standards for nitrogen and phosphorus related to water pollution in December 1982. In January 1983, it asked the Central Council for Environmental Pollution Control to recommend effluent standards regarding nitrogen and phosphorus in order to prevent the entrophication of lakes and reservoirs. After one year and eight months of deliberation, the council submitted a recommendation to the following effect:

1) Various problems due to entrophication are already occurring, at a considerable number of lakes and reservoirs, while some of other lakes and reservoirs also threaten to be beset with such problems.

2) Scientific observations in the past have made it clear that eutrophication is basically attributable to nitrogen and phosphorus.

3) From the foregoing, it is considered necessary to control the concentration of nitrogen and phosphorus in lakes and reservoirs with a view to preventing eutrophication.

In line with the recommendation, the Environment Agency revised, the cabinet order for implementation of the Water Pollution Control Law on May 17, 1985, and the enforcement regulation of the said law and the ordinance of the Prime Minister's Office stipulating effluent standards on May 27. Furthermore, the agency issued notifications on lakes and reservoirs subject to effluent standards for nitrogen and phosphorus and the methods for detection of those pollutants on May 30 to enforce the standards from July 15, 1985.

(2) Effluent Standards

The effluent standards for nitrogen and phosphorus are applicable to "specified facilities" under the Water Pollution Control Law, which discharge 50 cubic meters of effluent on the daily average. The effluent

standards have been set at 120 milligrams per liter for nitrogen (60 milligrams per liter on the daily average) and 16 milligrams per liter for phosphorus (8 milligrams per liter on the daily average). That is, in line with the recommendation by the Central Council for Environmental Pollution Control, the permissible limits have been fixed at the same levels of nitrogen and phosphorus concentration in water discharged from general households. However, regarding about 60 types of industry, including stock-raising, which will find it extremely difficult to meet the general effluent standards, provisional effluent standards, which are relaxed compared with the said effluent standards, will apply for five years after the date of enforcement.

Moreover, to ensure smooth implementation of the effluent standards, the existing specified facilities are exempted from their application for six months (one year for some industries) from the date of enforcement.

Furthermore, when the effluent standards are found inadequate to prevent eutrophication, the governors of the prefectures involved are empowered to fix stricter effluent standards.

(3) Water Areas Subject to Standards

The new effluent standards do not apply to rivers and the sea. But they are not applicable to all lakes and reservoirs. Standards for nitrogen are applied to a smaller number of lakes and reservoirs than is the case with the standards for phosphorus. The scope of application has been determined as follows:

The phosphorus standards are applicable to lakes and reservoirs where water stays for four days or longer (excluding those with a chlorine ion content of more than 9,000 milligrams per liter, and those where special dam operations are conducted) out of lakes and reservoirs with a drainage area of more than one square kilometer and a total surface area of more than 0.1 square kilometer (this limitation does not apply to lakes and reservoirs used as source of tap water). Also covered are rivers and other water bodies designated as "public water areas" flowing into the above lakes and reservoirs. The length of retention time is the most important indicator regarding lakes and reservoirs liable to eutrophication. Where

such time is short, phytoplanktons do not multiply significantly; therefore, it is not necessary to control nitrogen and phosphorus in lakes and reservoirs. On the other hand, when the retention time is more than four days, the influx of nitrogen and phosphorus results in the propagation of phytoplanktons, thus requiring control. Nevertheless, even if the retention time exceeds four days, phytoplanktons, which grow in fresh water, cannot multiply in those lakes and reservoirs where much sea water flows in, boosting the chlorine ion content to more than 9,000 milligrams per liter. Therefore, such lakes and reservoirs are exempted from the control. Among the lakes and reservoirs subject to the standards, Nakanoumi (Tottori and Shimane Prefectures) shows the highest chlorine ion content. Moreover, the standards are not applied, either, to dams with a high flood-adjusting capacity, where the retention time is limited.

The nitrogen standards are applicable to lakes and reservoirs where the figure obtained by dividing the nitrogen content of water by the phosphorus content is less than 20 and the phosphorus content of water exceeds 0.02 milligrams per liter of the lakes and reservoirs subject to the phosphorus regulation. Also covered are rivers and other water bodies designated as public water areas flowing into those lakes and reservoirs. Therefore, the concentrations of nitrogen and phosphorus are measured first, and the standards are applied only to those lakes and reservoirs which meet the above conditions.

The Environment Agency has selected lakes and reservoirs which meet the above-mentioned conditions with the cooperation of the prefectural governments and other quarters concerned. As a result, a total of 1,022 lakes and reservoirs will be subjected to the phosphorus regulations. Of them, 45 are also subject to the nitrogen regulation, including Hachirogata (Akita Prefecture), Kasumigaura, Inbanuma (Chiba Prefecture), Teganuma, Lake Biwa, Nakanoumi, Lake Shinji (Shimane Prefecture) and Lake Kojima (Okayama Prefecture). Among these are lakes and reservoirs in the upper and lower reaches of the same rivers where the drainage areas of those in the upper reaches are included in the lakes and reservoirs in the lower reaches. As to the nitrogen regulation, selection was made only concerning those lakes and reservoirs regarding which sufficient data for decision are available. Therefore, in case new or additional data become available for the other lakes and reservoirs, there will be additional designations.

AREAWIDE SEWAGE SLUDGE TREATMENT AND DISPOSAL PROJECT — ACE PLAN

by

Tsutomu TAMAKI

Executive Director, Japan Sewage Works Agency

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

Prepared for Presentation at:
11th United States/Japan Conference
on
Sewage Treatment Technology

October 1987
Tokyo, Japan

ABSTRACT

In the 1986 fiscal year, a new project for areawide sewage sludge treatment and disposal was started in Japan. Dubbed the Ace Plan, this project has the following features:

(1) Two or more local public bodies jointly treat and dispose of sewage sludge produced at their wastewater treatment plants. This project not only offers the merit of scale, but also is expected to facilitate the acquisition of sites for the treatment and disposal of sludge.

(2) Sewage sludge is regarded as a rich supply of energy and material, and its recycling and harnessing are promoted so that the sludge can be handled stably and perpetually, not as a burden, but as a useful resource. The joint treatment is preferable from the viewpoint of cost as well as of the recycling and harnessing operations.

(3) The Ace Plan is aided by the Japan Sewage Works Agency having abundant experience and advanced technical know-how regarding sludge treatment and disposal, and local public bodies can merely concentrate their efforts on the construction of sewers for the improvement of sanitary services to their constituents.

(4) Treasury investments and loans are available for the Ace Plan. Thus, local government bodies can push forward their areawide sewage sludge treatment and disposal projects without sustaining any initial cost.

CONTENTS

1. INTRODUCTION 54
2. STATUS QUO OF SEWAGE SLUDGE DISPOSAL 54
3. NECESSITY OF AREAWIDE SEWAGE SLUDGE TREATMENT AND DISPOSAL 57
4. JAPAN SEWAGE WORKS AGENCY, THE PRIME MOVER OF THE ACE PLAN 59
5. FUNDS FOR THE ACE PLAN 63
6. IMPLEMENTATION OF ACE PLAN 65
7. CONCLUSION 70

1. INTRODUCTION

At the Ninth United State - Japan Conference on Sewage Treatment Technology held in Tokyo in October 1983, I presented a paper titled "Current Topics on Sewage Works in Japan", and touched upon the concepts of an areawide sewage sludge treatment and disposal project which then was still in the study stages. About half the population of Japan lives in about 1/25 of the land area of the United States, and with the percentage of the sewered population being about 37%, all local governments are having difficulties in finding dump sites for sewage sludge. This situation is getting worse every year with the increase in the sewer coverage. To find a viable solution, the Japan Sewage Works Agency has so far promoted research and development for the implementation of an areawide sewage sludge treatment and disposal project at the request of the Ministry of Construction and local governments. The results of the research and development have been reflected in the Sixth Five-Year Program for Sewerage Construction started in FY 1986. As the Japan Sewage works Agency Act was amended in April 1986, the Japan Sewage works agency was commissioned to undertake the areawide sewage sludge treatment and disposal project.

2. STATUS QUO OF SEWAGE SLUDGE DISPOSAL

Table 1. Disposal of sewage sludge (April 1985 - March 1986)

(1,000 m^3/year)

Form of sludge \ Method of disposal	Landfill	Reclamation	Effective use	Others	Total (%)
Dewatered cake	845	430	183	63	1,521 (71)
Ash	165	103	7	4	279 (13)
Dry sludge	24	0	130	4	158 (7)
Digested sludge and thickened sludge	0	0	0	180	180 (9)
Total (%)	1,034 (48)	533 (25)	320 (15)	251 (12)	2,138 (100)

Note: Dry sludge includes composted sludge.

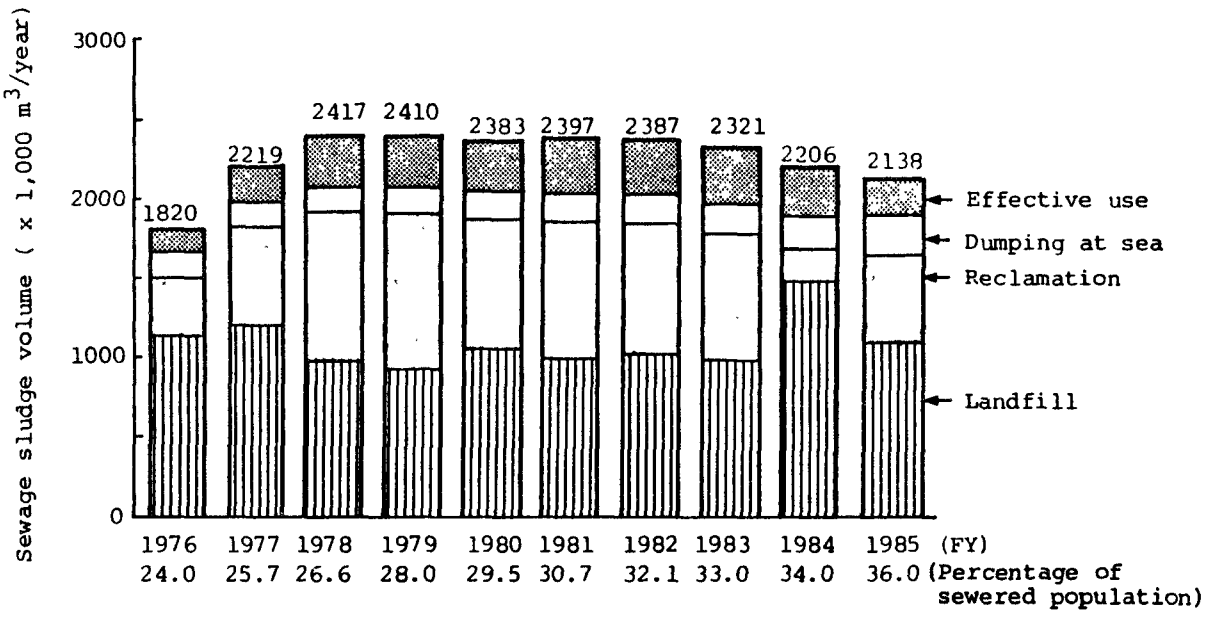


Figure 1. Annual transition of sewage disposal methods

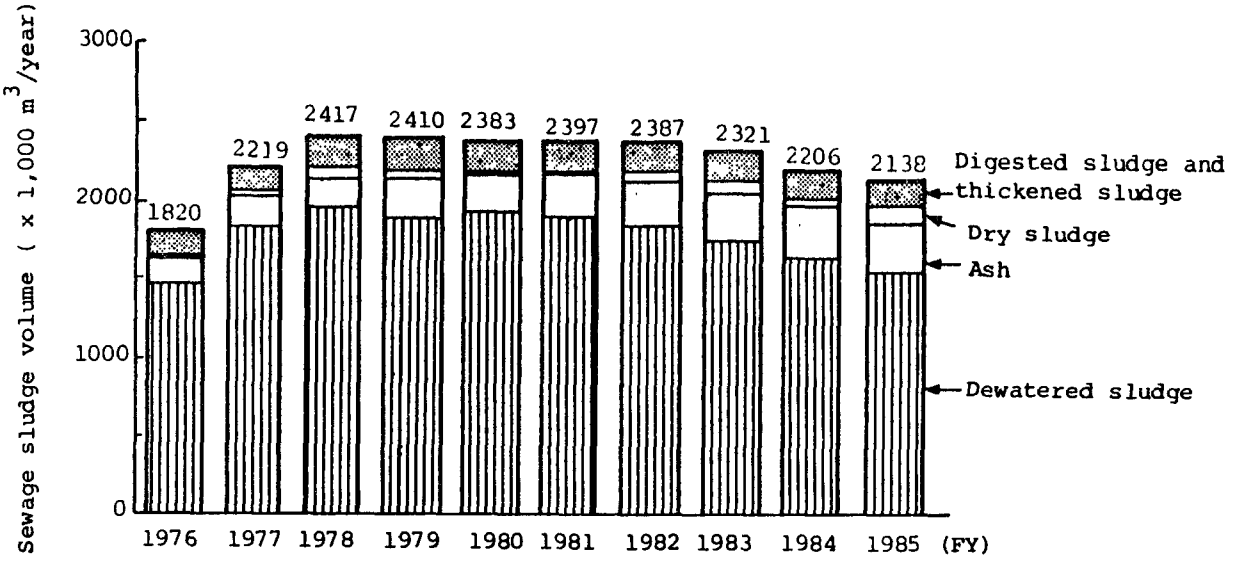


Figure 2. Annual transition of forms in which sewage sludge is disposed of

Table 1 shows how sewage sludge was disposed of in Japan in FY 1985, and Figures 1 and 2 show the annual transition of sewage disposal. From these data, the status quo of sewage sludge disposal in Japan can be summarized as follows:

a. The volume of sewage sludge disposed of in Japan peaked at about 2,400,000 m^3 in 1978 and 1979, and since then has been on a steady decline. In FY 1984, the total disposed volume fell to about 2,100,000 m^3. The decrease in the sewage sludge volume for disposal despite an increase in the sewered population is due to efforts to reduce the sludge volume by incineration.

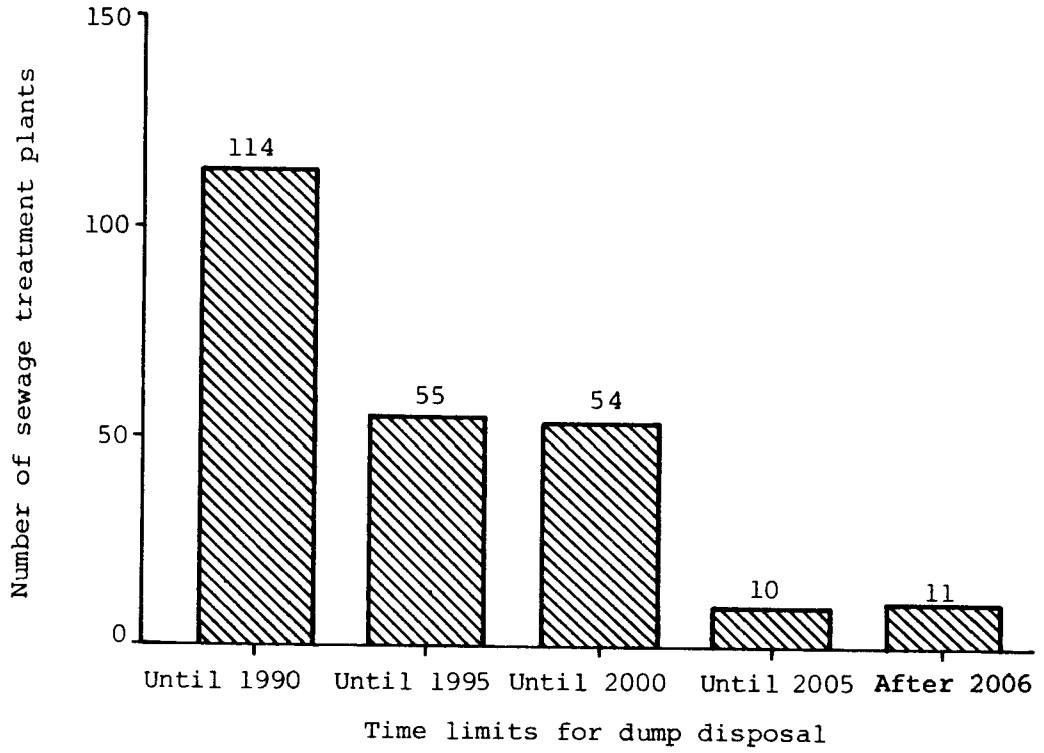
b. Whereas the volume of that part of the sludge which is disposed of as ash is only 13% of the total, it is inferred that more than half the total sludge produced is incinerated to reduce the volume.

c. Landfill and reclamation accounts for 73% of the total sludge volume, and what is used effectively is a mere 15%.

Although the percentage of sewered population still remains at about 37%, the sewer coverage is above 70 or 80% in many metropolises and their satellite cities. But almost every municipality with a high sewer coverage is in a quandary as to how to dispose of sewage sludge. Sewage sludge extracts heavy cost for treatment and disposal, and there are municipalities that have great difficulties in finding dump sites. In fact, a dearth of dump sites is compelling sanitary officers in large cities to implore distant municipalities to provide dump sites. (See Table 2.)

Table 2. Sewage sludge dump sites in greater Tokyo area (number in dumping routes)

Treatment plant (pref.) \ Dump site (Pref.)	A	B	C	D	E	F	G	H	I	J	Sea	Sewage treatment plant	Total
Pref. A	7	5			4	1	6				3	5	31
Pref. B		14			6	3						2	25
Pref. C		7	11		2	2		1	1		1	2	27
Pref. D	1	8		15	1					1	1	6	33
Total	8	34	11	15	13	6	6	1	1	1	5	15	116



(The above is the statistics for 244 sewage treatment plants out of the total 432 plants surveyed from which 188 plants not practicing landfill type disposal or having failed to respond to the questionnaire survey were excluded.)

Figure 3. Estimated availability of dump sites for sewage sludge (including private dump sites)

As the statistics in Figure 3 show, there are many sewage treatment plants for which no definite future can be see because dump sites are not available for their sludge.

3. NECESSITY OF AREAWIDE SEWAGE SLUDGE TREATMENT AND DISPOSAL

As discussed in the foregoing, every city in Japan, particularly in heavily populated areas, is having difficulties in finding sludge disposal, which has prompted the development and implementation of the areawide sewage sludge treatment and disposal dubbed the Ace Plan.

The Ace Plan is a project in which two or more local public bodies jointly treat and dispose of sewage sludge produced at their wastewater treatment plants.

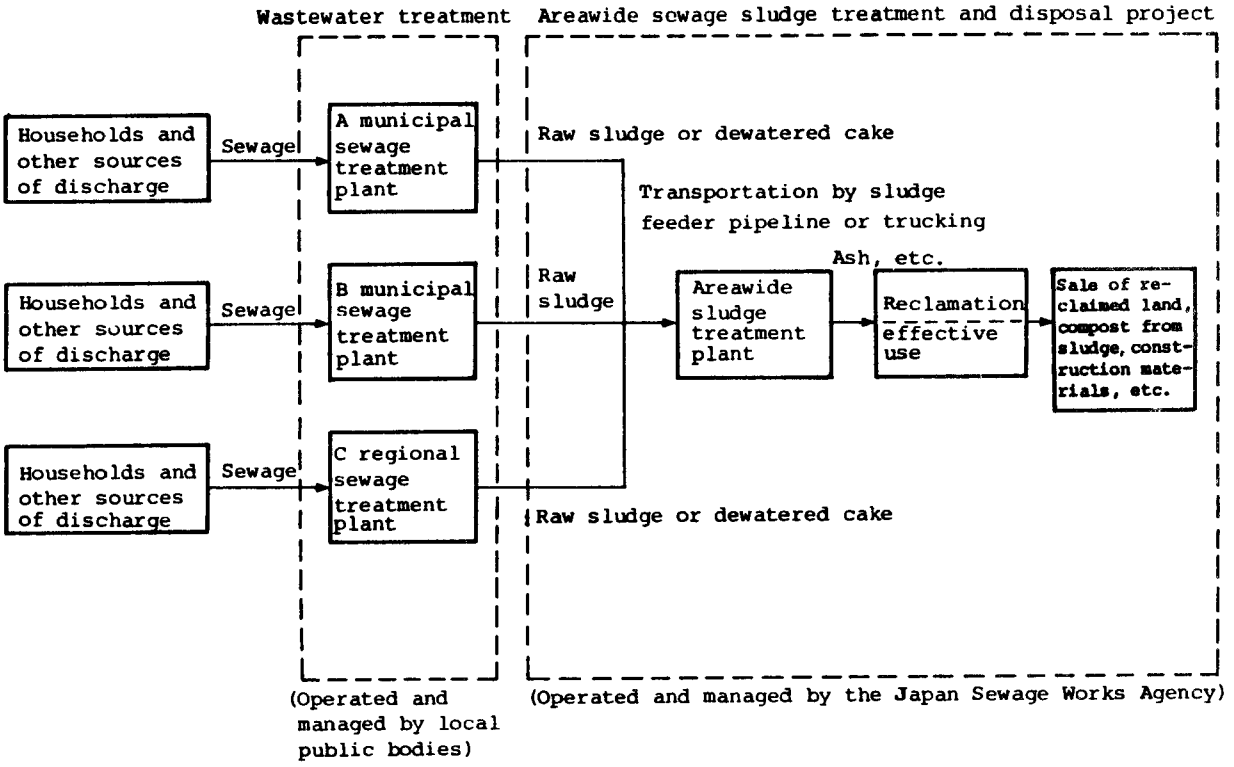


Figure 4. Schematic of Ace Plan

Figure 4 is a schematic representation of the Ace Plan. The reasons justifying the integrated treatment and disposal of sewage sludge on an areawide scale are as summarized below.

a. Necessity of joint incineration
It is of urgent necessity to install incineration plants to reduce the volume of sewage sludge. Nevertheless, there are few people who are receptive to the idea of having an incineration plant near them. In a heavily populated area, it is almost impossible for every municipality to have its own plant within its administrative district. Thus, these municipalities have no choice but to incinerate their sludge jointly at a place where there are no conflicting interests about the installation of an incineration plant.

b. The merit of scale brought about by joint treatment and disposal
The initial investment, and operation and management costs are less if the sewage sludge treatment and disposal are carried out jointly than if carried out separately by individual municipalities. As the expenses for the sewage treatment and disposal are proving difficulties for every municipality, joint treatment and disposal is imperative. In this respect, the Ace Plan provides a viable answer.

c. Acquisition of site for sewage sludge treatment
Existing wastewater treatment plants in metropolises have no room for expansion. Intense urbanization and soaring land prices make it very difficult for individual municipalities to install or expand incinerators and sewage treatment plants independently. Local municipalities in a region are obliged to install, operate, and maintain a joint facility for treatment and disposal of their sewage sludge.

d. Acquisition of dump site
There are few sites in metropolises for sewage sludge treatment, still less for the disposal of sewage sludge. Now a days, few municipalities can find any dump site for sewage sludge.

e. Harnessing of sewage sludge
As long as sewage sludge remains waste, its treatment and disposal is governed by the limiting restraints of the natural environment. If we are to continue the treatment and disposal of sewage sludge without problems, the only solution is to turn it to good account.

This dictates: ① cost reduction through joint processing which offers benefits of scale; and ② improvement of the marketability and competitiveness of products from sludge by intensive inventory control and demand-supply adjustment. Areawide sewage sludge treatment and disposal is indispensable for recycling and harnessing sewage sludge.

At present, the areas where sewage sludge are likely to find uses include agriculture, construction, and energy. Therefore, acronym of agriculture, construction and energy (ace) is used in the alternative name for the areawide sewage sludge treatment and disposal project (the Ace Plan). An ace is usually highest card in a card game, and the chief pitcher in baseball is also called an ace. And our wishes that the areawide sewage sludge treatment and disposal project become the definitive answer to the sanitary problems in this field are reflected in the word "Ace."

4. JAPAN SEWAGE WORKS AGENCY, THE PRIME MOVER OF THE ACE PLAN

The areawide sewage sludge treatment and disposal project presupposes the united efforts of several local municipalities. At the great request of many local municipalities that argued in favor of the Japan Sewage Works Agency being the principal promoter of the Ace Plan, the implementation of which calls for the coordination of local municipalities, harnessing of advanced technologies, and effective use of treasury investments and loans, the Japan Sewage Works Agency was elected to undertake the Ace Plan.

A profile of the Japan Sewage Works Agency is given below by way of reference.

To protect against water pollution problems resulting from the gravitation of people and industries towards the metropolises, the environmental water quality standards were established in 1970. As the development and improvement of sewerage were considered indispensable for meeting the standards, the construction of sewers and sewage treatment plants was pushed forward more vigorously than ever before. As a result, the percentage of the sewered population increased from a mere 16% in 1970 to about 37% in 1986.

Over the years, many municipalities had faced difficulties in the operation and management of their sewage works resulting from a serious dearth of sanitary engineers.

In 1972, the Japan Sewage Works Agency (then called the Sewage Works Center) was established to support such municipalities. Since then, the Japan Sewage Works Agency as a group of expert sanitary engineers has provided local municipalities with various services including the construction of wastewater treatment plants, technological research and development, and technical assistance. The number of wastewater treatment plants constructed the by the Japan Sewage Works Agency during the 11 years from 1975 is 214.

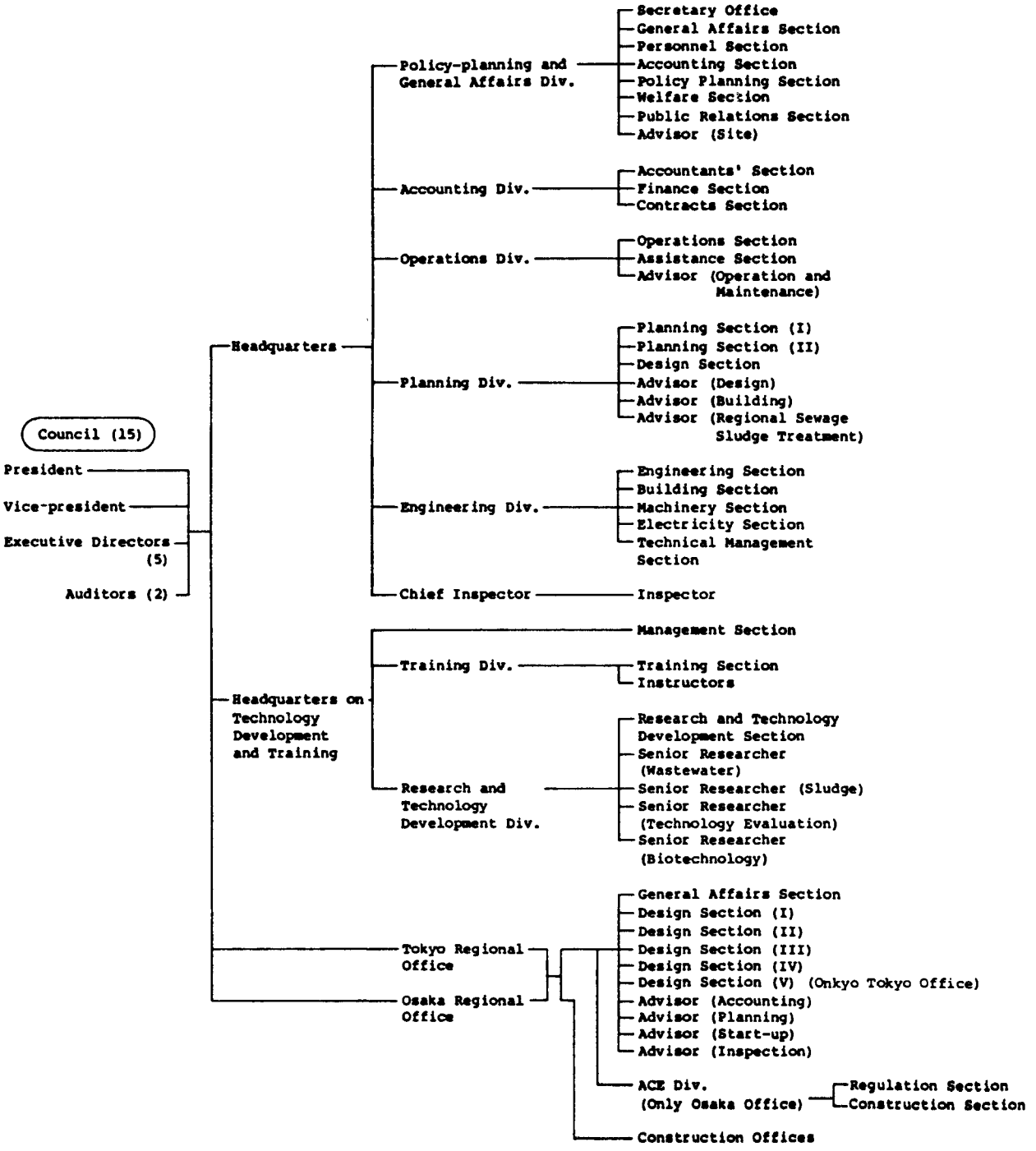


Figure 5. Organizational structure of the Japan Sewage Works Agency

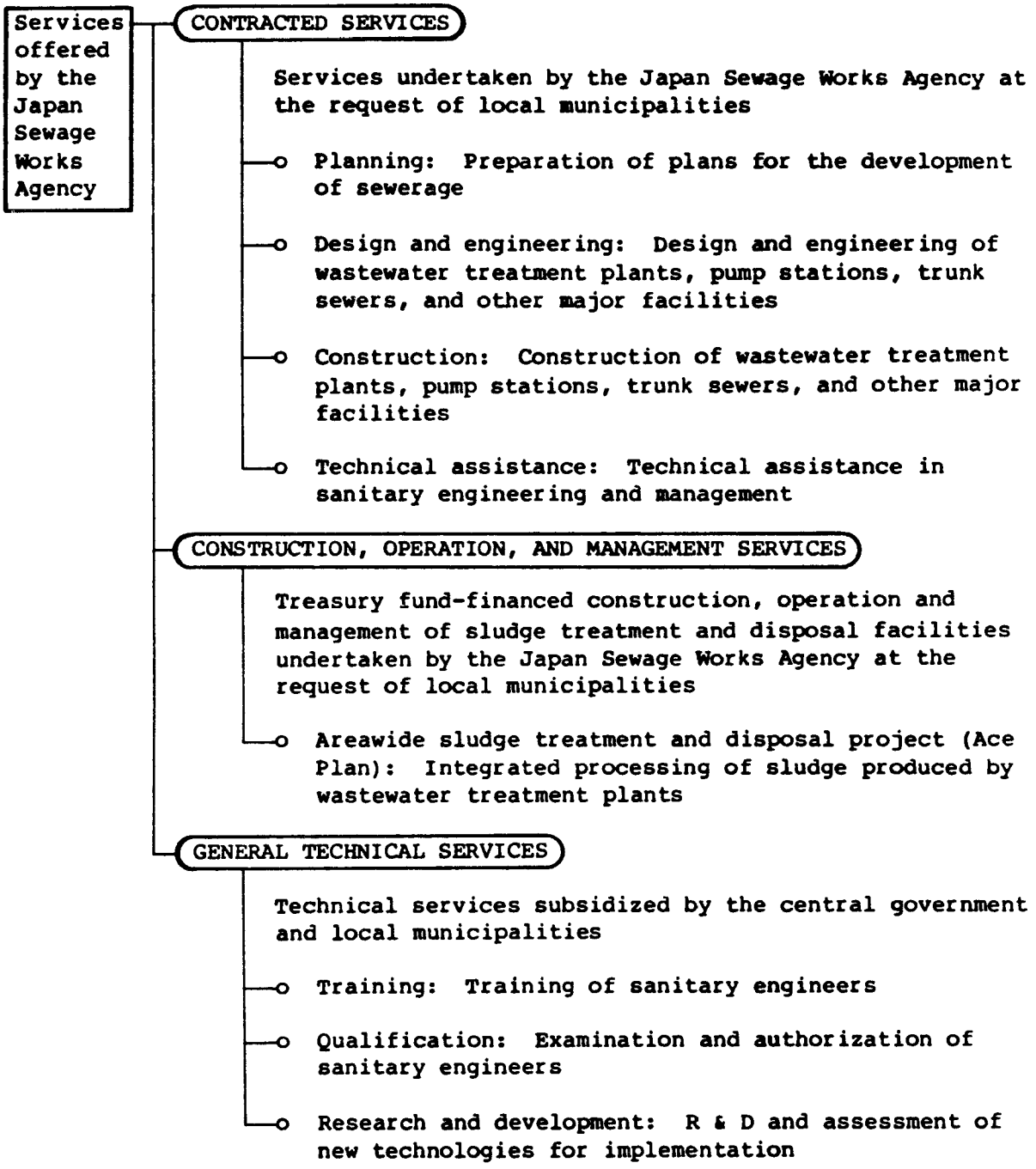


Figure 6. Services offered by the Japan Sewage Works Agency

Figure 5 is an organizational chart of the Japan Sewage Works Agency, and Figure 6 shows the services offered by the agency. The number of employees working for the Japan Sewage Works Agency is about 750 at present.

The benefits that the local municipalities will enjoy from the services of the Japan Sewage Works Agency as the principal promoter of the Ace Plan are itemized below.

a. Promotion of the construction and improvement of sewerage
As the Japan Sewage Works Agency is undertaking troublesome sludge treatment and disposal under the Ace Plan, the local municipalities can merely concentrate their efforts on the construction and improvement of sanitary facilities other than sludge treatment plants, making it possible to build up and improve the sewer network in a streamlined manner.

b. Use of advanced technologies
The Japan Sewage Works Agency is mobilizing all its technology and experience for the construction, operation, and management of facilities to process sewage sludge in an efficient way and to harness sewage sludge for effective use. This means that the local municipalities can take full advantage of the highly advanced sanitary engineering technologies and the very favorable conditions under which such technologies are available, which would be hard to come by if each of the local municipalities had to do the entire job.

c. Alleviation of difficult tasks involved in the coordination of conflicting interests between local municipalities
A joint undertaking between local municipalities often runs aground from the beginning owing to various ramified problems including the assignment of duties and responsibilities and cost sharing. As the principal promoter of the Ace Plan, the Japan Sewage Works Agency can alleviate and solve such problems an efficiently.

5. FUNDS FOR THE ACE PLAN

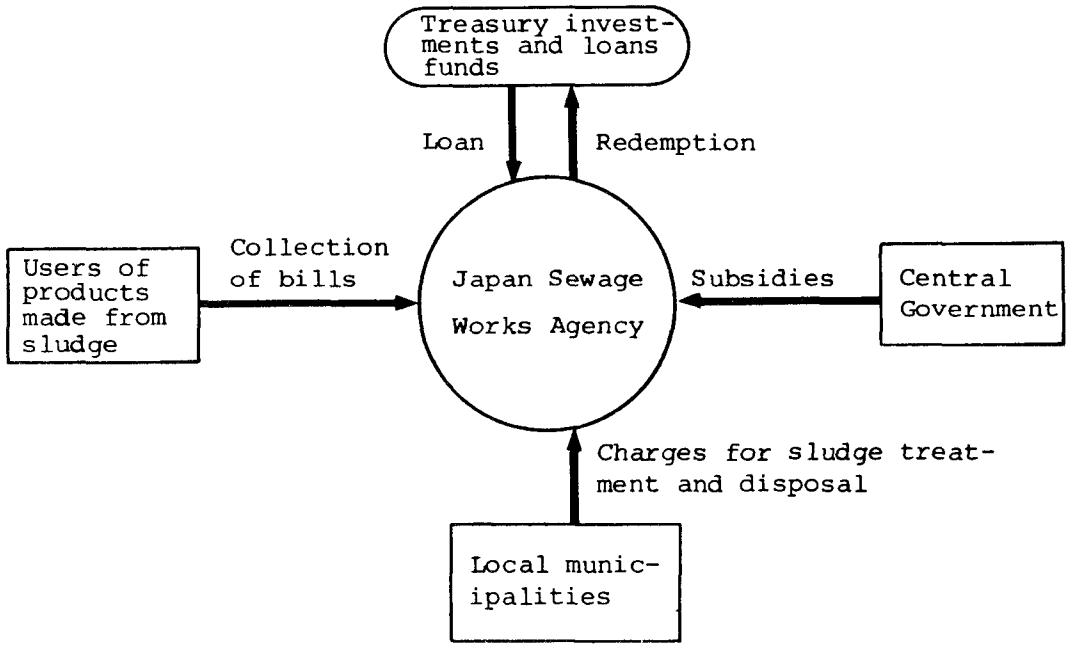


Figure 7. Cash flow in the Ace Plan

Figure 7 shows the cash flow in the Ace Plan. The funds available for the Ace Plan include:

① Treasury investments and loans
② Government subsidies
③ Charges for sewage sludge treatment and disposal
④ Revenues from the sale of construction materials

The Ace Plan has many attractive features, of which the most important is the availability of treasury investments and loans.

The treasury investments and loans are available form the Ministry of Finance using postal savings, pension reserve funds, other Fund Management Bureau's funds, postal life insurance reserve, government bonds, etc., which are kinds of revolving funds which are repaid or redeemed by the interest and principal being collected from the projects and organizations that the funds are invested in or loaned to. These funds are available only for public projects that can pay back the capital with interest from proceeds without fail. The major recipients of treasury funds include the Japan Highway Public Corporation and the Housing and Urban Development Corporation. The funds for treasury investments and loans which were as much as ¥31 trillion (US$200,000 million) as of FY 1987 are called the second national coffers. Indeed, the treasury investment and loan funds are indispensable for projects like the Ace Plan that need the intensive investment of large sums of money. In March 1987, the law concerning the Fund Management Bureau's funds was amended partly due to the appreciation of the yen and partly due to international pressures for the expansion of domestic demand, and the legal limit of the interest rate on treasury loans which had so far been set at 6.0% per annum was lifted. As a result, the interest rate was reduced to 5.2% in March 1987, and was further reduced to 4.6% in May.

In the past, the funds for treasury investments and loans have been used for the underwriting of local bonds for sewage works. The Japan Sewage Works Agency's Ace Plan is the first project that is credited with the direct introduction of funds for treasury investments and loans.

According to the Ace Plan, all the costs and expenses for the construction, operation, and management of the sludge treatment and disposal facilities not covered by government subsidies are defrayed by treasury loans which are to be redeemed in a semiannual equal-installment plan over 25 years (including a grade period of 5 years) from the charges collected from local municipalities for sludge treatment and disposal and from the revenues from the sale of construction materials and other products made from sludge.

6. IMPLEMENTATION OF ACE PLAN

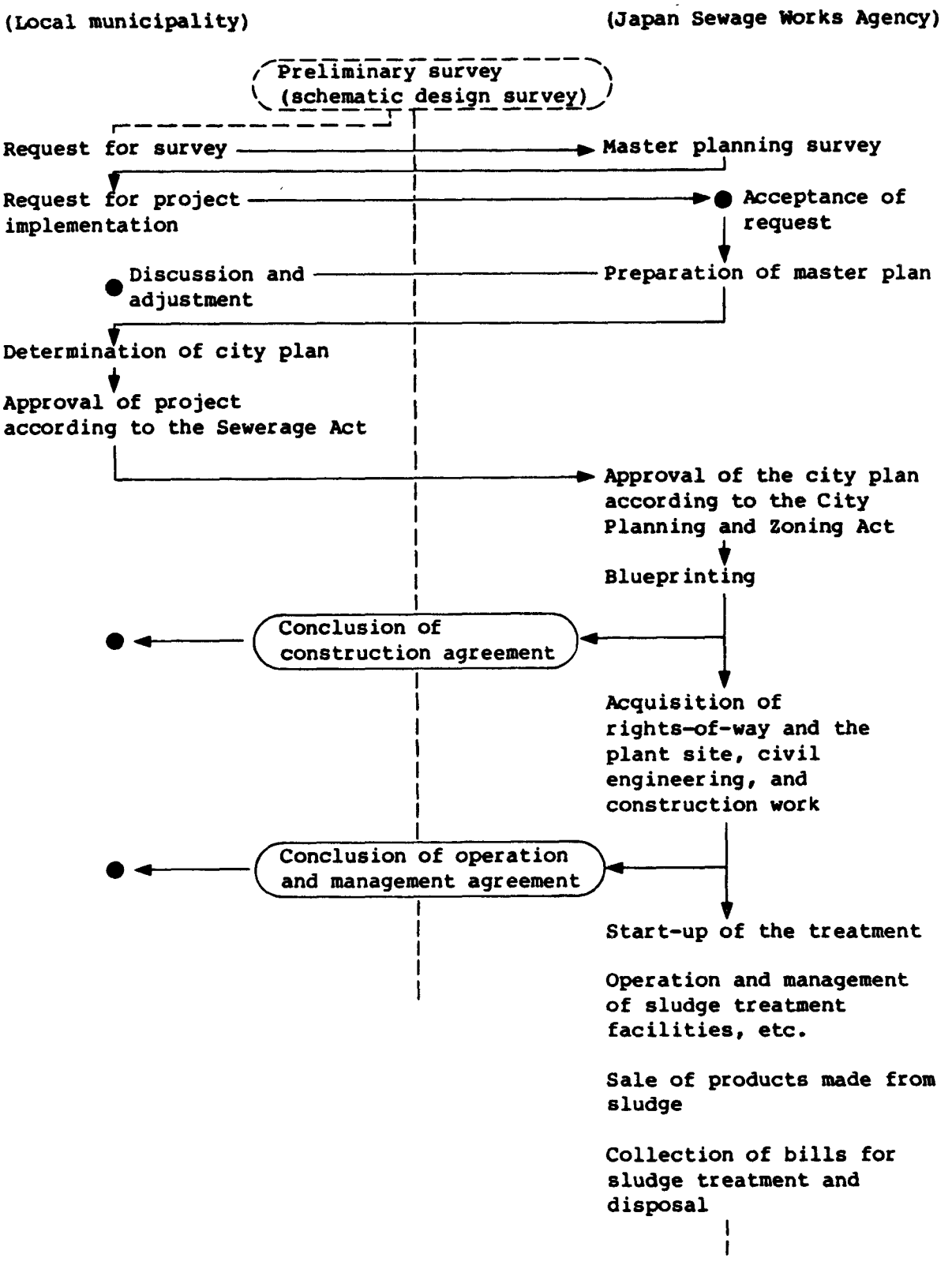


Figure 8. Activity flow in the Ace Plan

Figure 8 shows the activity flow in the Ace Plan. For the area for which the implementation of the Ace Plan is proposed, a preliminary survey (basic schematic design survey) is conducted. According to the findings of the preliminary survey and based on the future projection of sludge production in the proposed area, the treatment processes, disposal methods, sludge recycling methods, facility plan, and financial plan, etc. are studied. This is called the master planning survey. The Japan Sewage Works Agency undertakes the master planning survey at the request of local municipalities. The costs and expenses for the surveys are defrayed by loans from treasury funds, and the loans are repaid from the proceeds to be raised mainly by the commissioning of the sludge treatment and disposal facilities.

After the master planning survey, the Japan Sewage Works Agency undertakes the detailed design, engineering, and construction of the facilities at the request of the local municipalities. The local municipalities need not bear any cost or expenses until the areawide sewage sludge treatmnet and disposal facilities are put into operation. Only after the commissioning of the facilities, need they pay to the Japan Sewage Works Agency the charges according to the tariffs set with respect to the volume and water content of the processed sludge.

The Ace Plan is implemented in stages after a careful study of the situation and wishes of local municipalities, priorities of competing projects, and the economic and technical feasibility of the areawide sewage sludge treatment and disposal projects. In FY 1986, when the Ace Plan was given the go-ahead, the areawide sewage sludge treatment and disposal projects were begun in Eastern Hyogo (Amagasaki City: Mukogawa Regional Sewerage) and Western Hyogo (Himeji City: Ibigawa Regional Sewerage) where the implementation of the projects was sorely needed.

The projects cost ¥5,000 million (US $33 million) in FY 1986, and the budget for FY 1987 has been set at ¥14,000 million (US $93 million). In FY 1987, the Ace Plan will be implemented in Southern Osaka (Minami-Osaka Circum-Bay Regional Sewerage, Sakai City, etc.) and Northeastern Osaka (Regional Sewerage on the left of the Yodo River, Katano City etc.).

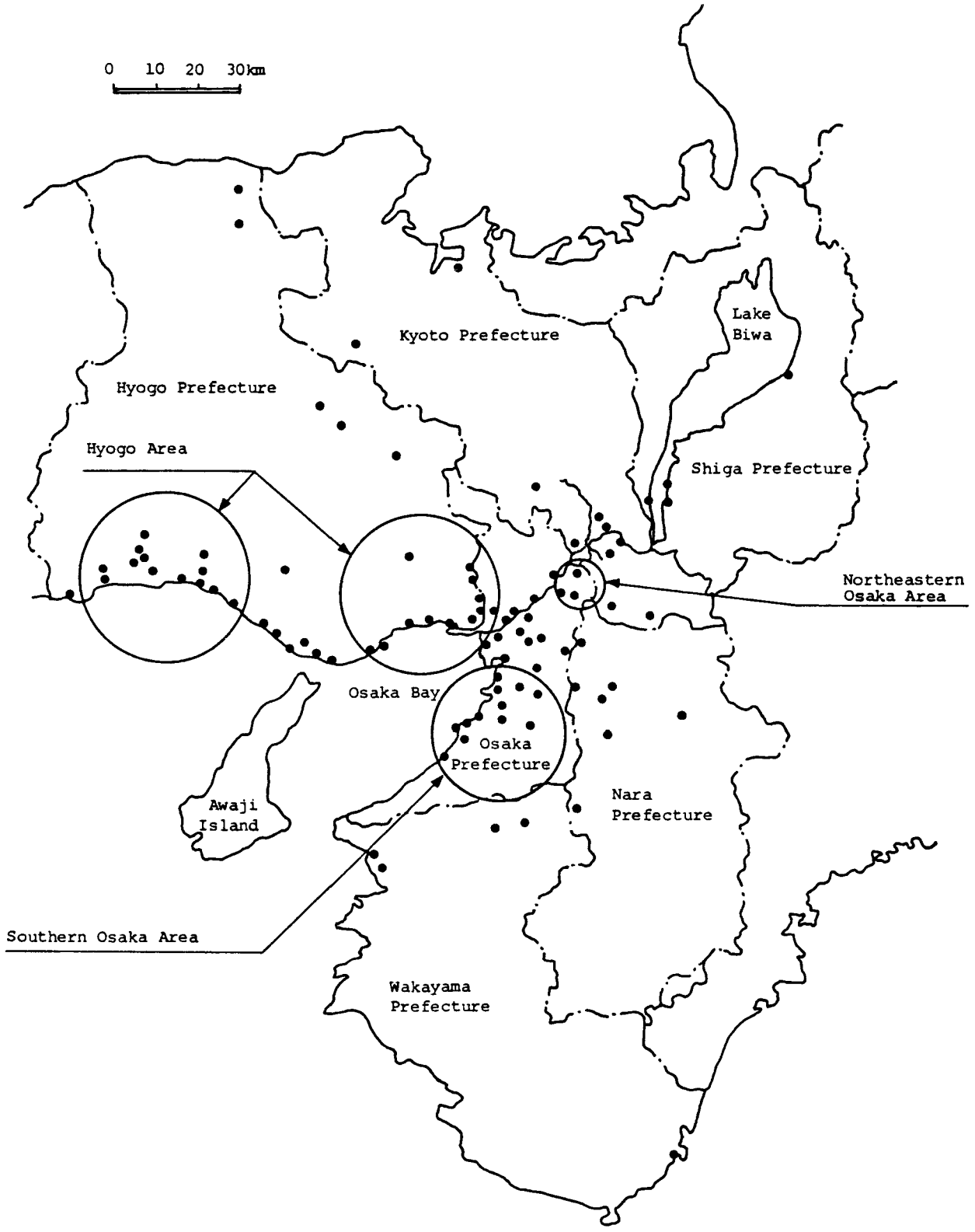


Figure 9. Location of areawide sewage sludge treatment and disposal plants constructed according to the Ace Plan

Figure 9 shows the location of the sites for the above four projects. For these areas, the facilities are planned as follows. (See Table 3.)

Table 3. Completed areawide sewage sludge treatment and disposal projects according to the Ace Plan

(A) HYOGO AREA (excerpts from master plan)

Item	Eastern Hyogo	Western Hyogo
Applicant municipalities (wastewater treatment plants)	Hyogo Prefecture (Muko River Upper and Lower Regional Wastewater Treatment Plants); Amagasaki City (Hokubu and Tobu Wastewater Treatment Plants)	Hyogo Prefecture (Ibo River Basin Regional Wastewater Treatment Plant); Himeji City (Chubu, Seibu, Tobu, Shikama, Takagimae, Fukuimae, Takagigawanishimae, and Shigomae Wastewater Treatment Plants)
Process flow	Sludge receiver → Centrifugal thickener → Centrifugal dewatering machine → Incinerator (Fluidized bed and Pyrolysis furnaces) → Disposal / Recycling and harnessing facilities → Products made from sludge	Sludge receiver → Centrifugal thickener → Centrifugal dewatering machine → Incinerator (Melting furnace) → Disposal / Recycling and harnessing facilities → Products made from sludge
Design raw sludge processing capacity	9,700 m^3/day (water content: 99%)	13,500 m^3/day (water content: 99%)
Disposal method	Effective use and landfill	Effective use and landfill
Project cost (FY 1986 - FY 2005)	approx. ¥37,000 million	approx. ¥58,000 million
Scheduled date for treatment start-up	FY 1989	FY 1989

(B) NORTHEASTERN OSAKA AREA (planned)

Item	Area
Applicant municipalities (wastewater treatment plants)	Osaka Prefecture (Yodo River Lefthand River Basin and Nagisa Regional Wastewater Treatment Plants); Hirakata City (Hokubu and Kori Wastewater Treatment Plants); Katano City (Kozu)
Scheduled date for treatment start-up	FY 1989

(C) SOUTHERN OSAKA AREA (planned)

Item	Area
Applicant municipalities (wastewater treatment plants)	Osaka Prefecture (Wangan Hokubu and Wangan Chubu Regional Wastewater Treatment Plants); Sakai City (Sanpo, Ishizu, Senpoku, and Tsukuno Wastewater Treatment Plants); Izumiohtsu City (Shiomi Wastewater Treatment Plant); Kishiwada City (Isonoue Wastewater Treatment Plant); Senpoku Environmental Facility Development Association (Takaishi Wastewater Treatment Plant)
Scheduled date for treatment start-up	FY 1990

As regards the northeastern Osaka and southern Osaka areas, the process flow and other details will be determined in the master plan if requested by the applicant municipalities.

Table 4. Project costs

Area	FY 1986	FY 1987 (budget)
Hyogo	¥5,000 million	¥11,000 million
Northeastern Osaka	-	¥2,000 million
Southern Osaka	-	¥1,000 million
(incl. treasury funds)	(¥4,700 million)	(¥12,900 million)

Plan for sludge feeder line

The sludge produced at wastewater treatment plants of respective local municipalities is forced to the areawide sludge treatment plant by a series of booster pump stations. Each wastewater treatment plant is installed with a sludge storage tank, sludge booster pump, etc. The feeder lines will be about ten km long, and will be installed with booster pump stations. Where the distance between the wastewater treatment plant and sludge treatment plant is too long (in excess of 20 to 30 km), trucking costs less than pipeline feeding.

Plan for areawide sewage sludge treatment and disposal

The sludge treatment follows thickening, dewatering, and incineration (melting) in turn. At the request of the local municipalities, the basic part of sludge incineration plants (melting furnaces) will be constructed first, and the sludge feeder lines, sludge thickeners, dewatering facilities, etc. will then be constructed in turn. In the first phase of the projects, the incinerators (melting furnaces) will be constructed which will be put in to operation in FY 1989 at all projected sites but in Southern Osaka. For the technical aspects of the sewage sludge melting system to be applied to the four projects, please see the paper contributed to the conference by Mr. Taigo Matsui, Director, Research and Technology Development Division, Japan Sewage Works Agency.

7. CONCLUSION

In my paper presented at the Ninth Conference, I likened sewage sludge to an ugly duckling in the hope that people would come to see that sewage sludge, which is usually thought of with extreme distaste, can be turned into a valuable thing. For details on sewage sludge recycling and harnessing, Mr. Yoshio Ohshima, Chief of the Ultimate Disposal Section, PWRI, Ministry of Construction, has presented an excellent paper which I suggest you read. As can be seen in his paper, research and development for the recycling and harnessing of sewage sludge has been actively pursued, and we at the Japan Sewage Works Agency are pushing forward with trial applications of sludge harnessing ideas and products made from sludge in the field. (See Table 5.)

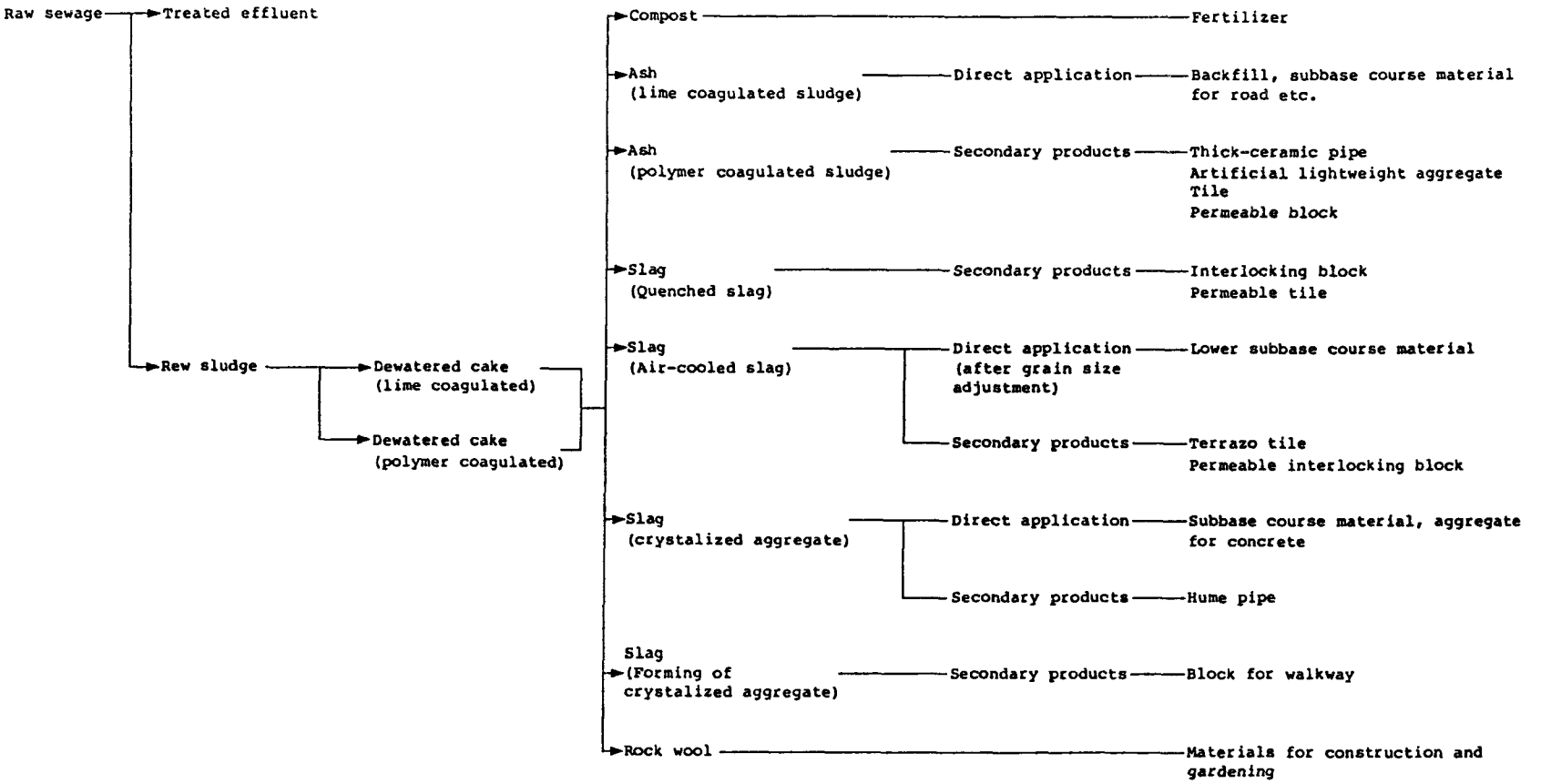


Figure 10. Merchandizable products made from sludge

Table 5. Current and planned use of products made from sludge by the Japan Sewage Works Agency (FY 1986 - FY 1988)

Prefecture	City or Town	Plant	Products made from sludge	Date of use	Applications
Iwate	Shiwa	Shiwa Purification Center	Interlocking block	Aug 1986	Porch approach, approx. 35 m^2
Kanagawa	Kamakura	Shichirigahama Relay Pump Station	Permeable block	Dec 1986	Curb, L = 50 m (3 courses), approx. 500 pcs.
Yamagata	Yonezawa	Yonezawa Purification Control Center	Permeable block	Dec 1986	Yard Walkway, approx. 36 m^2
Kanagawa	Yamato	Hokubu Wastewater Treatment Plant	Sludge tile (50 mm sq. mosaic; small type)	-	Round columns around building, 30 m^2; foot-washing place, 8 m^2
Kanagawa	Hakone	Shimogawara Pump Station; Gora Wastewater Treatment Plant	Sludge tile (small type) Sludge tile (small type)	1986 - 1987	Pump station exterior wall, 100 m^2; administrative office exterior wall, 1,200 m^2
Saitama	Higashi Matsuyama	No. 1 Relay Pump Station	Permeable block	Jan 1987	Porch, approx. 10 m^2
Gifu	Nakatsugawa	Nakatsugawa Municipal Purification Control Center	Sludge title (100 mm sq.)	Jan 1987	Machine building hallway wainscoting, approx. 10 m^2
Ohita	Saeki	Saeki Municipal Wastewater Treatment Plant	Permeable block	Jan 1987	Porch approach, approx. 200 m^2
Miyazaki	Hyuga	Hyuga Municipal Wastewater Treatment Plant	Permeable block	Feb 1987	Garden path, 150 m^2
Kochi	Ino	Ino Municipal Wastewater Treatment Plant	Sludge tile (small type)	Jun 1987	Administrative building-cum-machine room exterior wall, approx. 1.300 m^2
Saitama	Hidaka	Hidaka Purification Center	Sludge tile (floor), 100 mm sq.	Jul 1987	Administrative building, hallway, and other floors, approx. 45 m^2
Nagano	Maruko	Nishiuchi Purification Center	Sludge tile	Jul 1987	Rotary in front of administrative building (exterior), approx. 8 m^2
Akita	Omonogawa Region	Ohmagari Purification Center	Permeable flat plate, permeable block	Aug 1987	Yard walkway, approx. 50 m^2; Porch approach, approx. 20 m^2
Nagano	Matsumoto	Nishijima Purification Center	Sludge tile (floor), 100 mm sq.	Oct 1987	Sludge treatment plant building hallway floor, 70 m^2
Aichi	Misshin	Nisshin Municipal Northern Purification Center	Sludge tile	Dec 1987	Administrative building interior wall, 18 m^2
Toyama	Namerikawa	Namerikawa Municipal Wastewater Treatment Plant	Sludge tile (floor); interlocking block	Mar 1988	Main administrative building floor, approx. 20 m^2
Hyogo Area	Japan Sewage Works Agency	Eastern Hyogo Areawide Sludge Treatment Plant	Sludge tile (wall); sludge tile (floor); interlocking block	1987 - 1988	Administrative building, 300 m^2; administrative building, 50 m^2; administrative building porch approach, 80 m^2
		Western Hyogo Areawide Sludge Treatment Plant	Sludge tile (wall); sludge tile (floor)	1987 - 1988	Administrative building, 300 m^2; administrative building, 50 m^2;

As touched upon earlier in this paper, the Ace Plan integrates the efforts of all the participating local municipalities with the advanced know-how and experience of the Japan Sewage Works Agency to promote the recycling and harnessing of sewage sludge more effectively and efficiently than if they were conducted separately by individual local municipalities. It is strongly hoped that the Ace Plan will become a viable solution to sludge problems, and that it will also awaken the public to the fact that sewage sludge is actually a blessing in disguise.

Autogenous Incineration of Sewage sludge

by

Recovering the Heat of Exhaust Gas.

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

Kenichi OSAKO Yorimitsu YABU

Sewerage Bureau, Tokyo Metropolitan Government, Japan.

Table of Contents

page

1. Request for Energy Saving Measures. 77

2. Energy Saving Measures at Odai Sewage Treatment plant. 78

2–1 Renewal of Fine babble aeration apparatus. 78

2–2 Autogenous Incineration of Sewage sludge. 78

2–2–1 Reduce the moisture content and increase the heating value of dewatered sludge. ... 78

2–2–2 Fit incinerator with the polymer dosed sludge cake. 79

2–2–3 Recovering the heat of exhaust gas of fluidized bed incinerator. 79

3. Recovered Heat in Actual Operation. 81

4. Introduction of Digester Gas Generator. 81

Abstract

Odai sewage treatment plant is the one out of ten plants covering the 23 wards area of Tokyo Metropolitan, and treat the sewage with the capacity of 358,000 m^3/day using activated sludge process.

In regard to sludge treatment the sludge pumped out from Ochiai treatment plant (capacity 450,000 m^3/day) is also digested, mechanically dewatered, and incinerated at this plant.

According to the energy consumption in fiscal 1981, electricity constituted 23% and 77% was fuels for sludge process at Odai treatment plant.

For reduction of energy consumption, renewal of aeration equipment, introduction dissolved air floatation process for thickening, and conversion from vacuum filter to belt filter press for dewatering were applied from 1982. Owing to these measures, reduction of moisture content, and increasing the heating value of polymer cake were accomplished. We are now implementing the conversion of multiple hearth furnaces which need renewal, to fluidized bed furnaces step by step.

Also, we are trying to incinerate sludge without auxiliary fuels by recovering the heat of exhaust gas.

We expect to eliminate heavy oil which occupies 51% of energy consumption. And digester gas equivalent to 26% of total energy will plan to be recovered for electrical use.

1. Request for Energy Saving Measures.

The ward areas are divided into ten drainage areas owing to the topographical features such as rivers and ground elevations.

Under the present master plan for sewerage in the ward areas, 10.4 million peaple will be provided with sewerage system over a projected area of 53,827 ha when completed.

General facts and figures for sewerage planning in the ward areas are shown in Fig.−1 and Table−1.

The ratio of sewered population in the ward areas rose to 85% by the end of fiscal 1986.

The average volume of daily sewage accepted by the ten plants in the 23 wards was about 4.6 million cubic meters in fiscal 1985.

And the average volume of daily raw sludge was about 97,500 cubic meters.

The flow sheet of normal sludge treatment is thickening → digestion → dewatering → incineration → mixing → disposing for landfill.

Mechanically dewatered sludge cake is now mainly disposed to sanitary landfill in Tokyo Bay. But the sites available for landfill will be limitted, so about 70% of 2,785 t/day cake are incinerated to reduce its volume. In fiscal 1985, the average volume of 882 t/day of dewatered sludge, 98 t/day of incinerated ash, and 99 t/day of alumina cement were kneaded and the mixtures were disposed to Tokyo Bay for stable landfill.

After the experiences of world-wide energy crisises in 1974 and 1979, energy-saving has become an important item in sewage work.

Sewerage Bureau set up a project team in order to investigate the present condition of energy consumption and to promote the reduction procedures.

According to the investigations, the energy consumption such as electricity and fuels in sewage treatment, accumulated about 410 kwH for effluent 1,000 m^3 each in fiscal 1978.

Fig.−2 shows the component of total energy consumption in sewage treatment plants and pumping stations in ward area.

Fig.−3 shows the component of total energy consumed at treatment facilities in treatment plants.

Fig.−4 shows the component of electric energy consumed at treatment facilities in treatment plants.

Consequently, reduction of electricity which occupied the 77% of water treatment and reduction of fuels which occupied the 81% of sludge incineration were recognized as urgent problems to be tackled for saving energy in sewage treatment.

2. Energy Saving Measures at Odai Sewage Treatment plant.

Odai treatment plant began in operation in 1962. Fig.−5 shows the layout of facilities and Fig.−6 shows the flow sheet of treatment process.

Capacity of this plant is 358,000 m^3/day and raw sludge pumped out about 11 km from Ochiai treatment plant is also treated at this plant.

Activated sludge process is applied for water treatment. For sludge treatment, thickening digestion-dewatering-incineration is normal flow. Incinerated ash is trucked to mixing plant for reclamation.

Fig.−7,−8,−9 show the energy consumption of plant in fiscal 1981. 23% (14% for water treatment and 9% for sludge) of total energy cosumption was electricity and the rest 77% was fuels for sludge treatment.

Having been operated for more than 20 years, the renewal of treatment facilities were expected.

In 1982, we planned the renewal program and from now on, we are implementing the renewal of treatment facilities. This paper reports about the renewal of aeration equipment for increasing the effect of aeration system and renewal of sludge treatment facilities in order to eliminate auxiliary fuels.

2−1 Renewal of Fine babble aeration apparatus.

In spite of repairing the aeration apparatus from the beginning of operation of aeration tanks, the efficiency had declined gradually as shown in table.−2.

Therefore spiral flow aeration equipment was converted to whole plane placement aeration system in 1984.

Table.−3 shows about 50% increase in aeration efficiency with new appatratus at west-side tanks compared with the old ones at east-side tanks. In 1986, east-side tanks were also equipped with new apparatus.

2−2 Autogenous Incineration of Sewage sludge.

Final disposal of sewage sludge is now reclamation of Tokyo Bay. The sites of reclamation will be limitted, incineration of whole sludge is now scheduled.

In order to reduce the energy consumption for sludge treatment at Odai treatment plant, autogenous incineration is principal focus.

For autogenous incineration, reducing moisture content and increasing heating value of sludge cake will be necessary.

2−2−1 Reduce the moisture content and increase the heating value of dewatered sludge cake.

Till up to now, gravity process has been used for thickening at this plant, but the raw sludge from Ochiai treatment plant was difficult to settle. Furthermore organic content in raw sludge from Ochiai treatment plant has gradually increased. And as shown in Fig.−10, solid content in thickened sludge has declined.

For increasing the efficiency of dewatering machine and digester, improving the thickening process has become important. In 1982, dissolved air floatation process for thickening was introduced.

According to the data in fiscal 1985, the solid content of thickened sludge from Ochiai treatment plant by gravity thickening were 2.0%, on the othere hand mechanical thickening by dissolved air floatation process proved its effect up to 4.1%.

Average moisture content of dewatered sludge cake using a vacuum filter with 10% ferric chloride and 33% lime addition to solid content was 79%.

Adding 43% of inorganic matter for conditioning, and furthermore the moisture content of sludge cake being high, the heating value of dewatered cake resulted arround 2,300 kcal/DS·kg. So auxiliary fuels were needed for incineration.

For reduction of moisture content and increasing the heating value of dewatered cake,

belt filter press with polymer addition was introduced in 1982.

Table.−4 shows the differences of energy consumption and operational characteristics of four types of dewatering machines.

Table.−5,−6 show the characteristics of dewatered sludge cake using belt filter press compared to vacuum filter. The cake dewatered by belt filter press, improved its moisture content to 73% and its heating value to 3,600 kcal/DS·kg.

2−2−2 Fit the incinerator with the polymer dosed sludge cake.

Table.−7 shows the comparison between multiple hearth furnace and fluidized bed furnace in general in case of incinerating polymer dosed sludge cake.

Table.−8 shows the characteristics of exhaust gas in case of incinerating polymer cake at the outlet of furnace, investigated at Kasai treatment plant using fluidized bed furnace (capacity 100 t/ws-day), and at Odai treatment plant using multiple hearth furnace (capacity 180 t/ws-day).

According to the experimental data, fluidized bed furnace is superior to multiple hearth furnace in NOX. CO, and odor concentration.

Main component of odor concentration by fluidized bed furnace is SOX, so it will be easily scrubbed by alkali absorption. Fluidized bed furnace will be superior to multiple hearth furnace in stable operation and in characteristics of exhaust gas.

2−2−3 Recovering the heat of exhaust gas of fluidized bed incinerator.

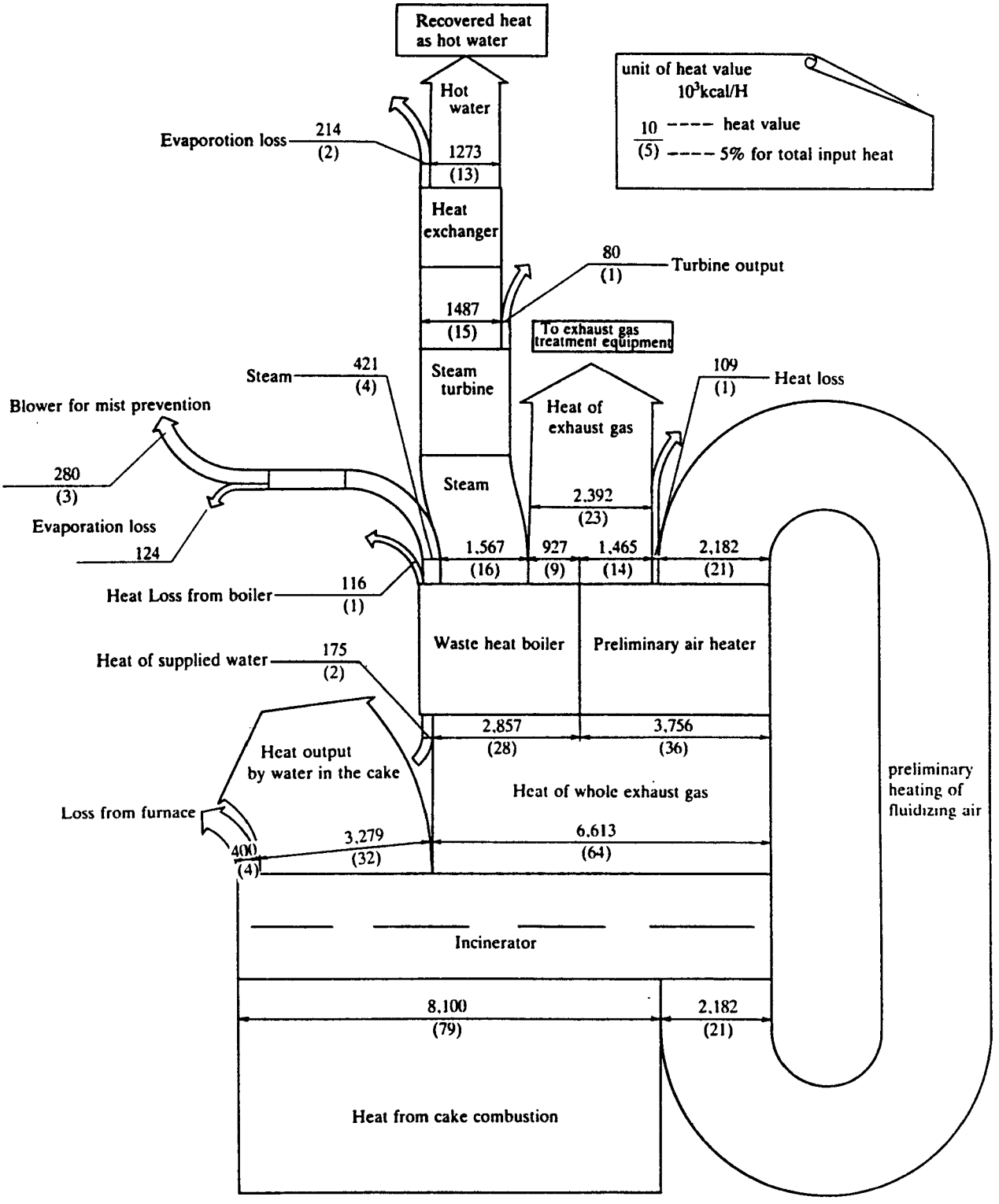
The average moisture content of polymer dosed sludge cake at Odai treatment plant is 73% (organic content 72%), and the heating value is 3,600 kcal/DS·kg.

Autogenous combustion will be accomplished if the fluidizing air were preheated using the recovered heat of exhaust gas.

The next figure shows the energy balance estimated in designing fluidized bed furnace-No 2. 64% of the heat output from the furnace will be available. And among them 23% will be sent to exhaust gas treatment equipment, so the rest 41% will be recovered for use.

21% will be recycled through preliminary heating the air for fluidization in the furnace, and recovered heat as hot water will be 13%.

This recovered heat as hot water will plan to be used for heating digester.


Recovered heat as hot water
unit of heat value
10³kcal/H
10 ---- heat value
(5) ---- 5% for total input heat
Hot water
Evaporotion loss
214
(2)
1273
(13)
Heat exchanger
80
(1)
Turbine output
1487
(15)
To exhaust gas treatment equipment
Steam
421
(4)
Steam turbine
Heat of exhaust gas
109
(1)
Heat loss
Blower for mist prevention
280
(3)
Steam
2,392
(23)
Evaporation loss
124
1,567
(16)
927
(9)
1,465
(14)
2,182
(21)
Heat Loss from boiler
116
(1)
Waste heat boiler
Preliminary air heater
Heat of supplied water
175
(2)
2,857
(28)
3,756
(36)
Heat output by water in the cake
Heat of whole exhaust gas
preliminary heating of fluidizing air
Loss from furnace
400
(4)
3,279
(32)
6,613
(64)
Incinerator
8,100
(79)
2,182
(21)
Heat from cake combustion

3. Recovered Heat in Actual Operation.

Fluidized bed furnace-No−3 (capacity 200 t/ws-day) began operation in april 1986.
Data in actual operation were as follows

Items	Design	Operation
Dewatered cake moisture content	74.0%WB	74.0%WB
〃 Organic content	75.0 % DB	72.0 % DB
〃 Lower heating value	3,800kcal/DS·kg	3,800kcal/DS·kg
Dewatered cake for Incineration	200T/day	193T/day
Temperature of fluidizing air	523°C	550°C
Recovered hot water	92.4m³/H(58°C → 72°C)	80m³/H(60°C → 75°C)
Auxiliary fuel	0 l/T	0 l/T

(11 ~ 26 July 1986)

Auxiliary fuels will not be needed in case of two NO−2, NO−3 fliuidized bed furnaces (capacity 200 t/day × 2) were in operation.

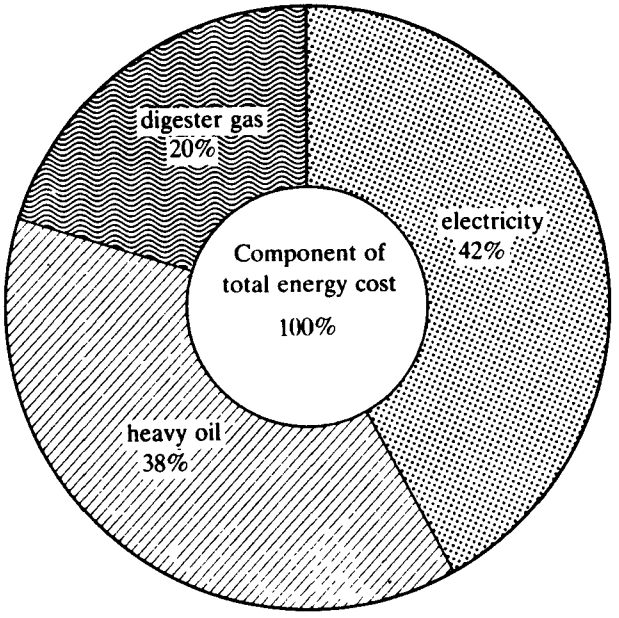
4. Introduction of Digester Gas Generator.

Recovered hot water from the heat of exhaust gas of fluidized bed furnaces will be used for heating digester.

The digester gas having been used as fuel for boiler will be available for other use.

So digester gas generator will plan to be introduced, and generated electricity will gain energy cost at this plant.

Implementing these renewal program at Odai treatment plant, reduction of energy cost will be estimated 38% (component of heavy oil) of total energy cost, based on the following figure (Fiscal 1981), and 20% (component of digester gas) will be recovered as electricity.



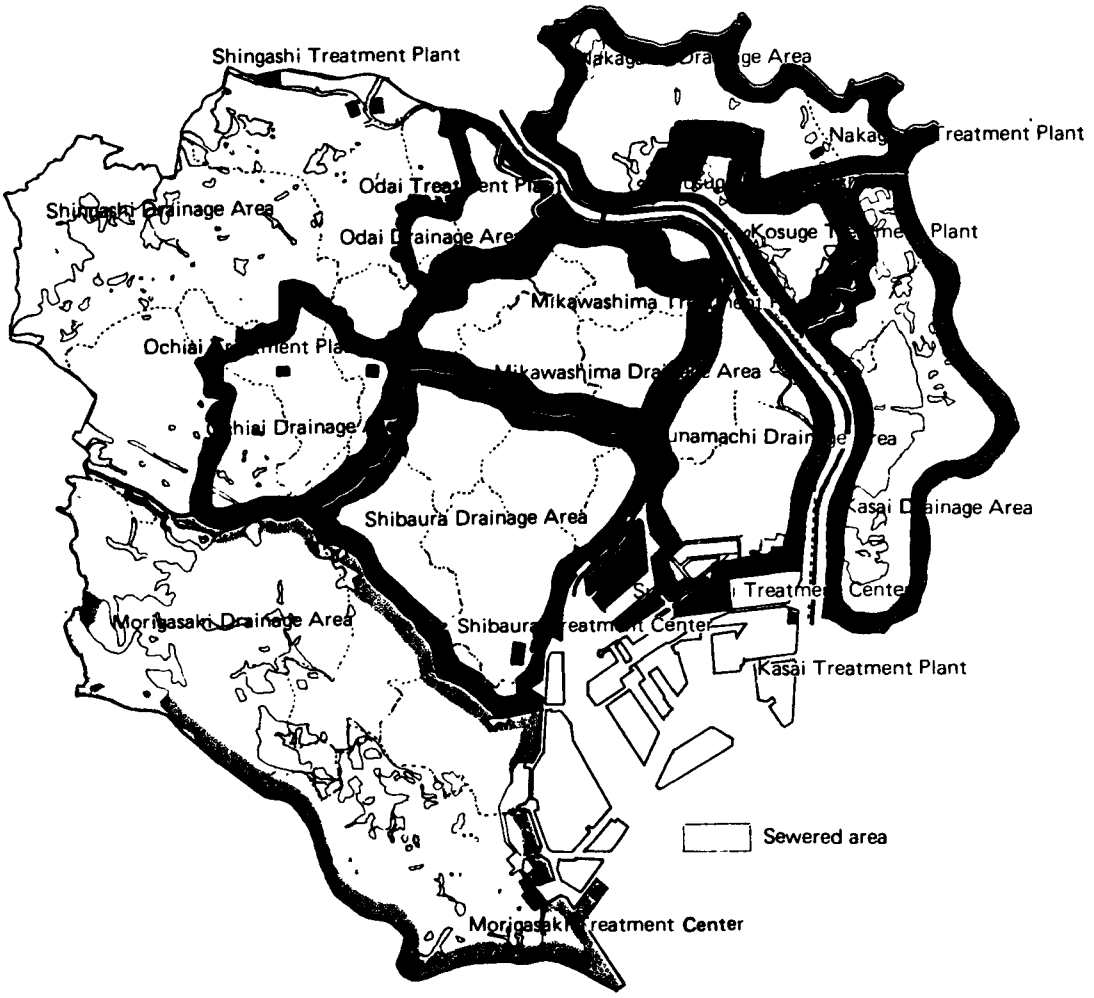


Fig.–1 Total Planning for the Ward Areas' (At end of March 1986) Sewerage System

Table−1. Recent Statistics of Drainage Areas (in the Ward Area)

(At end of March, 1986)

Drainage area		Mikawashima	Sunamachi	Shibaura	Odai	Ochiai	Morigasaki	Shingashi	Kosuge	Kasai	Nakagawa	Total
Sewage treatment Plant	Location	8-25-1 Arakawa Arakawa-ku	3-9-1 Shinsuna Koto-ku	1-2-28 Konan Minato-ku	2-1-14 Miyagi Adachi-ku	1-2-40 Kamiochiai Shinjuku-ku	5-2-25 Omorinnami Ota-ku	3-1-1 Shingashi Itabashi-ku	1-2-1 Kosuge Katsushika-ku	1-1-1 Rinkai-cho Edogawa-ku	5-1-1 Nakagawa Adachi-ku	
	Site (m^2)	195,413.44	827,173.32	177,190.58	112,605.21	66,917.50	(West) 140,500.81 (East) 262,774.43	189,376.28	130,517.48	(North) 166,163.00 (South) 164,979.00	317,425.26	2,751,036.31
	Date of operation	Mar. 1922	Feb. 1930	Mar. 1931	Apr. 1962	Mar. 1964	(West) Apr. 1966 (East) Mar. 1975	Sept. 1974	(West) Jun. 1977 (East) Dec. 1982	Sept. 1981	Apr. 1984	
Treatment capacity (m^3/day)		700,000	680,000	1,130,000	358,000	450,000	1,410,000	705,000	250,000	240,000	75,000	5,998,000
No. of pumping station		10	19	9	3	–	10	2	3	7	2	65
Total length of main sewer lines (m)		69,702	62,959	103,243	23,250	39,010	192,895	169,045	27,079	78,045	61,357	826,585
Diffusion of sewerage	Total population	771,500	729,600	787,300	315,700	642,800	2,035,900	1,600,100	223,000	635,000	584,200	8,325,100
	Sewered population	771,500	688,300	779,900	314,500	638,400	1,663,400	1,388,000	209,300	342,700	154,200	6,950,200
	Sewered rate (%)	100	94	99	99	99	82	87	94	54	26	83
Main facilities	Grit chamber	16	–	9	5	5	28	17	14	12	–	106
	Primary sedimentation tank	20	18	10	6	8	26	7	6	8	2	111
	Aeration tank	16	16	20	10	8	19	18	8	8	4	127
	Secondary sedimentation tank	35	24	30	8	10	38	13	6	8	2	174
Sludge treatment facilities	Sludge thickener	Mixed sludge is pumped through pipelines to the Sunamachi treatment center where the sludge is dewatered, and then disposed by incineration	5	3	6	Mixed sludge is pumped through pipelines to the Odai treatment plant where the sludge is dewatered and then disposed by incineration	2	2	4	3	Mixed sludge is pumped through pipelines to the Kosuge treatment plant where the sludge is dewatered and then disposed by incineration	29
	Sludge digestion tank		12	8	8		4	–	–	–		32
	Dewatering machine		32	12	21		24	6	5	8		108
	Gas holder		1	0	1		1	–	–	–		3
	Sludge incinerator		9 2,250 t/day	–	4 710 t/day		2 600 t/day	3 700 t/day	2 100 t/day	2 250 t/day		22 4,610 t/day

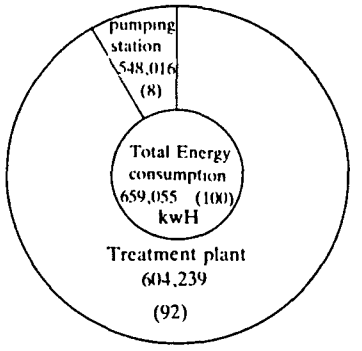


Fig.−2 Total energy consumption for 23 ward area

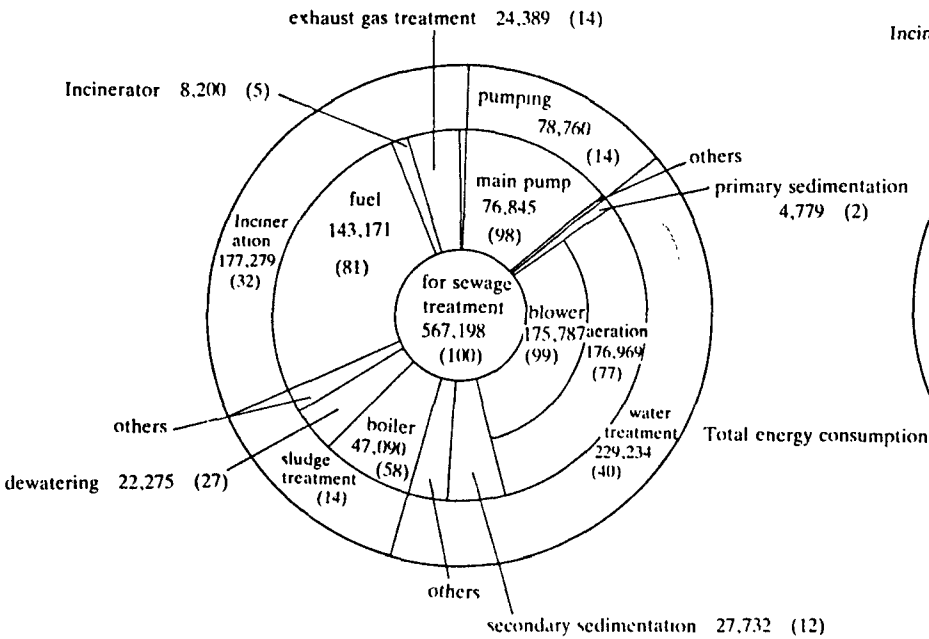


Fig.−3 The component of Treatment facilities to Total energy consumption

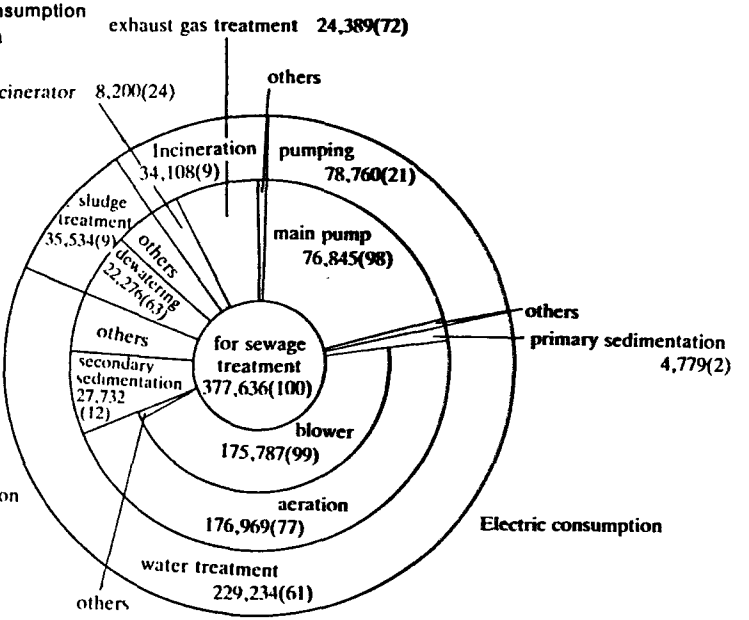


Fig.−4 The component of Treatment facilities to electric consumption

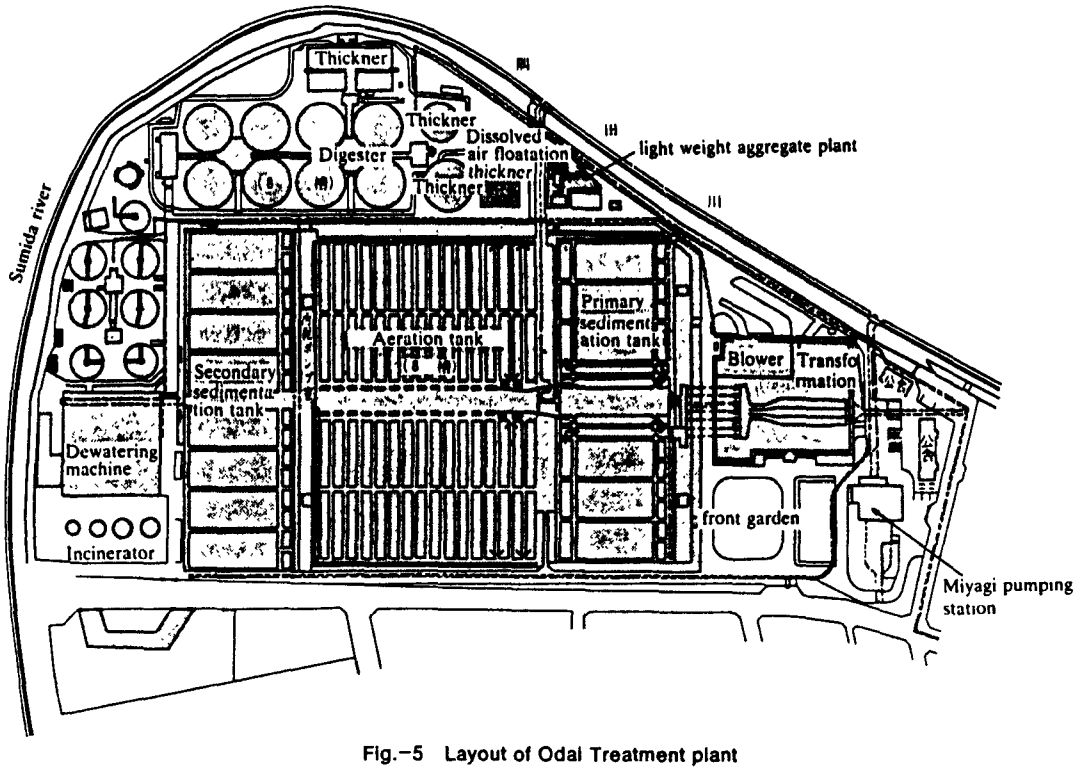


Fig.−5 Layout of Odai Treatment plant

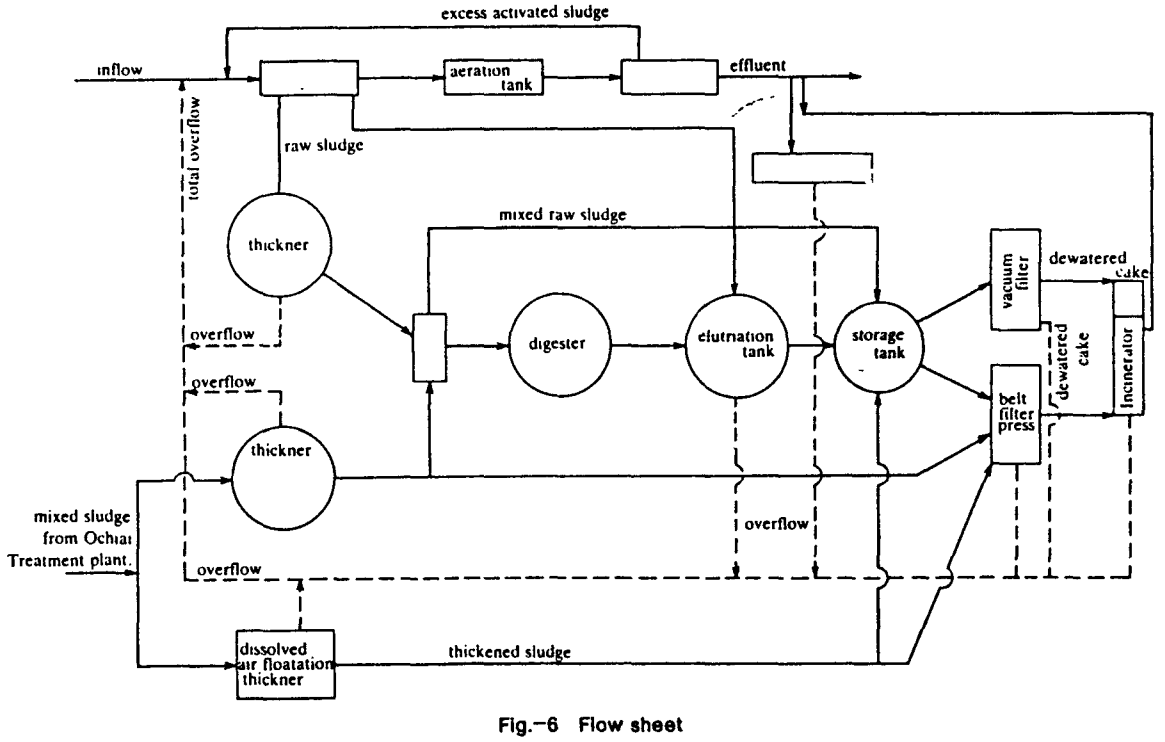


Fig.−6 Flow sheet

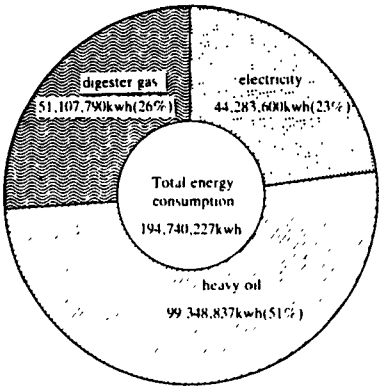


Fig.−7 Total energy consumption at Odai Treatment plant (Fiscal 1981)

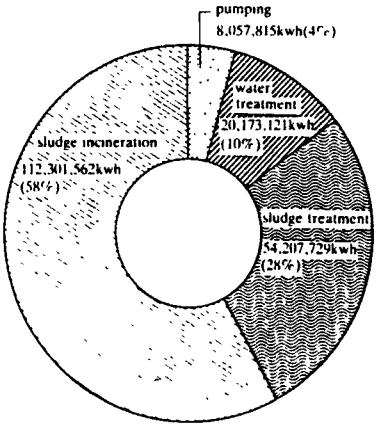


Fig.−8 Component of Treatment facilities to Total energy consumtion

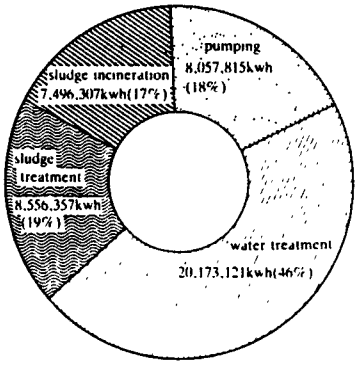


Fig.−9 Component of Treatment facilities to electric energy consumption

Table −2. Decline of aeration efficiency

year	summer									winter								
	sewage treated 1,000m^3/day	aeration ratio for unit sewage Nm^3/m^3	aeration ratio for unit volume tank Nm^3/M^3	DO at outlet of A·T mg/l	SV at outlet of A·T %	BOD load t/day	Removed BOD t/day	BOD−SS load kg/100kg·day	aeration for Removed BOD $Nm^3/kg·BOD$	sewage treated 1,000m^3/day	aeration ratio for unit sewage Nm^3/m^3	aeration ratio for unit volume tank Nm^3/M^3	DO at outlet of A·T mg/l	SV at outlet of A·T %	BOD load t/day	Removed BOD t/day	BOD−SS load kg/100kg day	aeration for Removed BOD $Nm^3/kg·BOD$
1962	11	6.9	0.80							25	4.6	0.49					19	
63	52	1.7	0.25					1.5		44	3.2	0.39					22	
64	66	5.5	0 60		1.0	—		22		64	3.2	0.57		12			16	
65	330	2.9	0.67		10	35	30	36	32	346	2.7	0.66		16	52	44	28	21
66	878	8.1	0.82		14	41	87	23	81	326	8.4	0.77		14	61	55	87	20
67	346	3.7	0.90		13	59	51	82	25	271	3.8	0.72		18	66	53	55	19
68	348	3.7	0.90	4.1	7	40	34	47	38	318	3.4	0 75	5.0	9	47	38	39	28
69	862	3.5	0.88	2.0	8	50	42	53	30	340	3 2	0.78	4.1	9	51	38	58	29
70	396	3.7	1.04	1.9	8	69	52	97	28	360	5.0	1.27	5.0	9	100	79	71	23
71	476	4.0	1.32	3.3	7	71	56	77	84	425	3.9	1.16	4 7	7	51	36	64	46
72	434	4.9	1.51	2.2	9	47	44	45	49	418	5.1	1.49	3.9	14	70	62	68	34
73	460	5.0	1.61	2.1	15	56	51	42	45	412	4.6	1.32	4.3	18	76	67	56	28
74	507	3.7	1.31	3.0	13	51	46	39	40	376	4.5	1.20	5.3	14	49	44	39	39
75	445	4.0	1.27	3.0	14	41	36	42	50	390	4.5	1.25	5.0	16	57	52	45	35
76	434	4.6	1.40	3.3	12	48	43	50	47	366	5.4	1.28	5.8	13	62	58	42	34
77	407	4.6	1.21	3.4	13	39	37	50	50	336	6.3	1.38	4.1	14	4.5	41	34	52
78	850	6.1	1.39	3.3	13	37	34	31	63	301	6.4	1.25	3.9	15	55	51	37	38
79	815	6.3	1.28	2.5	13	32	30	27	66	292	6.4	1.21	4.7	18	37	34	31	55
80	347	5.7	1.28	3.7	12	82	30	36	67	281	7.6	1.38	4.4	14	44	41	85	52
81	324	6.7	1.46	3.1	12	29	27	25	79	274	8.2	1.51	3.5	19	68	60	39	39
82	336	7.1	1.61	2.7	12	30	28	23	84	275	8.6	1.60	4.1	16	69	66	50	36
83	819	7.5	1.62	2.2	12	38	36	50	67	268	9.2	1.66	3.6	15	41	36	36	68

Table − 3. The increase in aeration efficiency with the renewal of aeration apparatus

(Nov. 1984 ~ Dec. 1985)

comparison renewal to old apparatus	Inflow sewage BOD_5	effeuent BOD_5	aeration ratio	power for unit sewage	power for removal BOD_5	BOD−SS load	aeration for removal BOD_5
renewal apparatus west-pide tank	mg/l 112	mg/l 4.6	m^3/m^3 6.9	kwH/m^3 0.15	kwH/kg ·BOD 1.42	kg/kg·day 0.34	m^3/kg·BOD 68
old apparatus east-side tank	112	4.5	10.9	0.21	2.00	0.38	104

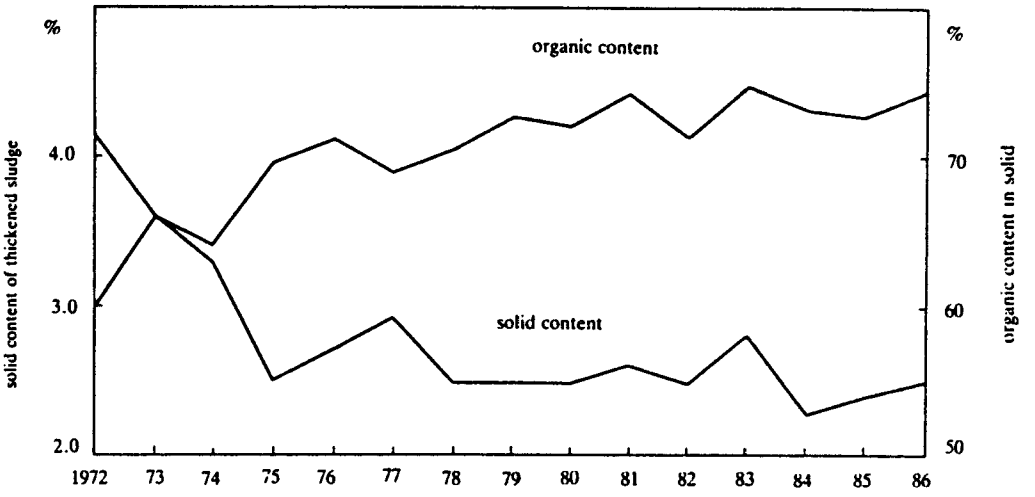


Fig.−10 Characteristics of sludge and thickening of Ochiai Treatment plant.

Table −4. The difference of energy consumption and characteristics of dewatering machines

	moisture content of dewatered sludge cake	capacity		energy consumption		
		unit capacity	capacity of equipped machine			
Belt flilter press	77%	130 SSkg/m·h	390 SSkg/unit h (width 3m)	80 kwh/SS·t	polymer	0.6%
vacuum filter	80%	10 SSkg/m^2·h	320 SSkg/unit·h (32m^2)	170 kwh/SS·t	lime ferric chloride	40% 10%
press filter	67%	35 SSkg/m^2·h	343 SSkg/unit·h (98m^2)	170 Kwh/SS·t	lime ferric chloride	35% 8%
centrifuge	79%	—	(700 SSkg/unit·h) (1.2 T/unit·h)	120 kwh/SS·t)	polymer	0.9%

sludge condition	solid content	organic cotent in solid
	2.5 ~ 3.0%	60 ~ 65%

Table −5. Characteristics of dewatered sludge cake (Belt filter press)

Run		unit	1984 11 15	1985 1 7	1 25	2 21	4 22	5 21	6 18	7 26	8 19	9 12	10 18	1985 11 13	average	variation range
Moisture content		%WB	75 3	75 9	73 6	76 2	71 6	69 4	71.7	71 1	73 1	73 5	72 0	72 8	73 0	69 4~76 2
Organic content		%DB	71 3			75 0	※1 70 3	67 3	71 6	64 6	※1 72 3	※1 71 3	※1 72.4	73 6	71.0	69.6~75 0
Inorganic content			28 7			25 0	29 7	32 7	28 4	35 4	27 7	28 7	27 6	26 4	29 0	25.0~32.7
Higher heating value per dry solid		kcal/kg ds	3,830	4,260	4,480	4,050	3,840	3,530	4,220	3,720	3,950	3,970	3,880	3,960		
Element composition	C	%DB	33 7			40 5		35 7			38 9			39 2		
	H		6 01			6 1		6 0			7 6			6 2		
	N		4 20			5 3		4 3			4 9			4 94		
	S		0 97			0 63		0 59			0 81			0 79		
	O		26 42			22 47		20 71			20 09			22 47		
lower heating value per dry solid		kcal/kg ds	3,505	※2 3,936	※2 4,156	3,721	※2 3,516	3,206	※2 3,896	※2 3,376	3,540	※2 3,646	※2 3,556	3,625	3,642	3,206~4,156
lower heating value per ignition loss		kcal/kg ds	4,916			4,961		4,764			4 896			4,925	4,892	

※1 Organic content = 100− Inorganic content

2. (lower heating value per dry solid) = (Higher heating value per solid) − $600\times9\left(\frac{\text{H — element in dry solid}}{100}\right)$

3 (lower heating value per ignition loss) = (lower heating value per solid) × $\left(\frac{100}{\text{Organic content}}\right)$

Table −6. Characteristics of dewatered sludge cake (Vacuum filter)

Run		unit	1984 11 15	1985 1 7	1.25	2 21	4 22	5 21	6 18	7 26	8 19	9 12	10 18	1985 11 13	average	variation range
Moisture content		% WB	80 8	83 8	82 8	83 6	85 6	81 4	82 3	75 3	76 0	76 8	79 9	78 8	80 6	75~86
Organic content		%DB	61 0			55 6		53 9	※1 57 0		※1 47 7	※1 48 0	※1 61 0	※1 53 4	54 7	48~61
Inorganic content			39 0			44 4		46 1	43 0		52 3	52 0	39 0	46 6	45 3	39~52
Higher heating value per dry solid		kcal/kg ds	2,510	3,050	2,990	2,320	2,630	2,700	2,790	1,880	2,030	2,530	3,190	2,330		
Element composition	C	%DB	25 1			23 5		26 9			24 4			24 3		
	H		4 51			4 6		5 3			4 9			4 4		
	N		2 94			2 8		2 9			3 3			2 64		
	S		0.24			0 01		0 08			0			0 04		
	O		28 21			24 69		18 72			15 1			22 02		
lower heating value per dry solid		kcal/kg ds	2,266	2,780	2,720	2,072	2,360	2,414	2,520	1,610	1,765	2,260	2,920	2,092	2,310	1,600~2,900
lower heating value per ignition loss		kcal/kg ds	3,715			3,727		4,479			3,700			3,918	3,910	

Table −7. Incinerator characteristics of polymer dosed sludge cake.

system / comparison items	Multiple hearth furnace	Fluidized bed furnace
Feature	In order to prevent the clinkers at drying and combusting, the exhaust gas circulating system is employed. The average combustion temperature is lowered and sufficient amount of up-coming heat gas is assured, and the incineration which is as reliable as the conventional system is made possible.	Since the dewatered sludge cake and the high-temperature air are stirred and mixed violently in the fluidized sand layer, drying and incineration of sludge can be carried out quickly and more completely.
Odor	The temperature of incinerating hearth is as high as 750°C to 850°C, but the exhaust gas coming up through drying hearth is around 250°C ~ 350°C, so odorous component remains in the gas.	Because the temperature inside the fluidized-bed furnace is high at 800°C ~ 850°C and all odorous organic substances are decomposed by heat, there is no need for the after-burner for deodorization.
Dust in exhaust gas	2 ~ 10 g/Nm^3	20 ~ 50 g/Nm^3
Nox	150 ~ 250 ppm	under 50 ppm
Sox	500 ~ 1,000 ppm	500 ~ 1,000 ppm
Co	under 500 ppm	under 100 ppm
Excessive amount of air	1.5 ~ 2.5 times of the thoretical volume required	1.3 ~ 1.5 times of the thoretical volume required
Exhaust gas treatment equipment	The exhaust gas treatment equipment will become larger.	The exhaust gas treatment equipment can be made compact.
Running operation	Easy running operation. In principle, 24 hr operation is preferable. It will take about 4hr for re-starting and needs fuel.	Easy running operation. Since the thermal capacity of the fluidized-bed is large, the temperature drop after stopping the furnace is small. Re-starting is smooth, and the intermitten operation can be performed easily and efficiently.
Noise	Main noise is exhaust gas fun.	Main noise is fluidizing blower and exhaust gas fun.
Maintenance	Regular check-up should be done every six or twelve months. In every 3 ~ 5 years, bricks of hearth, and rabble arms need to be exchanged.	Regular check-up should be done every six or twelve months. Twice a month, sand layer need to be supplemented.

Table −8. Characteristics of exhaust gas of furnace (before treatment).

	Fluidized bed furnace (Kasai Treatment plant 100 t/day)	Multiple hearth furnace (Odai Treatment plant 180 t/day)	Remark
Ratio of air per ignition loss	1.3 ~ 1.5	1.5 ~ 2.5	
Dust concentration	20 ~ 50 g/Nm^3	2 ~ 10 g/Nm^3	
NOX	~ 50 ppm(polymer cake)	150 ~ 250 ppm	
	200 ~ 500 ppm(lime cake)		
SOX	500 ~ 1,000 ppm	500 ~ 1,000 ppm	polymer cake
	200 ~ 500 ppm	200 ~ 500 ppm	lime cake
CO	~ 100 ppm	1,000 ~ 5,000 ppm (polymer cake)	
		~ 500 ppm (lime cake)	
Odor concentration	500 ~ 1,000 (SOX odor)	5,000 ~ 10,000 (polymer cake)	
		1,000~5,00 (lime cake)	

BENEFICIAL UTILIZATION OF SEWAGE SLUDGE

by

Yoshio Ohshima

Chief, Ultimate Disposal Section
Water Quality Control Division
Public Works Research Institute
Ministry of Construction

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

Prepared for Presentation at:
11th United States/Japan Conference
on
Sewage Treatment Technology

October, 1987
Tokyo, Japan

ABSTRACT

With the increase of sewage served population, the volume of sewage sludge also increases. Meanwhile, securing space for disposing of sludge produced is each year becoming increasingly difficult.

As a breakthrough in this situation, the reduction of sludge volume and the effective use of sludge are being conducted at the same time.

Incineration and melting are processes now being practiced to reduce the volume of sludge. Then, the effective use of sludge consists not only of using dewatered sludge and compost for greens and farms, as has been the practice from the past, but also includes many new applications which are being tested.

In this report, sludge is divided into inorganic and organic matters and ways of effective utilization taking advantage of the traits of each are discussed and, finally, the present situation and the future prospects of the effective use of sludge are stated.

What is referred to here as inorganic matter comprises incinerated ash and melted slag while organic matter comprises the biochemically analyzed items of organic matter contained in sludge. Regarding inorganic matter, the report shows its chemical composition and compositional change and the results of various surveys aimed to make effective use of it. As for organic matter, the report states how its chemical composition differs between mixed sludge, excess sludge and digested sludge and cites how to make ethyl alcohol from mixed sludge as an example of its effective use.

C O N T E N T S

1. INTRODUCTION 96

2. PRESENT SITUATION OF TREATMENT, DISPOSAL AND EFFECTIVE USE OF SEWAGE SLUDGE 96

2-1 Present Situation of Treatment 96

2-2 Present Situation of Disposal 96

2-3 Present Situation of Effective Utilization 99

3. EFFECTIVE USE OF INORGANIC COMPONENTS 101

3-1 Inorganic Components 101

3-2 Effective Use of Incinerated Components 103

4. EFFECTIVE USE OF ORGANIC COMPONENTS 107

4-1 Organic Components 107

4-2 Effective Use of Organic Components 115

5. CONCLUSION 119

1. INTRODUCTION

With the dissemination of the ratio of sewered population, the volume of treated wastewater and the volume of sewage sludge increases. An important problem in sewerage system is how to make effective use of these products. But at present, the proportion of effectively used sewage sludge is only ten-odd percent and the rest in disposed of by land reclamation, etc.

There are many obstacles in effectively using sewage sludge. For example, its use for greens and farms is largely restricted by such factors as the control of heavy metals in sewage sludge and soil.

This study concerns; (1) the present situation of effective utilization and its limitations, (2) detailed research into the components of sewage sludge and (3) the possibilities of effective use taking advantage of the characteristic components of sewage sludge.

2. PRESENT SITUATION OF TREATMENT, DISPOSAL AND EFFECTIVE USE OF SEWAGE SLUDGE

2-1 Present Situation of Treatment

Table 1 summarizes the present situation of Japanese sludge treatment from the Sewerage Statistics, fiscal 1984 edition, and the following can be pointed out:

a. The amount of sludge produced is 1,140,000 t/year in terms of dry solids in thickened sludge.

b. Of total sludge produced, 54% is incinerated and 40% is treated up to the stage of dewatering.

c. Of all stabilizing processes, anaerobic digestion is the most common and 38% of all sludge produced is treated anaerobically.

2-2 Present Situation of Disposal

The situation of Japanese sludge disposal in fiscal 1984 is shown in Tables 2 and 3 and its yearly changes are shown in Figs. 1 and 2. The present situation of sludge disposal may be extracted from these graphics as follows:

a. The volume of sludge that is disposed yearly in Japan decreased after a peak of about 2,400,000 m^3 in 1977 to about 2,230,000 m^3 in 1984. The reason why sludge disposal decreased in spite of the dissemination of the ratio of sewered population is because sludge reduction progressed by such means as incineration.

b. About half of the sludge produced was incinerated and, as sludge decreased through incineration, about 71% of sludge disposed of accounted for dewatered sludge.

Table 1 Present situation of sludge treatment in Japan

(1984 Sewerage statictics)

Ultimate state of sludge	Sludge treatment process	Dry solids based on thickened sludge (10^3 t/year)	(%)
Liquid	Withdrawal	0.6	0.05
	Thickening	0.6	0.05
	Thickening—Anaerobic digestion	12.5	1.11
	Total	13.8	1.21
Dewatered	Thickening—Dewatering	159.5	13.96
	Thickening—Anaerobic digestion—Dewatering	268.2	23.47
	Thickening—Aerobic digestion—Dewatering	2.5	0.22
	Thickening—Wet air oxidation—Dewatering	5.5	0.48
	Thickening—Heat treatment—Dewatering	17.4	1.52
	Total	453.1	39.65
Dried	Thickening—Drying	0.2	0.01
	Thickening—Anaerobic digestion—Drying	0.3	0.03
	Thickening—Dewatering—Drying	10.4	0.91
	Thickening—Anaerobic digestion—Dewatering—Drying	15.6	1.37
	Total	26.5	2.32
Composted	Thickening—Dewatering—Composting	10.1	0.88
	Thickening—Anaerobic digestion—Dewatering—Composting	9.7	0.84
	Total	19.8	1.72
Incinerated	Thickening—Dewatering—Incineration	445.3	39.98
	Thickening—Anaerobic digestion—Dewatering—Incineration	133.1	11.65
	Thickening—Aerobic digestion—Dewatering—Incineration	4.1	0.36
	Thickening—Heat treatment—Dewatering—Incineration	36.7	3.21
	Total	619.3	54.20
	Total	1,132.5	99.10
	Unknown	10.1	0.90
	Total (all)	1,142.6	100

Table 2 Outline of sludge disposal

(1984 4.1∿1985 3.31) (Unit: 1,000 m³/year)

Form of sludge \ Way of disposal	Land fill	Coastal reclamation	Utilization	Others	Total (%)
Dewatered sludge	970	430	177	10	1,587 (71)
Incinerated ash	142	140	34	0	316 (14)
Dried sludge	8	0	139	0	147 (7)
Digested or thickened sludge	1	0	0	178	179 (8)
Total (%)	1,121 (50)	570 (26)	350 (16)	188 (8)	2,229 (100)

Table 3 Outline of sludge utilization

(1984 4.1∿1985 3.31) (Unit: 1,000 m³/year)

Classification \ Sludge condition		Dewatered sludge	Ash	Dried sludge	Compost	Digested sludge	Total
Utilization in agriculture and parks	By municipalities	170	0	5	87	0	262
	Selling to fertilizer companies	7	3	1	46	0	57
	Sub-total	177	3	6	133	0	319
Utilization as a construction material		0	31	0	0	0	31
Total		177	34	6	133	0	350

c. By methods of disposal, 76% of sludge was used for reclamation on land and at sea. Only 16% was put to effective use. 31,000 m^3 were used as construction materials and this represents 1.4% of the total volume of sludge disposed. (Table 3)

2-3 Present Situation of Effective Utilization

The effective utilization of sewage sludge may be generally divided into (1) utilization as a soil conditioner and fertilizer for greens and farms and (2) utilization as construction materials because of inorganic matter contained in sewage sludge.

The former use of sludge is mainly in the forms of dewatered sludge and compost while the latter is presently used mostly in the form of incinerated ash but also as artificial light-weight aggregate, though used only in small quantities.

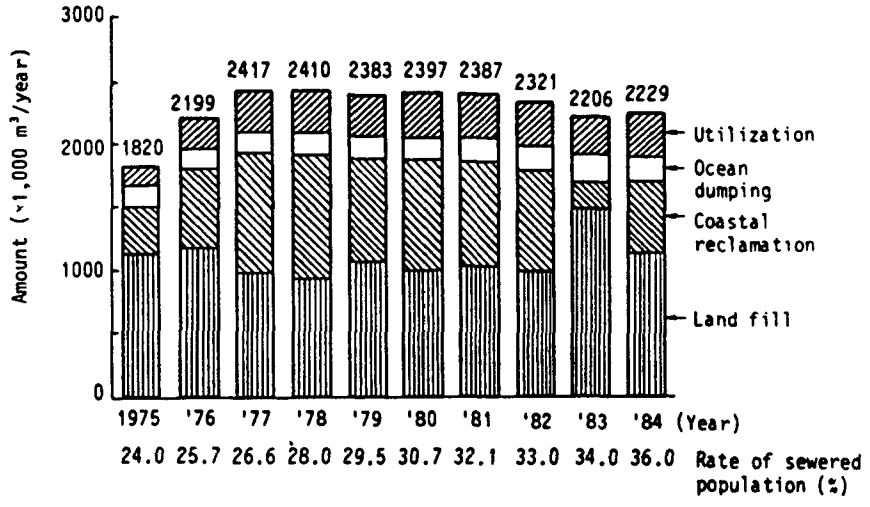


Fig. 1 Sludge disposal methods

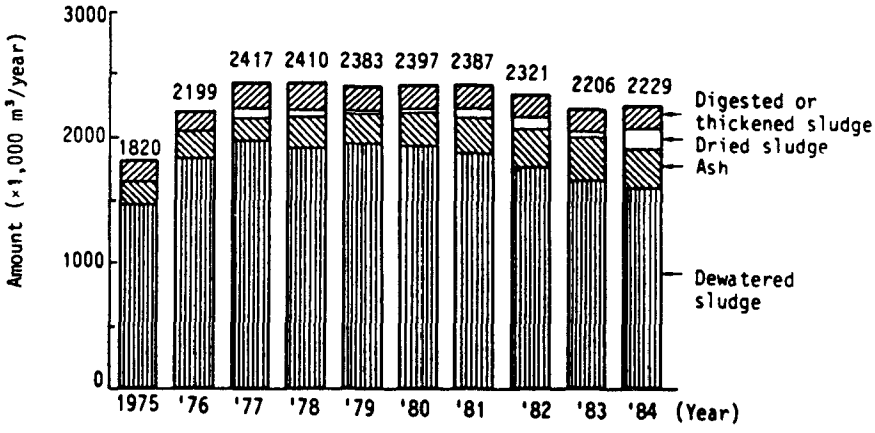


Fig. 2 Form of disposal sludge

The effective-use applications of sewage sludge as resource are shown in Fig. 3.

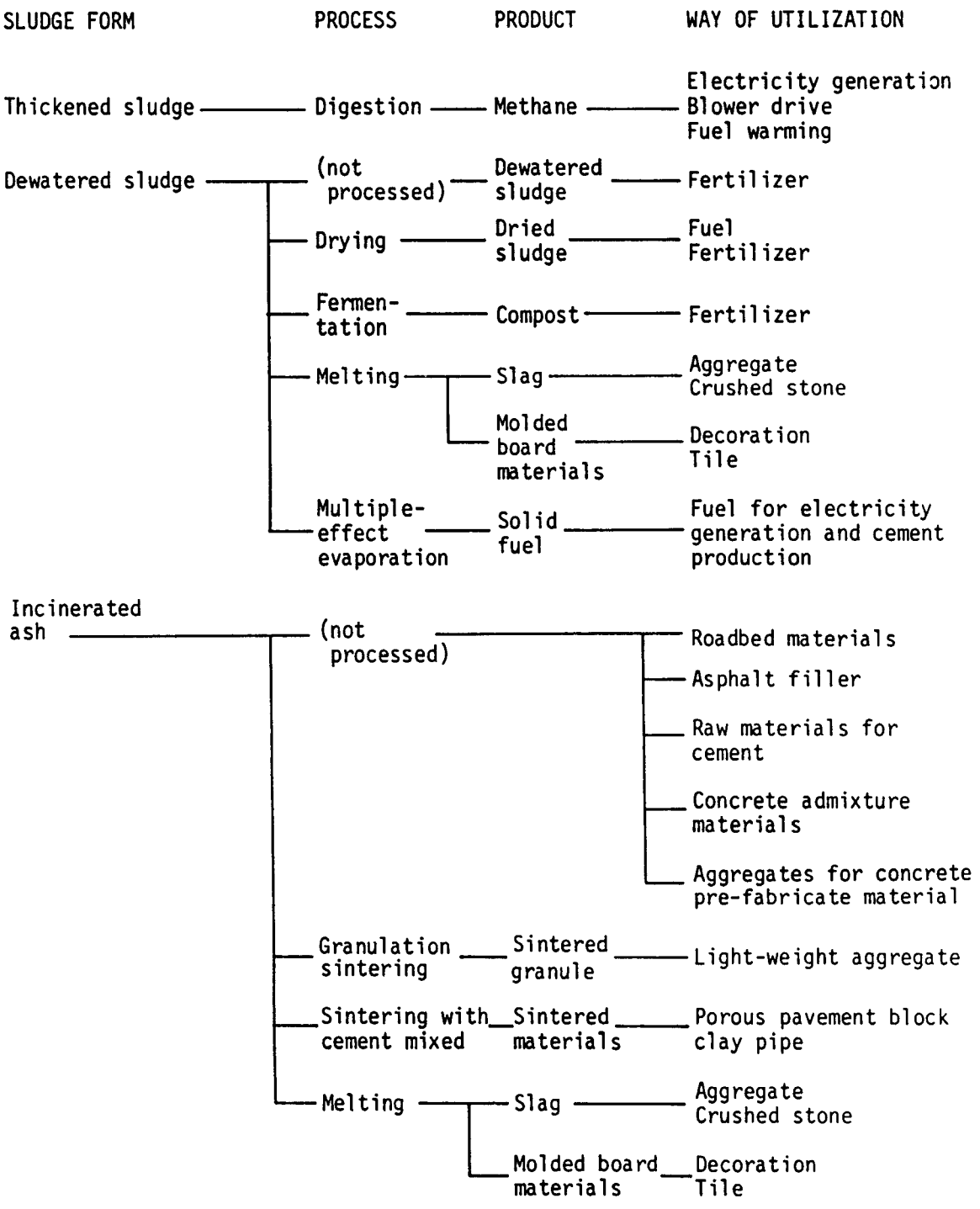


Fig. 3 Ways of utilization of sludge as resource

3. EFFECTIVE USE OF INORGANIC COMPONENTS

3-1 Inorganic Components

Inorganic components referred to here are components of incinerated ash and melted slag.

(1) Incinerated Ash

Incinerated ash are the residue left after incinerating sewage sludge. Their main components are silicon (Si), calcium (Ca), iron (Fe) and aluminum (Aℓ) and these four elements represent 70 ∿ 80% of all incinerated ash.

The composition of incinerated ash differs by (1) the properties of sewage and (2) by the type and quantity of sludge conditioner. Particularly, the lime component is profoundly affected by the conditioner used at the dewatering process. There are inorganic and organic conditioners. As for the former, the combination of ferric chloride and slaked lime are often used and for the latter, various organic polymer coagulants are used.

The lime component (as CaO) of incinerated ash of the dewatered sludge using both ferric chloride and slaked lime as conditioners is 15 ∿ 50%. By contrast, this is 1 ∿ 8% in incinerated ash using organic polymer coagulants. Table 4 shows examples of components of this incinerated ash. The true specific gravity of the incinerated ash is 2.5 ∿ 3.1 but their apparent specific gravity is about 1.0 and their grain size is distributed from silt to fine sand. Fig. 4 shows the yearly change of ash production by types of dewatered sludge.

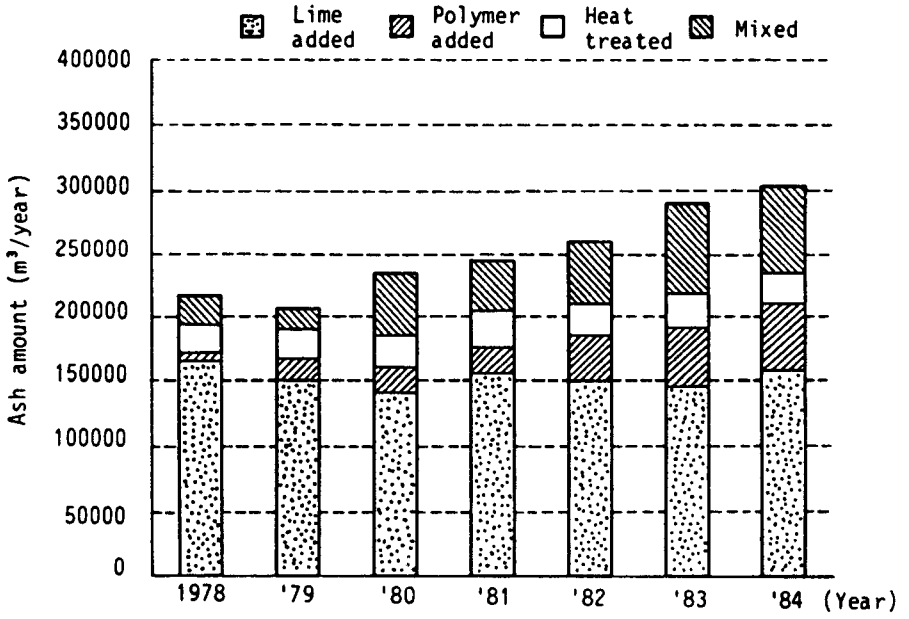


Fig. 4 Ash amount for each type of dewatered sludge

Table 4 Examples of ash analysis results

(%)

		CaO	SiO_2	Al_2O_3	Fe_2O_3
Lime added sludge (12 samples)	Mean	33	22	9	13
	Range	15∿51	9∿33	5∿17	7∿19
Polymer added sludge (9 samples)	Mean	5	44	16	13
	Range	1 ∿ 8	38∿50	12∿19	7∿20

(2) Melted slag

When sewage sludge is heated to 1,300 ∿ 1,500°C, the organic matter in the sludge is pyrolyzed and burned and the inorganic matter that remains is melted. Melted slag is obtained by cooling and solidificating this melted solution. Its true specific gravity is nearly equal to that of the incinerated ash. Melted slag is manufactured in different forms and strengths, depending on how it is cooled and whether crystallization is involved. Here, crystallization means that compounds in the slag are arranged regularly. What remains irregular is called amorphous (vitreous).

a. Granulated Slag ..... This is made by suddenly cooling the melted solution by either jetting water against it as it is discharged from the melting furnance or by dropping the solution into water. Because of sudden cooling, it becomes fine-grained, amorphous sludge slag.

b. Air-cooled Slag ..... This slag is made by cooling effected by leaving the melted solution from the melting furnance in the air. It is agglomerate because of gradual cooling. Though gradual, the solution cools in the air faster than the cooling velocity necessary for crystallization (slower than 3°C/min) and so becomes amorphous slag at a low degree of crystallization.

c. Crystal Slag ..... Slag crystallized to improve slag surface properties and strength is called crystal slag. The method of its crystallization are generally divided into; (1) method of maintaining a temperature best suited for crystallization for a certain period of time (e.g., 1,100 ∿ 1,150°C, 30 minutes) and; (2) a method of controlling cooling velocity and thereby securing temperature and time necessary for crystallization.

(3) Artificial Lightweight Aggregates

When granules of polymer incinerated ash are exposed to high temperature at 1,100 ∿ 1,150°C, their surfaces becomes half melted, air bubbles develop in their interiors and the granules expand as a whole. They become artificial lightweight aggregates when they are cooled in the air.

3-2 Effective Use of Inorganic Components

The following two are conceived as forms in which incinerated ash and melted slag are used as construction materials. One is the finally treated forms of sewage sludge (incinerated ashes and slag); these forms themselves are used as materials which include lime incinerated ash used as a backfilling material or as a roadbed material. The other is using incinerated ash and slag as part of the raw materials for making construction materials. To this end, there must be a separate material manufacturing process after the process of sludge treatment. This includes the use of non-lime incinerated ash as a raw material for clay pipes and water-permeable bricks.

Table 5 consolidates and evaluates past studies concerning the use of sewage sludge as construction materials. Below is an outline of utilization methods considered to have good prospects.

(1) Effective Use of Incinerated Ash

1) Foundation and Banking Materials

a. Banking Material

Lime incinerated ash is adequate as a banking material as far as strength is concerned because it can give sufficient strength immediately after the construction of a banking and becomes

Table 5 Evalution of beneficial uses

Sludge \ Application	Embankment	Soil improvement	Back filling	Base course	Concrete aggregate	Tile	Clay pipe, brick
Ash (Lime)	○	△	△	△			×
Ash (Polymer)							○
Slag (Air-cooled)				△	△		
" (Water-granulated)					△		
" (Crystallized)					○		
" (Formed)						○	

○ ..... use
△ ..... possible
× ..... impossible

increasingly strong for the next several weeks. But, with lime incinerated ash of especially fine grains, care must be taken readjusting the moisture content ratio used for handling; otherwise, rolling may be difficult and such problems as dust may well arise. Polymer incinerated ash is, as a whole, of such fine grains that they may be washed away by water or carried away by wind. Besides, they are rarely self-hardening. They are, therefore, undesirable as a banking material with respect to both strength and handling (Fig. 5).

b. Additive for Soil Conditioning

Lime incinerated ash can be adequately used as a soil conditioning additive singly or in combination with other components because they react well with soil. When using lime incinerated ash singly, they must be mixed with soft clayed soil or some other material. But strength does not increase significantly after conditioning. As a result, lime incinerated ash can only be used with the object of low strength improvement.

When using this ash in combination with other components, such as cement and lime, the combination, depending on its mixing ratio, has as much soil improving effect as any common soil conditioning additive. So, they can be used with the object of high strength improvement.

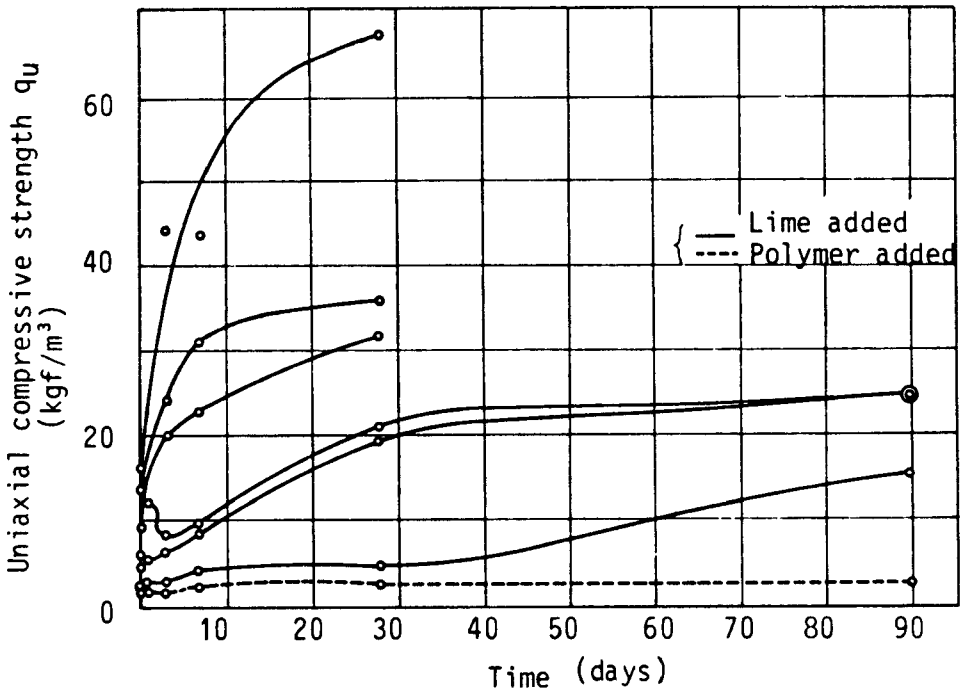


Fig. 5 Uniaxial compressive strength of ash

c. Backfilling Material

When using lime incinerated ash as a backfilling material, it provides great strength after the backfill, as in the above banking material in a. But from the point of view of easy re-excavation required of backfilling materials, it is inadvisable to use it singly as a backfilling material because used singly, it may give too much strength. Rather, it is better to use it as a soil conditioning additive by mixing it with soil. Using polymer incinerated ash as a backfilling material is undesirable for the same reason as in the case of its use for banking.

2) Paving and Roadbed materials

a. Roadbed Material

Lime incinerated ash can be used singly as a lower roadbed material or as road or sidewalk roadbed material because it improves strength by the reaction of lime.

b. Asphalt Filler

If lime incinerated ash is used in a standard mixing ratio of asphalt concrete, it makes an asphalt mixture.

3) Ceramic products

An increase in strength by the reaction of calcium compounds cannot be expected of non-lime incinerated ash (polymer incinerated ash), it can be used as a raw material for products such as, clay pipes, tiles and blocks.

When using non-lime incinerated ash as raw material for clay pipes, it is impossible for all clay to be replaced by incinerated ash as a raw material of clay pipes. Instead, the maximum is said to be about 10%. With regards to the working process, this can be performed conventionally except for changes in the mixing of clay and incinerated ash and the amount of water to be added.

Table 6 compares clay pipes containing incinerated ash with conventional products and JIS specifications. Even though both water absorption and compression strength tend to deteriorate with the increase of incinerated ashes added, these pipes fully satisfy the JIS specifications.

In a similar way, incinerated ash can be well utilized as water-permeable paving material. In this case, the rate of incinerated ashes to be added is 35 $\sim$ 40%, which is higher, compared with clay pipes. As product characteristics, this mixture has a bending strength of 35 $\sim$ 45 kg/cm^2 and a permeability coefficient of 0.03 $\sim$ 0.05 cm/sec.

Table 6 Characteristics of clay pipes made from ash

(φ250 mm, sintering temperature 1,100°C)

	External strength (kgf/m)	Water absorption (%)	Acid-resistant (%)
Clay pipe with no ash added	4,850	5.5	0
Clay pipe with 3% of crushed ash added	4,770	5.7	0
Clay pipe with 6% of crushed ash added	4,420	6.1	0
Clay pipe with 6% of not crushed ash added	4,090	6.8	0
JIS values	> 3,200	< 9.0	< 0.25

(2) Effective Use of Slag

1) Roadbed Material

Air-cooled slag has much the same strength as natural aggregates and can be adjusted to a necessary grain size. It is somewhat inferior to crushed stone in workability and stability after compaction. So, when using it singly, its utilization is limited to bottom-layer roadbeds and the beds of light-traffic roads, such as cycling roads, sidewalks and private roads. These characteristics of air-cooled slag are attributable to its smooth surface because of its vitreous nature and this can be improved by crystallization.

2) Concrete Aggregates

Slag aggregates obtained by air-cooling or granulating melted slag are vitreous. Therefore, concrete using these aggregates are inferior to concrete using ordinary aggregates in both strength and durability. Therefore, slag aggregates are recommended for use as concrete of relatively low strength only. The strength and durability of concrete using slag aggregates whose characteristics have been drastically improved -- which is possible -- by heat treatment and crystallization are nearly equal to those of concrete using ordinary aggregates. So, heat-treated sewage sludge slag aggregates can be used even for reinforced concrete (Fig. 6).

3) Molded Board Material, etc.

High-strength boards in different shapes can be manufactured by placing melted slag in molds of any shape and crystallizing them.

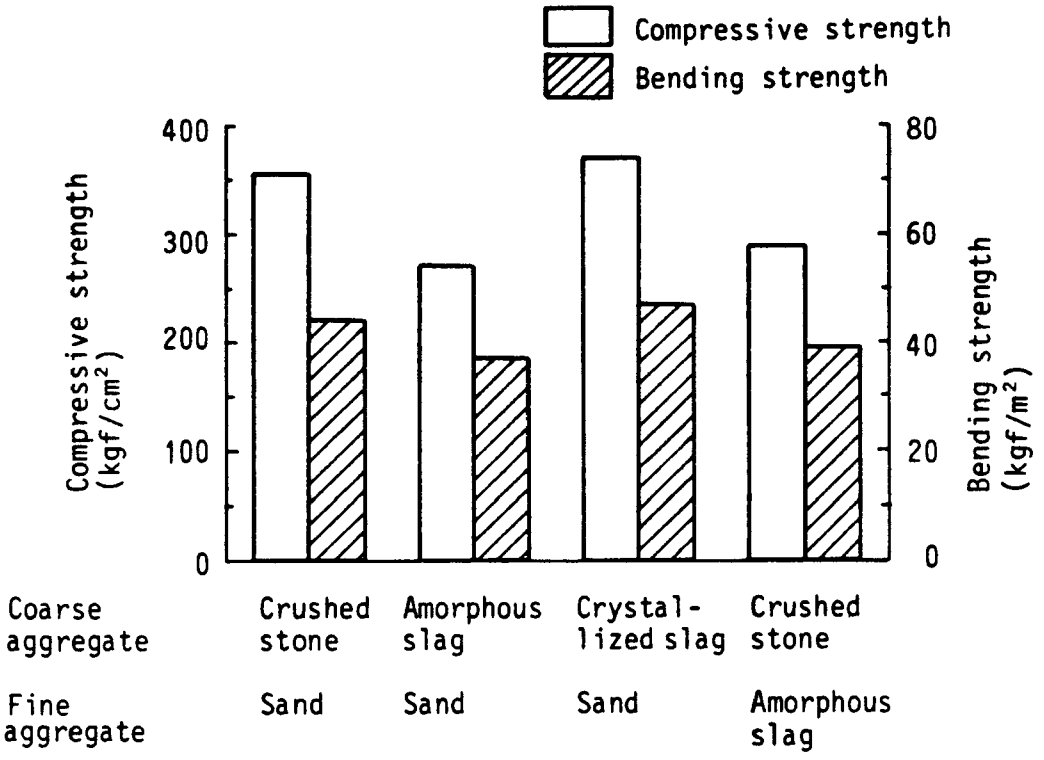


Fig. 6 Results of concrete tests

(3) Effective Use of Artificial Lightweight Aggregates

Artificial lightweight aggregates can be made by first granulating incinerated ashes and then heat-treating it at 1,100 ∿ 1,150°C. They can be used as lightweight aggregates for ordinary lightweight concrete buildings.

4. EFFECTIVE USE OF ORGANIC COMPONENTS

4-1 Organic Components

Table 7 shows the representative components of sludge used for greens and farms.

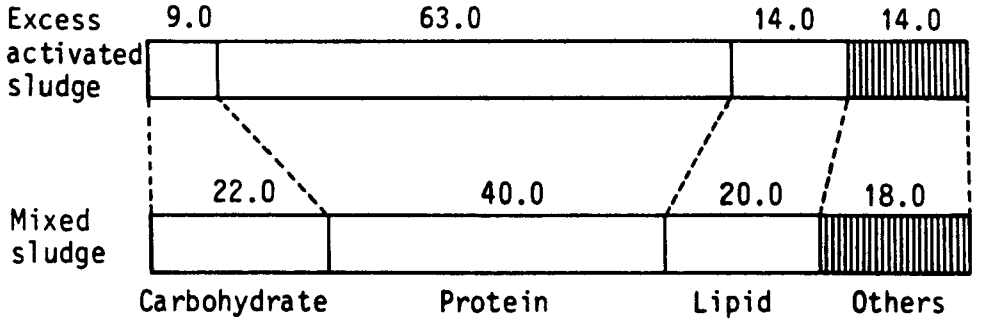


Fig. 7 Proportion of the components in VS

Table 7 Main components of sludge

Sample	Item	pH	Total nitrogen (DS %)	Phosphates (DS %)	Potassium (DS %)	C/N	Remarks	
Dewatered sludge	$Ca(OH)_2$ + $FeCl_3$	10.1	2.7	1.1	0.91	7.9	Ca 21.3%	1
		12.2	3.14	2.43	0.04	7.2	CaO 28.11%	3
		12.5	2.16	1.03	0.18	-	CaO 27.53%	4
	Coagulant Polymer	6.1	5.39	4.24	0.45	7.7		2
Composted sludge	Coagulant $Ca(OH)_2$ + $FeCl_3$	7.8	2.28	3.18	0.04	6.5	CaO 24.61%	3
		7.8	2.30	3.68	0.11	9.0	Ca 21.3%	3
		8.0	1.63	2.52	0.25	9.2	Ca 22.4%	3
	Polymer + Chaff	5.2	1.24	1.75	0.51	19.6		3
		6.6	1.90	1.86	0.57	16.1		
	Polymer + Bark	6.8	3.30	2.36	0.20	10.6		2
		6.6	3.57	1.7	0.60	9.3		3

1 1983, Association for Utilization of Sewage Sludge, Reuse and Utilization, No.21 P.44 (Dewatered Sludge of an STP in Toride City)

2 1980, Association for Utilization of Sewage Sludge, Index of Sewage Sludge Compost, P.143 (Ikenokawa STP, Hitachi City)

3 1983, Association for Utilization of Sewage Sludge, Sewage Sludge Composting, No.26 P.102

4 1984, Association for Utilization of Sewage Slude, Reuse and Utilization

Fig. 7 shows the average three component values of proportions occupied in the VS by excess sludge and mixed sludge produced at eighteen of Japan's sewage treatment plants. The chart does not show data on pure primary sludge not containing any excess sludge but this can be estimated if the mixing ratio of excess and primary sludge is assumed. Assuming excess: primary = 1:1, the proportions occupied in the VS by the carbohydrate protein and lipid of primary sludge are, respectively, 35%, 26% and 17%. It can be seen that whereas excess sludge is composed, mainly, of protein, primary sludge is composed, mainly, of carbohydrates.

To study these three components in further detail, mixed sludge, excess sludge and digested sludge were taken from three sewage treatment plants and a biochemical property analysis was made on these samples. The results of this analysis are indicated in Table 8. The results of the protein constituting amino acid composition analysis are shown in Table 9. In these results, there are no great differences among the three sewage treatment plants. Analysis methods are shown in Table 10.

(1) Carbohydrate

The carbohydrate content was higher in mixed sludge, 2 ∿ 5 times that than in excess sludge and digested sludge, and was comprised mostly of cellulose. Excess sludge contained 1.5 ∿ 2.0 times more monosaccharides and oligosaccharides than the other sludges. It also

Table 8 Biochemical comparison of dry sludge

Items	Unit		Ohgaki (Gifu)			Kohoku (Ibaraki)			Mito (Ibaraki)		
			Primary sludge	Excess sludge	Digested sludge	Primary sludge	Excess dludge	Digested dludge	Primary sludge	Excess sludge	Digested dludge
(1) Carbohydrate	-		-	-	-	-	-	-	-	-	-
(2) Degradable saccharide	mg/g as Glu	o	193	68.9	54.3	283	81.9	53.2	224	54.3	54.3
(3) Monosaccharide, oligosaccharide	mg/g as Blu	Δ	7.8	12.4	4.3	6.4	16.8	10.8	11.7	15.9	5.9
(4) Neutral monosaccharide	-		-	-	-	-	-	-	-	-	-
(5) Glucose	mg/g as Glu	o	0.1 >	0.1 >	0.1 >	0.1 >	0.1 >	0.1 >	0.1 >	0.1 >	0.1 >
(6) Amino sugar	ma/g as Glu	o	0.80	3.80	0.21	0.35	1.86	0.71	0.96	2.02	0.35
(7) Uronic acid	ma/g as Glu	o	0.37	0.97	0.12	0.18	0.91	0.18	0.61	0.71	0.72
(8) Others	mg/g as Glu	Δ	6.6	7.6	4.0	5.5	14.0	9.9	10.1	13.2	4.8
(9) Degradable polysaccharide	mg/g as Glu	o	185	56.5	50.0	277	65.1	42.4	212	38.4	48.4
(10) Cellulose	mg/g as Glu	o	172	22.1	24.8	193	20.2	17.8	174	20.5	27.1
(11) Others	mg/g as Glu	Δ	13.0	34.4	25.2	84.0	44.9	24.6	38.0	17.9	21.3
(12) Refractory saccharide	mg/g	o	109	118	165	121	129	171	94	131	139
(13) Lipid	-		-	-	-	-	-	-	-	-	-
(14) Water soluble lipid (organic acid)	mg/g as AA	o	46.3	78.8	11.2	42.8	50.7	6.1	58.7	64.8	6.2
(15) Acetic acid	mg/g as AA	o	21.2	36.3	0.5 >	20.1	0.5 >	0.5 >	26.8	27.2	0.5 >
(16) Others	mg/g as AA	Δ	25.1	42.5	11.2	22.7	50.7	6.1	31.9	69.5	6.2

(17) Water insoluble lipid	mg/g	o	114	124	83.1	148	111	63.7	136	127	55.8
(18) Fatty acid	mg/g as OA	o	105	118	66.2	137	104	38.1	126	120	32.5
(19) Free fatty acid	mg/g as OA	o	41.2	74.7	5.3	96.5	60.2	8.0	87.3	69.1	8.1
(20) Palmitic acid	mg/g as OA	o	15.9	8.2	0.7	34.9	7.1	2.4	31.8	9.8	1.9
(21) Stearic acid	mg/g as OA	o	4.2	1.8	0.3	8.6	1.6	0.8	6.7	2.1	0.6
(22) Oleic acid	mg/g as OA	o	6.6	4.9	0.6	19.3	5.3	1.8	12.4	5.8	1.5
(23) Linoleic acid	mg/g as OA	o	1.7	1.6	0.2	7.2	1.3	0.5	3.5	1.7	0.5
(24) Linolenic acid	mg/g as OA	o	0.2	0.2	0.1 >	0.6	0.2	0.1 >	0.3	0.1	0.1 >
(25) Others	mg/g as OA	Δ	12.6	58.0	3.5	25.9	44.7	2.5	32.5	49.6	3.6
(26) Ester fatty acid	mg/g as OA	Δ	63.8	43.3	60.9	40.5	53.1	30.1	38.7	50.9	24.4
(27) Others	mg/g	Δ	9.0	6.0	16.9	11.0	7.0	25.6	10.0	7.0	23.3
(28) Protein and its relatives	mg/g as GA	o	231	437	166	197	438	299	233	420	213
(29) Degradable protein	mg/g as GA	o	139	380	50.4	128	308	193	135	287	130
(30) Refractory protein	mg/g as GA	Δ	92	57	116	69	130	106	98	133	83
(31) Free amino acid	mg/g as GA	o	39.0	114	19.8	24.0	130	106	50.9	283	70.2
(32) Protein, peptide	mg/g as GA	Δ	192	323	146	173	308	193	182	137	143
(33) Amino acid of protein and its relatives	mg/g as GA	o	167	312	103	146	292	195	149	310	156
TOC	mg/g	o	349	386	263	410	383	287	399	393	263
TN	mg/g	o	46.1	74.6	32.9	35.3	72.2	45.8	40.3	74.3	38.5
TS	mg/g	o	68	46	62	147	75	98	132	66	97
VTS	mg/g	o	664	742	541	830	769	607	764	751	512
Humic acid	mg/g	o	18.4	24.6	13.2	19.9	18.4	11.6	27.7	30.7	14.0
Fulvic acid	mg/g as LSS	o	201	337	147	120	360	201	141	283	185

1. Glu (Glucose), AA (Acetic acid), OA (Oleic acid), GA (Glutamic acid), LSS (Lignin sodiumsulfonate)
2. o analysis, Δ calculation

Table 9 Contents of amino acid in the sludge

Items	Ohgaki (Gifu)			Kohoku (Ibaraki)			Mito (Ibaraki)		
	Primary sludge	Excess sludge	Digested sludge	Primary sludge	Excess sludge	Digested sludge	Primary sludge	Excess sludge	Digested sludge
Arginine	11.1 (6.7)	22.9 (7.3)	6.1 (5.9)	9.7 (6.7)	21.7 (7.4)	12.9 (6.6)	10.0 (6.7)	23.4 (7.5)	10.0 (6.4)
Lysine	10.1 (6.1)	21.3 (6.8)	5.1 (4.9)	9.3 (6.4)	18.4 (6.3)	11.9 (6.1)	8.8 (5.9)	20.4 (6.6)	8.6 (5.5)
Histidine	4.4 (2.6)	7.8 (2.5)	2.5 (2.4)	3.9 (2.8)	7.5 (2.6)	4.6 (2.5)	3.7 (2.5)	8.1 (2.6)	3.7 (2.4)
Phenyl alanine	10.7 (6.4)	20.5 (6.6)	6.8 (6.6)	8.9 (6.1)	18.2 (6.2)	12.4 (6.3)	9.6 (6.5)	18.9 (6.1)	10.6 (6.8)
Tyrosine	9.9 (5.9)	18.7 (6.0)	6.7 (6.5)	7.7 (5.3)	16.9 (5.8)	12.3 (6.3)	8.2 (5.5)	17.1 (5.5)	10.2 (6.5)
Leusine	13.8 (8.3)	25.7 (8.2)	8.7 (8.4)	12.9 (8.9)	23.7 (8.1)	15.2 (7.8)	13.1 (8.8)	25.7 (8.3)	12.8 (8.2)
Isoleusine	8.2 (4.9)	14.8 (4.7)	5.1 (4.9)	7.7 (5.3)	14.0 (4.8)	9.4 (4.8)	7.7 (5.2)	14.9 (4.8)	7.7 (4.9)
Methionine	4.8 (2.9)	9.3 (3.0)	3.2 (3.1)	4.4 (3.0)	7.8 (2.7)	5.3 (2.7)	4.4 (3.0)	8.6 (2.8)	4.5 (2.8)
Valine	10.8 (6.5)	19.2 (6.1)	7.1 (6.9)	9.0 (6.2)	18.2 (6.2)	12.1 (6.2)	9.6 (6.5)	19.2 (6.2)	9.9 (6.3)
Alanine	8.8 (5.3)	17.4 (5.6)	5.5 (5.3)	8.1 (5.6)	16.5 (5.6)	10.7 (5.5)	9.1 (6.1)	17.9 (5.8)	8.5 (5.4)
Glycine	6.7 (4.0)	11.5 (3.7)	4.4 (4.3)	5.3 (3.6)	11.0 (3.8)	7.6 (3.9)	5.4 (3.6)	11.7 (3.8)	6.0 (3.8)
Proline	6.1 (3.7)	11.3 (3.6)	3.5 (3.4)	5.0 (3.4)	12.3 (4.2)	7.6 (3.9)	5.1 (3.4)	11.2 (3.6)	6.1 (3.9)
Glutamic acid	21.5 (12.9)	37.2 (11.9)	12.5 (12.1)	20.7 (14.2)	34.4 (11.8)	23.0 (11.8)	18.5 (12.4)	38.0 (12.3)	18.2 (11.6)
Serine	6.6 (4.0)	12.0 (3.8)	4.3 (4.2)	5.6 (3.8)	11.1 (3.8)	8.0 (4.1)	5.8 (3.9)	11.9 (3.8)	6.3 (4.0)
Threonine	8.4 (5.0)	16.5 (5.3)	5.5 (5.3)	6.8 (4.7)	16.0 (5.5)	11.1 (5.7)	7.5 (5.0)	17.0 (5.5)	8.7 (5.6)
Aspartic acid	17.8 (10.7)	33.6 (10.8)	11.3 (10.9)	14.6 (10.0)	32.0 (11.0)	22.3 (11.4)	15.6 (10.5)	33.4 (10.8)	17.4 (11.1)
Tryptophan	4.8 (2.9)	8.8 (2.8)	3.4 (3.3)	3.8 (2.6)	8.7 (3.0)	6.3 (3.2)	4.3 (2.9)	9.1 (2.9)	5.0 (3.2)
Cystine	2.3 (1.8)	3.7 (1.2)	1.5 (1.5)	2.3 (1.6)	3.8 (1.3)	2.7 (1.4)	2.2 (1.5)	3.7 (1.2)	2.2 (1.4)
Total	166.8	312.2	103.2	145.7	292.2	195.4	148.6	310.2	156.4
Values according to the ninhydrin reaction	231	437	166	197	438	299	233	420	213

1. ( ): percentage

Table 10 Analysis method

	Item	Analysis method	Substances
Carbohydrate	Degradable saccharide	Hydrolysis of sulfuric acid, anthrone reaction	
	Refractory saccharide	Gravimetric method (weighing the residues after decomposion)	
	Degradable polysaccharide	Hydrolysis of sulfuric acid (after monosaccharide and oligosaccharide decomposed), anthrone reaction	
	Amino sugar	Elson-Morgan method	
	Uronic acid	Sulfuric acid and carbazole reaction	
	Cellulose	Modified Southgate method	
	Neutral monosaccharide	HPLC	Glucose
	Water soluble lipid	Steam distillaion, titrimetric method	
Lipid	Water insoluble lipid	Extraction gravimetry by chloroform-methanol	
	Fatty acid	Saponification valve method	
	Free fatty acid	Acid value method	
	Water soulble lipid composion	HPLC	Acetic acid
	Free fatty acid composion	GC	Palmitic acid Stearic acid Oleic acid Linoleic acid Linolenic acid
Protein and its relatives	Protein	Hydrolysis, ninhydrin reacion	
	Refractory protein	Enzyme reaction by pepsin	
	Free amino acid	Ninhydrin reaction	
	Amino acid composion	Hydrolysis, amino acid analyser	Natural amino acid
Humus	Humic acid	Fluorometric analyser	
	Fulvic acid	Fluorometric analyser	
	TOC	Elemental analysis	
	TN	Elemental analysis	
	TS	Gravimetric method	
	VTS	Gravimetric method	

contained more amino sugar and uronic acid but its contents of these were smaller than its content of polysaccharides. Digested sludge contained somewhat more refractory sugar than the other sludges.

(2) Lipid

Mixed sludge and excess sludge contained 1.5 ∿ 2.5 times more lipids than digested sludge and the lipids were comprised mostly of fatty acids including water-soluble ones and the amount of free fatty acids was large. Among the free fatty acids, the contents of palmitic acid and oleic acid were large and, in particular, mixed sludge abounded in palmitic acid. Digested sludge contained less water-soluble lipids and free fatty acids than the other sludges and more than half its lipids were ester fatty acids and other fatty acids.

(3) Protein

Excess sludge contained about twice as much protein substances as the other sludges and they included many free amino acids, protein and peptide. As a whole, the rate of readily degradable protein digested by pepsin was high among the protein substances. Glutamic acid and aspartic acid somewhat abounded as amino acids composing protein substances but the others were insignificant.

(4) General Properties

Mixed sludge and excess sludge exceeded digested sludge in TOC and VS indicating quantities of organic matter. In TN correlated to protein substances, excess sludge exceeded the other and showed a tendency similar to the quantitative relations of protein substances.

Regarding humuses, there was more fulvic acid than humic acid and this was particularly remarkable in excess sludge.

(5) Summary

Table 11 lists the composition of sewage sludge by the carbon (C) standard from the results of analyses.

Mixed sludge has a degradable sugar content of about 20 ∿ 30%, a fatty acid (including water-soluble lipids) content of about 30% and a protein content of about 20 ∿ 30% and these represent about 75% of TOC excluding refractory substances, such as refractory sugars, steroid and amorphous polymers. The degradable sugars are comprised mostly of cellulose.

Excess sludge has a degradable sugar content of about 5 ∿ 10%, a fatty acid (including water-soluble lipids) content of about 30% and a protein content of about 40 ∿ 50%, which is roughly half of that, and these represent about 80 ∿ 85% of TOC excluding refractory substances, such as refractory sugars, steroid and amorphous polymers. The presumed reason why the rate in TOC is higher in this sludge than in the others is because excess sludge is composed mainly of microorganisms.

Table 11 The composition of sludge (based on TOC)

	Degradable saccharide			Fatty acid			Protein and its relatives		Others
	Monosaccharide oligosaccharide	Cellulose	Polysaccharide	Water soluble lipid	Free fatty acid	Ester fatty acid	Free amino acid	Protein, peptide	
Primary sludge		24.0			29.4			23.5	23.1
	0.9	18.6	4.5	5.1	14.6	9.7	4.1	19.4	
Excess sludge		7.1			29.8			45.5	17.6
	1.6	2.2	3.3	6.7	13.4	9.7	18.4	27.1	
Digested sludge		7.9			14.1			33.9	44.1
	1.0	3.4	3.5	1.2	2.0	10.9	9.7	24.2	

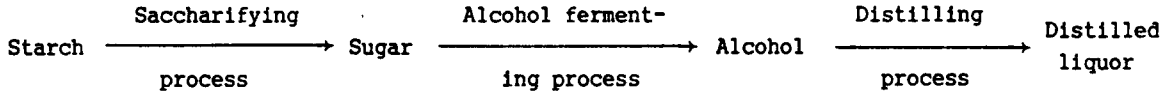
(Unit: % in TOC)

Digested sludge has a degradable sugar content of about 10%, a fatty acid (including water-soluble lipids) content of about 10 ∿ 20% and a protein content of about 30 ∿ 40% and these represent about 40 ∿ 50% of TOC excluding refractory substances, such as refractory sugars, steroid and amorphous polymers; thus, refractory substances account for roughly a half. Further, the rates of water-soluble lipids and free fatty acids are extremely low in digested sludge, compared with mixed sludge and excess sludge. Considering that digested sludge is the residue of anaerobic decomposition of mixed sludge and excess sludge, it seems that degradable sugars (including cellulose) are decomposed almost completely with ester fatty acids, protein and peptide remain, being difficult to decompose.

4-2 Effective Use of Organic Components

The organic components can only be used for greens and farms. But as stated in the preceding section, sludge contains various components and the use of these is a major problem of the future. Take an instance where ethyl alcohol was made, using mixed sludge which contained 20% cellulose in it.

Generally, the following procedure is necessary to make alcoholic beverages from such raw materials as grapes, barley, rice and sweet potatoes:



In the case of wine, the saccaharifying process in unnecessary because its raw material is fruit abounding in sugar (monosaccharides including glucose) and the raw material immediately undergoes alcohol fermentation using yeast. But in the case where the starch of cereals, potatoes, etc. is used as a raw material, starch (polysaccharide) is turned into glucose (monosaccharide), using amylase or some other saccharifying enzyme in the saccharifying process, and then the glucose undergoes alcohol fermentation using yeast. Ethyl alcohol is made from sludge by these processes of saccharification, fermentation and distillation.

(1) Saccharification

A problem with the saccharifying process is the decision regarding saccharifying conditions. Specifically, it concerns; (1) selection of an enzyme; (2) whether the sludge was sterilized or not and; (3) the quantity of the enzyme, rate of saccharification, etc. What is referred to here as the rate of saccharification (%) is the value of quantity of glucose produced (mg/ℓ)/degradable sugar (mg/ℓ) x 100.

Selection of enzyme: The rate of saccharification was best when three enzymes: cellulase, amylase and glucoamylase were used together. So, this trio was used in the experiment.

Sterilization of sludge: The preliminary experiment disclosed that, without heat treatment, saccharification did not progress even if an enzyme was added. So, heat treatment was adopted to sterilize the

Table 12 Results of saccharification and alcohol fermentation

Subtrates	Condition of sterilization	Saccharification process					Alcohol fermentation process				
		Conditions		Quality of product			Conditions			Quality of product	
		Enzyme dosage (mg/100 mℓ)	Reaction time (hr)	Content of degradable saccharide (mg/100 mℓ)	Content of produced glucose (mg/100 mℓ)	Ratio of saccharification (%)	Yeast	Yeast dosage (mg/100 mℓ)	Reaction time (hr)	Odor	Ethanol content (v/v%)
Primary sludge	120°C 15 min.	20	24	42.6	49.1	115	Baker's Yeast	100	24	Night soil-like	0.1
	60°C 15 min.	20	24	42.6	16.4	38.5	Baker's Yeast	100	24		0.1
	120°C 15 min.	20	17	85.2	16.9	19.8	Baker's Yeast	100	64		0.2
Excess sludge	120°C 15 min.	20	24	36.7	36.2	98.6	Baker's Yeast	100	24		0.1
Digester supernatant	120°C 15 min.	20	24	37.9	1.6	4.2	Baker's Yeast	100	24		+
Starch	120°C 15 min.	20	24	98.0	40.2	46.0	Baker's Yeast	100	24	Spirit-like	0.1
Cellulose	120°C 15 min.	20	24	102.3	12.3	12.1	Baker's Yeast	100	24	Sweet	+

sludge. As for conditions of heat treatment, comparison between two cases: 60°C and 120°C disclosed that the rate of saccharification was higher with 120°C, as indicated in Table 12. So this temperature was adopted.

Relation between quantity of enzyme added and rate of saccharification of degradable sugar: The test results are shown in Fig. 8. Results for test cases other than the case shown in Fig. 8 are given in Table 12. From these charts, the following conclusions may be made:

a. In the case of the same degradable saccharide concentration, the rate of saccharification is proportional to the quantity of the enzyme that is added (Fig. 8).

b. The rates of saccharification of mixed sludge, starch and cellulose, which have much the same degradable sugar concentration, are in the order of starch > mixed sludge > cellulose (Table 12).

c. The rate of saccharification of excess sludge is large but the rate of saccharification of the digestion supernatant liquid under the same conditions is very small (Table 12).

(2) Fermentation

The fermentation test this time used baker's yeast or saccharomyces. Table 12 shows some of the test results using bread yeast.

The ethanol concentration in this table is so low that the ethanol cannot be called alcohol. Hence, the necessity of the distilling process. In the case of sludge, the odor was similar to that of excrement.

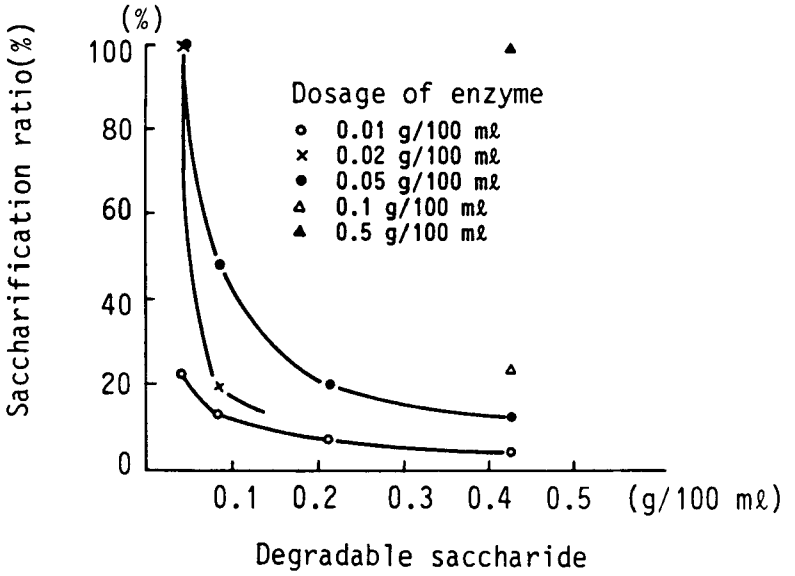


Fig. 8 Dosage of enzyme and saccharide content

(3) Distillation

Tests on the distilled concentration of the saccharified solution after saccharification and distillation of the fermented solution after fermentation were conducted using this concentrated solution because of a very low ethanol concentration of about 0.1%, as shown in Table 11. This operation flow is represented in Fig. 9.

Thus, 15 mℓ of an ethanol with a concentration of 15.3% was recovered as a final product from 15 g of dried sludge. However, in the distilling process, odorants in the sludge were concentrated along with ethanol.

(4) Summary

In this test, large quantities of expensive enzymes were used in the saccharifying process. Under these circumstances, the biotechnological culture and immobilization of microorganisms to secrete saccharifying enzymes are desired. Also, the immobilization of yeast

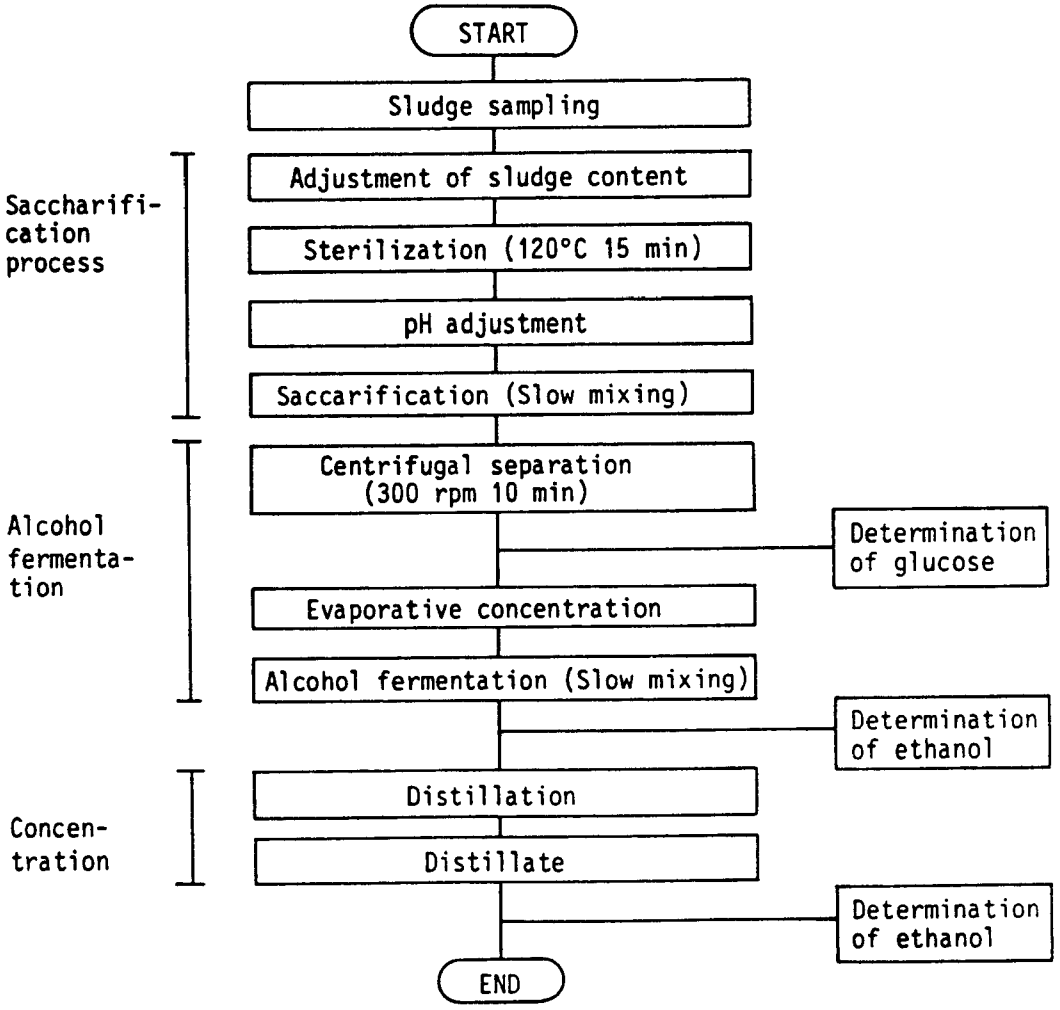


Fig. 9 Flow of alcohol production from sludge

in the fermenting process is necessary. The removal of odorants in ethanol produced must be incorporated as preliminary to the distilling process.

5. CONCLUSION

Once the usefulness of sludge as a raw material is recognized, as stated in all preceding sections, it will be an inexhaustible resource since it is produced as long as people continue to exist.

Here, when considering the importance of sewage treatment plants as a source of supply of raw materials, one can see that sewage pipes form a system not requiring the mining or transportation of raw materials since they criss-cross throughout areas of human habitation.

If there is a sewage treatment plant as a source of supply of sludge, advanced industries using sludge as a raw material will develop in its vicinities, the present sludge treating process will be modified and the sewer system will become an entirely new institution.

Therefore, the first step towards a future strategy on sludge must be careful consideration regarding:

a. What can be made from sludge?

b. How to ensure stability in the quality and quantity of sludge as a raw material?

c. How to cut the cost of making sludge into a product?

d. How to increase the market values of sludge products and where should they be marketed?

If, in this study, it is decided that excess sludge is more useful, it will be necessary to restructure the wastewater treatment system so as to increase the production of excess sludge and the entire sewer system will have to be changed in this and other ways.

As stated already, the nucleus of the future sewer system strategy is the sewage treatment plant -- and particularly the sludge treatment facility. A full array of advanced biotechnology-using industries standing around this nucleus -- this, it seems, should be the picture of what the future holds in store for us all.

EVALUATION OF ENERGY SAVING USING HEAT BALANCE MODEL FOR MELTING PROCESS

by

Taigo MATSUI, Director

Tadahiro MURAKAMI, Dr., Senior Researcher

Research and Technology Development Division
Japan Sewage Works Agency

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

Prepared for Presentation at:
11th United States/Japan Conference
on
Sewage Treatment Technology

October 1987
Tokyo, Japan

Abstract

The possible volume reduction and stabilization of the sewage sludge associated with the melting process are expected to be greater than with the incineration process. In addition, melted slag can be utilized. However, since the melting process requires a very high temperature to melt inorganics in the sludge, the technologies to minimize energy consumption, to establish system operation and to prolong durability of facilities should be developed. This paper discusses the auxiliary fuel consumption as follows.

(1) Preparation of a model that provides the auxiliary fuel consumption of the melting system on the basis of the mass and heat balances.

(2) Evaluation of the auxiliary fuel consumption in the above model using the cake moisture content, the volatile solids of the cake, the dried cake moisture content and the melting temperature as parameters.

(3) Examination of the operational conditions for an energy saving melting system based on the results of (1) and (2) above.

Contents

Page

1. INTRODUCTION 124

2. STATE OF THE ART OF SLUDGE MELTING SYSTEMS 125

3. PROPOSED HEAT BALANCE MODEL FOR THE MELTING SYSTEMS 129

3.1 Operational Conditions of Melting Systems 129

3.2 Basic Values for Mass and Heat Balances 130

3.3 Symbols Used for Mass and Heat Balances 133

3.4 Heat Balance Equations for Sludge Melting Systems 133

4. ENERGY EVALUATION OF SLUDGE MELTING SYSTEMS 141

5. CONCLUSION 150

1. INTRODUCTION

Since it has become increasingly difficult in recent years to secure final disposal sites for sludges, the selection of the sludge treatment process to be employed is often subject to a number of constraints. These circumstances strongly require the volume reduction and the stabilization of sewage sludge. In addition, the reuse of treated sludge is a possible alternative to final disposal.

Melted sludge (hereinafter referred to as slag) is currently the most stable form of sludge, as well as the most compact. Either dewatered cake or incinerated ashes is transformed to the slag by the melting process where the inorganics are melted at a high temperature and then solidified in a cooling process.

The melting process excels the incineration in terms of volume reduction and the stabilization of sludge and also in the possible utilization of the slag. Since the melting process needs heating the inorganics to a melting temperature in the melting furnace for even a short period of time, the technologies to carry out this process with low energy consumption, easy operation and high durability of facilities remain to be established.

This paper disusses the auxiliary fuel consumption from the following viewpoints.

(1) Preparation of a model that provides the auxiliary fuel consumption of the melting system on the basis of the mass and heat balances.

(2) Evaluation of the auxiliary fuel consumption in the above model using the cake moisture content, the volatile solids of the cake, the dried cake moisture content and the melting temperature as parameters.

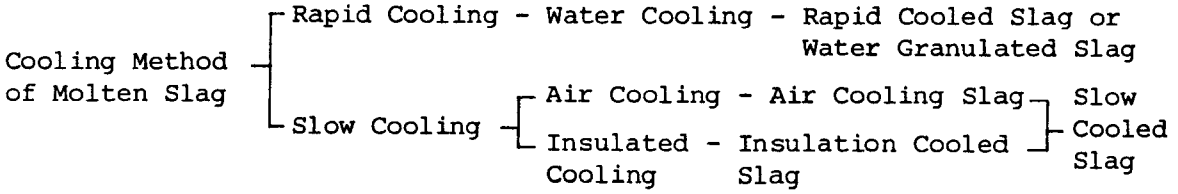
(3) Examination of the operational conditions for an energy saving melting system based on the results of (1) and (2) above.

2. STATE OF THE ART OF SLUDGE MELTING SYSTEMS

When sludge is heated, the water content evaporates and the combustible content is thermally decomposed and combusted. Eventually, even the inorganics begin to melt to form molten slag. This molten slag is then cooled down to form slag. The system where dewatered cake, dried sludge and ashes, etc. are recovered in the form of slag is called the sludge melting system.

In general, the inorganics of sewage sludge consist of the following metal compounds, i.e. silica (Si), iron (Fe), aluminium (Al), calcium (Ca), magnesium (Mg), potassium (K) and Sodium (Na). Of these, the Si, Fe, Al and Ca compounds often make up 70-80% of the inorganics content. The melting temperature varies between 1,300 and 1,500°C depending on the composition of the sludge.

As shown below, the molten slag is made into either rapid-cooled or slow-cooled slag depending on the method employed.

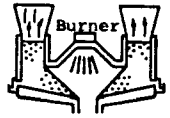
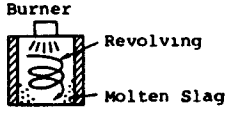
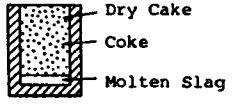
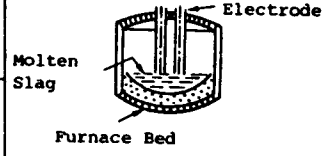
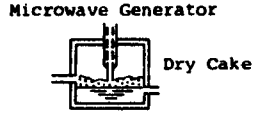


It is expected that the melting process achieves more volume reduction and the stabilization of sewage sludge than the simple incineration and that the slag will be used as aggregate.

A melting furnace with excellent heat resistance is required to recover the inorganics of the sludge as molten slag. Table 1 shows the types and characteristics of the melting furnaces currently developed for the sewage sludge treatment.

Sludge is fed to the melting furnace in the form of either dried cake or incinerated ash. All of the five types of melting furnaces referred to Table 1 are capable of melting both dried cake and ashes. This Paper deals with the reflector melting furnace, the cyclon melting furnace and the coke-bed melting furnace, all of which carry out the incineration and melting of cake in one stage in the melting system as shown in Figure 1.

Table 1 Types of Sludge Melting Furnaces

Type of Melting Furnace	(Type of Energy) Heating Method	Melding Subject	Outline of Furnace	Illustration
1 Reflector Melting Furnace	(Fuel) o Gas o Heavy oil o Powdered coal	Dried cake	Dried cake is placed around the core in a doughnut shape. When the fire is lit, the inside upper wall of the furnace is heated, forming a kind of reflector to maintain melting. The burner is used until the furnace reaches a steady-state. Once the steady-state has been reached, the furnace temperature is controlled by adjusting the cake input through changes in the ceiling height.	
		Ashes	The ashes are placed inside the furnace and the furnace is heated by the burner so that its upper part forms a kind of reflector in order to maintain melting. The heat to maintain melting is entirely supplied by the fuel combustion heat from outside.	
2 Cyclone Melting Furnace	Same as above	Dried cake	With the cake and air, a revolving flow along the inside wall of the cylindrical furnace is generated to lengthen the retention time of the cake. The cake combustion in turn heats the furnace wall to maintain melting.	
		Ashes	While the burner heats the furnace, a revolving flow of ashes and air is generated along the inside wall of the cylindrical furnace to maintain melting. The heat to maintain melting is entirely supplied by the fuel combustion heat from outside.	
3 Coke Bed Melting Furnace	(Fuel) Coke	Dried cake	Dried cake and coke are thrown into the furnace in turn to assure the permeability and energy supply required for continuous melting.	
		Ashes	Ashes and coke are thrown into the furnace in turn to assure the permeability and energy supply required for continuous melting.	
4 Arc Melting Furnace	(Electricity) o Arc heat to melting subject o Arc heat and reflected heat	Dried cake	When electricity is turned on to the carbon electrode in the furnace and the base metal at the bottom of the furnace, arc discharge occurs, to generate the heat to maintain melting.	
		Ashes	Same as above	
5 Microwave Melting Furnace	(Electricity) o Friction heat caused by oscillated rotation of molecules	Dried cake Ashes	Theoretically possible. When the microwave (electro-magnetic) radiation is applied to the ash (which is a dielectric substances) the ashes commence electrical oscillation and melt by their self-generated heat. The thermal conditions can be regulated by the microwave output.	

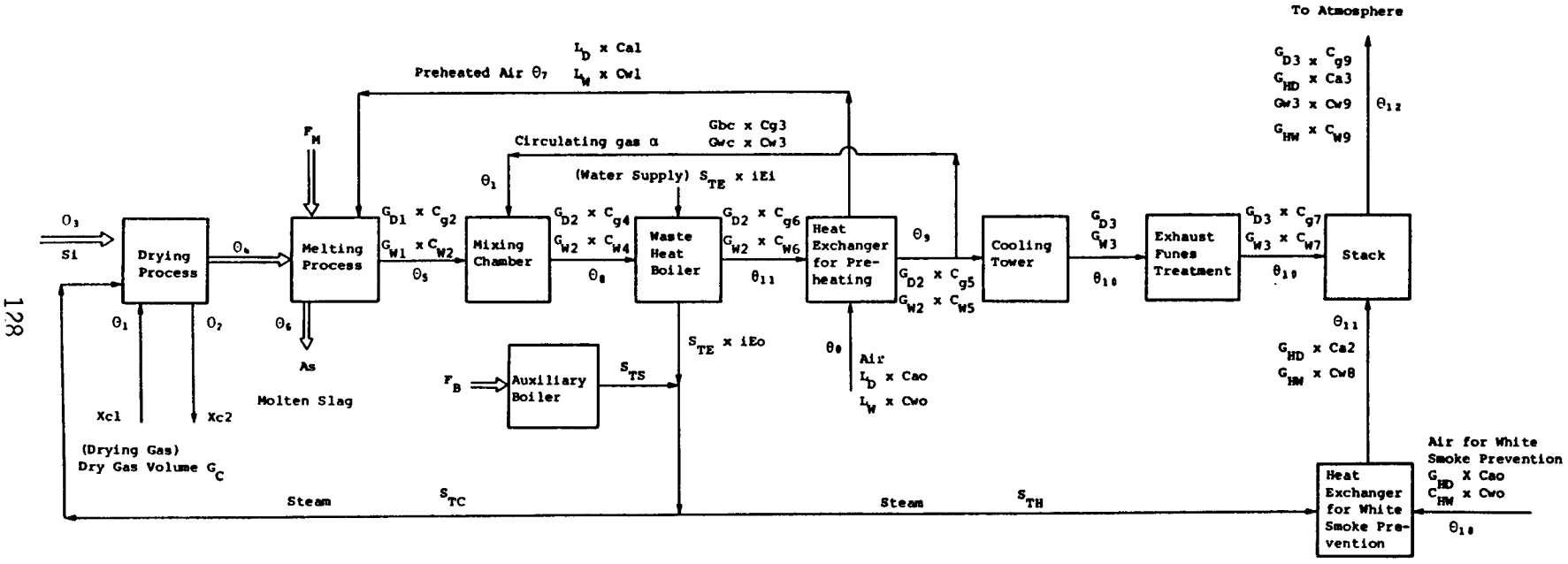


Figure 1 Sludge Melting System

3. PROPOSED HEAT BALANCE MODEL FOR THE MELTING SYSTEMS

The model melting furnace shown in Figure 1 was used to evaluate the auxiliary fuel consumption of the melting system. The operational conditions and the system variables for evaluation are given below on the basis of the operation results of the pilot plant.

3.1 Operational Conditions of Melting Systems

(1) Air Ratio ($1+\phi_S$) for Melting Furnace

If the melting furnace does not commence autogenous combustion or does not reach the melting temperature, the auxiliary fuel consumption (Fm) is calculated on the basis of the excess air ratio (ϕ_S) for cake melting fixed at 0.2. In the case where the melting furnace is capable of supporting autogenous combustion, the value of ϕ_S is increased to cool the furnace in order that the melting temperature does not exceed the set value.

(2) Steam Supply from Auxiliary Boiler

While the steam required for the dryer is supplied by the waste heat boiler, an auxiliary boiler is installed to provide an additional source of steam in the case of a shortage of the steam supply from the waste heat boiler. The required steam volume from the auxiliary boiler (S_{TS}) is then calculated.

(3) Cake Melting Temperature (θ_5)

Since the cake melting temperature varies with the cake composition, it is examined for the temperature range of 1,400-1,600°C.

(4) Mixing Chambers

The exhaust gas from the melting furnace is mixed with gas from the heat exchanger to prevent white smoke in order that the

gas temperature at the waste heat boiler inlet (θ_8) is as low as 1,000°C to stabilize the operation of the waste heat boiler and provide boiler protection from a thermal point of view. The ratio of the circulating gas with respect to the exhaust gas volume from the melting furnace is given as α.

(5) Heat Recovery

While the heat to produce steam for the dryer is recovered at the waste heat boiler by means of the gas-to-liquid heat exchange method, the heat for circulating air and white smoke prevention is recovered from the melting furnace exhaust fumes by the gas-to-gas heat exchange method.

(6) Dryer

An indirect type of dryer which utilizes the steam from the waste heat boiler is used with a fixed steam pressure of 7 kg/cm^{2}G.

3.2 Basic Values for Mass and Heat Balances

(1) Baseline for Balance Calculations

The lower calorific value is used as the baseline with a standard temperature of 0°C for the heat balance.

(2) Atmospheric Conditions

The standard temperature of the atmosphere and sludge is given as 20°C.

(3) Latent Heat of Vaporization

The value of 586 Kcal/kg is used to give the latent heat of vaporization for steam at 20°C.

(4) Specific Heat

The specific heat of each substance used in the balance calculation is given in Table 2.

Table 2 Specific Heat of Each Substance

Substance	Air	Steam		Exhaust gas	Dry solid in sludge	Melted slag
		20-200°C	at 250°C	at 250°C		
Specific heat (Kcal/kg·°C)	Ca	C_{W1}	C_{W2}	Cg	Cd	Cas
	0.24	0.445	0.453	0.252	0.4	0.4

(5) Calorific Values of Cake and Auxiliary Fuel

The calorific values of the cake and the auxiliary fuel used for heat balance calculation are given by the following equation.

Lower Calorific Value of Cake ..... (1)

H = 51.8 V (Kcal/kg)

V: Ignition loss of dried cake solids (%)

Lower Calorific Value of Auxiliary Fuel

Heavy Oil (A)

Density : 0.86 (kg/ℓ)

Lower Calorific Value: 10200 (Kcal/ℓ)

(6) Required Air Volume for Melting and Combustion Gas Volume

Theoretical Air Volume Required for Melting

Using the Rosin-Fehling formula, the following equations are obtained.

<Cake>

$$L_{SG} = \frac{1.10H_{\ell}}{1,000} \cdot \rho \text{ (kg/kg)} \quad \text{..... (2)}$$

<Auxiliary Fuel>

$$L_{FG} = (\frac{0.85H\ell}{1,000} + 2.0) \cdot \rho \ (kg/kg) \quad \ldots\ldots (3)$$

Here, $H\ell$: Lower Calorific Value (Kcal/kg)

ρ : Air Density (kg/Nm^3)

Air Volume Required for Melting Furnace

The excess air ratio with respect to the cake in the furnace is given as ϕ_S and the excess air ratio with respect to the auxiliary fuel is given as ϕ_F.

<Cake>

$$L_{SD} = (1 + \phi_S)\ L_{SG} \ (kg/kg) \quad \ldots\ldots (4)$$

<Auxiliary Fuel>

$$L_{FD} = (1 + \phi_F)\ L_{FG} \ (kg/kg) \quad \ldots\ldots (5)$$

Here, the value of ϕ_F is given as 0.05.

Theoretical offgas Volume Produced by Melting Furnace

Using the Rosin-Fehling formula, the following equations are obtained.

<Cake>

$$G_{SG} = (\frac{1.1H\ell}{1,000} + 0.4) \cdot \rho \ (kg/kg) \quad \ldots\ldots (6)$$

<Auxiliary Fuel>

$$G_{FG} = \frac{1.11H\ell}{1,000} \cdot \rho \ (kg/kg) \quad \ldots\ldots (7)$$

(7) Heat Loss and Efficiency

The following values are used to give the heat loss, etc. of the sludge melting system.

Heat Loss at Dryer (ℓ_d) (with respect to Heat Input) 5%

Heat Loss at Melting Furnace (ℓ_m)
(with respect to Heat Input) 15%

Heat Loss at Waste Heat Boiler (ℓ_b)
(with respe-t to Exchanged Heat) 5%

Efficiency of Auxiliary Boiler (Eb)
(with respect to Calorific Value of Heavy Oil) 86%

Heat Loss at Heat Exchanger for Preheating Air (ℓ_1)
(with respect to Exchanged Heat) 5%

Heat Loss at Heat Exchange for White Smoke
Prevention (ℓ_2) (with respect to Exchanged Heat) 5%

3.3 Symbols Used for Mass and Heat Balances

The symbols used for the mass and heat balances are shown in Table 3.

3.4 Heat Balance Equations for Sludge Melting Systems

(1) Drying Process

The steam volume required for the drying process (S_{TC}) is given by the following heat balance equation.

<Heat Input>

① Latent Heat of Steam (Q_{i1}) : $\gamma c \cdot S_{TC}$ (Kcal/hr)

② Calorific Value of Carrier Gas (Q_{i2}): $i_{c1} \cdot G_C$ (Kcal/hr)

Total Heat Input (Q_i) : $Q_{i1} + Q_{i2}$ (Kcal/hr)

Table 3 Symbols Used for Mass Heat Balance

S	Si (kg/hr)	Cake input to dryer	F	F_M (kg/hr)	Auxiliary fuel consumption by furnace	G	G_{D1} (kg/hr)	Dry gas weight from furnace
	So (kg/hr)	Cake output from dryer (= Cake input to melting furnace)		F_B (kg/hr)	Auxiliary fuel consumption by auxiliary boiler		G_{SD} (kg/hr)	Dry gas weight generated by cake melting
D	D (kg/hr)	Dry solids in cake		F (kg/hr)	Auxiliary fuel consumption $F = F_M + F_B$		G_{FD} (kg/hr)	Dry gas weight generated by auxiliary fuel combustion
W	Wi (kg/hr)	Water volume in cake supplied to dryer	f	f_M (kg/kg-DS)	Auxiliary fuel consumption per dry solids unit by furnace		G_{W1} (kg/hr)	Steam weight in gas from furnace
	ΔW (kg/hr)	Volume of evaporated water in dryer		f_B (kg/kg-DS)	Auxiliary fuel consumption per dry solids unit by auxiliary boiler		G_{SW} (kg/hr)	Steam weight generated by cake melting
	Wo (kg/hr)	Water weight in cake from dryer (= Water weight in cake supplied to melting furnace)		f (kg/kg-DS)	Auxiliary fuel consumption per dry solids unit $f=f_M+f_B$		G_{SSG} (kg/hr)	Theoretical steam weight generated by cake combustion Given as $G_{SSG}/G_{SG} = 0.085$
ω	ω_i (%)	Moisture content in cake supplied to dryer	L	L_{SD} (kg/hr)	Dry air weight for cake melting $L_{SD} = L_{SG}(1+\phi_S)\ V_S$		G_{FW} (kg/hr)	Steam weight in gas generated by fuel combustion
	ω_o (%)	Moisture content in cake from dryer (= Moisture content in cake supplied to melting furnace)		L_{SW} (kg/hr)	Steam weight in cake melting air $L_{SW} = X_A L_{SD}$		G_{FSG} (kg/hr)	Theoretical stream weight generated by fuel combustion Given as $G_{FSG}/C_{FG} = 0.081$
V	V (%)	Ignition loss of cake		L_{FD} (kg/hr)	Air weight for auxiliary fuel combustion $L_{FD} = L_{FG}(1+\phi_F)F_M$		G_C (kg/hr)	Dry carrier gas weight for dryer
	V_S (kg/hr)	Volatile solids $V_S = D(V/100)$		L_{FW} (kg/hr)	Steam weight in auxiliary fuel combustion air $L_{FW} = X_A L_{FD}$		G_{DC} (kg/hr)	Circulation weight of carrier gas
A	A_s (kg/hr)	Weight of slag $A_S = D - V_S = D(1-V/100)$		L_{SG} (kg/kg)	Air weight theoretically required for cake combustion		G_{WC} (kg/hr)	Circulation weight of steam
ϕ	ϕ_S (-)	Excess air ratio for cake melting		L_{FG} (kg/kg)	Air weight theoretically required for auxiliary fuel combustion $L_D = L_{SD} + L_{FD}$, $L_W = L_{SW} + L_{FW}$		G_{D2} (kg/hr)	Dry gas weight at waste heat boiler inlet
	ϕ_F (-)	Excess air ratio for auxiliary fuel combustion					G_{W2} (kg/hr)	Steam weight at waste heat boiler inlet
	$(1-\phi_S)$ (-)	Air ratio for cake melting					α (kg/kg)	Ratio of circulating gas (vis-a vis gas weight from furnace)
	$(1+\phi_F)$ (-)	Air ratio for auxiliary fuel combustion					G_{HD} (kg/hr)	Dry gas weight in white smoke prevention air
							G_{HW} (kg/hr)	Steam weight in white smoke prevention air

Table 3 - Continued

X	X_A (kg/kg)	Absolute humidity of standard air (°C, mb) given as 0.01	
	X_{C1} (kg/kg)	Absolute humidity at carrier gas dryer inlet	
	X_{C2} (kg/kg)	Absolute humidity at carrier gas dryer outlet	
	X_C (kg/kg)	Absolute humidity of wet saturated air at 10°C	
	X_H (kg/kg)	Absolute humidity of exhaust gas for white smoke prevention	
i · γ	i_{EI} (Kcal/kg)	Enthalpy of water supplied to waste heat and auxiliary boilers	
	i_{EO} (Kcal/kg)	Enthalpy of steam generated by waste heat and auxiliary boilers	
	γ_C (Kcal/kg)	Latent heat of steam in dryer	
	i_C (Kcal/kg)	Enthalpy of steam in dryer	
	i_{C1} (Kcal/kg)	Enthalpy of carrier gas at dryer inlet	
	i_{C2} (Kcal/kg)	Enthalpy of carrier gas at dryer outlet	
	γ_W (Kcal/kg)	Latent heat of vaporization of cake moisture content	
	γ_M (Kcal/kg)	Latent heat of melting of slag	
C	Ca (Kcal/kg°C)	Specific heat of air	
	Cw (Kcal/kg°C)	Specific heat of steam	
	Cg (Kcal/kg°C)	Specific heat of exhaust gas	
	Cd (Kcal/kg°C)	Specific heat of dry solid	
	Cas (Kcal/kg°C)	Specific heat of either slag or ashes	
ℓ	ℓd (%)	Heat loss at dryer (vis-a-vis heat input)	
	ℓm (%)	Heat loss at furnace (vis-a-vis heat input)	
	ℓb (%)	Heat loss at waste heat boiler (vis-a-vis exchanged heat)	
	Eb (%)	Efficiency of auxiliary boiler (vis-a-vis calorific value of heavy oil)	
	ℓ1 (%)	Efficiency of air heater (vis-a-vis exchanged heat)	
	ℓ2 (%)	Efficiency of white smoke prevention unit (vis-a-vis exchanged heat)	
S	S_{TE} (kg/hr)	Steam weight generated by waste heat boiler	
	S_{TS} (kg/hr)	Steam weight generated by auxiliary boiler	
	S_{TC} (kg/hr)	Required steam weight by dryer	
	S_{TH} (kg/hr)	Required steam weight for white smoke prevention	
θ	θ0 (°C)	Atomospheric temperature	
	(Dryer)		
	θ1 (°C)	Temperature at inlet	
	θ2 (°C)	Temperature at outlet	
	θ3 (°C)	Cake temperature at inlet	
	θ4 (°C)	Cake temperature at outlet	
	(Melting furnace)		
	θ5 (°C)	Temperature inside furnace	
	θ6 (°C)	Slag temperature at furnace outlet	
	θ7 (°C)	Temperature of preheated air	
	θ8 (°C)	Gas temperature at waste heat boiler inlet	
	(Cooling tower)		
	θ9 (°C)	Temperature at inlet	
	θ10 (°C)	Temperature at outlet	
	θ11 (°C)	Gas temperature at waste heat boiler outlet	
	θ12 (°C)	Gas temperature at stack outlet	
	θ13 (°C)	Temperature of white smoke prevention air after heating	

<Heat Output>

①	Sensible Heat of dried Cake (Q_{01})	: $(Cd \cdot D + Wo)\theta_4$	(Kcal/hr)
②	Calorific Value of Carrier Gas (Q_{02}):	$i_{c2} \cdot G_C$	(Kcal/hr)
③	Heat Radiation Loss (Q_{03})	: $Qi \cdot \ell d/100$	(Kcal/hr)
	Total Heat Output (Q_0):	$Q_{01} + Q_{02} + Q_{03}$	(Kcal/hr)

As the heat input equals the heat output ($Q_i = Q_0$),

$$S_{TC} = \frac{(Cd \cdot D + Wo) \cdot \theta_4 + \{i_{C2} - (1 - \ell d/100) \cdot i_{C1}\} G_C}{(1 - d/100)_C} \text{ (Kcal/hr)} \quad ..(8)$$

(2) Melting Process

The auxiliary fuel weight required for the melting process (F_M) and the cake-to-air ratio are obtained from the following heat balance equations.

<Heat Input>

①	Calorific Value of Cake (Q_{i1})	: $H_\ell D$	(Kcal/hr)
②	Calorific Value of Sensible Heat of Combustion Air (Q_{i2})	: $[(L_{SD} + L_{FD}) \cdot Cal + (L_{SW} + L_{FW}) \cdot (C_{W1})]\theta_7$	(Kcal/hr)
③	Calorifica Value of Auxiliary Fuel (Q_{i3}):	$H_F \cdot F_M$	(Kcal/hr)
	Total Heat Input (Q_i):	$Q_{i1} + Q_{i2} + Q_{i3}$	(Kcal/hr)

<Heat Output>

①	Sensible and Latent Heats of Slag (Q_{01}):	$(\gamma m + Cas \cdot \theta_6) \cdot As$	(Kcal/hr)
②	Latent Heat of Vaporization of Cake Moisture Content (Q_{02})	: $\gamma_W \cdot W_0$	(Kcal/hr)
③	Calorific Value of Offgas (Q_{03})	: $[(G_{SD} + G_{FD}) \cdot Cg_2 + (G_{SW} + G_{FW}) \cdot C_{W2}] \cdot \theta_5$	(Kcal/hr)
④	Heat Radiation Loss (Q_{04})	: $Q_i \cdot \ell m/100$	(Kcal/hr)
	Total Heat Output (Q_0):	$Q_{01} + Q_{02} + Q_{03} + Q_{04}$	(Kcal/hr)

As $Q_i = Q_0$

F_M is calculated based on the value of ϕ_s at 0.2 (where $F_M \geqq 0$). If $F_M < 0$ when the value of ϕ_s is given as 0.2, the new value of ϕ_s is calculated based on $F_M = 0$, $L_{FD} = 0$, $L_{FW} = 0$, $G_{FD} = 0$ and $G_{FW} = 0$.

(3) Mixing Chamber

The circulating gas weight is calculated as follows supposing that the mixture of circulating gas and offgas from melting furnace has the temperature of θ_8 (°C). α is the ratio of circulating gas to offgas at the furnace exit in weight.

(i) Equation of Gas Mixing

$$(G_{D1} \cdot C_{g2} + G_{W1} \cdot G_{W2}) \cdot \theta_5 + \alpha\ (G_{D1} \cdot C_{g3} + G_{W1} \cdot C_{W3}) \cdot \theta_9 = (1 + \alpha)\ (G_{D1} \cdot C_{g4} + G_{W1} \cdot G_{W4}) \cdot \theta_8$$

$$\alpha = \frac{(C_{g4} \cdot \theta_8 - C_{g2} \cdot \theta_5) \cdot G_{D1} + (C_{W1} \cdot \theta_8 - C_{W2} \cdot \theta_5) \cdot G_{W1}}{(C_{g3} \cdot \theta_9 - C_{g4} \cdot \theta_8) \cdot G_{D1} + (C_{W3} \cdot \theta_9 - C_{W4} \cdot \theta_8) \cdot G_{W1}} \quad \ldots (9)$$

(ii) Weight of Gases

Weight of Circulation Gas

Dry Gas :	$G_{DC} = \alpha \cdot G_{D1}$	(kg/hr)
Moisture Content:	$G_{WC} = \alpha \cdot G_{W1}$	(kg/hr)

Weight of Gas Input to Waste Heat Boiler

Dry Gas :	$G_{D2} = (1 + \alpha) \cdot G_{D1}$	(kg/hr)
Moisture Content:	$G_{W2} = (1 + \alpha) \cdot G_{W1}$	(kg/hr)

(4) Weight of Gas Output from Cooling Tower

The absolute humidity of the gas flowing from the cooling tower (Xo kg/kg) is obtained by comparing the absolute humidity of wet saturated air Xc (kg/kg) of θ_{10} (°C) at the cooling tower outlet with the absolute humidity of the gas flowing into the cooling tower Xi (kg/kg).

$Xi = G_{W1}/G_{D1}$ (kg/kg)

If $Xx > Xi$, $Xo = Xi$

If $Xc < Xi$, $Xo = Xc$

Therefore, the weight of gas output from the cooling tower is:

Dry Gas	:	$G_{D3} = G_{D1}$	(kg/hr)
Moisture Content	:	$G_{W3} = G_{D1} \cdot Xo$	(kg/hr)

(5) Air Heater

The gas temperature at the air heater inlet is given by the following heat balance.

<Heat Input>

① Calorific Value of Exhaust Gas (Q_{i1})	:	$(G_{D2} \cdot C_{g6} + G_{W2} \cdot C_{W6}) \cdot \theta_{11}$	(Kcal/hr)
② Calorific Value of Air (Q_{i2})	:	$(L_D \cdot C_{ao} + L_W \cdot C_{W0}) \cdot \theta_0$	(Kcal/hr)
Total Heat Input (Q_i)	:	$Q_{i1} + Q_{i2}$	(Kcal/hr)

<Heat Output>

① Calorific Value of Offgas (Q_{01})	:	$(G_{D2} \cdot C_{g5} + G_{W2} \cdot C_{W5}) \cdot \theta_7$	(Kcal/hr)
② Preheating Air (Q_{02})	:	$(L_D \cdot C_{a1} + L_W \cdot C_{W1}) \cdot \theta_7$	(Kcal/hr)
③ Heat Radiation Loss (Q_{03})	:	$(Q_{02} - Q_{i2}) \ell_1/100$	(Kcal/hr)
Total Heat Output (Q_0)	:	$Q_0 = Q_{01} + Q_{02} + Q_{03}$	(Kcal/hr)

The value of θ_{11} (°C) can be obtained as $Q_i = Q_0$.

(6) Waste Heat Boiler

The volume of steam generated by the waste heat boiler (S_{TE}) is given by the following equation.

<Heat Input>

① Calorific Value of Offgas (Q_{i1})	: $(G_{E2}\cdot C_{g4} + G_{W2}\cdot C_{W1})\cdot\theta_8$	(Kcal/hr)
② Calorific Value of Supplied Water (Q_{i2})	: $i_{E2}\cdot S_{TE}$	(Kcal/hr)
Total Heat Input (Q_i)	: $Q_i = Q_{i1} + Q_{i2}$	(Kcal/hr)

<Heat Output>

① Calorific Value of Offgas (Q_{01})	: $(G_{D2}\cdot C_{g6} + G_{W2}\cdot C_{W6})\cdot\theta_{11}$	(Kcal/hr)
② Calorific Value of Steam (Q_{02})	: $i_{E0}\cdot S_{TE}$	(Kcal/hr)
③ Heat Radiation Loss (Q_{03})	: $(Q_{02} - Q_{i2})\ell_b/100$	(Kcal/hr)
Total Heat Output (Q_0)	: $Q_0 = Q_{01} + Q_{02} + Q_{03}$	(Kcal/hr)

The value of S_{TE} (Kcal/hr) is given as $Q_i = Q_0$.

(7) Heat Exchanger for White Smoke Prevention

The air weight required for white smoke prevention (G_{HD}), the temperature of the air for white smoke prevention (θ_{13}) and the required steam weight (S_{TH}) are given by the following mass and heat balance equations.

① Air weight for White Smoke Prevention G_{HD} (kg/hr):

Given the air gas and moisture content of the air, shown as G_{HD} and G_{HW} respectively, and the absolute humidity of the exhaust gas after the generation of white smoke X_H (kg/kg) is prevented, supposing that the air weight is classified into dry gas weight (G_{HD}) and moisture ($G_{HW} = G_{HD}\cdot X_A$) and that the absolute humidity of offgas after white smoke prevention is X_H (kg/kg) the following equation is obtained.

$$\frac{G_{W3} + G_{HD}}{G_{D3} + G_{HW}} = X_H \quad \text{Therefore, } G_{HD} = \frac{G_{D3} \cdot X_H - G_{W3}}{X_A - X_H} \qquad \ldots (10)$$

② Temperature of Air for White Smoke Prevention θ_{13} (°C):

The value of θ_{13} is given by the heat balance of the calorific value of the gas as follows.

$$(G_{D3} \cdot G_{g7} + G_{W3} \cdot C_{W7}) \cdot \theta_{10} + (G_{HD} \cdot C_{a2} + G_{HW} \cdot C_{W8}) \theta_{13}$$
$$= [G_{D3} \cdot C_{g9} + G_{HD} \cdot C_{a3} + (G_{W3} + G_{HW}) \cdot C_{W9}] \cdot \theta_{12}$$

Therefore θ_{13} =

$$\frac{[GD_3 \cdot C_{g9} + G_{HD} \cdot C_{a3} + (C_{W3} + G_{HW}) \cdot C_{W9}] \cdot \theta_{12} - (G_{D3} \cdot C_{g7} + G_{W3} \cdot C_{W7}) \cdot \theta_{10}}{G_{HD} \cdot C_{a2} + G_{HW} \cdot C_{W8}} \qquad (11)$$

③ Required Volume of Steam S_{TH} (kg/hr):

<Heat Input>

①	Calorific Value of Steam (Q_{i1}):	$\gamma_C \cdot S_{TH}$	(Kcal/hr)
②	Calorific Value of Air (Q_{i2}) :	$(H_D \cdot C_{a0} + H_W \cdot C_{W0}) \cdot \theta_0$	(Kcal/hr)
	Total Heat Input (Q_i):	$Q_i = Q_{i1} + Q_{i2}$	(Kcal/hr)

<Heat Output>

①	Calorific Value of Air (Q_{01}) :	$(H_D \cdot C_{a2} + H_W \cdot C_{W8}) \cdot \theta_{13}$	(Kcal/hr)
②	Heat Radiation Loss (Q_{02}) :	$(Q_{01} - Q_{i2}) \ell_2 / 100$	(Kcal/hr)
	Total Heat Output (Q_0):	$Q_0 = Q_{01} + Q_{02}$	(Kcal/hr)

The value of S_{TH} is given as $Q_i = Q_o$.

(8) Steam Balance

The steam balances for the processes (1)-(8) are given as follows.

Requirements

Steam Weight Required by Dryer	S_{TC} (kg/hr)
Steam Weight Required by Heat Exchanger for White Smoke Prevention	S_{TH} (kg/hr)

Weight Generated

Steam Weight Generated by Waste Heat Boiler	S_{TE} (kg/hr)
Steam Weight Generated by Auxiliary Boiler	S_{TS} (kg/hr)

$$S_{TC} + S_{TH} = S_{TE} + S_{TS}$$

(9) Auxiliary Boiler

If $S_{TC} + S_{TH} > S_{TE}$, the fuel consumption by the auxiliary boiler (F_B kg/hr) is given by equation (12) below.

$$F_B = \frac{S_{TS}\ (i_{E0} - i_{Ei})\ \times 100}{H_F \cdot E_b} \text{ (kg/hr)} \qquad \ldots\ldots (12)$$

In the following Chapter 4, the fuel consumption by the auxiliary boiler is examined using those heat balance equations obtained in this Chapter.

4. ENERGY EVALUATION OF SLUDGE MELTING SYSTEMS

Based on the heat balance model described in the previous Chapter, the auxiliary fuel consumption by the melting furnace system is examined in terms of the following four parameters. The ranges of the parameter values used in this examination are as follows.

①	Ignition loss (V)	:	40 - 80%
②	Melting Temperatures (θ_5)	:	1,400-1,600°C
③	Moisture Content of Dewatered Cake (ω_i)	:	50-80%
④	Moisture Content of Dried Dewatered Cake (ω_o)	:	0-ω_i%

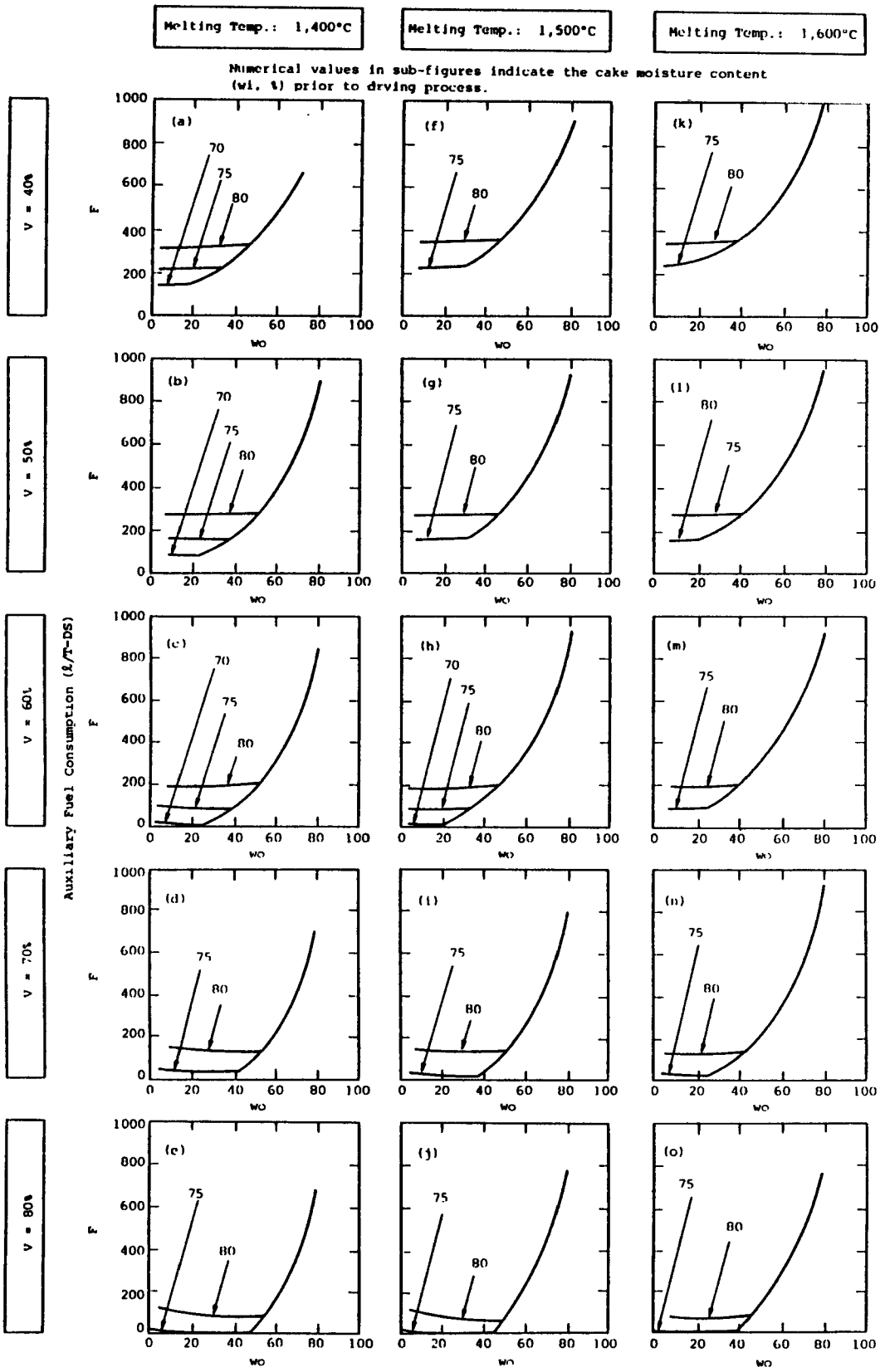


Figure 2 Relationship Between Parameters and Auxiliary Fuel Consumption in Melting System

Figure 2 shows the relationship between those four parameters and the auxiliary fuel consumption. In this Figure, the different degrees of the ignition loss in the cake (V) are plotted against the different melting temperatures (θ_5). When the values of V and θ are given, the corresponding figure showing the relationship between the parameters and the fuel consumption can be found among the sub-figures (a) to (o) by referring to the values in Figure 2. In all the sub-figures, the auxiliary fuel consumption of the melting system (F) is plotted against the cake moisture content (ω_o) at the drying process outlet. The numerical values in each sub-figure indicate the moisture content of the inflow cake to the drying process (ω_i). While those sub-figures located in the upper part of Figure 2 relate to lime-added cakes where the ignition loss is low, those located in the lower part relate to polymer-added cakes where the ignition loss is high. Those figures to the right of Figure 2 related to the high degree of alkalinity due to the lime addition where the melting temperatures are accordingly high. The general relationship between the four parameters and the auxiliary fuel consumption is therefore obtained.

The representative trend of the auxiliary fuel consumption with respect to the cake conditions is examined next.

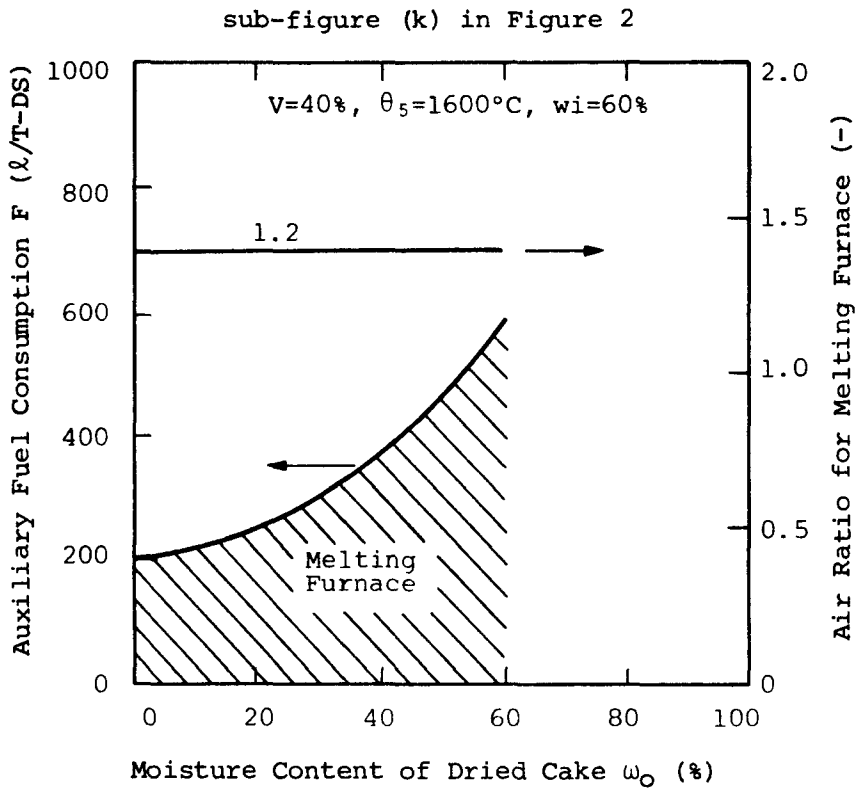


Figure 3 Example of Auxiliary Fuel Consumption of Melting System Using Lime-added Cake

Figure 3 shows the relationship in the case of lime-added cake where both the moisture content (ω_i) and the ignition loss are low and the melting temperature is high. Under these conditions, the total auxiliary fuel consumption by the system can be reduced if the cake moisture content is made as low as possible in the drying process before it is fed to the melting furnace. In this case, the steam weight required by the drying process is reduced using the heat recovered from the offgas from the furnace. However, even if the cake is completely dried, auxiliary fuel is still required at the rate of 80 liters per ton of wet

cake and 200 liters per ton of dry solids. These consumption rates are high compared with the 67 liters per ton of wet cake that is consumed when filter press dematered cake is incinerated in a multiple hearth furnace with an afterburner.

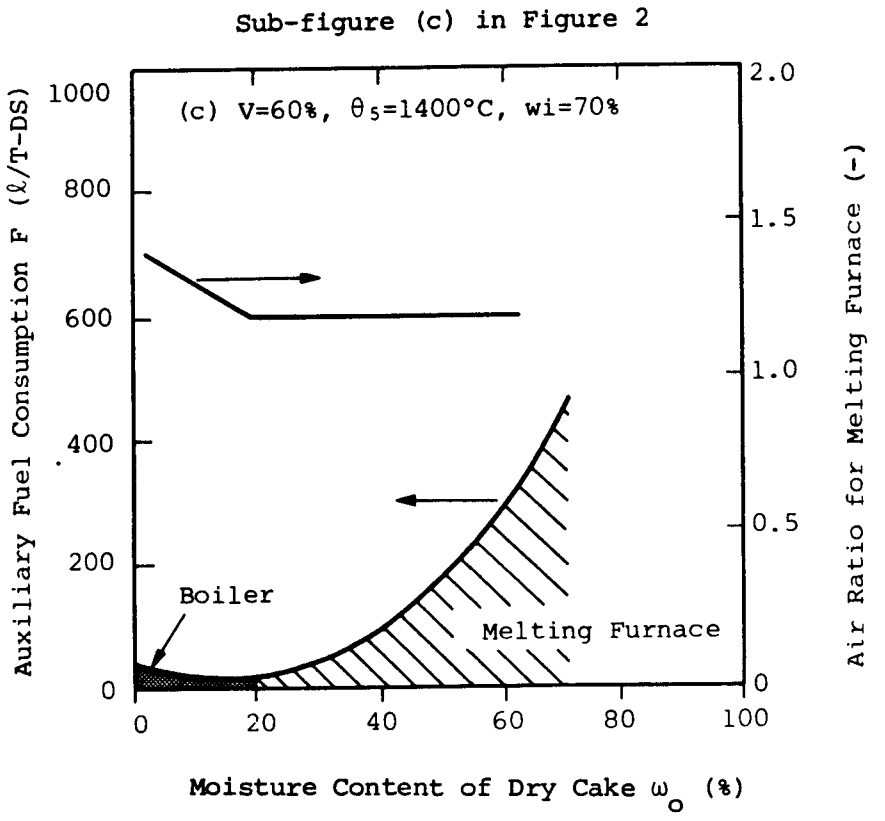


Figure 4 Optimum Conditions of Operation to Minimize Auxiliary Fuel Consumption of Melting System

Figures 4 shows the auxiliary fuel consumption for polymer-added cake (V = 60%, ω_i = 70%) at a low melting temperature of 1,400°C. The fuel consumption is lower for the lower cake moisture content down to ω_i of 20%. When the cake is dried to the moisture content of less than 20%, however, the following two phenomena appear.

(1) Since a cake moisture content of less than 20% cannot be achieved by steam from the waste heat recovery process, auxiliary fuel for the auxiliary boiler (F_B) is required to provide supplementary steam.

(2) Since the combustion of the cake with a ω_i of less than 20% increases the temperature in the furnace to over 1,400(°C), the furnace temperature should be controlled at 1,400°C or lower by increasing the air flow to the furnace. Consequently, the air-to-cake ratio in the melting furnace becomes high, resulting in a disadvantageous situation from the energy saving viewpoint.

In other words, under the conditions given in Figure 4 (sub-figure (c) in Figure 2), the optimal operational conditions to minimize the auxiliary fuel consumption can be found. The minimum auxiliary fuel consumption under the optimal conditions is either 4.7 ℓ per ton of wet cake or 15.6 ℓ per ton of dry solids. This is a great improvement on 26.6 ℓ per ton of wet cake in a multiple hearth furnace without an after-burner.

Figure 5 gives an example of rather high ω_i of 80% due to the low dewaterability, in turn due to the high organic content. The results are similar to those shown in Figure 4. However, when the ω_o is between 50-55%, auxiliary fuel is required for both the melting furnace (F_M) and the auxiliary boiler (F_B). Under the optimum operating conditions, the fuel consumption is either 30 litres per ton of wet cake or 60 litres per ton of dry solids, not exceeding the actual consumption shown in Figure 7.

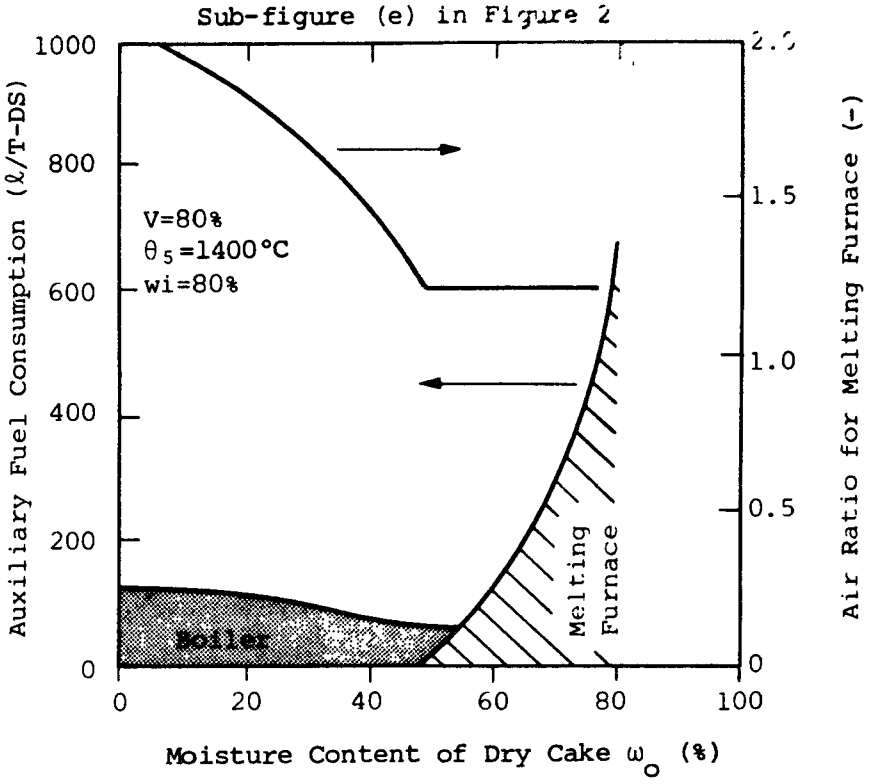


Figure 5 Optimum Conditions of Operation to Auxiliary Fuel Consumption of Melting System

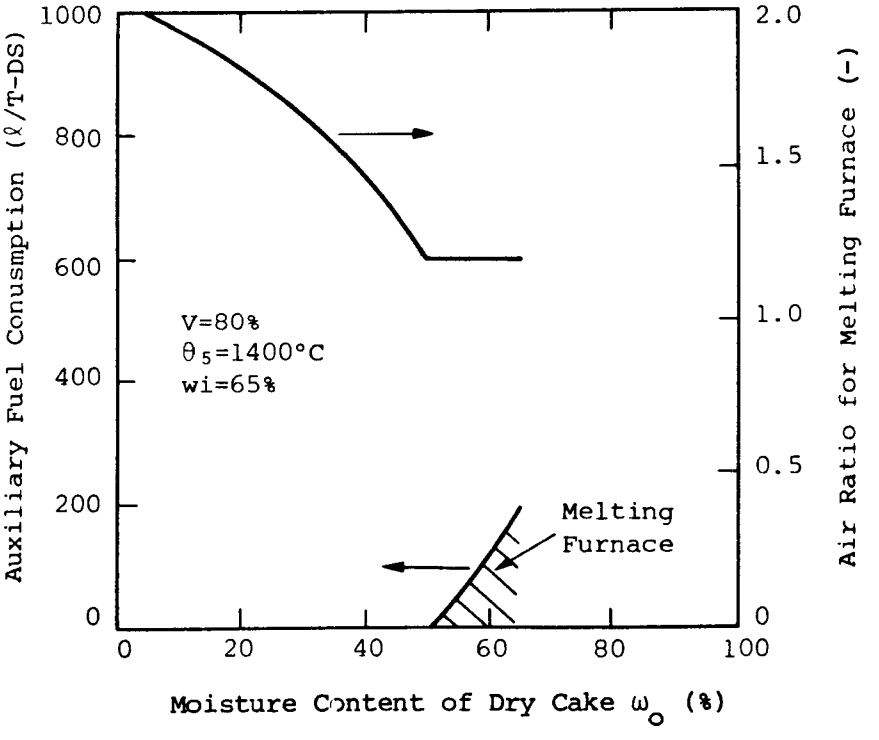


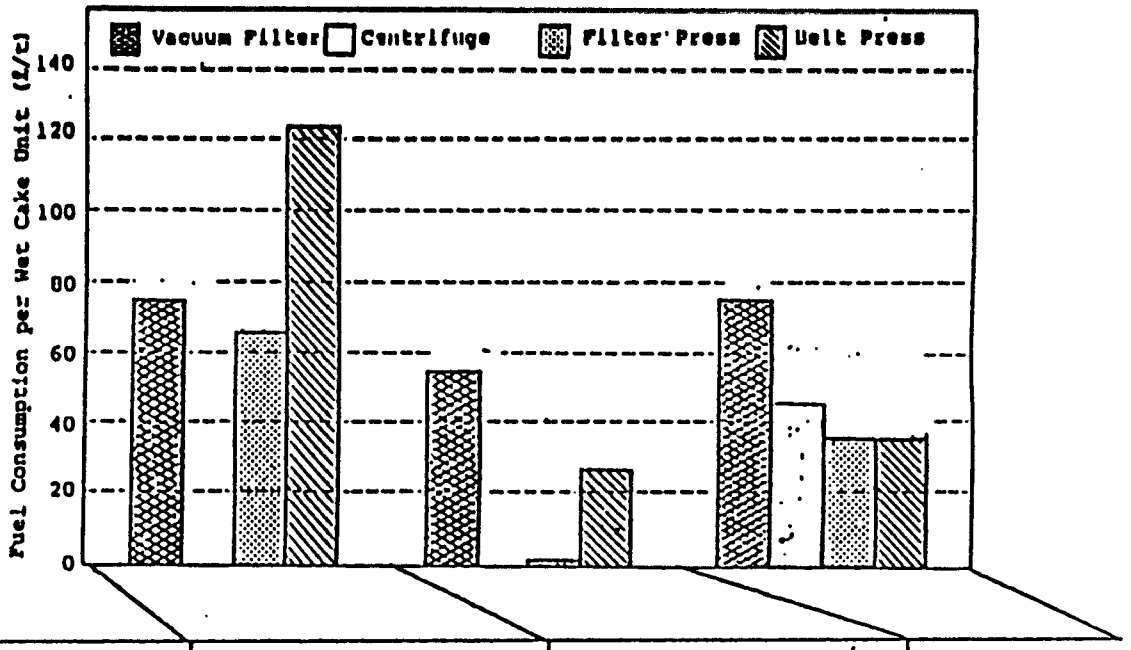
Figure 6 Example of Autogenous Combusion and Melting

Figure 6 gives an example high ignition loss and low moisture content (ω_i). Since these conditions are favourable for the melting operation, melting through autogenous combustion is possible with the value of ω_o at 50% or lower. When the value of ω_o is lower than 50%, however, the excess air ratio in the melting furnace should be raised in order to prevent the furnace temperature from rising too far.

Based on the above examination, the effects of the various parameters on the auxiliary fuel consumption are summarized below.

(1) Moisture Content of Dewatered Cake fed to Drying Process (ω_i)

The ω_i gives much effect on the auxiliary fuel consumption especially when the ω_i value is great. This can be clearly seen in the case where a reduction of 5% in the ω_i, from 80% to 75%, results in a 25% reduction in the water volume.



Furnace type / Dewatering method	Multiple hearth with secondary combustion chamber (ℓ/t)	Multiple hearth without secondary combustion chamber (ℓ/t)	Fluidized bed (ℓ/t)
Vacuum filter	75.1 (Furnace 46.4)	53.6	75.0
Centrifuge	-	-	45.7
Filter press	66.8 (Furnace 19.8)	0.7	35.6
Belt press	123.8 (Furnace 32.2)	26.6	35.9

Figure 7 Fuel Consumption per Cake Unit by Different Furnaces and Dewatering Methods

(2) Moisture Content of Dry Cake (ω_i) fed to Melting Process (ω_i)

It is preferable that the value of ω_i be low enough in the melting furnace to maintain the melting temperature without auxiliary fuel (F_M) requirement (unless the calorific value of the dry solid is primarily low, as shown in sub-figure (k) in Figure 2). However, there is an optimum point at which the total system including the drying and melting processes operates at its most efficient.

(3) Ignition Loss (V)

A greater ignition loss gives a lower fuel consumption. In reality, however, when the value of V is high, the value of ω_i is also high. In the case of dewatering by highly efficient belt press for example, autogenous combustion melting somewhat difficult to achieve as the values of V and ω_i are 70% and 75%, respectively.

(4) Melting Temperature (θ_5)

High basicity requires a higher melting temperature that results in higher auxiliary fuel consumption (F_M). At present, the study of additives to lower the basicity and thus to lower the melting temperature is in progress.

5. CONCLUSION

As heat is quantitatively dealt with in this paper, the thermal irreversibility in relation to high or low temperatures is not considered. However, the experiment proved that the quantitative treatment of heat does not cause any problems in a temperature range of up to 1,600°C in the melting furnace.

Electricity, water and chemicals, etc. should be considered as utilities for the melting system in addition to an auxiliary fuel. This will remain the subject of future study. The electricity consumption of the drying and offgas treatment processes in particular is expected to be much larger than that in the melting process.

In addition, the durability of various equipment, optimum operational conditions and a combination of unit processes to achieve an efficient system should also be carefully examined in the future.

This paper is concluded as follows.

(1) A heat balance model for the sludge melting system has been successfully proposed.

(2) The auxiliary fuel consumption is possibly calculated using the four parameters, i.e, V, ω_o, ω_i and θ_5 in the model.

(3) Consequently, optimum operational conditions and preferable cake characteristics will be obtained for an energy-saving melting system.

REUSE OF TREATED WASTEWATER IN FUKUOKA CITY

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

HIROSHI ONIKI

DIRECTOR, PLANNING DIVISION

SEWAGE WORKS BUREAU

FUKUOKA CITY

CONTENTS

1. Preface 155

2. Reclamation of Treated Sewage in Japan 155

3. Outline of Sewage System in Fukuoka City 156
   3.1 Outline of Fukuoka City 156
   3.2 Sewage System in Fukuoka City 156

4. Water Demand/Supply and Its Prospectus 158

5. Water Saving City 159
   5.1 Water Shortage in Fukuoka City 159
   5.2 Water Saving Policy of Fukuoka City 162
   5.3 Promotion of Miscellaneous Use Waterworks 162
   5.4 Present Situation of the Miscellaneous Use Waterworks 163

6. Efficient Use of Treated Sewage 167

   6.1 Treated Sewage in Fukuoka City 167

7. Model Project for a Wastewater Reclamation and Reused System 168
   7.1 Purpose 168
   7.2 Outline of the Model Project 168
   7.3 Operation 171
   7.4 Consciousness of the Users 176
   7.5 Prevention of Misconnecting Pipe 177
   7.6 Service Charge 178
   7.7 Follow-up Survey 179

8. Expansion Program of Model Project for a Water Reclamation and Reused System 187
   8.1 Background 187
   8.2 Program Planning 187
   8.3 Facility Plan 193

9. Construction Schedule and Financial Plan 196
   9.1 Construction Schedule 196
   9.2 Financial Plan 196
   9.3 Expansion of Availability 198

10. Conclusion 205

1 PREFACE

Japan is one of the few countries with much rain and its annual precipitation accounts for 1,800 mm, twice as much as that of the world average. However, the amount of the annual rainfall per capita of the country accounts for approximately 6,000 m^3, one sixth of that of the world average. Therefore, it cannot be said that our country is not blessed with water resources.

Especially in Fukuoka City, the amount of the annual rainfall per capita is approximately 500 m^3, one twelfth of that of the national average, which is very low. In the Fukuoka area including Fukuoka City and its surrounding area, there is no large river enough to meet the water demand and there is few underground water geologically. Because of these geographical conditions, the area is not abundant in water resources. In other words, it stands on a fragile structure as regards water supply. In 1978, the city was severely suffered from a heavy water shortage and the citizens were obliged to think of the water problem more seriously.

Since then, a new water policy to realize a water saving city had been needed. As a part of the new policy, we worked on the expansion of the miscellaneous use water supply including the effective use of treated sewage.

This report will explain the present situation and prospectus of the project.

TABLE 1. Condition of precipitation

ITEM	Population (×10,000)	Area (×1,000 km^2)	Annual precipitation (mm/year)	Gross of precipitation (Billion m^3/year)	Quantity of precipitation per capita (m^3/year/capita)
World Average	389,000	135,830	973	132,162	33,975
Japan	11,193	377	1,788	675	6,030
Fukuoka City	117	0.337	1,690	0.6	512

2 RECLAMATION OF TREATED SEWAGE IN JAPAN

Sewage treatment plants in Japan generates the treated sewage of approximately 7.4 billion m^3 annually, and this figure will increase to 31 billion m^3 per year eventually, accompanied by the expansion of sewage system.

This treated sewage is also one of the reliable and abundant water resources which is available in the city.

The integration of growing population and industry in big cities is causing troubles in water supply.

Therefore, the Ministry of Construction is promoting model project for a wastewater reclamation and reused system, which is to reuse treated sewage

for miscellaneous use such as flush toilet water.

This new project has started its service in the Shinjuku urban subcenter, Tokyo, where a lot of high buildings were constructed under the redevelopment plan, and in the Tenjin district, Fukuoka. In addition, in the Makuhari district, Chiba, the Rokko Island district, Kobe, the Yamate village, Okayama, and Takamatsu City, Kagawa, the project are on a way and expecting to start operation within a few years.

3 OUTLINE OF SEWAGE SYSTEM IN FUKUOKA CITY

3.1 OUTLINE OF FUKUOKA CITY

Fukuoka City is located on the southwestern part of the country, at a distance of about 1,100 km from Tokyo. Facing to Hakata Bay in the north, surrounded by mountains in the south, and blessed with a rather moderate climate, the city is favored with natural environment. Also, it has a long history and has developed as an exchange point with China in politics, economy, and culture for about 2,000 years.

In 1889, the city was started with a population of about 50,000 under the organization of municipalities. Now, it is a center of politics and economy in West Japan, covering an area of 340 km^2 with a population of 1,180,000, the 8th largest city in the country.

In order to be an international city, the city government is promoting the ranging exchange with other cities in foreign countries through friendship or sister-city affiliation: Oakland in the United States (in 1962), Kuangchou in China, Bordeaux in France, and Auckland in New Zealand. Moreover, under the slogan of "A Vital City Open to the Asian Pacific", it will hold "the Asian Pacific Exposition in Fukuoka, 1989" to make a rapid progress to the future.

3.2 SEWAGE SYSTEM IN FUKUOKA CITY

The first sewarage in the city was started in 1930, but this was the drain sewer system to drain rainwater and home wastewater.

The real sewerage was started in 1963 when the 1st Five-year Sewerage Improvement Plan was enforced. In recent years, we appropriate 40 billion yen in the budget of the year for the construction of sewage system, taking it as one of the important 3 policies of the city.

As March, 1987, the construction work was completed in the area of 9,689 ha out of the entire planned area of 14,595 ha, serving 830,000 people, a 70% of the total population of the city.

Under the long term plan, we are planning to improve the sewage system in the entire downtown area and to increase the diffusion rate up to 97% by 1995.

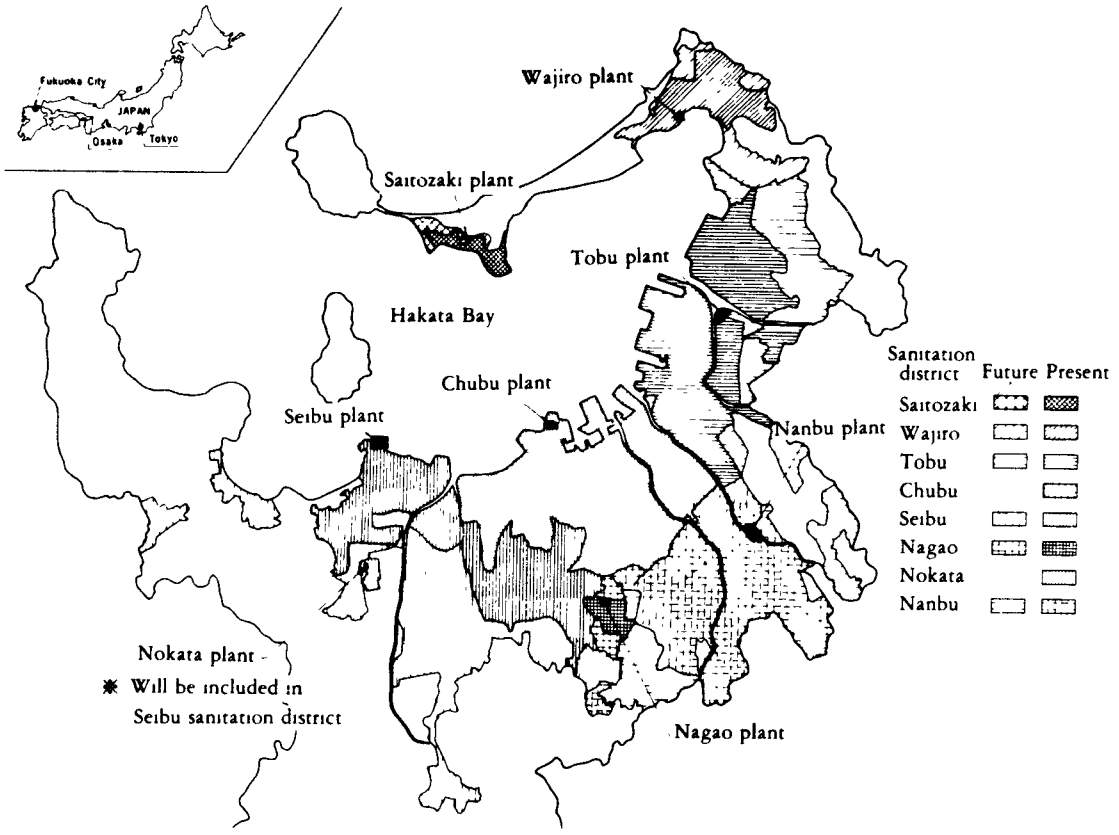


Figure 1. Location map of treatment plants and sewerage districts.

TABLE 2. Service of Fukuoka City

(as of March 1987)

Total area of the city	33,712 ha
Total area of urbanized districts	14,595 ha
Total population	1,172,577
Total service area	9,689 ha
Service area in urbanized districts	66.4 %
Population in sewage service area	829,300
Percentage of population sewage-service area to total population	70.4%
Sewer length	2,620,000 m
Treatment capacities (Total of 7 sewages)	507,100 m^3/ day

4 WATER DEMAND/SUPPLY AND ITS PROSPECTUS

In 1986, the water demand was 436,000 m^3 and there is no significant increase to the consumption of 433,000 m^3 in 1977, a year before we were severely suffered from the heavy water shortage in 1978. We believe that it is because we have been trying to achieve the water saving city. Although the capacity of the existing water supply system is a little more than the demand, the city is still in a severe condition for the stable water supply and can be easily affected by drought because of the following features.

TABLE 3. General Plan for Water Supply and Demand
(Unit: 1,000 m^3/day)

YEAR		1986 F/Y	1995 F/Y
Max. demand per a normal day		633	809
Plan for Facility	Capacity of existing facility	651	651
	Extension plan at present	-	97
	Extension plan in future	-	151
	TOTAL	651	899

TABLE 4. Filtration Plants by water source and their supply cpapcity

(as of March 1987)

Name of Filtration Plant	Supply capacity (m^3/day)
Matsuzaki Filtration Plant	35,000
Takamiya Filtration Plant	199,000
Otogane Filtration Plant	110,500
Meotoishi Filtration Plant	174,000
Zuibaiji River Filtration Plant	15,000
SUB TOTAL	533,500
Supply from Chikugo River (from Egawa & Terauchi Dams)	118,000
TOTAL	651,500
Developing of Water Sources	
Name of Project	Supply capacity (m^3/day)
General Development of Chikugo River	21,800
General Development of Tatara River	74,800 (incl. receiving water 9,800)
TOTAL	96,600
Grand TOTAL	748,100

a. High dependance on small rivers

Not like other cities which depend on the 1st grade rivers with stable and large water flow for their water resources, Fukuoka City depends on the 2nd or lower grade rivers with a shorter length and smaller water flow. Especially during drought, the water flow of the rivers tends to extremely decrease.

b. High dependance on dams

The city depends on dams for its another water resources so that the rate of intake-water limitation tends to become high during drought.

c. Confined groundwater is not available

Groundwater, especially confined groundwater, is a water resource which is hardly affected by a temporary water shortage because it takes a long period to be produced. But in the city the rock formation lies too shallow under the ground to count on as a reliable water resource.

Therefore, it is necessary to secure the existing water resources, to promote water saving campaigns, and to develop new water resources including the reuse of treated sewage, in order to establish a real water saving city and to supply water which is inevitable for the citizens' daily lives and urban activities.

5 WATER SAVING CITY

5.1 WATER SHORTAGE IN FUKUOKA CITY

Although we are short of water resources in the city due to its geographical conditions as mentioned before, we have a considerably large amount of rainfall, which is called Natane-Tuyu (or spring rainy reason) during three months from March to May each year. And the early summer rainy season Tuyu rapidly increases the water level of dams, which makes it possible to supply water during summer time.

TABLE 5. Precipitation
(By record at Fukuoka Weather Bureau)

Month	Average Precipitation (latest 30 years) mm (A)	1977		1978	
		Precipitation mm (B)	(B)/(A) %	Precipitation mm (C)	(C)/(A) %
1	87.0	38.5	44.3	75.0	86.2
2	79.1	48.0	60.7	59.0	74.6
3	91.5	132.0	144.3	62.0	67.8
4	140.6	161.0	144.5	57.0	40.5
5	143.5	211.0	147.0	42.5	29.6
6	259.8	251.5	96.8	353.0	136.9
7	269.6	88.0	32.6	44.5	16.5
8	161.5	120.0	74.3	99.5	61.5
9	205.0	65.0	31.7	125.5	61.2
10	94.3	35.0	37.1	84.5	89.6
11	77.7	163.5	210.4	54.5	70.1
12	74.1	40.0	54.0	80.5	108.6
Total	1,683.7	1,353.5	80.4	1,137.5	67.6

However, the amount of rainfall during the period of September 1977 to May 1978 was merely 599 mm, 61% of 993 mm of the equivalent period of the average year. Especially, it should be noted that the rainfall during the period of March 1978 to May 1978 was 116 mm, 43% of 376 mm of the equivalent period of the average year. This was the lowest record of the rainfall that the Fukuoka Weather Bureau had ever seen. So, the amount of intake-water from rivers decreased. In mid July, 1978, the water storage tremendously decreased to 18% of the capacity so that we could not help limiting the water supply. This limitation lasted for 287 days in total, and we asked other cities to help us with water carts. Workers at the city office worked hard on delivering water in plastic containers to each household as the alternative of the waterworks. The water shortage evaluation - (the water saving rate %) x (water supply limitation days) - was 8,160%. It is said that the evaluation of 1,500% drives 80% of people to their tolerance limit, so that the figure of 8,160% can show how serious the water shortage was to the people in the city.

TABLE 6. Experience of water shortage supply in Fukuoka

| Year | 1978 | | |

5.2 WATER SAVING POLICY OF FUKUOKA CITY

Taking this water shortage as a lesson, the city council has worked hard to realize a water saving city by means of appointing "The General Plan of Water Saving City of Fukuoka" (enforced on February 1, 1979) as well as incorporating "The Promotion of Water Saving" in the City Improvement Master Plan (revised in 1981).

This General Plan clarifies the responsibility of administration, citizens, and companies, as well as promotes the efficient use of water: how to save water, how to deal with drought. It consists of the following sections.

1) Promoting water saving awareness

   Make individual citizens and companies recognize that water is a limited and valuable resource. Publicize the necessity of the thorough water saving.

2) Promoting water saving appliances

   Request the R&D, manufacutring, and sales of water saving appliances such as water saving plunger and flush toilet.
   Appoint certain appliances and promote its use to reduce the water consumption.

3) Leak prevention on water distribution line

   Minimize leaks and realize the efficient distribution by reducing the surplus water pressure in the distribution pipe.

4) Spread of miscellaneous use waterworks

   Promote the miscellaneous use waterworks and the reuse of treated sewage to large buildings.

As a result of these, the successful water saving has been performed by large users such as governmental offices, schools, department stores, and business offices.

5.3 PROMOTION OF MISCELLANEOUS USE WATERWORKS

As a concrete measure to spread the miscellaneous use waterworks, we are also providing the following administrative guidance.

1) Introduction of the miscellaneous use waterworks to the facilities and buildings controlled by the city government.

2) The applicant who plans to build a new large building (with the floor area of more than 5,000 m^2, and the diameter of the water-supplying pipe of more than 50 mm, not including warehouse and parking space) should submit the water saving plan including the miscellaneous use waterworks and the efficient water use for the new building.

3) In order to reuse the treated sewage which is raw water of the miscellaneous use waterworks, the reuse plan should be proposed.

Especially, if the miscellaneous use waterworks with the inbuilding reclaimed wastewater supply system is introduced to large buildings, it will increase the construction and equipment cost of those buildings. Therefore, the Miscellaneous Use Waterworks Financial Aid has been provided to reduce the builders' liability on the interest of the loan for the related facilities (for five years) and on the equivalent property tax (for three years) since 1980. This aid was originally enforced in our city. The total amount of the aid (practiced in 52 cases) has reached to 190 million yen as of March 1987.

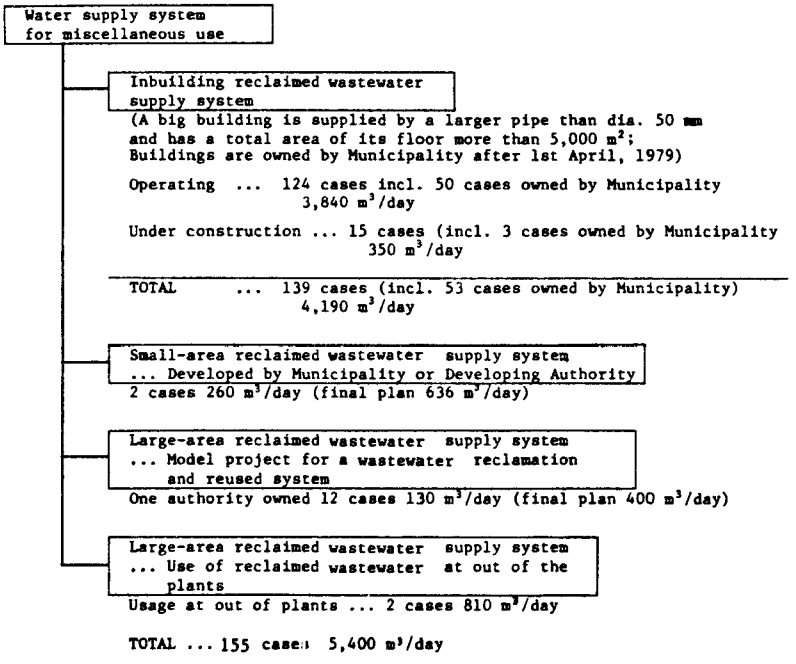
As a result, the miscellaneous use waterworks has been installed in 139 buildings and the amount of reused water has reached to 4,190 m^3/day. Although this figure accounts for less than 1% of the total amount of water supply of 436,000 m^3/day, it is considered to be very significant because it could not be done without cooperation and understanding of citizens and companies.

5.4 PRESENT SITUATION OF THE MISCELLANEOUS USE WATERWORKS

In Fukuoka City, the miscellanous use waterworks was started in 1979, but its use is limited only to flush toilet for the time being. The miscellaneous use waterworks can be divided in three types: inbuilding reclaimed wastewater supply system, small area reclaimed wastewater supply system, and large-area reclaimed waste water supply system.

TABLE 7. Water supply system for miscellaneous use

(as of March, 1987)



5.4.1 INBUILDING RECLAIMED WASTEWATER SUPPLY SYSTEM FOR MISCELLANEOUS REUSE

As of March, 1987, the number of facilities which applied the inbuilding reclaimed wastewater supply system for miscellaneous reuse was 139 in total, 124 in operation (including 50 public facilities) and 15 under construction (including 3 public faciliteis), and it has been increasing by about 20 cases each year. The types of those buildings vary:
business office use (40%), school (30%), and commercial use (20%), accounting for 90%. The amount of reused water at each building varies from 6 m^3/day to 520 m^3/day, but in most cases 20 - 30 m^3/day.

TABLE 8. Progress of inbuilding reclaimed wastewater supply system for miscellaneous reuse

Unit: cases (as of March, 1987)

Use of building \ Completion year	1979	1980	1981	1982	1983	1984	1985	1986 in operation	TOTAL			Classification	
									In operation	Under construct.	TOTAL	Public	Private
Primary, Secondary and High school	4	5	1	4	5	7	3	4	33	1	34	33	1
College & University	-	1	1	1	-	3	1	1	8	-	8	-	8
Public	-	1	-	2	3	-	-	1	7	1	8	8	-
Private	-	1	1	7	6	8	8	10	41	5	46	-	46
Business building	-	2	3	1	1	6	3	2	18	7	25	-	25
* Public	-	-	3	1	3	1	-	-	8	1	9	9	-
* Private	-	3	-	1	-	-	-	-	4	-	4	-	4
** Others	-	2	-	2	-	1	-	-	5	-	5	3	2
TOTAL	4	15	9	19	18	26	15	18	124	15	139	***53	****86

Note: * Facility for public use ... Bus terminal, meeting hall, pool, library & exhibition hall
** Others ... Train inspection plant, sewerage plant & facility for miscellaneous water use in a factory
*** Including 3 cases in under-construction
**** Including 12 cases in under-construction

1) Raw water

3 types: waste water, sewage (in case of the outside area of sewage treatment system), and the combination of wastewater and rain water.

2) Method of treatment

In case of the combination use of wastewater and rain water, physical-chemical treatment is applied to treat the water. Most of the facilities are equipped with filters, disinfection, and activated carbon absorption process. As for sewage, the combination of biological method and physical-chemical method is usually applied. The most popular treatment flow consists of active sludge treatment, contact oxidation, sand filtration, and disinfection.

3) Control

The inbuilding reclaimed wastewater supply system for miscellaneous reuse is mostly maintained and controlled by the contractors in charge of the septic tank control, although it is done partly by the building maintenance service companies. According to the Law of Sanitary Environment for Buildings, the proper control has been done without any deterioration in the performance.

4) Cost

Because the size of the miscellaneous use waterworks is still small, the charge is usually high, more than 1,000¥/m^3, which is rather expensive comparing to that of water supply (750 ¥/m^3 for large users.)

5.4.2 SMALL AREA RECLAIMED WASTEWATER SUPPLY SYSTEM FOR MISCELLANEOUS REUSE

Multi-housing units such as apartments are excepted from the application of the miscellaneous use waterworks in order to balance with ordinary houses. However, newly built municipal housing complex and other housing complex which is newly developed on the reclaimed land are encouraged to be a model case of the application of the miscellaneous use waterworks. The outline of each case is shown in Table 9.

TABLE 9. Existing facilities of small-area reclaimed wastewater supply system for miscellaneous reuse

(as of March, 1987)

	Tamura Residential Zone	Kashiihama Residential Zone
Area	10.6 ha (incl. one school)	90 ha (incl. 5 schools)
No. of House	Planned for 650 Houses (Living 250 in existing 390)	Planned for 5,000 Houses (Living 2,500 houses)
Population	Planned 2,700	Planned 15,000 ... 16,000
Construction period	From 1980 to 1985	From 1979 to 1989
Quantity of miscellaneous water	136 m^3/day (Existing 20 m^3/day)	500 m^3/day (Existing 240 m3/day)
Completion Date	June 1981	April 1983
Owner	Residence Managing Authority	Residence Managing Authority

5.4.3 LARGE AREA RECLAIMED WASTEWATER SUPPLY SYSTEM FOR MISCELLANEOUS REUSE

This is a large scale waterworks to reclaim the secondary treated water at sewage treatment plants, to be used for miscellaneous use water in a large area according to the demand frm companies and households.

Model Project for a wastewater reclamation and reused system, which was enforced in 1979 with cooperation of the Ministry of Construction, uses this method.

6 EFFICIENT USE OF TREATED SEWAGE

As of March, 1987, 600 sewage treatment plants throughout the country generates treated sewage of 7.4 billion m^3 in total annually, and at half of those sewage treatment plants treated sewage is used for defoaming water. 53 plants among them deliver treated sewage to the outside for industrial use and landscape use, accounting for approximately 50 million m^3/year.

6.1 TREATED SEWAGE IN FUKUOKA CITY

In Fukuoka City, 7 sewage treatment plants are in operation and discharging treated sewage of 320,000 m^3/day. Most of the water required at plants, 200,000 m^3, is covered by treated sewage. Also, these plants are distributing the secondary treated water of 1,200 m^3/day to other public facilities such as refuse treatment plant and processed marine product public corporations.

TABLE 10. Outline of using reclaimed wastewater at out of the purification plant

(as of March 1987)

Purification plant	Customer's facility	Reclaimed wastewater quantity	Usage	Remark
Tobu	Tobu refuse treatment plant	254 m^3/day	Flying ash conditioning	Reclaimed in the facility
	Marine product public corporation	908 m^3/day	Deodrizing machinery-washing	

7 MODEL PROJECT FOR A WASTEWATER RECLAMATION AND REUSED SYSTEM

7.1 PURPOSE

Since the heavy water shortage in 1978, treated sewage has been recognized again as a valuable water resource existing in the city. In order to utilize it aggressively, the Model Project for a wastewater reclamation and reused system (following as the Model Project), large area reclaimed wastewater supply system was executed as a sewerage works, firstly in Japan.

7.2 OUTLINE OF THE MODEL PROJECT

The purpose of this Model Project is to reclaim the secondary treated water at the Chubu Sewage Treatment Plant (with the maximum capacity of 350,000 m^3/day, by the activated sludge process) for commercial use, mainly for flush toilet water. The Model Project was enforced in 1979, and a part of it started its operation in June, 1980. Now it supplies the reclaimed wastewater to all 12 planned buildings. The outline and operating result will be reported in the following section.

7.2.1 RECLAMATION METHOD

Closed type large area reclaimed wastewater supply system.

7.2.2 USE

Commercial use.

For commercial use water except drinking water, there are flush toilet water, car washing water, sprinkling water, and air conditioning water. Among them the flush toilet use was chosen to apply to the system, because it can be used without being touched by hands and it is much use.

7.2.3 APPLICABLE FACILITIES

12 public buildings.

Because this method is more applicable to the area with much commercial use water and with the potentiality of efficient water circulation, it applied to 12 public buildings located in the Tenjin area which forms central business area with many large commercial buildings.

7.2.4 RECLAMATION FACILITIES

Sand filtration, ozonation, and chlorination.

The reclamation facilities were selected with full consideration of performance, reliability, cost, maintenance of the planned process, and plottage, on the assumption that the facility should not discomfort the users and should secure their safety.

The main role of each process is as follows.

Sand filtration is to eliminate floatage and to prevent the pipe and equipment from being clogged.

Ozonation is to eliminate the odor, color, and disinfect the turbidity.

And chlorination is to keep disinfection longer.

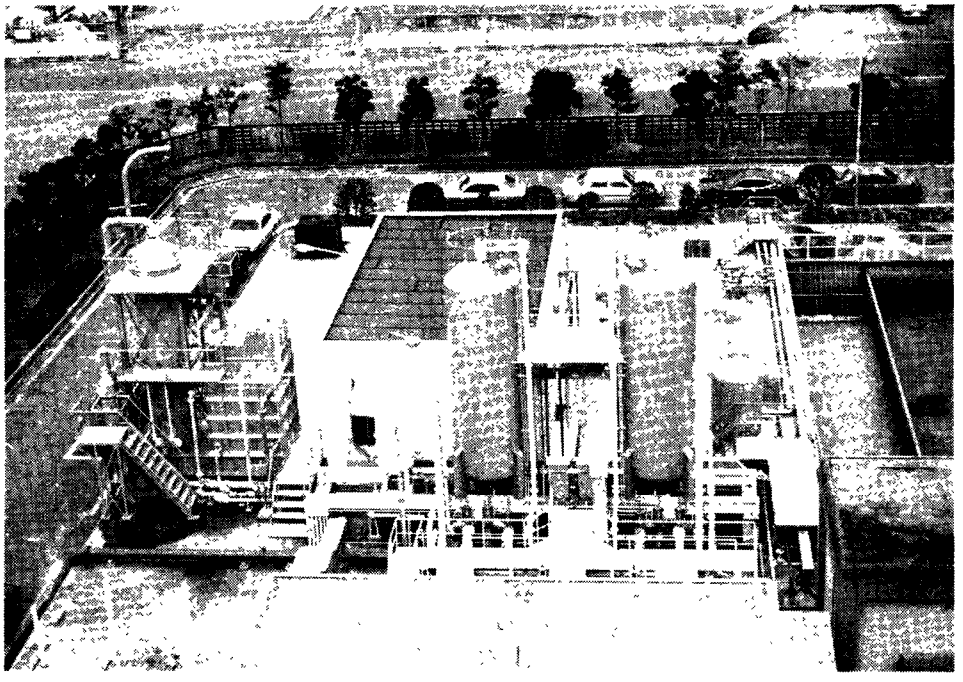


Photo. 1 Reclamation facilities

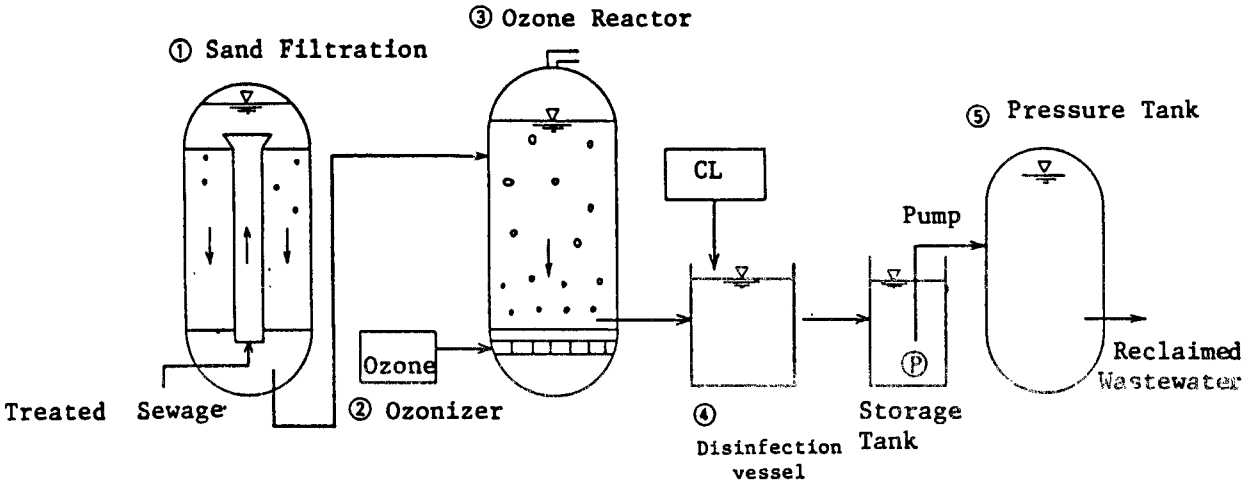


Fig. 2 Location of Reclamation Facilities

7.2.5 OUTLINE OF FACILITY

The outline of the facility is shown in Table 11. The capacity is 400 m^3/day and the distribution pipe can be extended up to 6 km.

TABLE 11. Outline of Facility

Name of Facility	Style and Structure	Capacity	Remark
Sand filtration tank	Decendent type	1,440 m^3/day	Using existing facility
Ozone reactor	Silent discharge system	q=300 ∿50g/H	
Disinfection	Injection of sodium hypochlorite	retension time 15 minutes	
Storage tank	RC construction	V=100 m^3	
Pressure tank	Auto-pressure system by pump	Q=400 m^3/day	Effluent pressure = 3...6 kg/cm^2
Distribution pipe	Ductile cast iron pipe	Pipe caliber 100 ∿ 200 mm	Pressure at end of pipe = 1.5kg/cm^2

7.3 OPERATION

This Model Project was initially started at 3 buildings in June, 1980, and completed at all 12 buildings in December, 1982. It has been working out properly so far.

7.3.1 VOLUME OF RECLAIMED WASTEWATER SUPPLY

The volume of supplied water in 1987 is shown in Table 12. And the consumption of water of each building is shown in Table 13. A 40% of the total water consumption is covered by the reclaimed wastewater.

TABLE 12. State of operation

(as of March 1987)

Item \ Month	4	5	6	7	8	9	10	11	12	1	2	3	Ave.	Max.	Min.
Quantity of raw water (m^3/day)	161.4	176.9	170.0	181.0	168.5	179.2	180.6	158.7	137.7	141.5	171.2	176.0	166.9	181.0	137.7
Transmitted quantity to out of plant (m^3/day)	132.8	152.7	141.8	152.3	139.4	146.5	149.8	149.1	133.1	132.8	163.8	176.5	147.6	176.5	132.8

TABLE 13. Quantity of annual consumption

(as of March 1987) (Case: m^3)

Name of Customer	Reclaimed wastewater (A)	Potable water (B)	Total (C)	(A)/(C) (%)
Chuo Citizen Center Chuo Citizen Gymnasium	2,576	2,951	5,527	46.6
Chuo Ward Office	6,657	16,001	22,658	29.4
Kyushu branch office Japan Tobacco Industry	3,355	10,407	13,762	24.4
Daimyo Public Hall	249	1,637	1,886	13.2
Co-operative Public Office of Legal Affairs	2,201	5,110	7,311	30.1
Chuo Post Office	6,927	9,716	16,643	41.6
Chuo Police Office	4,598	5,975	10,573	43.5
Municipal Office	2,137	1,782	3,919	54.5
Tenjin Station (Subway)	5,003	5,740	10,743	46.6
Akasaka Station (Subway)	3,627	558	4,185	86.7
TOTAL	37,330	59,877	97,207	38.4

7.3.2 QUALITY OF THE RECLAIMED WASTEWATER

In order to ensure the safety and the efficiency of the reclaimed wastewater, periodical inspection of water quality is done at both reclamation facility and the users' sites. The result of the daily examination and the close examination are shown in Table 14 and Table 15, respectively. The daily examination is done by automatic water quality monitoring equipment.

TABLE 14. Data of daily examination

(as of March 1987)

ITEM \ Month	4	5	6	7	8	9	10	11	12	1	2	3	Ave.	Max.	Min.
Water Temperature (°C)	20.6	23.2	25.9	27.1	30.3	28.5	25.2	21.9	18.5	16.6	16.6	16.7	22.6	30.3	16.6
PH	7.4	7.4	7.4	7.3	7.5	7.6	7.7	7.5	7.6	7.6	7.5	7.5	7.5	7.7	7.3
Color grade	5	6	6	4	6	7	8	9	8	8	9	9	7	9	4
Chloride Ion (mg/l)	1,100	870	870	750	1,400	1,000	470	600	320	550	600	560	760	1,400	320
Electric conductivity (μU/cm)	2.5	2	2.1	1.8	3.6	3.3	3.2	3.1	2.9	2.4	2	1.8	2.5	3.6	1.8
Turbidity grade	1.1	0.9	0.7	0.5	1	0.7	1.1	2.4	2.3	3	3.2	4.5	1.8	4.5	0.5
Residual Chloride (mg/l)	3.9	-	3.6	3.2	3.5	3.6	3.3	3.3	3.6	3.6	3.5	3.5	3.5	3.2	3.9

TABLE 15. Data of detailed examination

(as of March 1987)

I T E M	Raw water (treated sewage by A system)	Effluent at pressure tank	Chuo Citizen Center
Water temperature (°C)	-	22.6 (13.8∿30.3)	19.5 (13.0∿26.1)
PH	7.2 (6.8∿7.6)	7.8 (7.3∿8.0)	7.6 (7.0∿8.0)
Odor (grade)	-	Not unpleasant	Not unpleasant
Color (grade)	12 (5∿15)	7 (3∿9)	7 (3∿12)
Turbidity (grade)	-	<2	<2
SS (mg/l)	5.7 (<5∿11)	<5	<5 (<5∿9)
COD (mg/l)	9.9 (5∿13)	8.6 (7.0∿10.0)	7.8 (4.8∿11.0)
BOD (mg/l)	22 (3.9∿51)	1.8 (<1.0∿3.0)	2.6 (<1.0∿8.6)
Fe (mg/l)	-	0.15 (<0.1∿0.6)	0.2 (<0.1∿0.4)
Mn (mg/l)	-	0.2 (<0.1∿0.3)	0.1 (<0.1∿0.2)
Ca (mg/l)	-	69 (36∿98)	-
Mg (mg/l)	-	33 (15∿56)	-
No. of Colon Bacilus group (pc/ml)	-	0	0
Residual chloride (mg/l)	-	3.8 (2.9∿4.5)	0.3 (0.05∿0.5)
Consumed quantity of Acid (mg/l)	115 (50∿170)	113 (73∿170)	-
Soluble material (mg/l)	-	1,260 (770∿2,000)	-
Interface activated negative ion material (mg/l)	<0.1 (<0.1∿0.2)	-	-

a) Efficiency of the reclamation facilities

The exclusion rate of each item in the sand filtration tank and in the ozonation tank are calculated according to the result of the close examination at reclamation facilities, as shown in Table 16.

TABLE 16. Effect of clarification in reclamation plant

(as of March 1987)

Item	Quality of reclaimed water		Total removal (Sand filtration + Ozone reaction)
	Treated sewage	Reclaimed wastewater	
Color	12 (grade)	7 (grade)	42 (%)
SS	5.7 (mg/ℓ)	3.2 (mg/ℓ)	44 (%)
COD	9.9 (mg/ℓ)	8.6 (mg/ℓ)	13 (%)
BOD	22 (mg/ℓ)	1.8 (mg/ℓ)	92 (%)

i) Sand filtration

For this Model Project, the existing sand filtration tank which was built for reusing treated sewage at a treatment plant is used. The filtration speed of the tank is approximately 540 m/day, faster than the standard speed of 150 - 200 m/day. However, the average exclusion rate of suspended solids (SS) is 44%, which means that the tank functions satisfactorily.

ii) Ozonation

On average, the ozon of 8.6 mg/ℓ is injected into the tank, and the exclusion rate of color is 42%, which can decrease the color grade of treated sewage from the 12th grade to the 7th grade.

b) Efficiency of disinfection

The water quality standard specified by the Ministry of Construction requires the residual chlorine at the user's site. So, a solution of sodium hypochlorite, 6.4 mg/ℓ , is injected, which makes the residual chlorine of 3.8 mg/ℓ at the storage tank (just before distribution) and that of 0.3 mg/ℓ at the user's site. The main factor to affect this residual chlorine is ammonia nitrogen. When the amount of ammonia nitrogen is under a certain level (1 - 2 mg/ℓ), chlorine becomes free. When it is over the level, chlorine is combined to be the residual one. The free chlorine vanishes more easily than the combined chlorine. In order to

keep the residual chlorine, it is required to stabilize ammonia nitrogen above a certain level and make chlorine the combined residual chlorine.

If the amount of distributed water is small, the retention time in distribution pipe will be long. Also, if the amount of reclaimed wastewater at the user's site is small, the retention time will be long. Table 17 shows the retention time which was actually measured. The retention time varies from 1 day to 28 days and this may cause the residual chlorine to vanish at user's site which retention time is long. We had a research to make sure that the decrease of the residual chlorine accelerates the elimination of coliform group bacteria. As a result of this research, we found that there were no coliform group bacteria both in a upper tank and a low tank of the tested flush toilet, in spite the fact that no residual chlorine had been detected. We believe that coliform group bacteria were extinguished by ozonation and chlorination. However, other bacteria may grow when the residual chlorine vanishes, or the supplied water may be polluted at the user's site. Therefore, it is required to keep the residual chlorine even at the user's site and to shorten the retention time as much as possible.

TABLE 17. Description on retention period

(as of March 1987)

Name of Customer	Facilities of Customer			Quantity of used water (ave. m^3/day)	Retention period (day)	Retention period in supply pipe (day)	Total of retention period (day)
	Capacity of Tank (m^3)						
	Receiving basin	Vessel on Upper tank	Total				
Hamanomachi pump station	8	2	10	10.3	0.97	0.6	1.57
Co-operative public office of Legal Affairs Bureau	20	9	29	6.3	4.6	0.66	5.26
Akasaka St. (subway)	-	-	-	8.7	-	0.76	0.76
Japan Tobacco Industry	48	12	60	10.8	5.56	0.76	6.32
Chuo ward office	36	9	45	17.4	2.59	0.8	3.39
Daimyo public hall	2	1	3	0.6	5	0.86	5.86
Tenjin St. (subway)	-	-	-	15.8	-	1.05	1.05
Chuo post office	27	4	31	24.9	1.25	1.44	2.69
Chuo police office	50	10	60	10.8	5.56	2.8	8.36
Municipal office	130	8	138	5.3	26	2.8	28.8
Chuo Citizen Center	20	6	26	6.1	4.26	1.4	5.66

c) Quality of supplied reclaimed wastewater

The actually measured water quality is shown in Table 18, in comparison with the quality standard in "The Technical Instruction of wastewater reclamation and reused system" specified by the Ministry of Construction. There is no foaming, and the color grade and the turbidity are low so that you cannot tell the difference between the drinking water and the reclaimed wastewater. So far there has been no problem in the quality of reclaimed wastewater.

TABLE 18. Comparison between quality of reclaimed wastewater and standard water

Item on quality	Quality of supplied water		Technical Manual (draft) for sewer water recycling use by Sewer Dep., Civil Ministry (water for lavatory), 1981	Temporary Standard on Quality for miscellaneous water at lavatory which uses recycled sewer water, by Sanitation for Environment Dep., Welfare Ministry, 1981	Technical Manual (draft) Miscellaneous Water Supply by Fukuoka City, Apr. 1985
	At reclamation facilities	At Chuo Citizen Center			
Odor	Not unpleasant	Not unpleasant	Not unpleasant		Not unpleasant
Color (grade)	7	7	Not unpleasant		Not unpleasant
PH	7.8	7.6	5.8∿8.6		5.8∿8.6
COD (mg/l)	8.6	7.8	---		<30
Residual Chloride (mg/l)	3.8	0.3	*(A few quantity contained)		*(A few quantity contained)
No. of Colon Bacilli group(pc/ml)	0	0	< 10		<10
Turbidity (grade)	< 2	< 2	---		---
BOD (mg/l)	1.8	2.8	---		---

Note: * According to the supplier's Standard

7.4 CONSCIOUSNESS OF THE USERS

We had the user research on the color, odor, and turbidity of the flush toilet water which reclaimed wastewater is used for. As shown in Figure 3, 80% of the users answered, "I don't feel any difference between reclaimed wastewater and portable water supply." On the other hand, some answered, "Comparing with portable water supply, it seems to have some color, odor, and turbidity." However, considering the quality of reclaimed wastewater, we believe that the level of its color and turbidity is beyond the limit of man's sight, and that the level of its odor may vary very much according to the users.

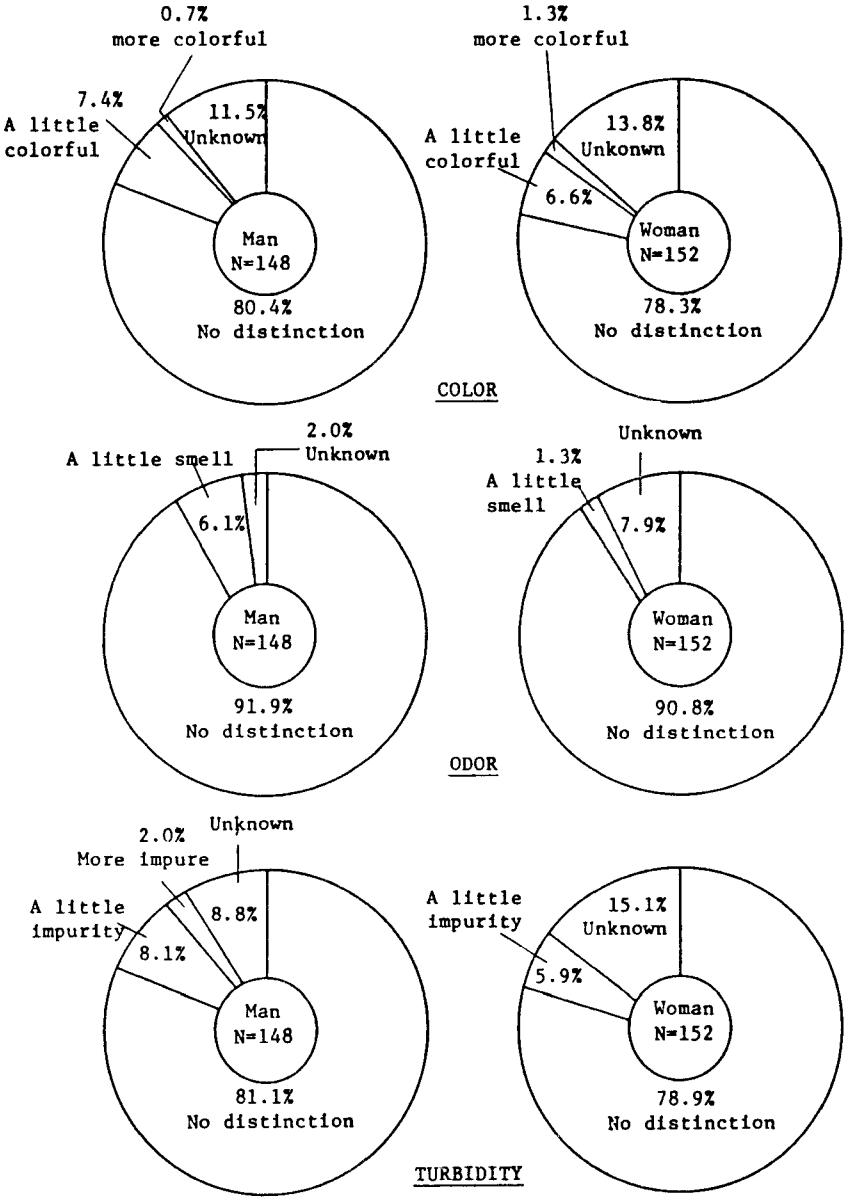


Fig.3 Feeling to reclaimed wastewater after using the flush toilet

Also as shown in Figure 4, 88% of the users said, "It should be used widely." So, we think that there is no problem either in the users' acceptance or in the users' feeling of using reclaimed wastewater for flush soilet.

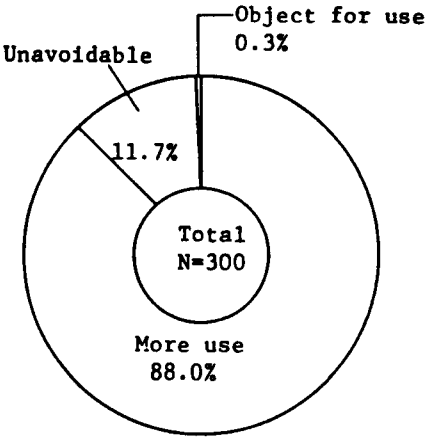


Fig.4 Consensus on the reuse of sewage treated water

7.5 PREVENTION OF MISCONNECTING PIPE

In enforcing this project, what was most concerned about was the accident which would be caused by misconnecting pipe to waterworks. Therefore, the following measure is taken for understreet pipeline and internal pipeline (installed in the building), to prevent such misconnection.

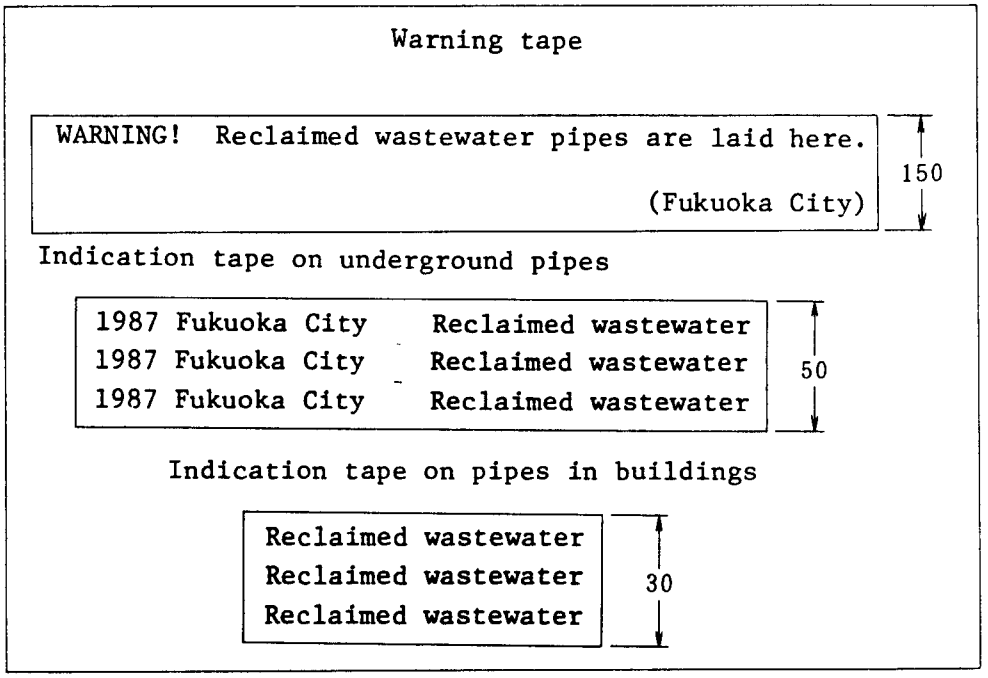


Figure 5 Prevention of mispiping

a) Understreet pipe

1. The understreet pipe should be covered with polyethylene sleeve and put a yellow tape on it.

2. The understreet pipe should be buried with a yellow indication tape to indicate the location clearly.

3. The valves for the understreet pipe should be engraved with the mark of 'Reclaimed Wastewater'.

b) Internal pipe

1. The internal pipe should be put the green tape on and the use should be written on the important sections of the pipe.

2. The indication posts should be put up at the pipe location.

c) Flush toilet

1. The indication boards should show that reclaimed wastewater is used.

Before starting distribution, we ensured to prevent misconnection by putting the artificial color into the upper tank,

7.6 SERVICE CHARGE

To receive the reclaimed wastewater service, the user should sign an agreement and the water supply rating will apply to this service. Since the present facility is still small in size, the service cost is 458 yen/m^3 (see Table 19 for details). As for the operating cost, 70% of that is labor cost and 15% of that is electricity. All the income from service charge is spent for operating expense and depreciation.

TABLE 19 Cost for Reclaimed Waste Water Supply

(As of March 1987)

	Item	Actual value in 1986, fiscal	
		Annual Amount (¥1,000)	Water Cost (¥/m^3)
Expense	Operating & Maintaining cost	10,386	228
	Depreciation	4,873	106
	Interest	5,649	124
	Total	20,908	458
	Quantity of supplied water	45,625 m^3/year	
Revenue	Revenue	¥13,237 × 1,000	
	Unit cost per actual sale	¥ 354	

7.7 FOLLOW-UP SURVEY

Although there has been no major problem since we started service, we are making various knids of follow-up survey to prevent equipment troubles caused by slime or corrosion because of the lack of experience in this field.

7.7.1 OUTLINE OF THE FOLLOW-UP SURVEY

We picked slime generated on the bottom and side face of the receiving basin, the overhead tank, and the tank of flush toilets at the Chuo Citizen Center and the Hamanomachi Pump Station. Then we had the constituent test and the microscopic water examination.

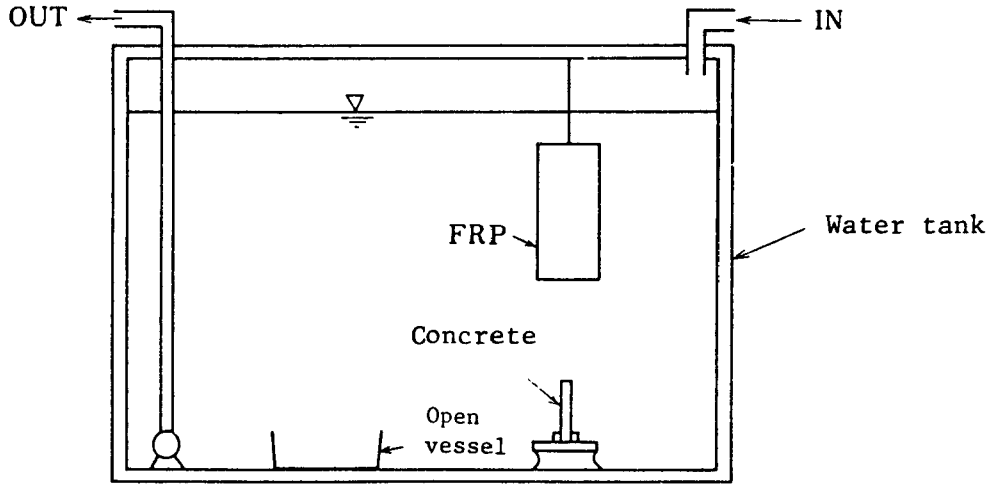


Fig. 6 Outline location of Measuring Equipment

7.7.2 COMPOSITION AND FORMATION OF SLIME

Composition of the extraneous matter on the side face of the tank and the sediment on the bottom of the tanks are as follows. (Also see Figure 7.)

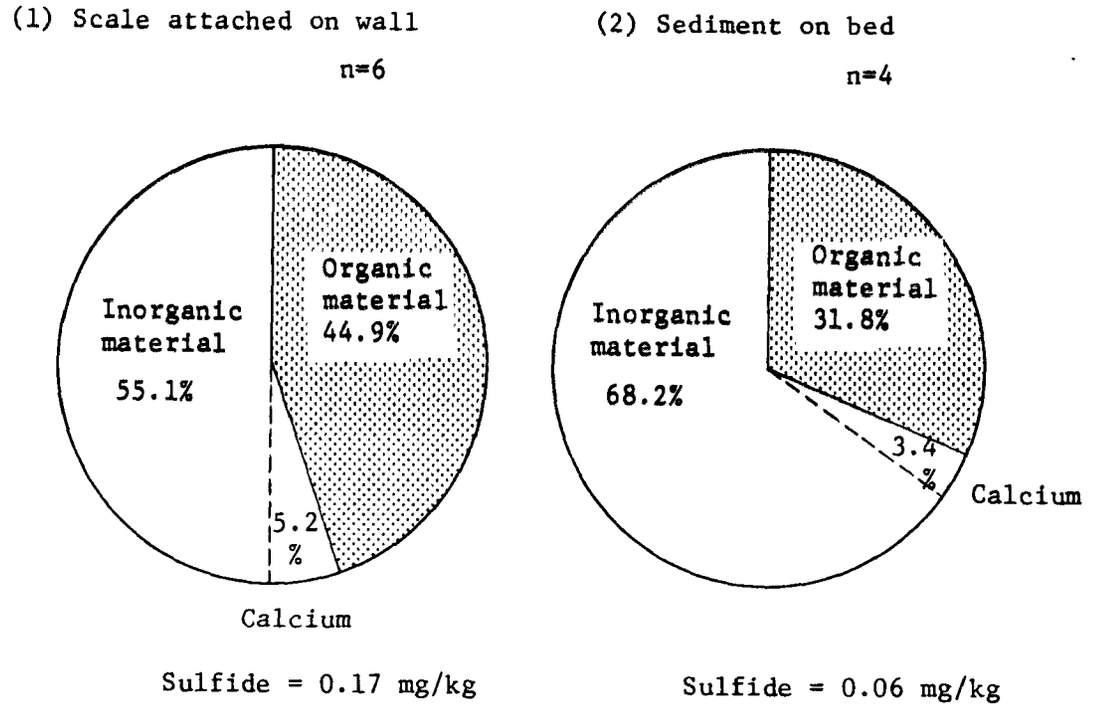


Fig 7. Comparison of components between Scale on Wall and Sediment on Bed

a) The main constituents of slime are minerals including a few percent of calcium scales. Usually the constituents of slime are organic substance. The reason we had the different type of slime with minerals is that it is composed of inorganic salts of high conductivity, high hardness, and high chlorine ionization (cl^-).

b) Most of the slime accumulated on the bottom of the receiving tank are considered to be the deposit of the suspended solids which has remained in the reclaimed wastewater or which has formed during distribution. On the other hand, the rest of the slime is considered to be produced when inorganic salts contained in the reclaimed wastewater was affected by the environmental change in the tank.

7.7.3 BACTERIA COMPOSING SLIME

Most of the organic matter of the slime are bacteria and yeasts which are lower than animal or plant plankton, and those bacteria and yeasts take organic or mineral nourishment to reproduce themselves and form the slime.

7.7.4 SLIME ADHESION TO CERTAIN MATERIALS OF THE TANK WALL

The following formula shows that the slime adhesion depends on the material of the tank and on the condition of its surface.

Concrete $\doteqdot$ FRP with rough surface > FRP with smooth surface

7.7.5 PHYSICAL FACTORS AFFECTING THE FORMATION OF SLIME

Major physical factors are the retention time and the depth of the reclamation. Also, the current of the water, the condition of the incoming light, the size of the side face contacting the water, the time of such contact, and the change of the water level seem to affect the formation of slime.

a) Relation between the contact space and the contact time

The generation speed of the slime on the side face of the tank is corelative to two factors of the contact time and the contact space. In comparing these two factors, the contact time (retention time) plays a more important role in the generation of the slime than the contact space does.

Table 20. The relation between the volume of SS and the contact space/time

Item	x	y	z	Correlation coefficient (r) [Contribution rate]		
	Generation speed (mg/day)	Tank wall space [Contact space] (m^2)	Retention time [Contact time] (Hr)	x: (y)	y:(z)	x:(y,z)
Chuo Citizen Center:						
Receiving basin	216	16.6	113.2			
Upper tank	73	10.5	32.2			
Hamanomachi Pump Station:				0.575 [33%]	0.920 [85%]	0.999 [99%]
Receiving basin	14	12.6	21.2			
Upper tank	70	5.2	5.4			

b) Relation between the formation of slime and the change of the stored water level

As for the generation of suspended solids on the contact space, the more suspended solids can be seen where more often the water level changes. This is because the contact space on the side face is exposed to the air, when the water level goes down, and the impurities in the water contacting the side face can be easily detected.

On the other hand, in the tank which has no change in the water level, more suspended solids can be seen at a lower part of the side face.

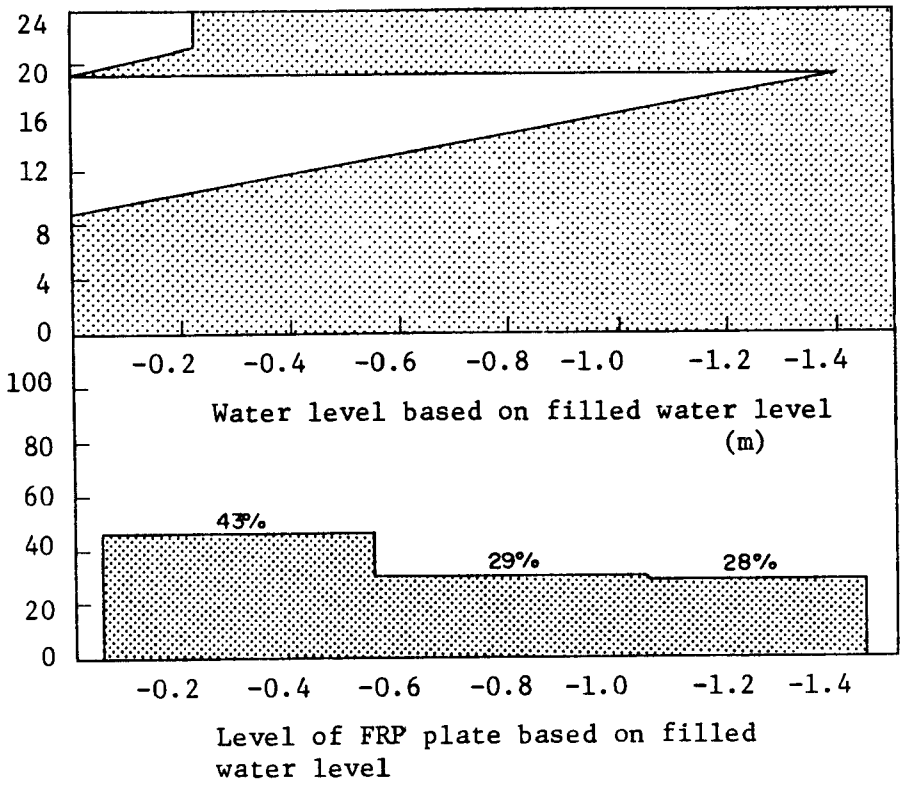


Fig. 8 Relation between distribution of produced scale and variation of water level (At Chuo Citizen Center: Water tank located upper tank)

7.7.6 COUNTERMEASURE TO WATER SUPPLY EQUIPMENT TROUBLES

a) Possible troubles due to the quality of water

When we reuse the reclaimed wastewater, water supply equipment may be troubled by 1) slime, 2) scales, 3) corrosion, or 4) clogging.
The most possible trouble caused by the quality of water is corrosion. As an index of corrosion rate, the Langelier's index is used. Table 21 shows the comparison of corrosion in the reclaimed wastewater VS that in potable water. Although both the reclaimed wastewater and potable water are categorized in the corrosion group, potable water is more corrosive than the reclaimed wastewater.

TABLE 21 Comparison on corrosion effect between Potable Water and Reclaimed Wastewater

Item		p H	Alkalis [M] (mg/ℓ)	Calcium [Ca] (mg/ℓ)	Soluble substance [Sd] (mg/ℓ)	Langelier Index
Reclaimed Waste-water	Max.	8.0	101	84	2144	0.240
	Ave.	7.6	76	61	1565	-0.377
	Min.	7.2	50	37	484	-1.004
Potable Water	Max.	6.8	27	55	187	-1.467
	Ave.	6.7	16	24	77	-2.073
	Min.	6.5	10	13	32	-2.754

Note: Langelier Index = $pH-8.313+\log(Ca/20)+\log(M/50)-2\times(\sqrt{2.5\times10^{-5}\times Sd})/(1+\sqrt{2.5\times10^{-5}\times Sd})$

Evaluation Case (1) Index >0 ... able to protect metal (producing scale type)

Case (2) Index =0 ... stable for chemical action

Case (3) Index <0 ... able to dissolve (active in corrosion type)

Other troubles such as equipment troubles and pipe corrosion are expected to occur in future. Because these are deeply related to the conductivity and the chlorine ion density of the reclaimed wastewater, it is required to improve the quality of the water and to keep the ion density below the limit.

However, even if such improvement can be made technically, it may be difficult to carry out economically. Therefore, the selection of the most appropriate equipment would be very important. To be more concrete, the countermeasures should be focused on two points: selecting anticorrosive metal for the material and eliminating physical factors to generate slime.

a. Selecting anticrrosive metal for the material

The item which is most easily affected by corrosion is distribution pipe. Table 22 shows the result of the anti-corrosion test for four typical materials of pipe. It tells that vinyl chlorine lining steel pipe or copper pipe has an excellent anticorrosion feature, although the joint part of those types of pipe should be also anticorrosive, which is another problem to be solved.

TABLE 22 Test data on anti-corrosion faculty for materials of distribution pipe

Kind of water pipe			Cast iron pipe (CIP 80A)		Zinc coated steel pipe {SGP(W) 80A}		Vinyl CL lining steel pipe {VLP(W) 80A}		Cupper pipe {CUP(L) 80A}	
Item \ Test water			Reclaimed Wastewater	Potable Water	Reclaimed Wastewater	Potable Water	Reclaimed Wastewater	Potable Water	Reclaimed Wastewater	Potable Water
Corrosion Ratio	Max. depth of corroded pit (mm)		0.8	0.86	0.10	0.37	0	0	0	0
	Max. corrosion ratio		8.0	8.6	1	3.7	0	0	0	0
	Ave. depth of corroded pit (mm)		0.16	0.18	0.07	0.15	0	0	0	0
	Ave. corrosion ratio		2.28	2.57	1	2.14	0	0	0	0
	Condition of corrosion		Rust on all surface (water proof rust)	Rust on all surface (absorbable water)	Rust on all surface (water proof rust)	Rust on all surface (absorbable water)	Rust only on a joint part	-ditto-	-ditto-	-ditto-
Block Ratio	Pipe	Block ratio on pipe dia. (%)	2.5	2.5	1	1	0	0	0	0
	Joint	Block ratio(%)	40	12.5	22	22	20	20	40	40
		Material	Cast iron	Cast iron	Cast iron	Cast iron	Cast iron	Cast iron	Cast iron	Cast iron
Quantity of Sediment	Unit sediment (g/m^2)		1031.05	630.18	54.53	132.17	3.81	4.12	5.7	1.2
	Pipe weight after cleaning scale(g/cm^2)		7.58	7.67	3.30	3.20	4.49	4.47	1.85	2.18
	Material condition after cleaning scale		corroded	corroded	corroded	No change	No change	No change	No change	No change

b. Countermeasures to prevent slime

The physical factors of the slime formation and the restraint conditions are shown in Table 23. The following is the summary of check points in planning the receiving basin and the upper tank.

TABLE 23 Causes of producing Slime and inhibitory conditions

Item \ Cause of producing \ Condition		Plus (active) conditions	Negative (inhibitory) conditions
Scale on wall surface	Surface condition of material	Rough	Smooth
	Variation of water level in tank	Big	Little
	Area contact with water	Big	Little
	Time of contact (retention time)	Long	Short
Sediment on bottom	*Depth of water tank	Shallow	Deep
	Retention time	Long	Short
Common	*Residual Chloride	None	Some
	*Light	Some	None

Note: * Marked item could not check in the survey. However, these items may be one of causes of Slime production from existing documents.

1) Material

The material of tanks should be excellent in anticorrosion and shading, and the finish of the inner face should be carefully checked.

2) Shape

The tank with a large surface of the side face or with a shallow bottom should be avoided.

3) Capacity

The tank should have an appropriate capacity so as to keep the retention time within the limit, and the other equipment such as lift pump should be enhanced as much as possible so that a smaller tank can be installed.

4) Method of water supply

The water supply by upper tanks has many factors to generate slime such as the water level change and the direct rays of the sun (which increase the temperature and the photosynthesis of plant plankton), so that other method like a pressure tank should be developed.

5) Others

Although adding chlorine chemicals in one of the most efficient way to prevent slime, the residual chlorine will decrease during distribution. Especially for the facilities distant from the reclamation plant, it become hard to secure the residual chlorine enough to prevent slime. Therefore, it may be necessary to consider to add the chlorine chemical at each user's site.

We have reported the result of the follow-up research on slime, scales, and corrosion. We plan to have a long term research on how those affect the safety of the water supply and the equipment.

8. EXPANSION PROGRAM OF MODEL PROJECT FOR A WASTEWATER RECLAMATION AND REUSED SYSTEM

8.1 BACKGROUND

As mentioned before, we have supplied the reclaimed wastewater of 400 m^3/day since June, 1980. Teh citizens have been very conscious of the reuse of treated sewage for miscellaneous use, and the need of this reclaimed wastewater service is increasing at the buildings surrounding the public offices which is now in service. Expecially in the Tenjin area, Chuo-ku, where there are a lot of large buildings of banks, business offices, and department stores, forming a center of politics and economy in western Japan, the construction of new large buildings and the reconstruction of old buildings are planned. There is a large potentiality of the miscellaneous use water.

Also we started the reclamation work in 1981 on the Hakata Bay General Project, and in a part of this area, Jigyo and Momochi area with 138ha, educational facilities (such as university and museums) and urban facilities (such as hospitals and houses) will be constructed. Moreover, on this reclaimed land the "Asian-Pacific Exposition in Fukuoka 1989" will be held to celebrate the 100th anniversary of the city during March to September in 1989. Because this exposition is expecting six million visitors, a large amount of miscellaneous use water will be needed temporarily. We expand the Model Project from 1986 in order to promote the efficient reuse of treated sewage and to meet the large needs of miscellaneous use water, based on the result of the present models.

This will bring us the following benefits and to ease the strained demand of water supply.

a. Treated sewage as a permanent water resource

b. Stable supply of miscellaneous use water during drought

c. Switch the inbuilding reclaimed wastewater system to the large area reclaimed wastewater supply system in order to decrease the users' cost and make an efficient use of equipment space

d. Enhance the cost performance of the existing models project

8.2 PROGRAM PLANNING

8.2.1 SETTING THE SERVICE AREA

In order to realize the cost efficient project, it is important to minimize the construction cost of reclamation facilities and distribution pipe by selecting the service area with high density of users. This also benefits the users in that the users do not have to equip the reclamation facilities in their buildings, when they build new buildings in the area with the treatment sewage reclamation and reused system. So, all they need only install double piping system.

Taking these facts into consideration, researching on the construction plan of large buildings and on the location of buildings to be served, and studying the cost efficient expansion of area, we finally set the service area

in the central urban area and the reclaimed land.

8.2.2 BUILDINGS TO BE SERVED

Because it is more beneficial to start the service at buildings with larger water demand, we set up the buildings to be served according to the standard in the Water Saving Plan. Since the reclaimed wastewater miscellaneous use water supply requires double piping in the building which needs a space for pipe shaft as well as the additional equipment cost, we excluded the existing buildings without double piping.

The buildings to be served can be categorized as follows.

a) Central urban area (178 ha)

1. Buildings served by the present Model Project (12), public buildings

2. Buildings served by the inbuilding reclaimed wastewater supply system (17)

   These are the buildings with the inbuilding reclaimed wastewater supply system according to the Water Saving Plan. The reclamation cost at those buildings is rather high due to its small size, so that it is better to switch it to the large area reclaimed wastewater supply system.

   Before we made the final decision, we sent the users a questionnaire to survey their attitudes to switching the existing system to the system of this Model Project.

3. Planned buildings (26)

   These are the buildings of which construction was officially announced in the Tenjin Area Redevelopment Plan.

4. Prospective buildings (20)

   These are the buildings which can be expected to build in the site with 5,000 m^2 or more space as specified in the Water Saving Plan. (The size of the space can be calculated by the following formula: plottage area x volume ratio specified by law)

5. Building to be rebuilt (9)

   The large old buildings which will be possibly rebuilt by 1995 because of deterioration.

6. Other buildings which has double piping system and applied for the service. (2)

Photo. 2 Central urban area of Fukuoka City (Around Tenjin & Watanabe street)

b) Reclaimed land (138 ha)

Public facilities, business buildings, and educational facilities which are planned to build on the reclaimed land will be supplied with miscellaneous use water. As for houses, only housing complexes will be supplied because it is hard to control individual houses to follow the administrative instruction. It may cause mispiping to potable waterworks when it will be rebuilt, or other sanitary problems. Also, facilities in the exposition area which will be held in 1989 will be supplied with miscellaneous use waterworks.

Photo. 3 One of reclaimed lands (Jigyo & Momochi Zone)

8.2.3 PLANNED SUPPLY

a) Unit consumption of water

The unit consumption of water for flush toilet of non-dwelling units was given to each building type according to the Miscellaneous Use Waterworks Technical Standard which was appointed in March, 1985, to apply to miscellaneous use water facilities of the inbuilding reclaimed wastewater supply system. As for the standard unit for dwelling houses, we used 25 ℓ/capita each day, based on the reclaimed wastewater demand research. This figure is considerably lower than the standard of other cities, because of the wide spread of water saving equipment encouraged by the government.

TABLE 24 Technical manual on miscellaneous water system: Unit consumption of water by building's use

Building use		Water quantity for flush toilet	
		Subject	Unit quantity
Primary school		Fixed number of boys & teachers	15∿24ℓ /man-day
Secondary school		Fixed number of boys & teachers	18∿28ℓ /man-day
High School		Fixed number of boys & teachers	18∿28ℓ /man-day
Office	None of restaurant	Total floor space	1.2∿1.8ℓ/m²-day
	Existing restaurant	Total floor space	1.3∿1.9ℓ/m²-day
Hospital		No. of bed	90∿117ℓ/bed-day
Library		Total floor space	1.4∿1.8ℓ/m²-day
Restaurant		Total floor space	2.4∿3.2ℓ/m²-day
Market		Total floor space	1.1∿1.3ℓ/m²-day
Citizen Center		Total floor space	1.2∿2.0ℓ/m²-day
Exhibition Hall		Total floor space	2.0∿3.4ℓ/m²-day
Bus Terminal		Total floor space	1.8∿2.6ℓ/m²-day
Hotel		Total floor space	4 ∿ 5 ℓ/m²-day

b) Peaking factor

1. Daily peaking factor (the rate of daily average to daily maximum demand)

According to the research on the actual condition of the present Model Project, although the volume of flush toilet water does not fluctuate very much seasonally, it was found that it fluctuates by days of the week and by the use of buildings. So, the rate of daily average to daily maximum demand was set in 0.8 by reviewing these fluctuation patterns of each building.

2. Hourly peaking factor (the rate of hourly maximum to daily maximum demand)

   Considering the operation hours of each building, the unit consumption of the water per capita, and the share of floor size of each building in the service area, the rate of hourly maximum to daily maximum demand was set in 2.5 of weighted average.

c) Planned water volume

The planned water volume is calculated as follows, using the daily peaking factor, the hourly peaking factor, and the efficiency ratio 0.9 (considering the leakage from distribution pipe).

Average daily water consumption = Standard unit × Floor size × Estimated population

Max. daily water supply = Average dialy water consumption ÷ Daily peaking factor 0.8 ÷ Efficiency ratio 0.9

Max. hourly water supply = Max. daily water supply × Hourly peaking factor 2.5

Among non-dwelling buildings, the estimated water volume for the buildings of the inbuilding reclaimed wastewater supply system and of the Model Project was calculated by the past record, and that for the buildings of which the use was clarified was calculated by the unit consumption of water for each building type, and that of the buildings of which the use was not clarified (such as prospective buildings) was calculated by the weighted average of the unit consumption of water for the commercial area. And for multi-dwelling complex, the estimated water volume was calculated by the standard unit per capita each day. Table 25 shows each calculated value.

TABLE 25 Scheduled water consumption by customer's facility

(Unit: m^3/day)

Customer's Facility		Used Water Quantity Ave./day	Supply Water Quantity Max./day	Supply Water Quantity Max./hour
Central Urban Area	Office building existing model	117	164	409
	Building using Inbuilding reclaimed wastewater supply system	225	315	788
	Building under planning	450	630	1,576
	New building	367	514	1,284
	Renovated building	290	407	1,017
	Candidate building for use of sewerage	28	38	95
	Sub total	1,477	2,068	5,169
Reclaimed Land	Non-residential use	285	399	998
	Residential use	368	515	1,288
	Sub total	653	914	2,286
	Grand total	2,130	2,982	7,455

Also, the volume of water supply for the Exposition in 1989 was calculated by the estimated number of visitors.

Average daily water consumption 432 m^3/day

Max. daily water supply 908 m^3/day

Max. hourly water supply 2,270 m^3/day

Flush toilet water, daily : 35,087 capita × 9 ℓ/day, capita × 1/1000

= 316 m^3/day

peak : 88,000 capita × 9 ℓ/day, capita × 1/1000

= 792 m^3/day

Sprinkle on streets : 50,000 m^2 × 1.8 ℓ/day, m^2 × 1/1000

= 90 m^3/day

Sprinkle on planted trees : 26 m^3/day

On the other hand, the service plan does not take sprinkling water into consideration if it is done duirng night or early morning when the water consumption decrease. Because it can be covered by flush toilet water. But for the trees of park and street in the area where new distribution pipe will be laid, sprinkling water may be considered.

The volume of sprinkling water was calculated by the size of parks and street trees, and the standard unit of sprinkling (4 ℓ/m^2, day), accounting for 36.5 m^3/day. The period of sprinkling is expected to be for three months in summer.

8.3 FACILITY PLAN

a) Raw water

The secondary treated water at the Chubu Sewage Treatment Plant, as the same raw water as the present Model project.

b) Capacity

Max. daily capacity: 3,000 m^3/day
(existing 400 plus extending 2,600)

c) Reclamation method

The reclamation method is the same as the present Model Project is using, considering the required quality of water and the following matters.

Sand filtration + Ozonation + Sodium hypochlorine disinfection

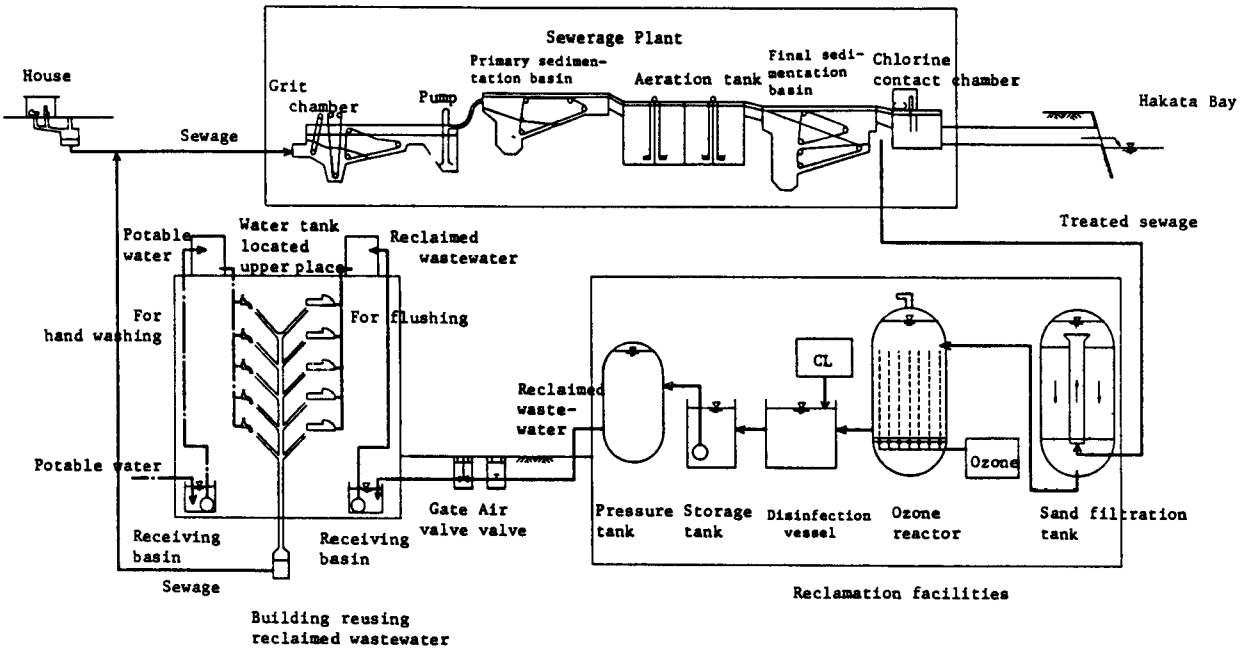


Fig. 9 Flow Diagram of Sewage Treated Water Circulating Reuse System

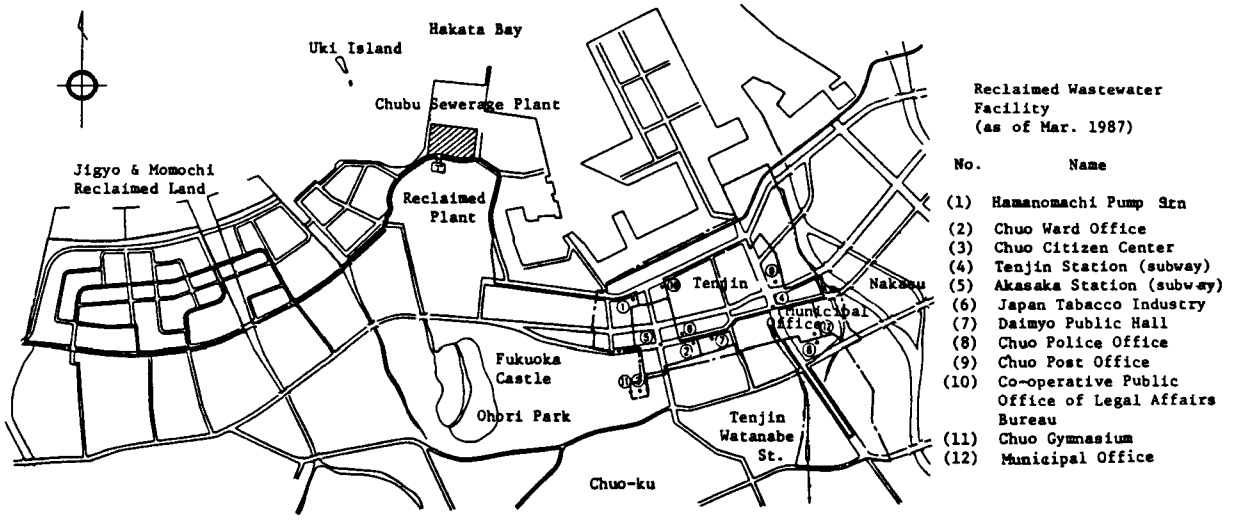


Fig. 10 Lane diagram of the model project for a wastewater reclamation and reused system

Criteria

1. Maintain the required quality of the reclaimed wastewater

2. Secure the safety in a sanitary aspect

3. Cause no troubles in on equipment such as corrosion and clogging

d) Distributed plan

The distribution reservoir will be built at reclamation facilities, so that reclaimed wastewater will be pumped to the business area and the reclaimed land through two piping route.

In order to stabilize the condition of water supply and to prevent the deterioration of water quality caused by stagnation, distribution piping system net should be used as much as possible.

e) Outline of the facility

After reviewing the factors mentioned above, the outline, capacity of the facility and distributed piping route was decided as shown in Figure 10 and Table 26.

TABLE 26 Outline of Design

Name of Facility	Construction	Shape and measure	Capability & others
Raw Water Vessel	R C		Retention time > 20 min.
Sand Filter Vessel	Moving floor type filter (Stainless St.)	2,600φ×5,900H×3 units	Filtering speed 200m/day
Ozone Reactor	FRP made cylindrical vessel	2,300φ×5.0m × 1 unit 1,800φ×5.0m × 1 unit	Retention time > 15 min.
Ozone Generator	Silent discharge with water cooling	600g/H × 3 units	Injection rate 15PPM
Chloride mix Vessel	R C	W1.0×L3.4×D3.0 3 water lanes	Retention time > 15 min.
Distributing Reservoir	R C		> 1,200 m^3
Pump for distribution of water	Underwater pump with pressure tank	φ100mm ×1.8m^3/min. ×30kw ×3 units φ 80mm ×0.9m^3/min. ×15kw ×3 units	for Central urban area for Reclaimed land
Pipe for water distribution	Cast iron pipe with duct-tile	φ100∿φ200 L=7.7km (incl. 3.9km exist.) φ150∿φ200 L=5.0km	for Central urban area for Reclaimed land

9 CONSTRUCTION SCHEDULE AND FINANCIAL PLAN

9.1 CONSTRUCTION SCHEDULE

We are attempting to start operation in March, 1989, on a step-by-step base to minimize the cost of preinvestment; the 1st period (1986-1988) includes the extension of 1,600 m^3/day and the 2nd period (1989-) includes the extension of 1,000 m^3/day. And the total construction cost will be approximately 1.8 billion yen.

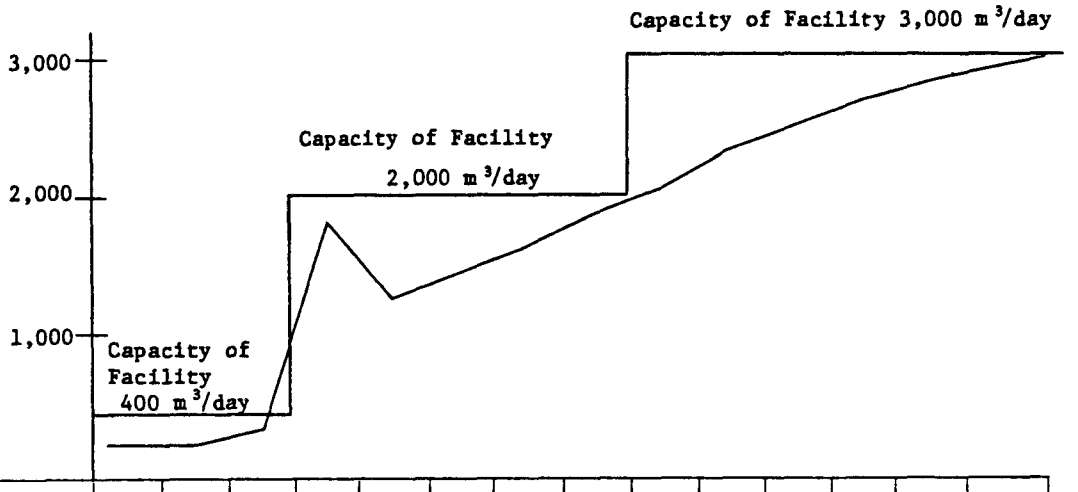
9.2 FINANCIAL PLAN

If the reclaimed wastewater resources can be secured as planned, the Expense and Revenue of each year will go into surplus in 1900 due to the demand from the Exposition and large buildings. We believe that the expansion program will considerably increase the payability of Model Project for a wastewater reclamation and reused system.

TABLE 27 Plan of Finance

	ITEM	1989 Estimation		1995 Estimation	
		Annual Amount (¥1,000)	Water Cost (¥/m^3)	Annual Amount (¥1,000)	Water Cost (¥/m^3)
Expense	Operation & Maintaining Cost	54,046	136	82,605	143
	Depreciation	15,499	39	24,299	42
	Interest	30,818	78	45,859	79
	Total	100,363	253	152,693	264
	Quantity of Supply Water	397,485 m^3/year		577,795 m^3/day	
Revenue	Revenue (¥1,000)	126,149		265,429	
	Unit Cost by Water Sale (¥/m^3)	418		605	

Plan for Supply & demand



	Fiscal Year	1986	1987	1988	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999
Central Urban Area	Model authority (existing)	175	175	175	175	175	175	175	175	175	175	175	175	175	175
	Inbuilding reclaimed waste-water building				91	91	91	91	91	91	91	91	91	122	122
	Building wants future use				38	38	38	38	38	38	38	38	38	38	38
	Increment demand forecast			111	515	606	697	788	930	1,021	1,196	1,287	1,378	1,469	1,560
	Sub total	175	175	286	819	910	1,001	1,092	1,234	1,325	1,500	1,591	1,682	1,804	1,895
Reclaimed Land	Residential				50	101	151	202	252	302	354	406	458	458	458
	Non-residential				43	88	132	176	220	265	308	351	399	399	399
	Sub total				93	189	283	379	472	567	662	757	857	857	857
Water for sprinkle						146	146	146	146	146	146	146	146	146	146
For Exhibition					908										
Grand total in plan		175	175	286	1,820	1,245	1,430	1,617	1,852	2,038	2,308	2,494	2,685	2,807	2,898

Fig. 11 Plan for Supply and Demand of Reclaimed Wastewater
Note: Max. Water Quantity per day (m^3/day)

9.3 EXPANSION OF AVAILABILITY

In order to increase the water supply and ease the demand of waterworks, it is necessary to expand the service area as well as the type of use.

As mentioned before, in the present Model Project the reclaimed wastewater supply flush toilet use in which the users will not touch the supplied water directly. However, there will be other use such as 1) cooling water of air conditioners, 2) car washing, 3) sprinkling for streets and trees, and 4) landscape use (playing fountains and ponds).

The basic conditions for these use are shown in Table 28. Since the concentration of the chlorine ion and the dissolved substance of the treated water at the Chubu Sewage Treatment Plant is high, the desalination process is necessary if it is used for cooling water of air conditioners or car washing water. The equipment of desalination is very expensive so that the installation of this kind of equipment may decrease the payability of the project. Also if it is used for landscape water, nutrient salts such as nitrogen and phosphorus included in the treated water may help the generation of alga even after the sand filtration. So, more study will be needed before the practical use.

As for the use of sprinkling for planted trees, we thought that the salt in the reclaimed wastewater might affect the plants which are week to salt. But the result of the test, actually sprinkling the treated water to the planted trees, showed us there would be no problem in using it. So, we added the sprinkling use of the reclaimed wastewater to the available use. The outline of this test is as follows.

Table 28. Basic requirements for treated water use

Use	Quality Level	Purpose	Requirements		
			Equipments	Human body and Environment	Feeling and Odor
1. Flush toilet	Low	Conveyance of sewage	Not to cause any damage on equipment.	There is no harm on the human body. Environmental effect should be considered.	No discomfort or odor should be caused.
2. Cooling water for airconditioners	Low	Room temperature control	Not to cause any damage on equipment. (Selected materials and maintenance can prevent most of the possible damage.)	Proper equipment and facilities can prevent the possible damage.)	There is no particular requirement.
3. Car washing	Medium	Car washing	Not to cause rust, not to damage on luster, or not to include much salt.	There is no particular requirements when using car washing machines. For hand-washing, the intake through skin and mouth should be prevented.	There is no particular requirement when using car washing machines. For hand-washing, no discomfort or ordor should be caused.
4. Sprinkling	Medium	Water supply to plants and trees. Laying dust.	Not to cause any damage on equipment. (Selected materials can prevent most of the possible damage.)	The intake through skin and mouth should be prevented. The water quality will vary according to the type of sprinkling: on-street, plants and trees.	No discomfort or odor should be caused.
5. Water works and playing fountains		Recreation (to ease mental stress)	Not to cause any damage on equipment. (Selected materials can prevent most of the possible damage.)	The intake through skin and mouth should be prevented. Fish should be able to live in the water. Environmental effect caused by scattering of the treated water should be taken into consideration.	No discomfort or odor should be caused.

(Reference: Guidelines on sewage reuse for buildings)

9.3.1 HOW DOES THE SPRINKLING OF THE RECLAIMED WASTEWATER AFFECT PLANTED TREES FOR ORNAMENTATION?

a) Purpose

We had a test to study how the reclaimed wastewater affects the growth and action of young trees when it has been sprinkled, because the reclaimed wastewater contains more chlorine and nitric acid than potable water.

b) Test materials

Among trees which are commonly planted in gardens and street-side, five different species were selected according to their tolerance to chlorine.

1. Trees which are hardly affected by chlorine.

   Camphortree (Cinnamomum camphora), Sasanqua (Camelia sasanqua)

2. Trees which are easily affected by chlorine.

   Fragrant olive (osmanthus asiaticus)

3. Trees which are categorized in the middle class between the two above.

   Azalea (Phododendron indicum), Hirado azalea (Phododendron mucronatum)

c) Test method

Those five different trees were cultivated in a large pot from April to December of the year, while the reclaimed wastewater had been sprinkled on them to see its affect on their growth compared with the case of potable water and rainwater.

Table 29 shows each ion concentration of five different ingredients of the reclaimed wastewater which may affect the plants. After repeated measurements (14 times), no significant differences was found in the level of each concentration.

Comparing with potable water, the reclaimed wastewater contained extremely high percentage of Na^{+}, Cl^{-}, and No_3^{-}.

TABLE 29 Ion density of reclaimed wastewater which were used for the test (unit: ppm)

Date of sample	Na^+	K^+	NO_3^-	CL^-	NH_4^+
6 Jun.	332.0	19.8	313.0	562.0	-
19 Jun.	550.0	15.2	325.0	398.0	-
4 Jul.	250.0	20.0	385.0	466.0	-
7 Jul.	450.0	24.0	306.0	275.0	4.5
31 Jul.	330.0	36.0	339.4	735.0	20.2
12 Aug.	410.0	22.0	403.0	834.0	11.3
28 Aug.	352.0	25.2	293.0	782.0	15.2
11 Sep.	325.0	31.3	315.0	765.0	11.2
25 Sep.	330.0	20.5	245.0	756.0	8.5
9 Oct.	445.0	25.5	288.0	839.0	11.8
23 Oct.	355.0	23.5	361.0	683.0	8.0
6 Nov.	510.0	32.5	365.0	899.0	5.4
20 Nov.	437.5	27.5	318.0	721.0	14.3
4 Dec.	557.5	42.0	334.0	976.0	17.9
Average	388.2	26.1	327.9	692.2	11.7
Minimum	330.0	15.2	245.0	275.0	4.5
Maximum	557.5	42.0	403.0	976.0	20.2
Potable Water	18.5		24.6	31.8	

d) Result

1. The trees which had been sprinkled with the reclamed wastewater grew rapidly. Also, nitrogen in the reclaimed wastewater increased the nitrogen contents in the leaves, so that the color of the leaves became dark green and the leef dry weight of trees was increased.

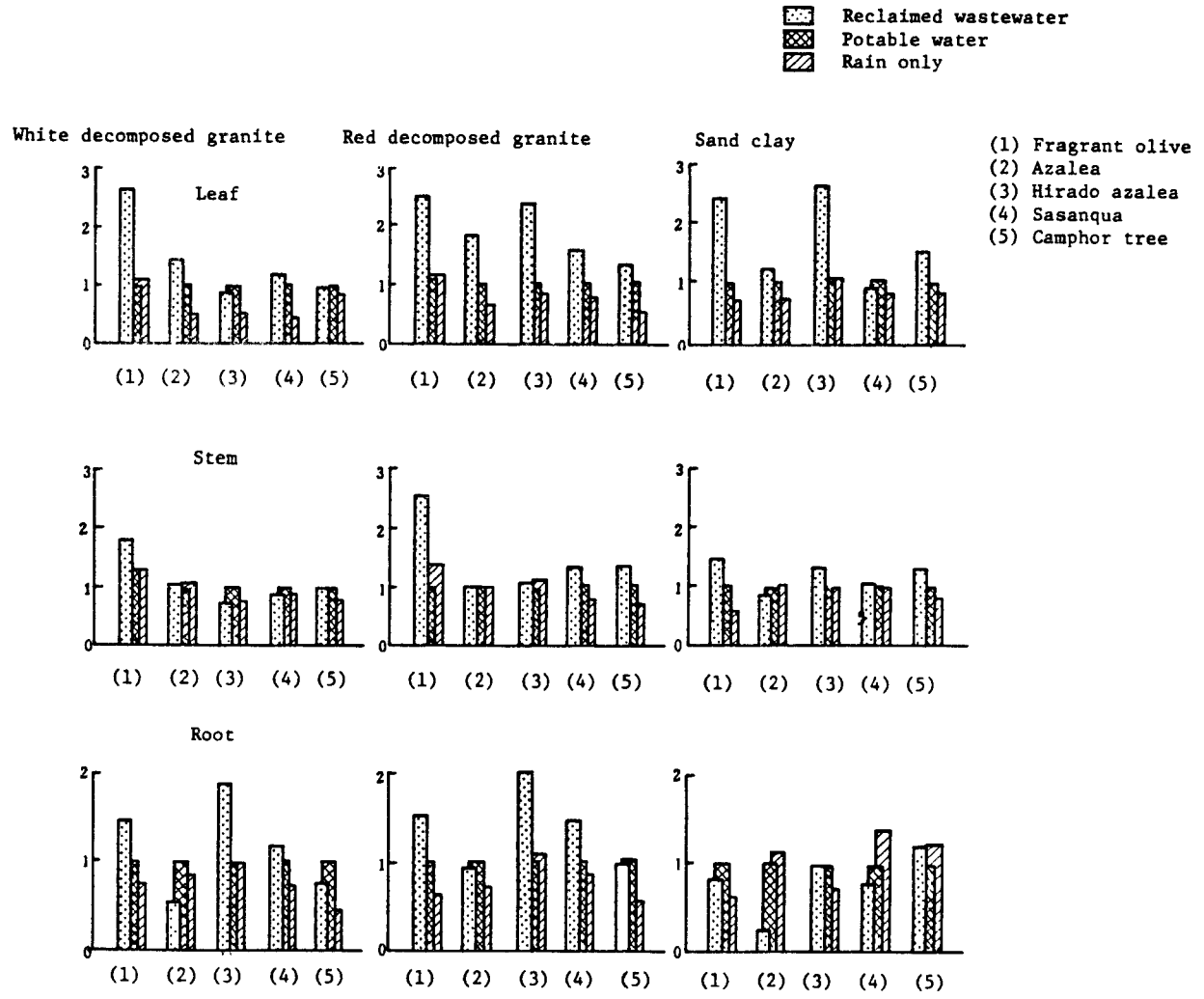


Fig. 12 Effects of Water on plantation by bed clay kinds
(Base: Potable water as 1.00)

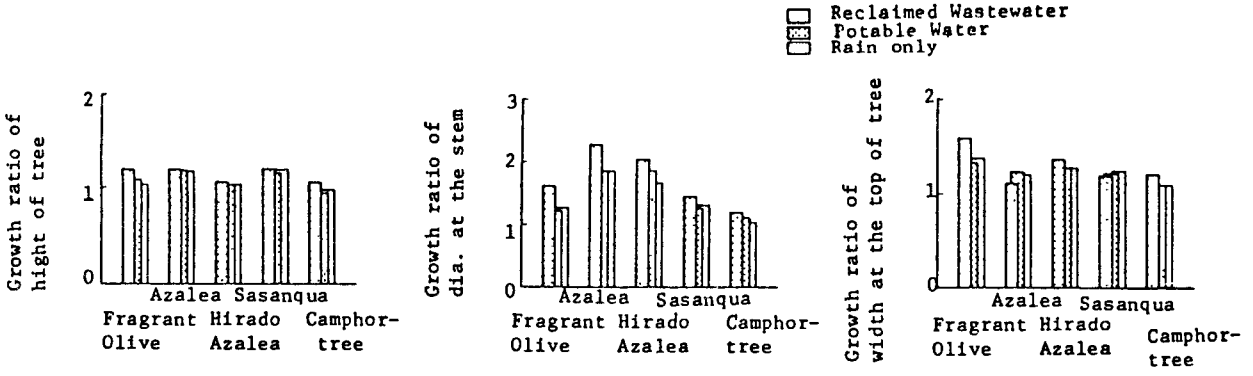


Fig. 13 Growth ratio of young trees on the end of Test
(Compared with each time of young trees before the start of Test)

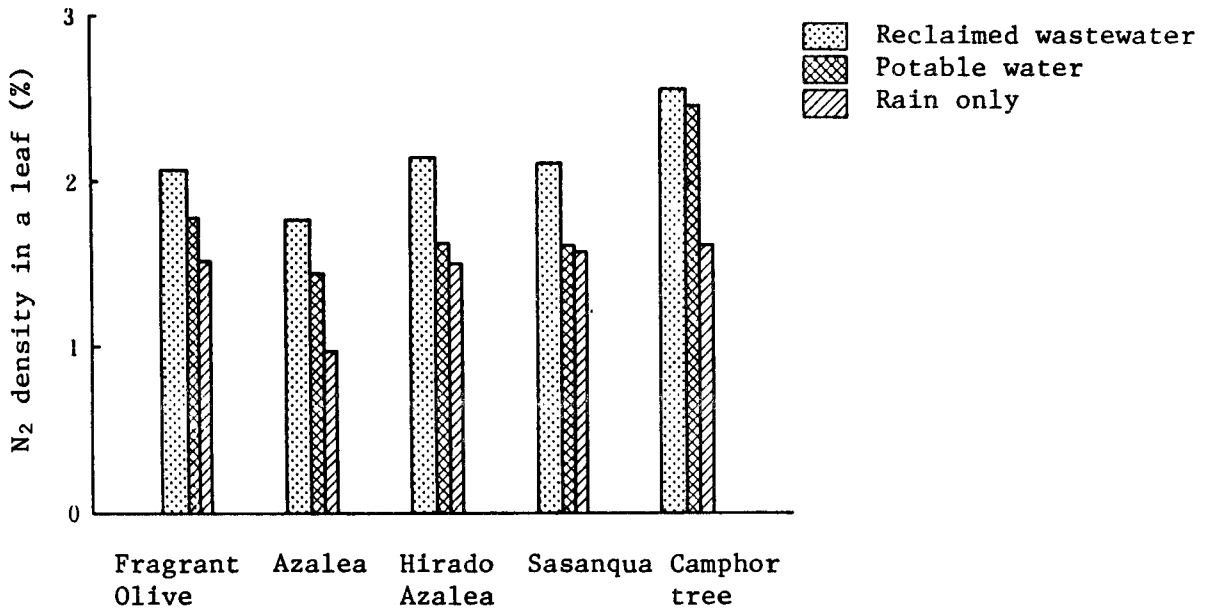


Fig. 14 Residual N_2 density remained in a leaf of plants on the end of test

2. The percentage of chlorine and sodium contained in the leaves of Fragrant olives (which may be more sensitive to chlorine) and Hirado azaleas/Azaleas (which may be categorized in the middle class) increased more than that of Camphortrees and Sasanquas (which may be hardly affected by chlorine) did. Especially in the leaves of Fragrant olives, there were a remarkable increase of chlorine. This type of tree would grow well due to the nitrogen of the reclaimed wastewater, but would be easily affected by the chlorine of the reclaimed wastewater.

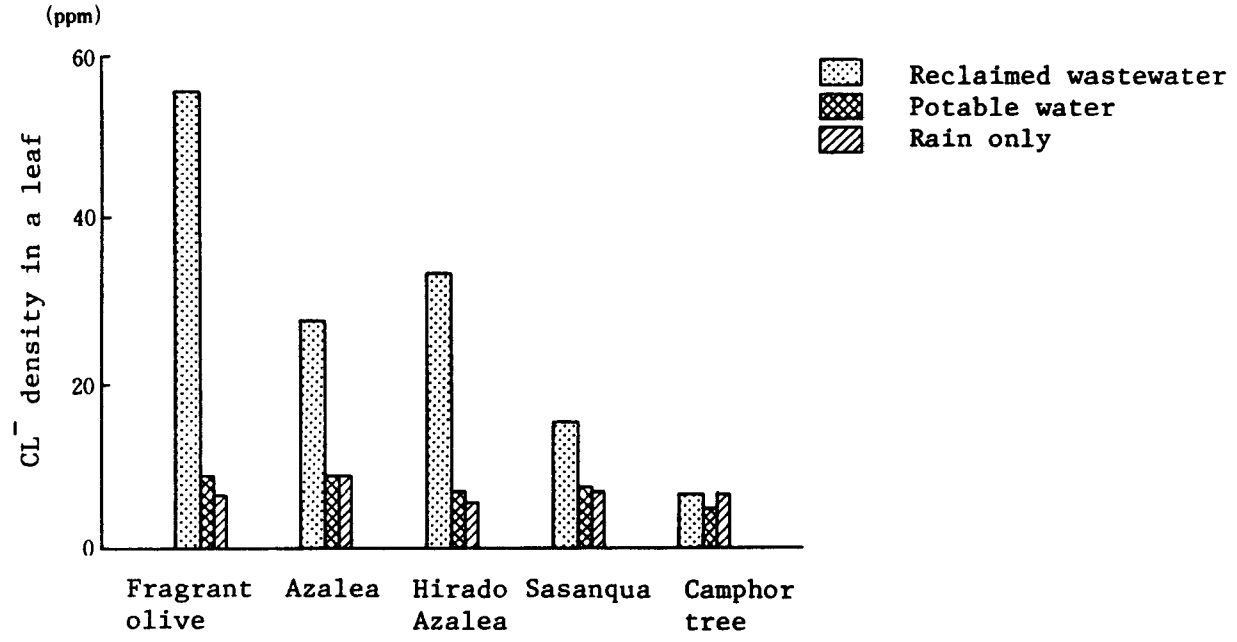


Fig. 15 Residual CL Ion density remained in a leaf of plants on the end of test

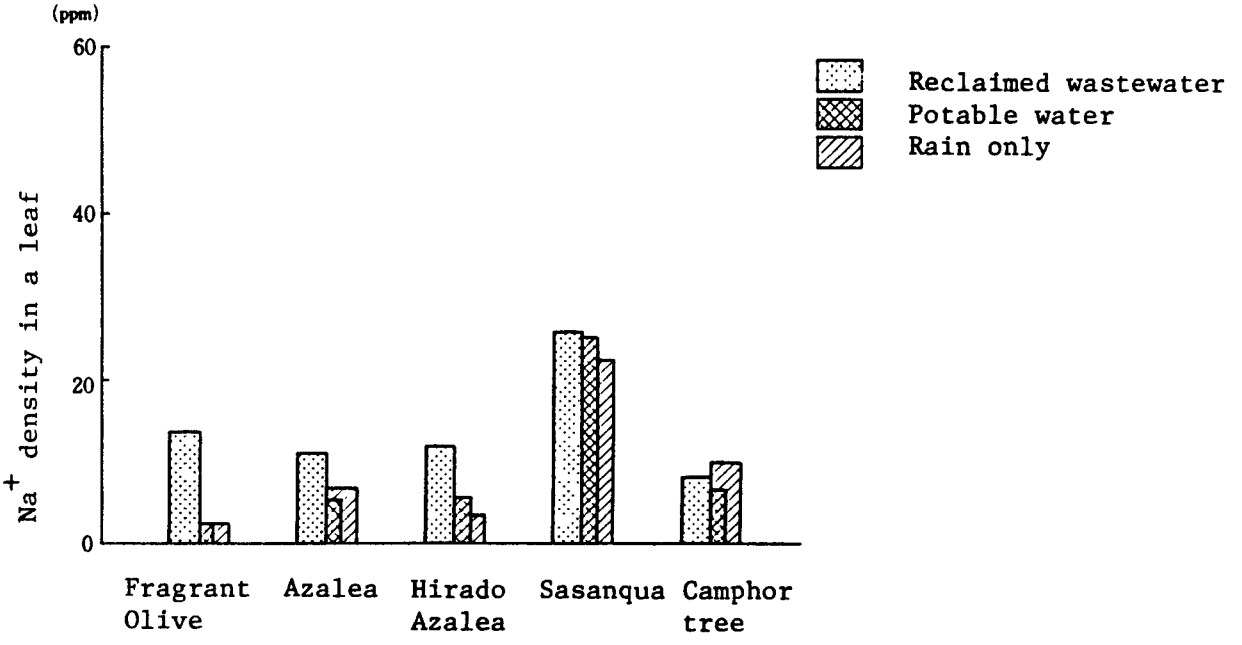


Fig. 16 Residual Na Ion density remained in a leaf of plant on the end of test

e) Conclusion

After the test, we concluded that the sprinkling of the reclaimed wastewater would give a plus effect on the growth of plants and trees due to its high nutrient content. Therefore, salts and chlorine in the reclaimed wastewater won't cause any serious troubles on them, as long as the sprinkling of the reclaimed wastewater is limited to the period of dry season. However, more study should be needed for the trees which are easily affected by salts (Fragrant olives) to see whether or not any chronic trouble will be caused by the sprinkling of the reclaimed wastewater.

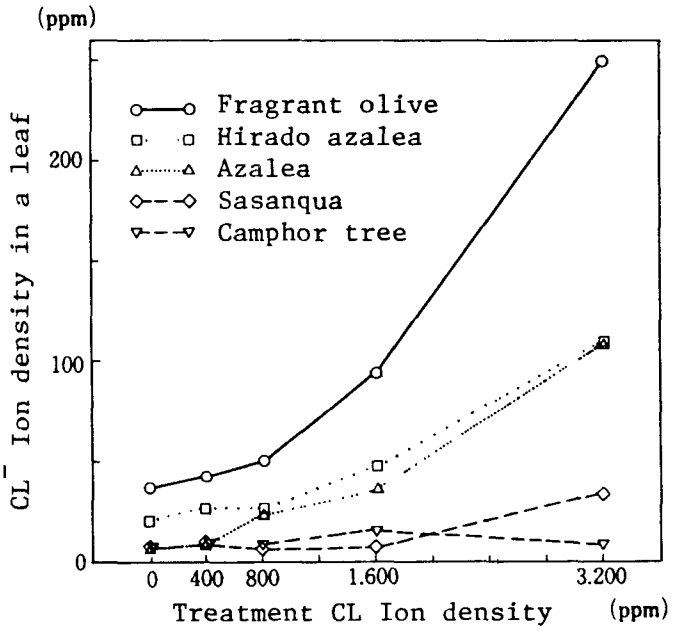


Fig. 17 Residual CL Ion density remained in a leaf of plants on the end of test which used different density of water

The soil which was used for the test is often used as planting soil because of its permeability, which might have prevented any trouble on trees. Therefore, before using treated water for irrigation, the permeability of the soil or the land should be checked and conditions should be provided to prevent the accumulation of salts.

10 CONCLUSION

We have reported Model Project for a wastewater reclamation and reused system as well as the present situation of miscellaneous use water of Fukuoka city and the expansion program of the project. In future, this Model Project will be expanded to other private buildings and the spread of miscellaneous use water will greatly contribute not only to the water saving policy, but to the prosperous city with better water environment.

We are planning the Aqua-Recycle Town Project which is the miscellaneous use water project including Model Project for a wastewater reclamation and reused system the central urban area of the city. This will increase the amount of available miscellaneous use water to approximately 12,000 m^3/day. We will make all the efforts to realize the multiple use of treated sewage and the effective use of precious water resources.

FLOOD CONTROL OF URBAN RUNOFF IN THE EAST AREA OF OSAKA PREFECTURE (NEYA RIVER BASIN)

by

Katsuhiko Kitai
Director
Sewerage Division
Osaka Prefectural Government
Otemaeno-cho, Higashi-ku, Osaka 540, Japan

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

Prepared for Presentation at:
11th United States/Japan Conference
on
Sewage Treatment Technology

October 1987
Tokyo, Japan

ABSTRACT

The eastern part of Osaka Prefecture, comprising the Neya Basin which is to be covered by Japan's first regional sewerage system, is a low-lying area which has frequently suffered flood damage since ancient times. Though damage has, over many years, been gradually reduced due to sewerage construction and river improvement, the recent rapid urbanization of this region has increased rainwater runoff from the region far beyond original estimates.

To cope with this situation, we have made a rainwater runoff model using the Espey Model with an increased run-off coefficient. As a result, it has been proved that the pump and pipe capacity is now insufficient due to the increase in peak discharges. In addition, calculation using the SKM Model revealed the possibility of overflow on a fairly large scale.

Osaka Prefecture plans to solve these problems by improving discharge capacity, as well as by constructing storage facilities for sewerage pipes, which may prove to be a more efficient measure. It is also expected that the runoff load during rainfall will be greatly reduced by means of these facilities.

Furthermore, the Comprehensive Flood Disaster Prevention Plan for the region covering sewerage, river improvement and facilities for runoff control is now under preparation. This plan will coordinate related sections by assigning clearly-defined tasks to each of them, and enhance safety precautions against flooding in the Neya Basin.

CONTENTS

		Page
1.	OUTLINE OF THE EASTERN PART OF OSAKA PREFECTURE	210
2.	RAPID INCREASE OF POPULATION AND DECREASE OF AGRICULTURAL LAND	212
3.	IMRPOVEMENT OF SEWERAGE	212
3.1	PRESENT SEWERAGE PLAN	212
3.2	SITUATION OF SEWERAGE CONSTRUCTION	214
3.3	EFFECT OF SEWERAGE CONSTRUCTION	214
4.	RIVER CONSERVATION WORKS	216
5.	INCREASE OF RAINWATER RUNOFF	220
6.	ESTIMATION OF OVERFLOW QUANTITY ACCORDING TO THE PRESENT SEWERAGE PLAN	222
7.	MEASURES AGAINST RUNOFF INCREASE	227
8.	REDUCTION OF POLLUTIONAL LOAD OF RAINWATER DISCHARGE FROM COMBINED SEWERAGE THROUGH SEWERAGE STORAGE	228
9.	COMPREHENSIVE FLOOD DISASTER PREVENTION PLAN	231
10.	COORDINATION OF SEWERAGE PLAN AND RIVER CONSERVATION PLAN	233
11.	SUMMARY	234

1. OUTLINE OF THE EASTERN PART OF OSAKA PREFECTURE

Osaka Prefecture has great population density. Though the Prefecture is the smallest in Japan, it is the second most populated with an estimated 8.7 million people. Since the Middle Ages, Osaka has developed as a center of the economy. Its population exceeded 2 million in the beginning of this century. During the post-war period, Osaka's population increased rapidly, especially after about 1960. The city area expanded with the efflux of population from Osaka City, the center of the Prefecture, into surrounding cities.

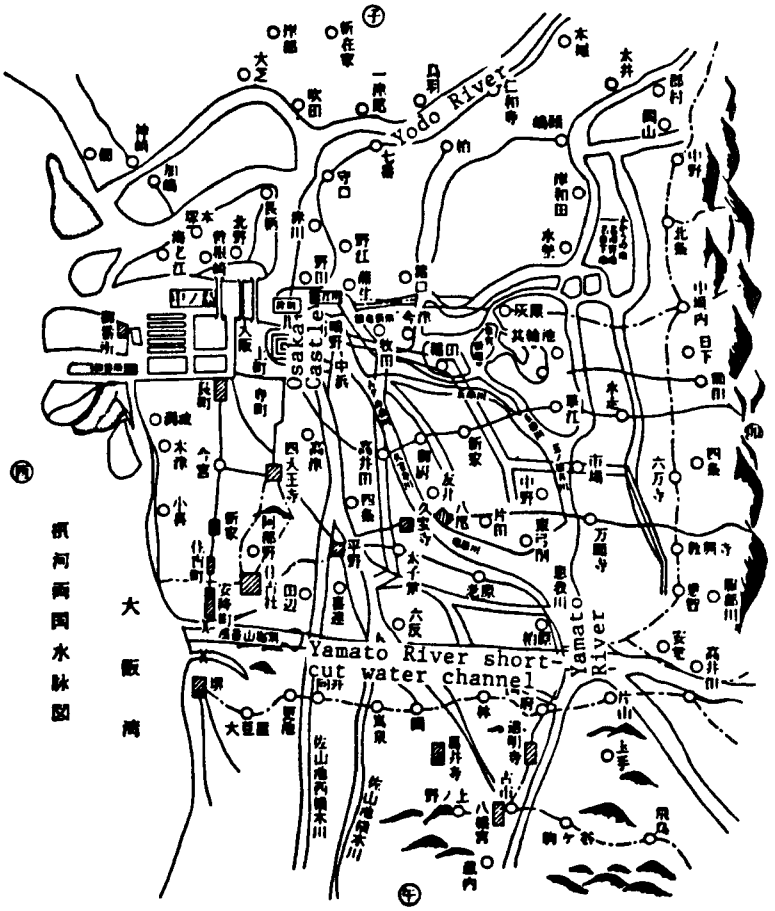


Figure 1 Eastern Osaka in the 17th Century

The eastern part is situated on a plain with an area of approximately 200 km^2, bordering the Yodo River to the north and the Yamato River to the south. In ancient times, these two big rivers joined in the vicinity of Kyobashi. The Uemachi Plateau is found in the southwestern part of the plain; a low and humid area extends east of the Plateau.

As a result of the excavation of the southern part of the Uemachi Plateau in the beginning of the 18th Century, the Yamato River now flows directly into Osaka Bay. Also, a new tributary of the Yodo River which flows into Osaka Bay was opened toward the end of the 19th Century. As a result, the Neya Basin was secured against overflow from the Yodo and Yamato Rivers. However, as the formation of alluvion by these rivers ceased, the plain has remained a low land up to the present. Thus, only the Neya River has served to

drain rainwater for low land between the Yodo and Yamato Rivers, which may be called the Neya Basin. With the advance of urbanization, drainage of the interior land and the improvement of the deficient draining capacity of the Neya River have become urgent tasks.

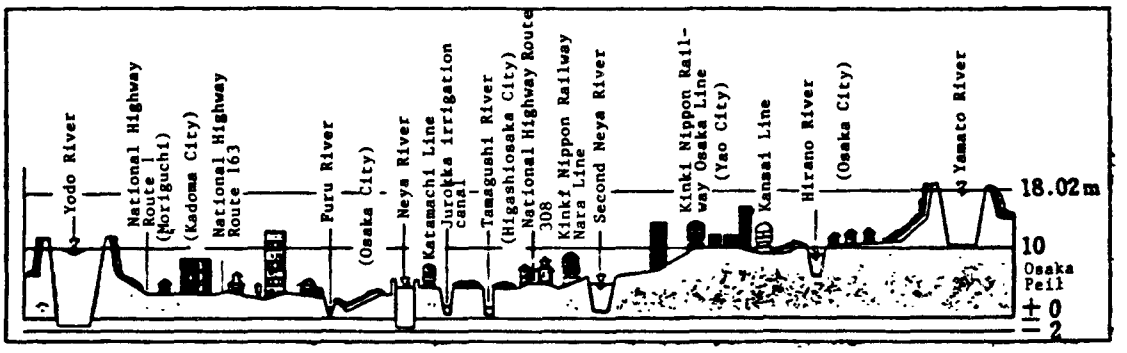


Figure 2 Height of Ground (Sectional View along the Central Loop Line)

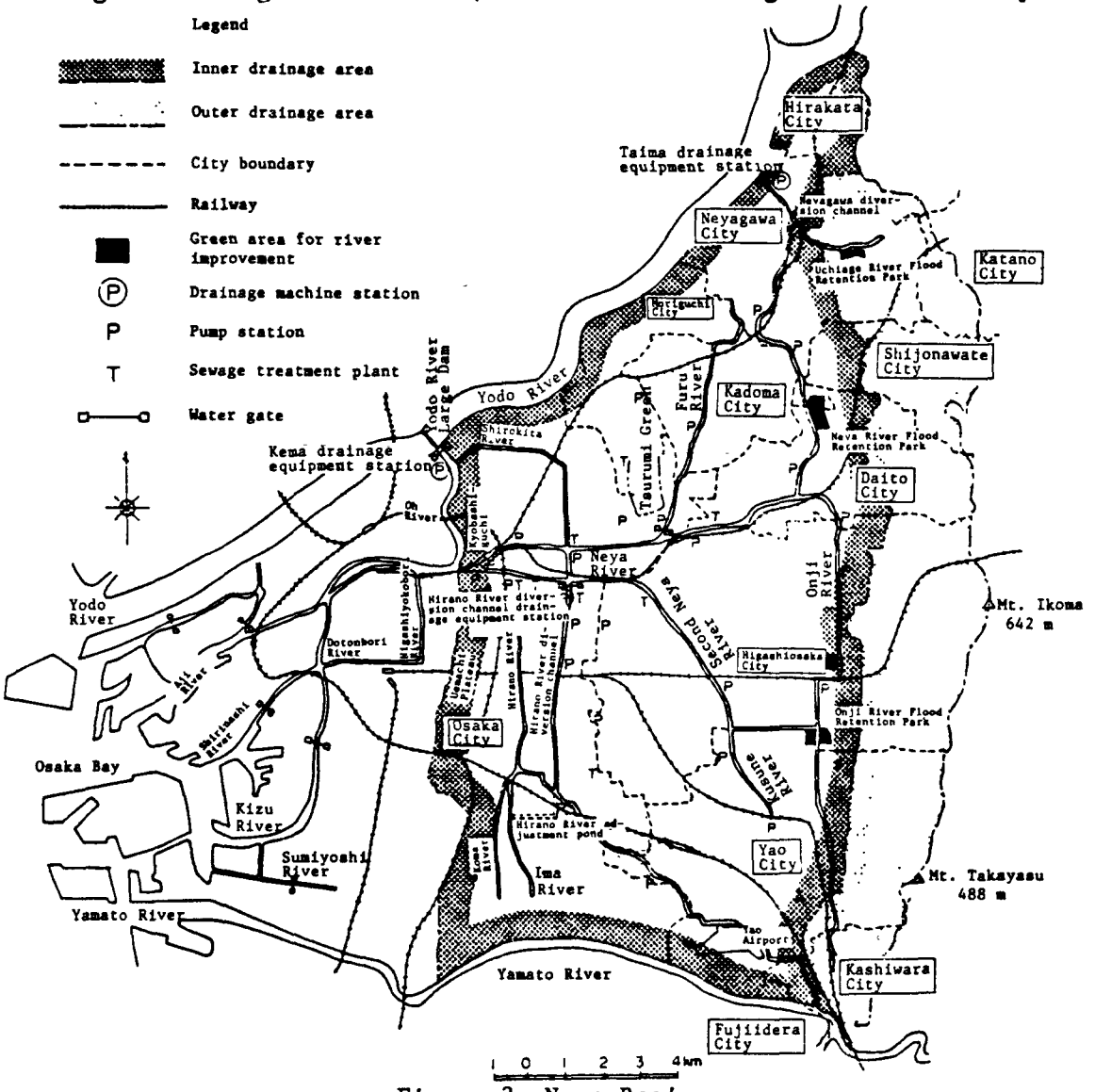


Figure 3 Neya Basin

2. RAPID INCREASE OF POPULATION AND DECREASE OF AGRICULTURAL LAND

The population of the Neya Basin in the east of Osaka Prefecture has been rapidly increasing since 1960. This population increase has led to the conversion of agricultural land into housing lots, resulting in a rapid decrease of the former.

This urbanization has increased the quantity of effluent rainwater, reduced the time before flooding, and worsened the damage by inundation. Accordingly, a small inundation can now cause great damage.

To cope with this situation, we have devoted ourselves to improving the sewerage system for hastening drainage and river conservation works for preventing floods.

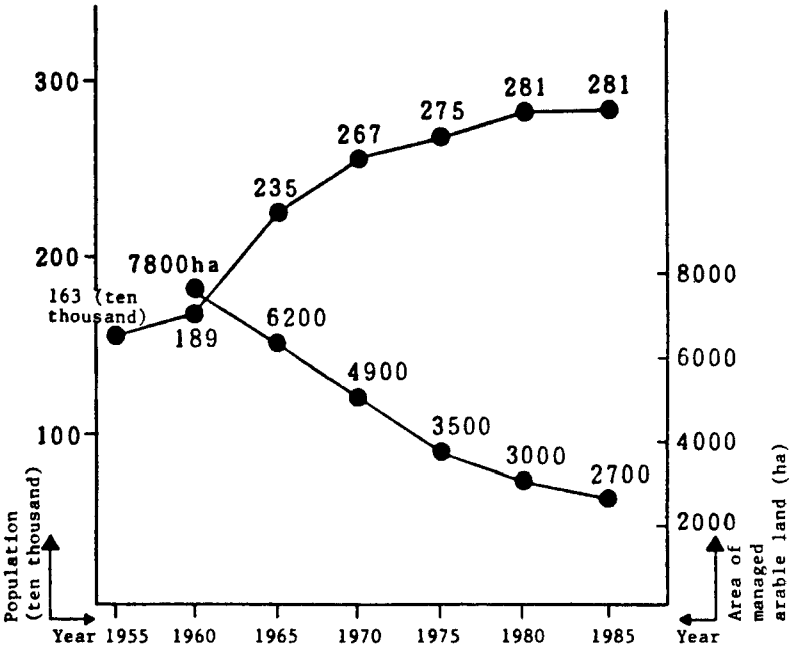


Figure 4 Long-term Change of Population and Agricultural Land in Neya Basin

3. IMPROVEMENT OF SEWERAGE

3.1 PRESENT SEWERAGE PLAN

The sewerage in the Neya Basin consists of the separate public sewerage systems of Osaka and Moriguchi Cities and sewerages in the northern and southern Neya Basin. The outline of each sewerage plan is shown in Table 1. After two years of planning, construction of the sewerage for the Neya Basin commenced in 1965 as the first regional sewerage system in Japan. As city area has sprawled in adjoining medium- and small-sized cities in the Neya Basin sewerage area, a new plan covering a larger area was required. To this end, the construction of the regional sewerage system was promoted.

Table 1 Sewerage Plan for Neya River Basin

		Design Area (ha)	Design Population	Trunk Sewer Length (m)	Treatment Capacity (m^3/day)
Neya River regional sewerage system	Konoike treatment district	6,725	590,000	56,380	236,000
	Kawamata treatment district	8,917	950,000	67,960	380,000
Osaka City	Imafuku treatment district	1,616	321,000	39,827	320,000
	Nakahama treatment district	1,869	281,000	39,850	333,000
	Hanaten treatment district	540	87,000	18,353	154,000
	Hirano treatment district	2,486	456,000	58,274	323,000
Moriguchi City	Moriguchi treatment district	563	100,000	200,387	65,000

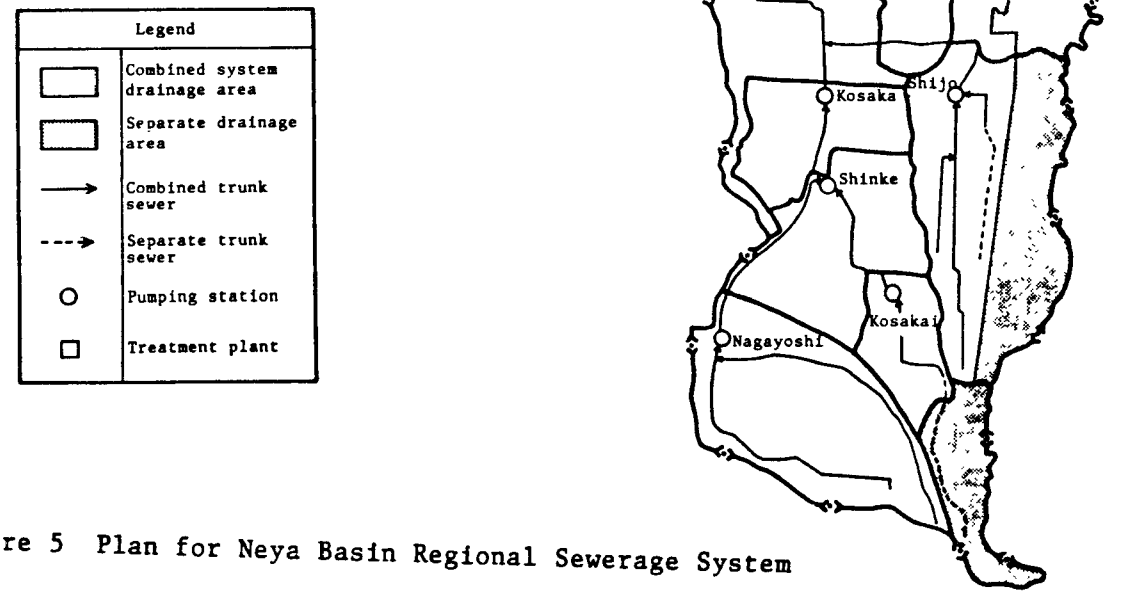


Figure 5 Plan for Neya Basin Regional Sewerage System

The regional sewerage system is designed to cope with an intensity of rainfall which may occur approximately once in five years. The effluence rate of rainwater is calculated by multiplying the rate by the proportion of the area used for city planning for each drainage subbasin, and then adding the products.

3.2 SITUATION OF SEWERAGE CONSTRUCTION

Table 2 Sewerage Construction Status (as of the end of 1985)

		Treatment Area		Ratio of Sewered Area/Population	
		Area (ha)	Population	Area (%)	Population (%)
Neya River regional sewerage system	Konoike treatment district	2,008	321,713	29.9	45.4
	Kawamata treatment district	2,136	260,935	24.0	32.7
Osaka City	Imafuku treatment district	1,616	314,300	100	100
	Nakahama treatment district	1,869	323,900	100	100
	Hanaten treatment district	540	88,200	100	100
	Hirano treatment district	2,445	354,900	98.4	98.4
Moriguchi City	Moriguchi treatment district	563	80,401	100	100

Sewerage construction status as of the end of 1985 is shown in Table 2. Separate public sewerage systems for Osaka and Moriguchi Cities have almost been completed. However, the Neya Basin regional sewerage system was only 29.9% complete in the northern region and 24.0% in the southern region at the end of 1985. Further construction is necessary. The investment in the regional sewerage system from 1965 to 1985 is shown in Table 3. The amount of investment is not proportionate to the proportion of sewered area, because priority has been given to the construction of large-scale pumping stations for storm water and trunk pipes.

Table 3 Past Investment in Neya River Regional Sewerage System

(Unit: ¥1 million)

Year	1965 ∿ 1970	1971 ∿ 1975	1976 ∿ 1980	1981 ∿ 1985	Total
Neya River regional sewerage system	16,721	39,679	46,744	38,767	141,911
Konoike treatment district	10,092	16,999	21,217	17,525	65,833
Kawamata treatment district	6,629	22,680	25,527	21,242	76,078

3.3 EFFECT OF SEWERAGE CONSTRUCTION

Though only 61.6% of the planned piping has been constructed for the Neya Basin regional sewerage system as of the end of 1986, the inundation

area has been greatly reduced. As an example, Figure 6 shows the comparison between the situation of inundation and the sewered area in the region. As can be seen from this figure, the inundation area has decreased with the progress of sewerage construction.

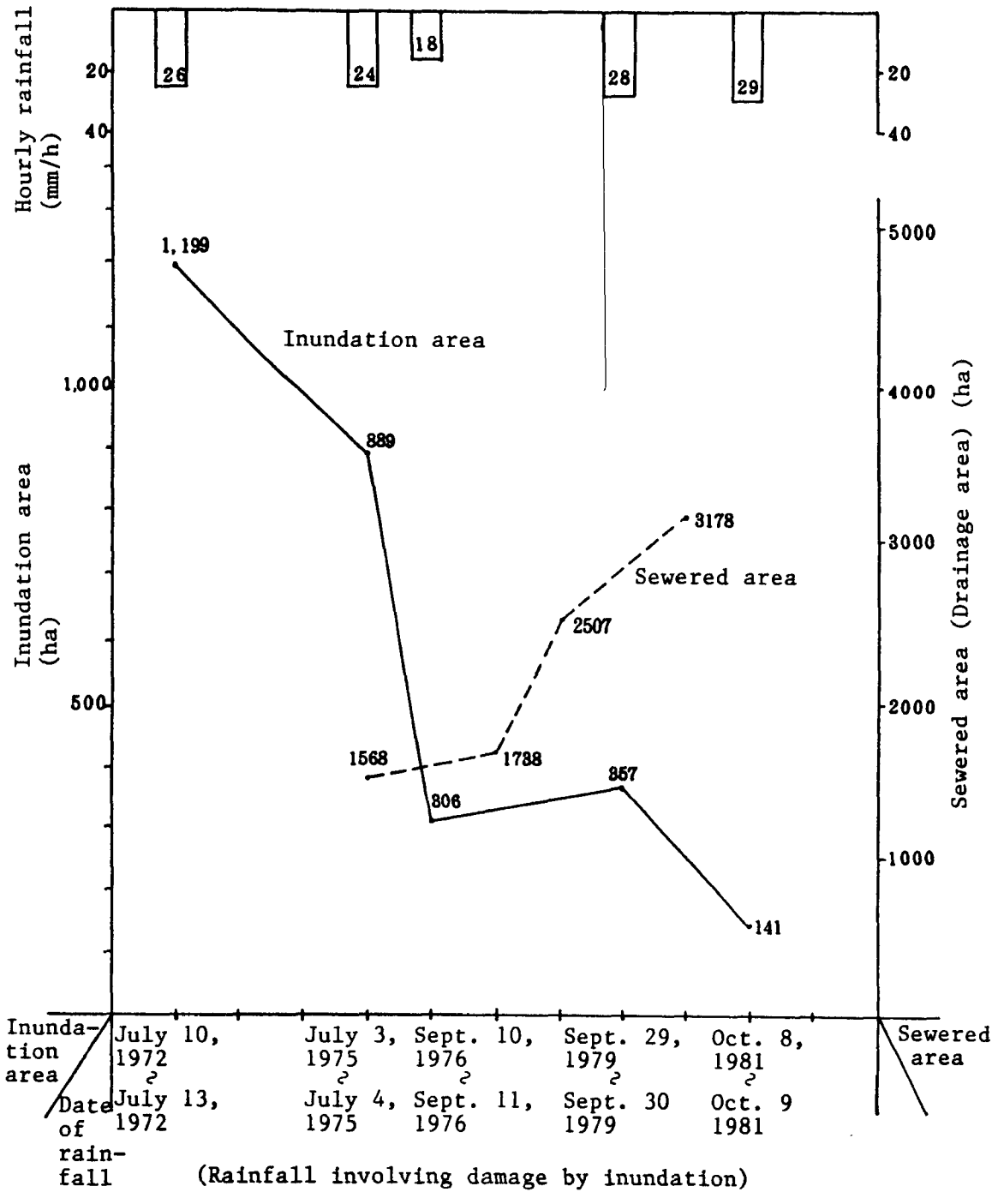


Figure 6 Change of Inundation Area in the Region in which Neya Basin Regional Sewerage System Has Been Constructed

Besides mitigating the damage caused by inundation, the sewerage has also contributed to the improvement of the water quality of the Neya River. In about 1970, the Neya River was one of the most polluted rivers in Japan. However, its water quality has been greatly improved due to the construction of sewerage and the introduction of water from the Yodo River etc. Figure 7 shows the quality of water discharged from the Konoike and Kawamata treatment plants and the improvement of the water quality of the Neya River.

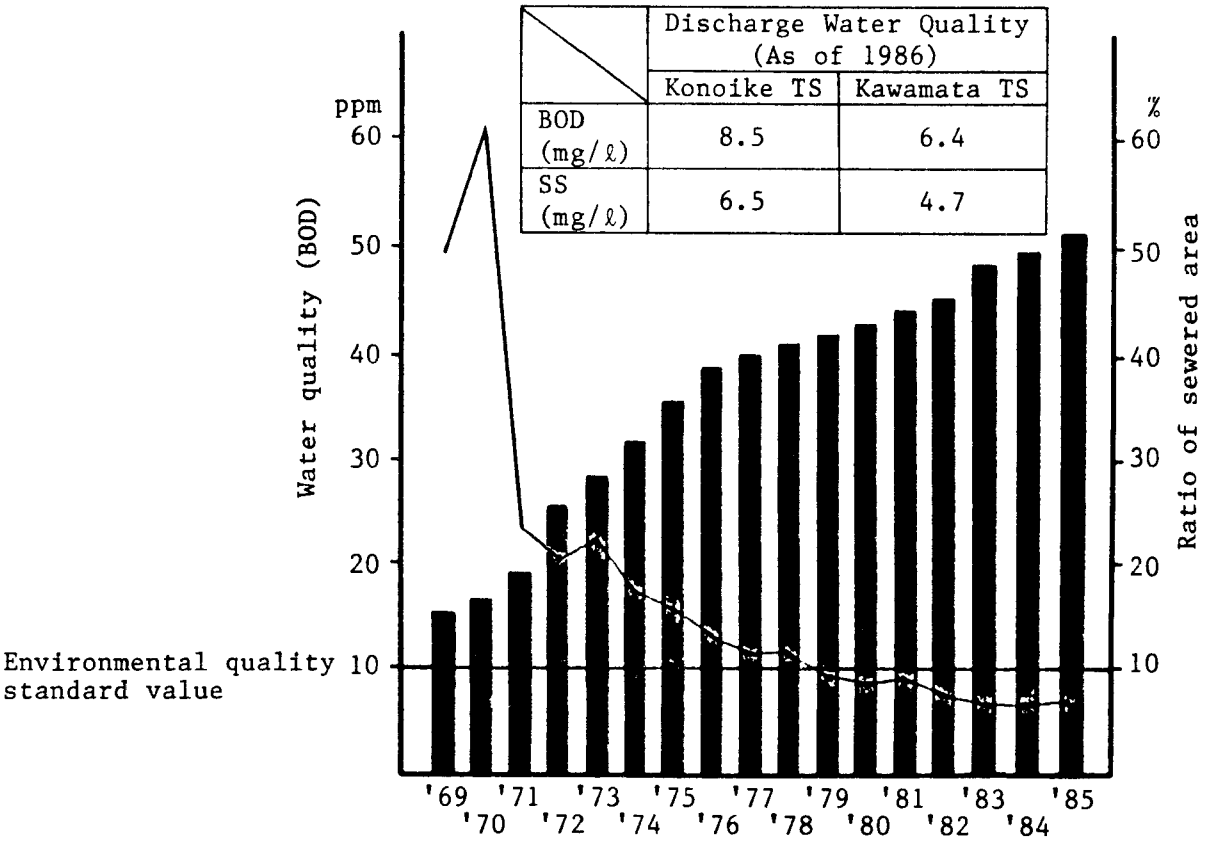


Figure 7 Sewerage Construction and Water Quality Improvement in Neya River

4. RIVER CONSERVATION WORKS

The present plan for the construction of the Neya River was revised in 1968, with the maximum rainfall recorded in Yao City on June 26 and 27, 1957, (see Figure 8), as its design rainfall. The unit hydrograph method utilizing the rational method is used for the calculation of runoff, and the storage function method is used for the inside of river channels. The runoffs from inner drainage area to river channels are those determined by the sewerage plan.

(Maximum rainfall
On June 26 and 27, 1957 (recorded in Yao))

Maximum rainfall intensity: 62.9 mm/hr Return period 1/29 (Tomas method)
Total rainfall : 326.0 mm Return period 1/250 (Tomas method)
Uniform rainfall throughout the entire drainage basin

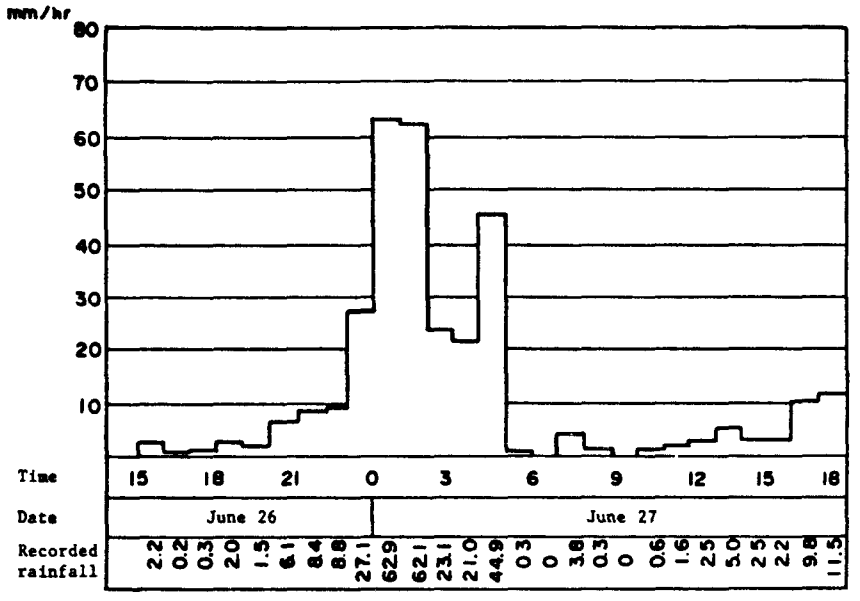


Figure 8 Design Rainfall

The basic design flood discharge obtained by the above calculation are shown in Figure 9. According to the flood-control plan, with a flow rate at Kyobashiguchi of 1,650 m^3/s, water will be discharged into the Yodo River at 425 m^3/s, into reservoirs in the Neya River Flood Retention Park etc. at 405 m^3/s and into improved river channels at a rate of 850 m^3/s. The planned distribution of design flood discharge is shown in Figure 10.

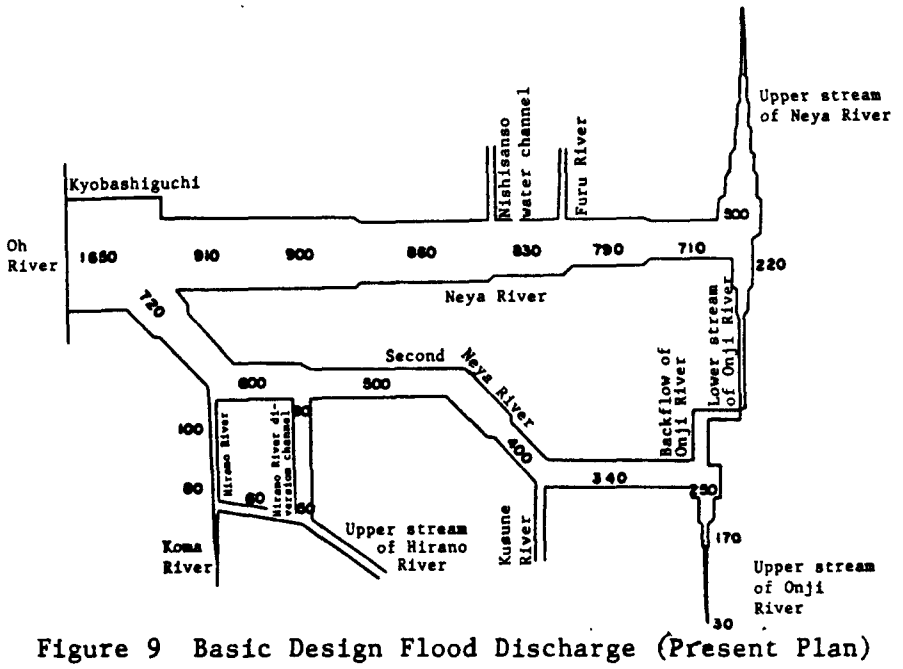


Figure 9 Basic Design Flood Discharge (Present Plan)

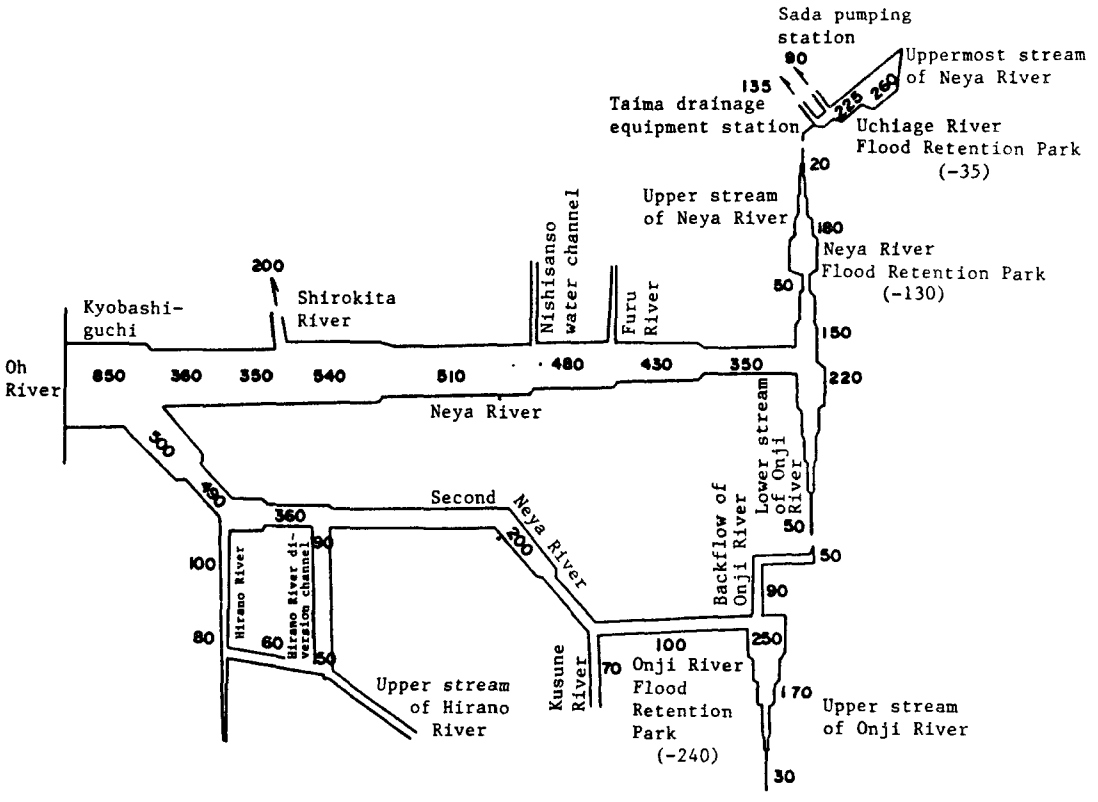


Figure 10 Distribution of Design Flood Discharge (Present Plan)

Table 4 Flood Control Plan

Basic design flood discharge 1,650 m^3/s	Improved river channels 850 m^3/s	
	Drainways 425 m^3/s	Kema Ⓟ 200 m^3/s Taima Ⓟ 135 m^3/s Sada Ⓟ 90 m^3/s
	Reservoirs 405 m^3/s	Onji River 240 m^3/s Neya River 130 m^3/s Uchiage River 35 m^3/s

As is shown in Tables 5 to 6, 82% of works for river channel improvement have already been completed with regard to the capacity of facilities for flood control. However, only 12% of works for the construction of drainways and 15% for reservoirs respectively have been completed so far, though 51% of needed land has already been acquired for reservoirs in the course of plans for the construction of multi-purpose retarding basins or flood retention park project.

Table 5 Progress of Bank Protection Works (with regard to the capacity of flood-control facilities)

(Unit: m^3/s)

	Total Plan	Capacity as of the End of 1986	Proportion of Completed Works (%)
Total	1,650	810	49
River channel improvement	850	700	82
Drainway	425	50	12
Kema P.S. Shirokita canal	200	0	0
Taima P.S. Neya River water channel	135	30	22
Sada P.S.	90	20	22
Reservoir	405	60	15
Neya River	130	60	46
Onji River	240	0	0
Uchiage River	35	0	0

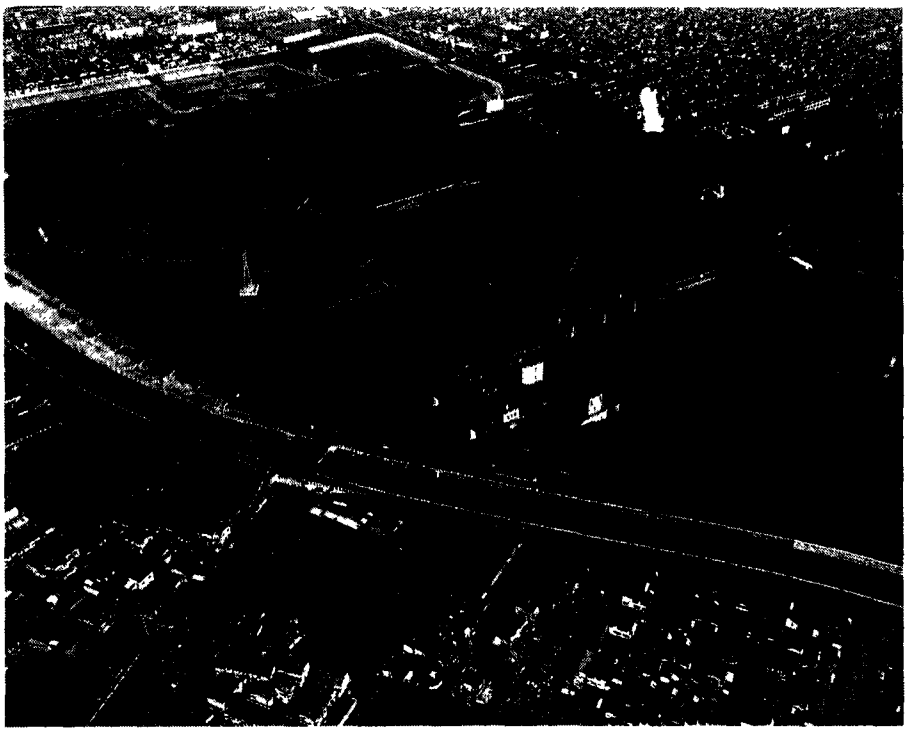
Table 6 Outline of Construction of River Flood Retention Park

1. Details of plan

River Flood Retention Park / Type		Neya River		Onji River			Uchiage River
		Total Plan	First-phase Plan	Total Plan	Ikeshima, Fukumanji	Hanazono	Total Plan
Scale	Total area (ha)	50.3	16.0	82.0	40.2	14.0	13.4
	Flood control capacity (m^3/s)	130	60	240	120	50	35
	Storage capacity (10,000 m^3)	146	42.5	290	130	30	36
	Area to be purchased (ha)	43.3	14.0	74.9	37.6	14.0	13.3

2. Progress of construction

	Neya River	Onji River	Uchiage River	Total
Acquisition of land	$\frac{42.9}{43.3}$ ha (99%)	$\frac{17.3}{74.9}$ ha (23%)	$\frac{6.7}{13.3}$ ha (50%)	$\frac{66.9}{131.5}$ ha (51%)
Progress of construction	• First-phase plan (16.0 ha) completed in 1983 • Second-phase plan started in 1983	• Under investigation as cultural assets • Started in 1985	• Land purchase underway	
Remarks	Multi-purpose retarding pond	Flood Retention Park project	Flood Retention Park project	



Picture 1 Neya River Flood Retention Park and Fukonokita Pumping Station

5. INCREASE OF RAINWATER RUNOFF

As explained above, flood-control facilities for sewerage and rivers have been drastically improved over the last twenty years. However, the increase in population and urbanization have exceeded the expectations of the later 1960s. The resulting increase in runoff and other factors have been causing damage by inundation almost every year.

To cope with this situation, the plan for the regional sewerage system has been revised drastically for the enhancement of drainage capacity, in anticipation of the level of urbanization at the beginning of the 21st century.

Table 7 shows run-off coefficients for each drainage basin according to present and revised plans. According to the present plan, the coefficient is 0.22 to 0.36, while it is 0.44 to 0.66 according to the revised plan.

In conventional sewerage plans, the rational method has been used for the calculation of rainwater runoffs, with emphasis laid on peak flow rates. However, though this method is suitable for the determination of the pipe section, it cannot be used to estimate rainwater runoff change within a time period. Accordingly, we have used the Espey Model for the evaluation of rainwater runoff control at reservoirs. This model has been chosen because it agrees with actual rainwater runoff data, requires only easy-to-prepare input data, and is applicable to a wide drainage basin. The Espey Model, as shown in Figure 11, is a method of synthesizing unit hydrographs for an

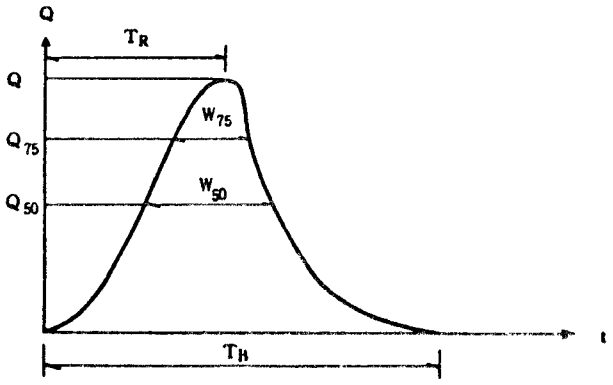
entire study area that was developed by Espey who collected data from urban watersheds in Houston and other locations throughout the United States.

Table 7 Rainwater Runoff of Each Drainage Basin

River basin	Drainage Basin (New)	Pumping Station	*1) Area (ha)	Pipe Length (m)	Pipe Gradient (‰)	Revised Plan			Present Plan		River Allowable Discharge Quantity (m^3/sec)
						Proportion of Impervious Area (%)	Runoff Coefficient	Peak Runoff (m^3/sec)	Runoff Coefficient	Peak Runoff (m^3/sec)	
Northern	Neyagawa 2nd	Taihei	549.58	4,320	1.07	54	0.57	35.236	0.24	19.125	16.000
	Neyagawa 1st	Kayashima	865.43	6,975	0.49	61	0.61	57.485	0.22	20.917	25.000
	Fukonokita	Fukonokita	224.12	3,167	0.87	54	0.57	15.065	0.30	10.049	9.000
	Kuwazai	Kuwazai	826.80	5,997	1.06	63	0.63	58.254	0.30	28.845	33.000
	Kadoma 1st	Kikusui	196.23	3,056	0.74	68	0.66	16.660	0.30	9.525	8.000
	Daito 1st Daito 2nd	Hino	693.82	3,976	0.82	45	0.51	37.047	0.26	24.102	26.000
	Matta	Matta	382.72	2,889	0.87	44	0.49	21,617	0.30	17.605	15.000
	Kadoma 3rd Daito 3rd	Konoike	1,053.01	5,916	0.73	51	0.55	59.409	0.27	33.509	36.000
Southern	Hiraoka-kawachi-minami (1)	Shijo	1,002.69	6,735	0.67	43	0.49	48.479	0.27	26.121	23.800
	Uetsuke	Uetsuke	320.33	2,711	0.87	36	0.44	14.936	0.29	15.135	15.000
	Fukono	Fukono	361.25	3,389	0.93	40	0.47	17.609	0.29	14.998	18.000
	Hiraoka-kawachi-kita (2)	Terashima	936.16	6,385	0.81	46	0.52	47.108	0.33	37,227	38.000
	Kawamata	Kawamata	1,189.54	6,332	0.86	46	0.51	60.007	0.36	49.050	53.000
	Kosaka	Kosaka	651.90	3,603	0.88	58	0.60	44.672	0.36	30.106	31.000
	Kosakai	Kosakai	444.87	3,440	0.75	47	0.53	25.017	0.32	20.997	20.900
	Shinke	Shinke	935.65	4,365	0.86	52	0.56	57.457	0.32	38.892	39.700
	Hikouzyou	Nagayoshi	1,372.70	7,653	0.75	53	0.56	78.311	0.30	38.436	36.820

*1) The area does not include rivers.

Note) The Espey Model is used for the revised plan, and the rational method for the present plan.



T_R : Time from the start of runoff to peak (min.)
T_B : Time from the start to the end of runoff (min.)
Q : Peak flow rate (m^3/sec.)
W_{50}: Time during which flow rate is more than 1/2 of peak (min.)
W_{75}: Time during which flow rate is more than 3/4 of peak (min.)

Figure 11 Espey Model

Expressions obtained via statistical analysis by Espey, with m as unit, are as follows:

$$T_R = 2.7981L^{0.315} \cdot S^{-0.0488}$$
$$Ao = AI/100$$
$$Q = 0.2477 \times 10^4 T_R^{-1.10}(Ao/640)$$
$$T_B = 0.1708 \times 10^5 (Ao/640)^{1.14} Q^{-1.15}$$
$$W_{50} = 0.2581 \times 10^4 (Ao/640)^{1.03} Q^{-1.04}$$
$$W_{75} = 0.1080 \times 10^4 (Ao/640)^{0.92} Q^{-0.94}$$

A: Subject district (ha)
L: Length of trunk sewer (m)
S: Gradient of trunk (‰)
I: Proportion of impervious area in subject district (%)

Application ranges are as follows:

A: 3.28 ∿ 23,600 ha
L: 168 ∿ 36,900 m
S: 0.47 ∿ 146 ‰)

As the peak flow rates of rainwater runoffs calculated with the Espey Model using the new run-off coefficient greatly exceed those of the present plan, the capacity of pumping stations and pipes will be insufficient if the design rainfall occurs. The rainwater runoffs according to the present and revised plans are shown in Table 7.

6. ESTIMATION OF OVERFLOW QUANTITY ACCORDING TO THE PRESENT SEWERAGE PLAN

If the design rainfall occurs under the run-off coefficient of the present sewerage plan, overflow will result in each drainage basin due to the shortage of the drainage capacity of pumping stations and pipes. Accordingly, we have estimated overflow quantity using the Espey Model for the calculation of rainwater runoff. We have used the SKM model for flow in pipes, and conducted unsteady flow analysis which represents the artesian condition.

The SKM model, used for the calculation of unsteady flow with pressure pipe conditions, was made by modifying the pipe runoff model named "SURKNET", which was introduced by Yen and others at the Second International Conference on Urban Storm Drainage held at University of Illinois in 1981. In this model, the outflow system is divided into the flow in open channel conditions, the transition state from open channel to pressure pipe and the flow in pressure pipe conditions.

The basic expressions of the model are as follows:

① Open-channel flow

$$\frac{I_1 + I_2}{2} - \frac{Q_1 + Q_2}{2} = \frac{S_2 - S_1}{\Delta t}$$

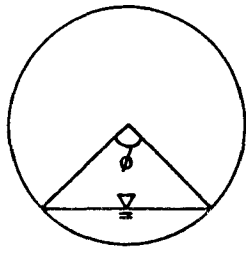
$$S = 0.182 \cdot n^{3/5} \cdot So^{-3/10} \cdot \Delta L \cdot (\phi \cdot D)^{2/5} \cdot (I^{3/5} + 5Q^{3/5})$$

② Flow in pressure pipe conditions

$$Hu - Ku \cdot \frac{V^2}{2g} - L \cdot \left(Sf + \frac{1}{g}\frac{\partial v}{\partial t}\right) = Hd + Kd\frac{V^2}{2g}$$

$$Q = Af \cdot V$$

I : Inflow
Q : Runoff
S : Storage quantity
So: Pipe gradient
L : Pipe length
ΔL: Pipe infinitesimal interval
ϕ : (See the right figure.)
n : Roughness coefficient
D : Pipe diameter
H : Manhole water level
Sf: Friction gradient
K : Ratio of loss
V : Velocity of flow
Affix 1, 2: Time
Affix u, d: Upper stream, lower stream



Pipe section

The flow sheet of the SKM model is shown in Figure 12. This model is used to calculate pipes and manholes in the regional sewerage system. Rainwater runoffs are calculated with the Espey Model for each junction between the public sewerage and the regional sewerage system.

According to the simulation using this SKM model, the overflow quantity of the entire river basin is approx. 930,000 m^3, approx. 60% of which comes from the northern basin. Overflow quantities are especially large in Taihei, Kuwazai and Konoike drainage basins in the northern river basin and Shijo and Nagayoshi drainage basins in the southern basin. The overflow quantity of each drainage basin is shown in Table 8.

Table 9 shows the overflow area for each overflow level shown in Table 10 obtained from these overflow quantities.

The overflow area in the entire northern basin is 387 ha: namely, overflow occurs in approx. 10% of the combined drainage basin area in the northern river basin. While the level of overflow is 0 to 5 cm in approx. 70% of the area (263 ha), the rest of the area mostly suffers with a heavy overflow level of 30 cm or more.

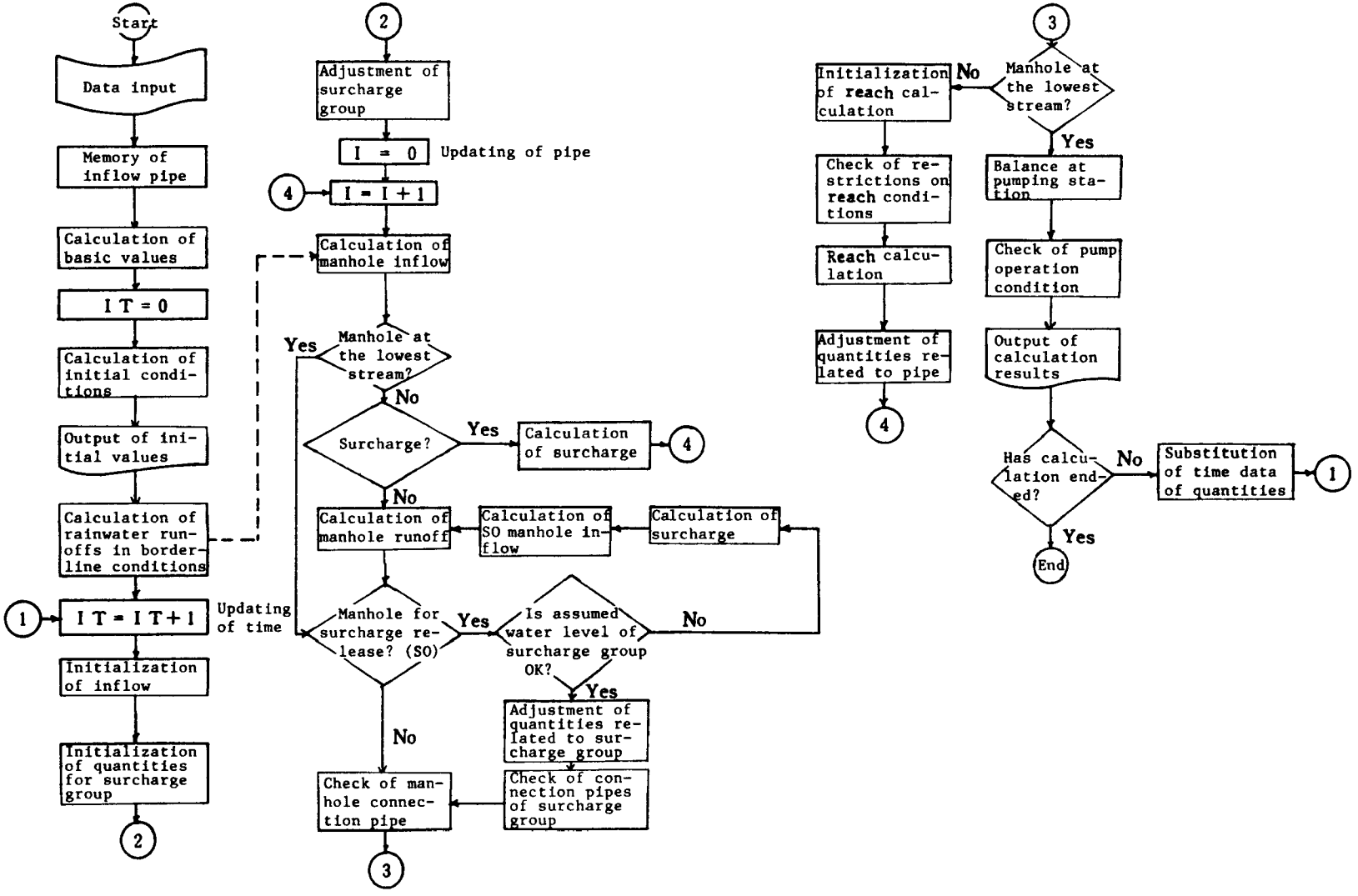


Figure 12 Flow Sheet of SKM Model

Table 8 Estimated Overflow Quantity of Each Drainage Basin

River basin	Pumping Station	Drainage Basin	Overflow Quantity (m^3)
Northern	Taihei	Neyagawa 2nd	174,100
	Fukonokita	Fukonokita	6,400
	Kuwazai	Kuwazai	91,400
	Kikusui	Kadoma 1st	8,500
	Hino	Daito 1st Daito 2nd	37,800
	Matta	Matta	0
	Konoike	Kadoma 3rd Daito 3rd	236,100
	Total		554,300
Southern	Shijo	Hiraoka-kawachi-minami (1)	130,600
	Uetsuke	Uetsuke	0
	Fukono	Fukono	0
	Terashima	Hiraoka-kawachi-kita (2)	3,300
	Kawamata	Kawamata	58,600
	Kosaka	Kosaka	47,900
	Kosakai	Kosakai	2,800
	Shinke	Shinke	22,000
	Nagayoshi	Hikouzyou	111,900
	Total		377,100
Total			931,400

Table 9 Overflow Level and Area (ha)

River basin	Pumping Station	Drainage Basin	Overflow Level					
			① (cm) 0 ∿ 5	② 5 ∿ 10	③ 10 ∿ 30	④ 30 ∿ 50	⑤ Over 50	Total
Northern	Taihei	Neyagawa 2nd	—	—	—	46.0	1.6	47.6
	Fukonokita	Fukonokita	—	—	7.2	—	2.4	9.6
	Kuwazai	Kuwazai	—	—	—	13.8	1.8	15.6
	Kikusui	Kadoma 1st	—	—	0.8	—	—	0.8
	Hino	Daito 1st Daito 2nd	—	—	3.2	5.0	0.1	8.3
	Matta	Matta	—	—	—	—	—	—
	Konoike	Kadoma 3rd Daito 3rd	263.5	—	—	—	41.5	305.0
	Total		263.5	—	11.2	64.8	47.4	386.9
Southern	Shijo	Hiraoka-kawachi-minami (1)	—	—	79.4	—	4.6	84.0
	Uetsuke	Uetsuke	—	—	—	—	—	—
	Fukono	Fukono	—	—	—	—	—	—
	Terashima	Hiraoka-kawachi-kita (2)	12.5	—	—	—	—	12.5
	Kawamata	Kawamata	—	—	—	18.3	—	18.3
	Kosaka	Kosaka	—	—	16.8	—	4.0	20.8
	Kosakai	Kosakai	32.8	—	—	—	—	32.8
	Shinke	Shinke	68.0	—	—	—	2.8	70.8
	Nagayoshi	Hikouzyou	—	—	90.1	—	3.5	93.6
	Total		113.3	—	186.3	18.3	14.9	332.8
Total			376.8	—	197.5	83.1	62.3	719.7

Table 10 Overflow Level

Overflow Level	Overflow (cm)
1	0 ∿ 5
2	5 ∿ 10
3	10 ∿ 30
4	30 ∿ 50
5	Over 50

In the southern basin, the overflow area is 333 ha, which forms 5% of the drainage basin area. The degree of overflow is less serious than in the northern basin: the level of overflow is 0 to 5 cm in 34% of the area (113 ha), and 10 to 30 cm in 56% (186 ha).

7. MEASURES AGAINST RUNOFF INCREASE

The increase in rainwater runoff not only exceeds the drainage capacity of sewerage pipe and pump equipment, but also gives rise to a more serious problem, namely the necessity to control rainwater discharge via sewerage drainage pumps, as the drainage capacity of rivers also becomes insufficient.

Measures to cope with the insufficiency of the drainage capacity of sewerage pipes include:

① Construction of bypass routes
② Discharge via hydraulic gradient
③ Construction of reservoir
④ Introduction of partial separate sewer system

The problems resulting from each measure are listed in Table 11. As each measure has its own merits and demerits, these measures should be used in combinations appropriate for each case.

Table 11 Measures for the Improvement of Pipe Drainage Capacity

Measure	Method	Problem
1. Bypass	• To convey water which cannot be drained by conventional trunk sewers through a new route	• In many cases, it is difficult to use the same route for this purpose. • A separate route must be selected.
2. Hydraulic gradient method	• To discharge using hydraulic gradient, taking backwater into consideration	• Pressure pipe condition may occur partially. • Abrupt pressure increase occurs. • Overflow from manhole may occur.
3. Change of connection points	• To change the connection points of the public sewerage connected to regional sewerage system into the lower stream (bypass route scheme using public sewerage connected to regional sewerage system)	• The burden on trunk sewers in the lower stream increases. • The plans of public sewerage connected to regional sewerage system must be changed.
4. Partial separate sewer system	• To discharge water which cannot be drained by the sewerage through existing water channels etc.	• Though the water-retaining effect will improve, the insufficiency of the drainage capacity of rivers cannot be addressed. • The water channel network must be restructured.
5. Extension of pipes	• To reconstruct pipes	• It is difficult to execute drainage while reconstructing pipes.
6. Reservoir	• To temporarily store water which cannot be drained into reservoirs	• The maintenance of reservoirs is required.

Measures for the control of rainwater discharge are listed in Figure 13. We deem the off-site storage superior to other measures because:

① the effect on related public sewerage is minimal,
② improvement of constructed pipes is difficult, and
③ this measure allows a centralized and manageable maintenance and operation system.

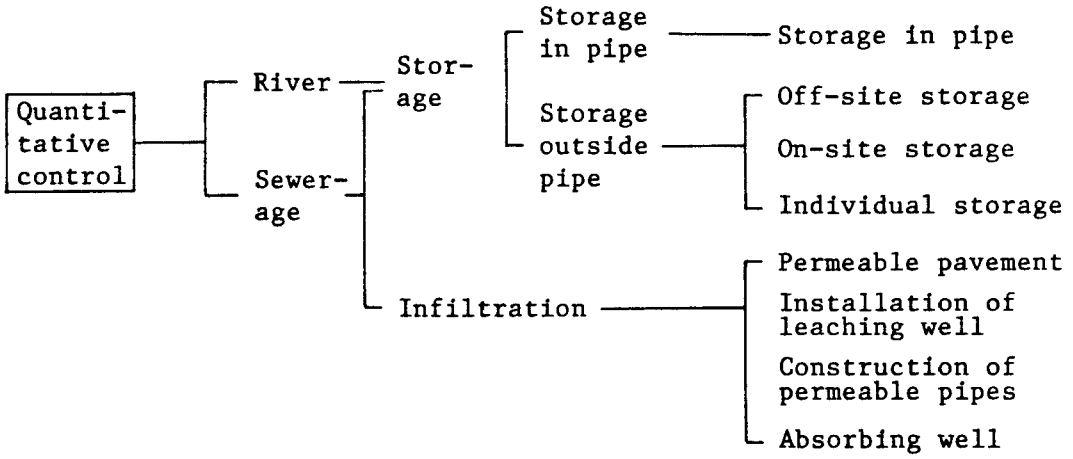


Figure 13 Quantitative Control Method

Accordingly, off-site storage should be considered as the primary measure against stormwater with regard to sewerage, though the first task is to improve the discharge capacity of rivers.

After adjusting the river conservation plan, we have taken the following steps to revise the sewerage plan.

① To increase the run-off coefficient to cope with the expansion of city area
② To change the probability of the occurrence of the design rainfall from once every five years to once every ten years

We plan to cope with the runoff increase caused by step ① by increasing drainage pumps, and that caused by step ② by establishing storage facilities in the sewerage pipe system.

8. REDUCTION OF POLLUTIONAL LOAD OF RAINWATER DISCHARGE FROM COMBINED SEWERAGE THROUGH SEWERAGE STORAGE

The effect of the pollutional load of rainwater discharge from the combined sewerage on the public water body is estimated at a considerably high level. This load can be reduced by making reservoirs to prevent floods.

We examined the reduction of the river discharge pollutional load via reservoirs, using the pile load model for the runoff pollution at rainfall.

In this model, it is assumed that runoff load L from a drainage basin is proportional to the αth power of rainwater runoff quantity Q and pile load P in the drainage area. The pile load, as the pollutional load under dry weather is assumed to be constant, is represented as follows:

$$\frac{dP}{dt} = -L + (DWL) = -KPQ^{\alpha} + (DWL)$$

This equation can be worked out as:

$$P = P_0 e^{-\int KQ^{\alpha} dt} + e^{-\int KQ^{\alpha} dt} \cdot \int (DWL) e^{\int KQ^{\alpha} dt} \, dt$$

P_0 : Initial value of pile load
K, α : Coefficient
(DWL): Pollutional load under dry weather per unit time

According to a study by the Public Works Research Institute of the Ministry of Construction, K and α can be determined as follows:

$$K_{BOD} = 1.14326 \times 10^{-3} \cdot I^{0.75082} / e^{0.00789A}$$

$$\alpha_{BOD} = 1.35122 + 0.05026 \, \ell nS + 0.10739 \, \ell nI$$

A: Drainage area (ha)
S: Surface gradient (‰)
I: Trunk sewer gradient (‰)

Among various methods to control reservoirs, we have examined reservoirs of the kind shown in Fig. 14. By this method, an influent exceeding the dry weather flow quantity is let into reservoirs until they become full, in order to reduce the peak influent load at the initial stage of rainfall.

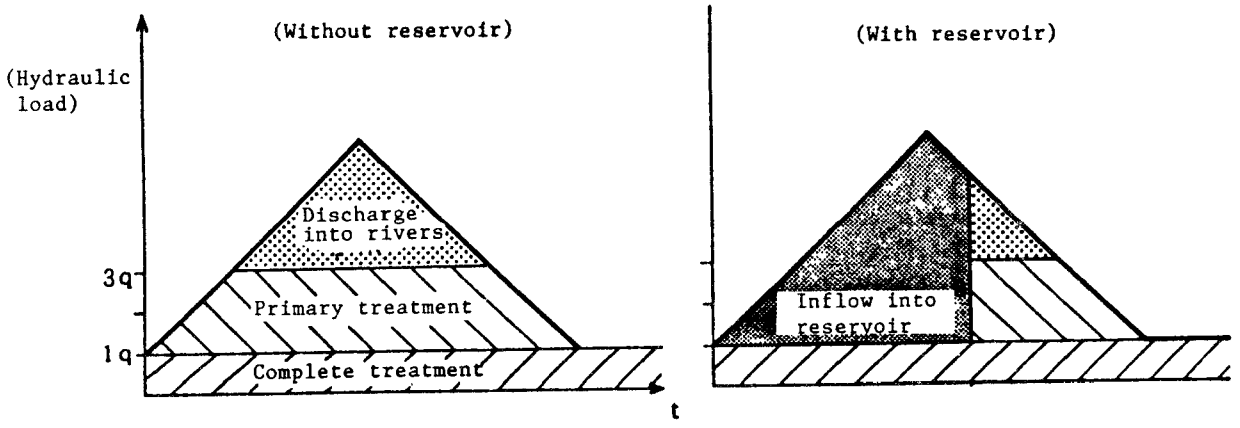


Figure 14 The System of Reservoir

The capacity of reservoirs in each drainage basin is determined as in Table 12 on the assumption that underground reservoirs under roads and parks are used as well as channels.

Table 12 Capacity of Reservoir

Pumping Station	Capacity of Reservoir (m^3)	Method	Location
Taihei	74,000	Underground storage	Area for regulation, park
Fukonokita	10,000	"	Flood Retention Park
Kikusui	10,000	Storage in tunnel	Central loop line
Kuwazai	60,000	Underground storage	Area for regulation, fields
Hino	7,000	"	Flood Retention Park
Konoike	86,000	Storage in tunnel	Central loop line
Shijo	94,000	Underground storage	Flood Retention Park
Kawamata	36,000	Storage in tunnel	Central loop line
Kosaka	49,000	"	"
Shinke	34,000	"	"
Nagayoshi	68,000	Storage in channel	Hirano River, channel in airport

Rainfall data collected at the Kuwazai pumping station in 1976 are taken as design rainfall, as they approximate the average value. Among a total 103 rainfalls, 7 with a maximum hourly rainfall exceeding 20 mm are excluded as the objective for flood control measures. The rest are divided into 5 ranks according to rainfall, each conceived after the centralized rainfall model.

Under these assumptions, the runoff pollutional load can be reduced at each pumping station, as is shown in Table 13. This result shows that reservoirs are effective in reducing the pollutional load at rainfall, as well as in controlling rainwater runoff.

Table 13 Reduction via Reservoir

(Unit: %)

Pumping Station	Design Rainfall Flow (mm) / Non-exceedance Probability	0∿5 / 49%	5∿10 / 65%	10∿20 / 80%	20∿30 / 90%	Over 30 / 100%	Annual Reduction
Taihei		—	—	100	99	90	92
Kuwazai		—	60	29	14	10	15
Fukonokita		—	100	86	71	58	76
Kikusui		—	100	92	57	33	61
Hino		—	63	40	21	18	29
Konoike		—	71	47	22	17	29
Kawamata		—	56	48	31	17	28
Kosaka		—	26	17	11	11	14
Shinke		—	76	49	28	18	30
Shijo		—	100	100	100	79	90
Nagayoshi		—	—	—	100	64	66

Note: — indicates that the river discharge load is 0 even without reservoirs.

9. COMPREHENSIVE FLOOD DISASTER PREVENTION PLAN

In consideration of this revision of the sewerage plan, a comprehensive flood disaster prevention plan for the Neya River has also been prepared to promote flood control by facilities in river and sewerage and through measures for facilities for runoff control. In this plan, the design rainfall is the maximum recorded rainfall in Yao in June, 1957, so that the recurrence of disasters of past scale will be prevented. A hydrograph based on the runoff from the entire basin comprising inner and outer drainage areas is used to define the river basin basic design flood discharge as the basic quantity for the treatment plan for the entire basin. The calculation of runoff is based on the run-off coefficient $f = 0.8$ in the outer drainage area (61.9 km^2), as in the present plan, and $f = 0.42$ to 0.8 in the inner drainage area (205.7 km^2), as in the revised drainage plan. The hydrograph at Kyobashiguchi according to this plan is shown in Figure 15: the peak flow rate of the river basin basic design flood discharge is 2,700 m^3/s.

This runoff increase comes solely from the inner drainage area in the Neya River basin, which is wholly comprised by the sewerage design area. As shown in Figure 16, the increase will be countered by flood-control facilities in river and sewerage, as well as those for the facilities for runoff control.

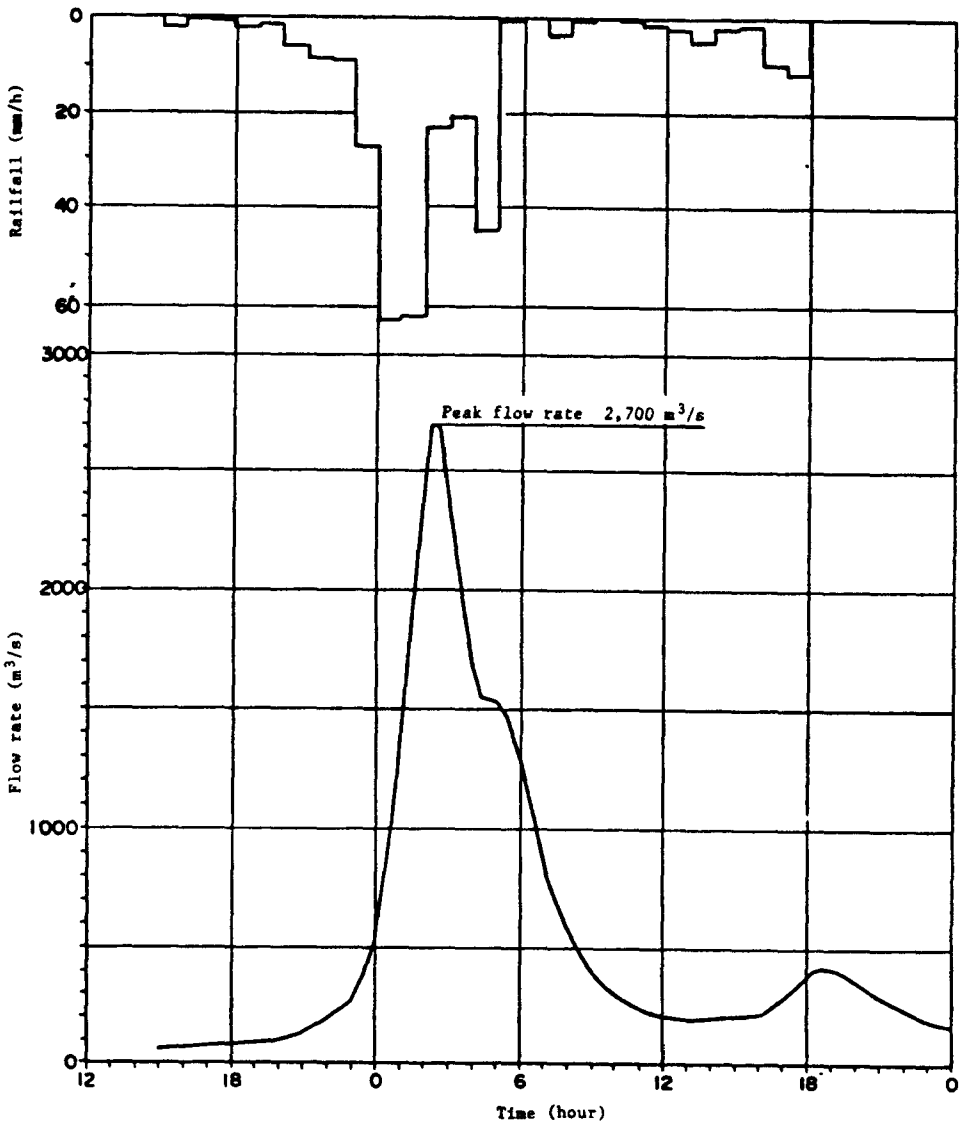


Figure 15 Basin Basic Design Flood Discharge (at Kyobashiguchi)

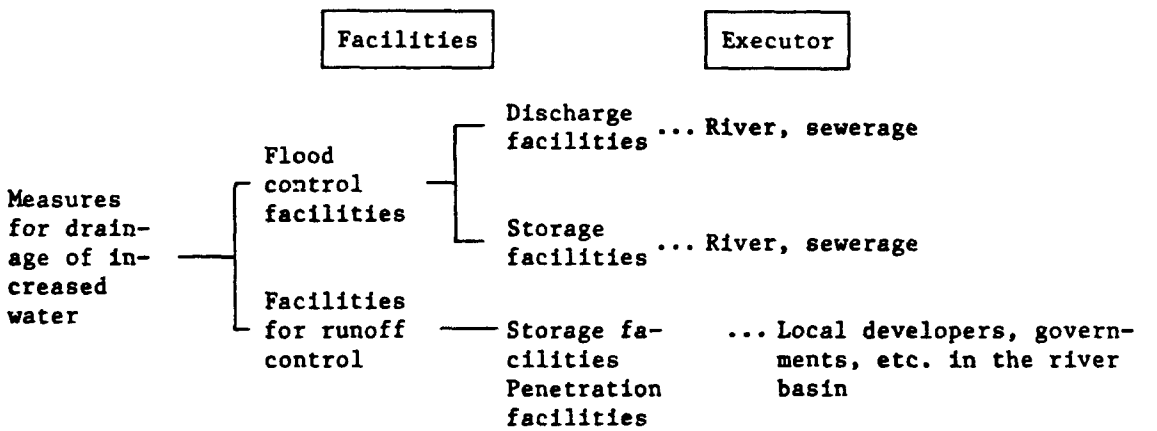


Figure 16 Measures for Drainage of Water in Inner Drainage Area

10. COORDINATION OF SEWERAGE PLAN AND RIVER CONSERVATION PLAN

The runoff at rainfall in the present Neya River regional sewerage system plan was determined during an investigation made in 1963 and 1964. The design rainfall was calculated according to the rainfall intensity method using the Talbot type formula (probability: once every five years), and the run-off coefficient was set at very low values of 0.22 to 0.36.

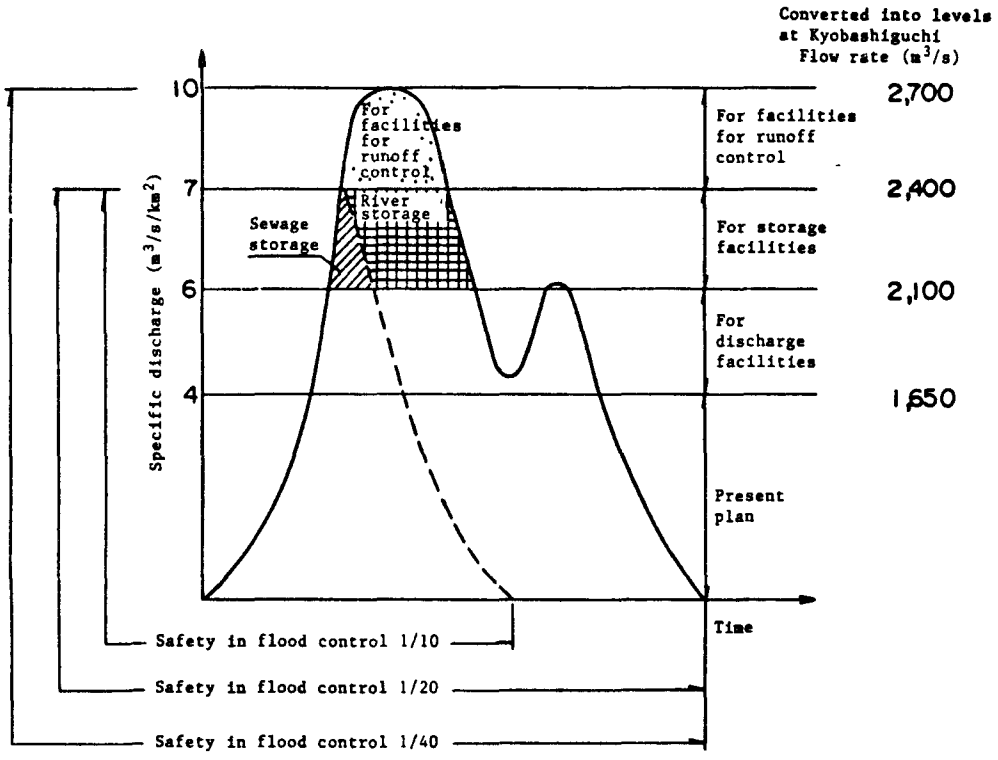


Figure 17 Flood Control Task Allotment

Since then, the run-off coefficient has increased due to the progress of urbanization in the Neya River basin. Also, the return period of the design rainfall will be changed from the present 5 years to 10 years, the long-term goal for sewerage improvement set at the Central City Planning Council held in 1985. As a result, the peak flow rate of the runoff at the Neya River will be 2,400 m^3/s. This value corresponds to the basic design flood discharge as the basic quantity for the flood control plan.

The mean specific discharge derived from the peak runoff for the design rainfall in the new sewerage plan (probability: 1/10) is 7 $m^3/s/km^2$. Accordingly, the comprehensive flood disaster prevention plan will cope with runoff up to a flow rate of 7 $m^3/s/km^2$ by the following flood control facilities:

(1) New discharge channels such as tunnels
(2) Reservoirs created through multi-purpose utilization of public facilities such as parks and roads

Runoffs at a level between 2,400 and 2,700 m^3/s, which are beyond the scope of the new sewerage plan, will be countered through runoff control measures which are implemented in the course of the effective utilization of public facilities such as roads and parks, the land use plan for housing, agriculture etc., and the redevelopment of the city area.

Also, as the second of the two peaks of the hydrograph for the comprehensive flood disaster prevention plan roughly corresponds to the runoff level whose probability is once every five years with regards to flood control facilities, we will cope with runoffs up to the second peak using discharge facilities such as new tunnels in view of the efficiency of operation and the cost of the facilities. Runoffs exceeding this level will be managed by storage facilities.

11. SUMMARY

Archaeological excavation has proved that men lived together in the Neya River basin since prehistoric times. It is probable that river improvement was the greatest concern for people in that era as well. As civilization progressed, river improvement also made great advancements: the changing of the course of the Yamato River in the beginning of the 18th century was the most large-scale enterprise.

While irrigation works have been carried out continuously since that time, city drainage projects have occurred on a far greater scale since the end of World War II. The effectiveness of the flood control measures in this river basin is proven by the fact that the benefit of the efficient land use and the accompanying rise in the price of land exceeds the investment for flood control by tens of times.

It is expected that the comprehensive flood disaster prevention plan comprising the revised sewerage and river conservation plans, and efforts by local governments and developers will enhance safety against flood in this river basin and promote the development of this region.

FULL-SCALE EVALUATION OF PHOSPHORUS MASS BALANCE IN BIOLOGICAL PHOSPHORUS REMOVAL PROCESS

by

Naohiro TANIGUCHI
Senior Researcher, Research and Technology Development Division, Japan Sewage Works Agency

Takao MURAKAMI
Senior Research Engineer, Ultimate Disposal Section, Public Works Institute, Ministry of Construction

Shuzou KOIKE
Researcher, Research and Technology Development Division, Japan Sewage Works Agency

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

Prepared for Presentation at:
11th United States/Japan Conference
on
Sewage Treatment Technology

October 1987
Tokyo, Japan

ABSTRACT

A series of experiments was conducted in the field to clarify the phosphorus removal efficiency and phosphorus mass balance of biological phosphorus removal systems, including the sludge treatment process, and to determine the system with the best biological phosphorus removal efficiency. In checking the adverse effect of the phosphorus load of the return flow from the sludge treatment process on the phosphorus removal efficiency, it was found that the phosphorus load of the return flow from the sludge treatment process should be kept less than 50 percent of the phosphorus load of the raw sewage. Experiments disclosed the following requirements to achieve sufficient phosphorus removal.

(1) Inorganic coagulant should preferably be dosed to the sludge if gravitational thickening is used.

(2) The primary settling tank sludge and the excess sludge should be thickened separately, and the excess sludge should preferably be thickened mechanically. For dewatering, however, it is recommended that the separately thickened sludge be mixed.

CONTENTS

1. INTRODUCTION 238

2. CONFIGURATION AND OPERATION OF THE WASTEWATER TREATMENT PLANTS USED FOR STUDY 239
2-1. OUTLINE OF THE PLANTS 239
2-2. OPERATING CONDITIONS 241

3. TEST RESULTS AND DISCUSSIONS 243
3-1. AN OUTLINE OF PHOSPHORUS REMOVAL 243
3-2. PHOSPHORUS CONTENT IN SLUDGE 246
3-3. CHANGES IN PHOSPHORUS CONCENTRATION OF SLUDGE TREATMENT PROCESS RETURN FLOW 247
3-4. PHOSPHORUS MASS BALANCE IN PHOSPHORUS REMOVAL PROCESS 248
3-5. ACCUMULATION OF PHOSPHORUS IN THE PROCESSES 251
3-6. PHOSPHORUS LOAD BY RETURN FLOW 252
3-7. QUANTITY OF PHOSPHORUS DISCHARGED BY THE DEWATERED CAKE FROM THE PROCESS 254
3-8. EFFECT OF CHANGES IN INFLUENT LOAD UPON PHOSPHORUS REMOVAL .. 255

4. SLUDGE TREATMENT IN BIOLOGICAL PHOSPHORUS REMOVAL PROCESS 256
4-1. SLUDGE THICKENING 256
4-2. FIXATION OF PHOSPHORUS IN THE SLUDGE 258
4-3. DEWATERING OF SLUDGE 260

5. CONCLUSION 263

6. ACKNOWLEDGMENTS 264

1. INTRODUCTION

In the 1960s, the course taken by the Japanese Government for ecomic growth was accompanied by aggravation of water pollution in lakes, swamps and other public waters. Most serious things among the others were closed water bodies where eutrophication went from bad to worse. In Kyoto which relies as a water source on Lake Biwa, the largest lake in Japan, citizens were beginning to complain of musty water in 1969. As eutrophication progressed, the Lake suffered red tide for the first time in 1977. Since then, the red tide contamination has been recurring almost every year.

At present, this kind of water pollution problem is more or less seen everywhere in Japan. Concerned about the situation, the Government amended the environmental standards in 1982 to add provisions relating to nitrogen and phosphorus, and designated 45 lakes and swamps by types.

Wastewater treatment plants which are discharging into closed water bodies are studying the introduction of advanced wastewater treatment processes to provide against eutrophication, pursuant to the tightened water quality requirements. Of the advanced wastewater treatment processes, the biological phosphorus removal process is attracting attention as it can be implemented comparatively easily by modifying the existing conventional activated sludge process.

It is reported, however, that the biological phosphorus removal process will not work if the sludge treatment process is incorrectly implemented, because phosphorus removed in the biological reactor is carried back to the wastewater treatment process together with the return flow to impair the overall phosphorus removal efficiency.

Research and development for the implementation of biological phosphorus removal processes has so far centered around the removal function of phosphorus in the wastewater treatment process. There have been few, if any, pilot plant studies on the behavior of phosphorus in sludge treatment process and the effect of phosphorus load in return flow.

This paper deals with a field study conducted on a biological phosphorus removal process to clarify the phosphorus removal performance and phosphorus mass balance of the entire wastewater treatment system, including sludge treatment process, and to provide a process that can maximize the biological phosphorus removal efficiency.

2. CONFIGURATION AND OPERATION OF THE WASTEWATER TREATMENT PLANTS USED FOR STUDY

2-1. OUTLINE OF THE PLANTS

This study was conducted at The Shinjiko Tobu Purification Center (hereinafter referred to as S plant), Shimane Pref., and the Kase Sewage Treatment Plant (hereinafter referred to as K plant), Kawasaki city.

The S plant is designed with the conventional activated sludge process to treat municipal wastewater from a separate sewer system. Its sludge treatment system is made up of a gravity thickeners, non-heated anaerobic digestion tanks and belt press filters.

The K plant is operating on the conventional activated sludge process for the treatment of municipal wastewater from a combined sewer system. During our tests, half was operated on the anaerobic aerobic process for comparison with the remaining half operated on the conventional activated sludge process. The sludge treatment system is made up gravity thickeners, filter presses and a sludge melting furnace. A centrifugal thickener was introduced when the study was in progress, and the sludge from the primary settling tank was processed in the gravity thickeners as usual while the excess sludge was thickened centrifugally.

Figures 1 and 2 show the process flow sheets of these plants. The discussions in this paper are made more about the S plant than about the K plant which is dealt with supplementarily.

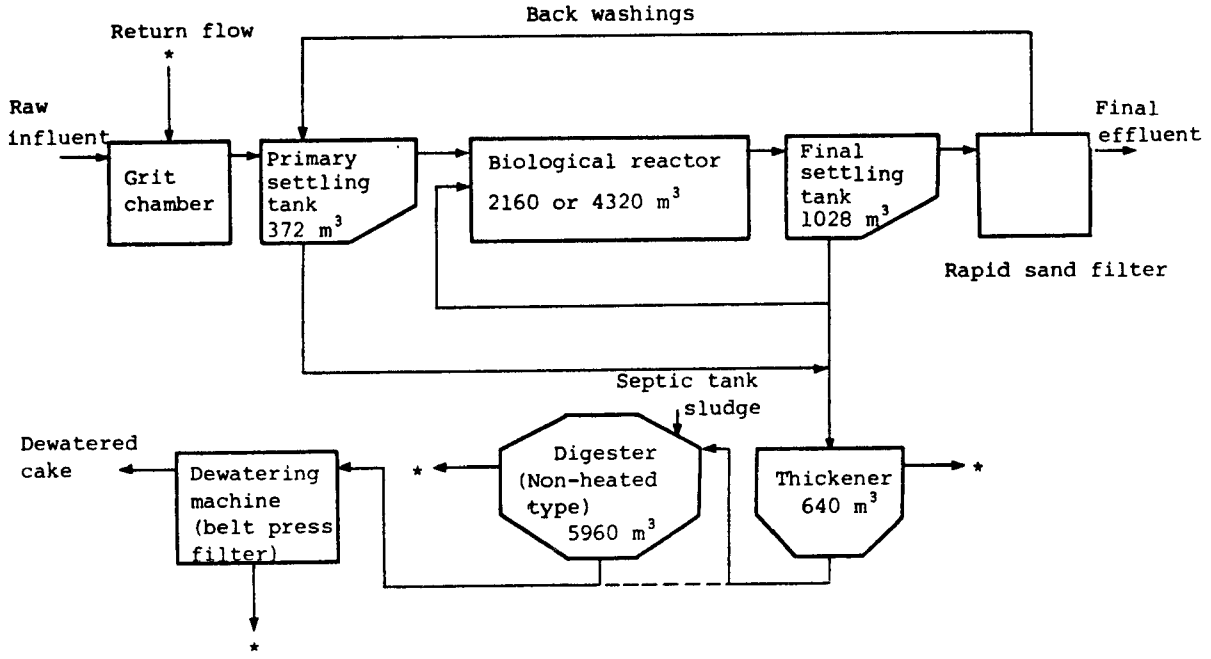


Figure 1. S plant flow sheet

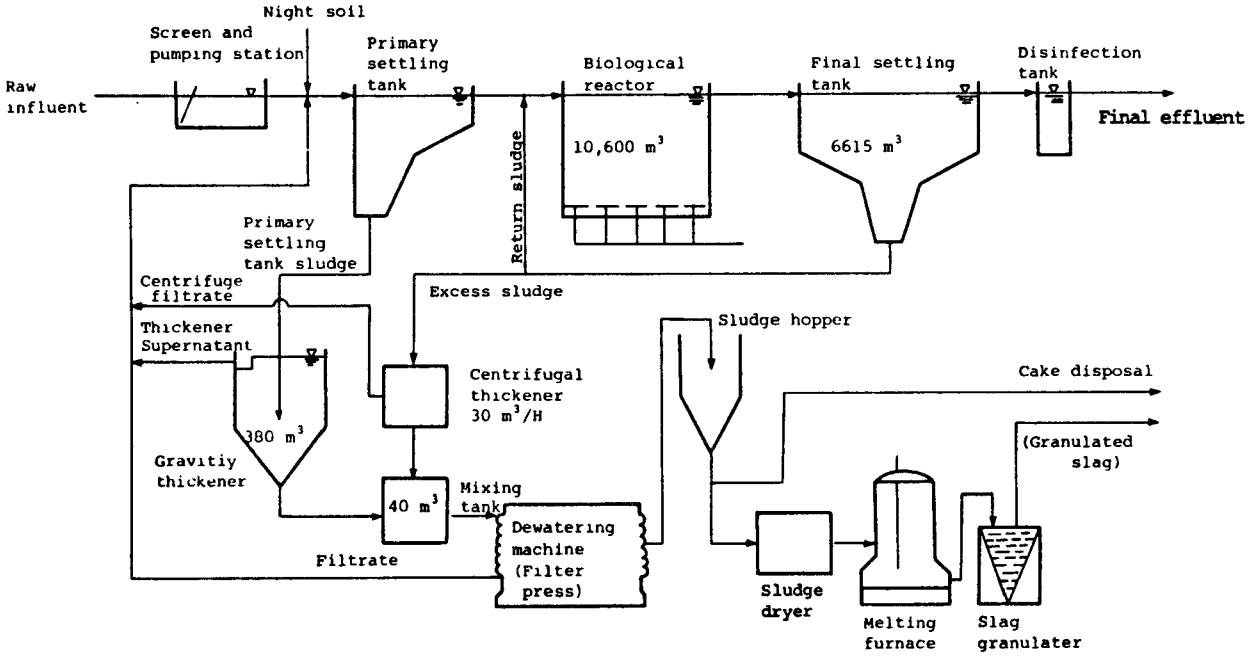


Figure 2. K plant flow sheet

2-2. OPERATING CONDITIONS

Table 1. Running conditions for S plant

Mode	RUN	Reactor water temperature (°C)	Primary settling tank outflow (m^3/day)	Return sludge (m^3/day)	Reactor retention time (hrs.)	Reactor MLSS (mg/l)	Reactor DO (mg/l)	SRT (day)
I	1	26.6	9,436	3,698	5.5	2,380	1.4	9.6
	2	24.8	9,337	3,706	5.6	2,380	1.1	10.6
	3	23.4	9,181	3,605	5.6	3,430	3.6	6.8
	4	22.3	9,244	3,604	5.6	3,120	4.2	6.0
	5	20.6	9,308	3,668	5.6	3,020	4.3	5.8
	6	19.0	9,276	3,590	5.6	3,130	5.0	4.9
	7	17.0	9,150	3,417	5.7	2,840	5.6	4.5
	8	15.4	9,443	3,491	5.5	2,840	4.1	4.7
	9	14.4	8,005	3,249	6.5	3,020	6.0	7.0
II	10	14.1	8,900	3,453	11.6	2,840	5.4	14.7
	11	14.0	8,681	3,776	11.9	3,590	5.6	19.8
	12	13.6	9,376	4,433	11.1	3,820	3.6	18.9
	13	14.0	9,239	4,642	11.2	4,050	4.5	18.1
	14	15.0	10,169	5,139	10.2	3,680	3.4	15.7

Note 1: The data are given in mean values.

Note 2: Reactor retention time refers to the flow not including return sludge.

Note 3: Reactor MLSS and aeration tank DO refer to the values at the tail-end compartments in Modes I and II.

Given in Table 1 are the conditions under which the S plant was operated. The span of each test run was about 15 days. For Runs 1 through 9, the S plant was operated in Mode I in which the inflow was set to meet the rated plant capacity (9,000m^3/day). For Runs 10 through 14, the S plant was operated in Mode II in which the reactor capacity was doubled with the inflow fixed the same as in Mode I.

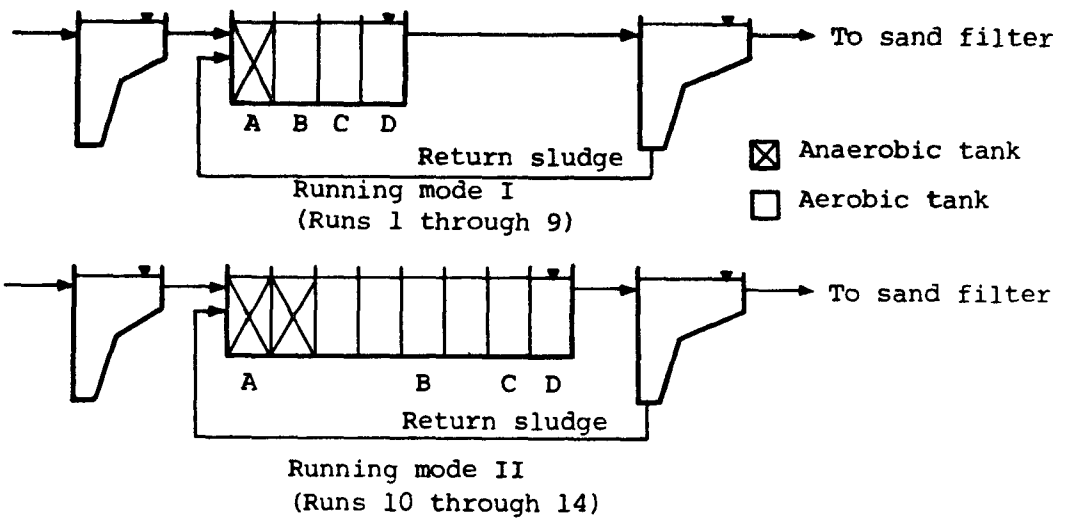


Figure 3. Running modes (S plant)

Table 2. Running conditions for K plant

RUN	Reactor water temperature (°C)	Primary settling tank outflow	Return sludge (m^3/day)	Volumetric aeration ratio (times)	Reactor retention time (hrs.)		MLSS (mg/l)	SRT (day)	SAT (day)
					Anaerobic tank	Aerobic tank			
1	20,8	45,100	12,100	3.67	2.11	3.59	1,960	4.04	2.55
		45,100	12,000	4.39	-	5.70	2,000	4.33	4.33
2	24,2	51,700	12,000	2.98	1.84	3.14	2,010	6.01	3.79
		45,600	12,000	3.72	-	5.67	2,080	4.75	4.75
3	21,4	46,800	12,000	3.22	2.03	3.46	2,400	12.85	8.10
		42,400	12,100	3.79	-	6.06	2,400	9.78	9.78
4	17,9	43,600	10,300	2.89	2.17	3.70	2,470	10.74	6.77
		40,900	10,400	3.55	-	6.25	2,790	13.59	13.59
5	15,5	42,200	10,500	2.41	2.56	4.35	2,710	9.81	6.18
		40,800	10,600	2.55	-	6.26	2,650	9.31	9.31
6	13,7	44,800	10,700	1.98	2.11	3.59	2,440	7.27	4.58
		39,500	11,600	2.51	-	6.47	2,270	6.80	6.80
7	13,7	45,300	11,600	2.39	2.09	3.56	2,360	5.90	3.72
		40,000	13,800	3.41	-	6.40	2,320	11.06	11.06

Note: Upper row; Anaerobic-aerobic process
Lower row; Conventional activated sludge process

Table 2 shows the operating conditions of the K plant. The volumetric aeration ratio (volumetric flow ratio of air to treated wastewater) was changed from run to run. From Run 1 to Run 7, the volumetric aeration ratio was reduced while observing the changes in effluent quality. From Run 3 on, the centrifugal thickener was operated for the thickening of excess sludge.

3. TEST RESULTS AND DISCUSSIONS

3-1. AN OUTLINE OF PHOSPHORUS REMOVAL

In the S plant, an experiment on the anaerobic-aerobic process was started in August, 1985. Just after the start of the experiment, phosphorus was liberated in the anaerobic tank, and the phosphorus concentration in the effluent started falling. The effluent from the primary settling tank was mixed with the return sludge, and was allowed to fall from a height of about 50 cm into the biological reactor. While the DO level at the foot of the fall was about 2 mg/l, the ORP level could be maintained within a range of -60 to -210 mV at other locations of the anaerobic tank.

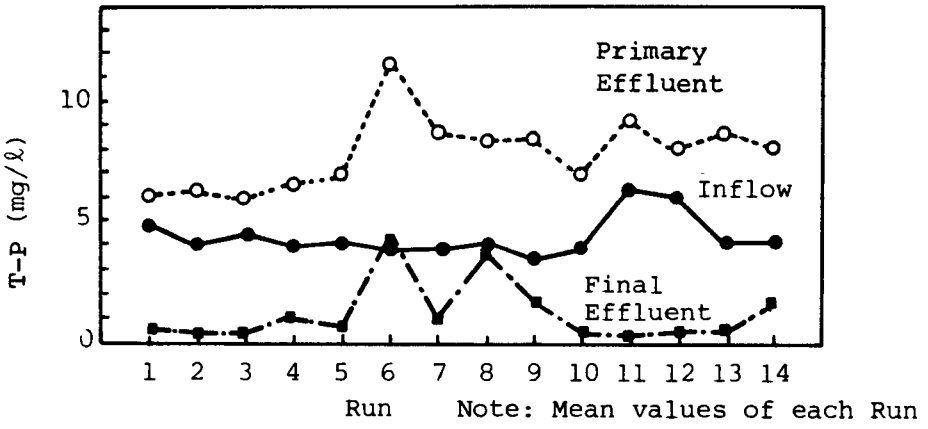


Figure 4. T-P Concentration of Inflow, Primary Effluent and Final Effluent of Each Run

Figure 4 shows the change in T-P concentration of the influent, primary settling tank effluent and the final effluent in relation to the runs. Table 3 summarizes the dephosphorization performance of the S plant.

Table 3. Performance of S plant

RUN	Primary Effluent					Final Effluent				
	BOD	SS	NH_4-N	T-P	PO_4-P	BOD*	SS	T-N	T-P	PO_4-P
1	166.5	98.0	31.0	6.09	3.72	3.1	5.6	27.2	0.28	0.09
2	87.0	102.0	31.9	6.32	3.75	2.9	5.7	27.5	0.19	0.17
3	97.5	86.8	29.2	5.96	4.50	2.9	3.4	23.2	0.24	0.21
4	99.7	87.6	30.8	6.59	5.24	3.4	4.1	28.2	0.88	0.82
5	109.0	85.7	32.3	7.04	6.06	7.9	6.7	31.6	0.60	0.41
6	111.0	101.2	34.6	11.63	8.54	2.5	8.5	34.7	4.26	4.17
7	142.3	93.2	32.5	8.78	7.41	3.6	7.9	30.7	0.99	0.76
8	199.0	88.1	31.8	8.48	7.13	3.0	8.8	30.7	3.77	2.07
9	156.0	143.3	27.4	8.59	5.81	1.9	6.0	32.8	1.49	1.19
10	151.0	101.0	32.1	7.14	5.30	2.9	3.3	35.8	0.23	0.07
11	136.0	106.3	31.8	8.29	4.70	3.2	3.3	36.1	0.16	0.03
12	147.0	104.9	32.0	8.06	4.59	4.1	3.9	32.9	0.27	0.17
13	194.5	95.8	32.1	8.72	7.06	3.9	5.4	32.1	0.38	0.14
14	233.5	98.9	32.2	8.13	7.27	15.8	32.8	32.0	1.54	0.48

BOD* refers to ATU injection.

From the start to Run 3, the T-P concentration in the final effluent was 0.2 mg/l. on the average. In Runs 4 and 5, the T-P concentration in the final effluent increased with increase in the phosphorus load in the return flow from the sludge treatment process. amounting to 0.5 to 0.6 mg/l. on the average.

In Run 6, activated sludge stored in the other systems which had been suspended since the start of the experiment was charged into the experimental system. This resulted in a temporary rise in the T-P concentration in the final effluent.

In Runs 7 to 9, the return flow phosphorus load from the sludge treatment process exceeded 100% of the phosphorus load in the inflow, and the T-P concentration in the final effluent often ran in excess of 1 mg/l.

From Run 10 on, phosphorus removal efficiency improved as the capacity of the biological reactor was doubled. In Run 14, however, SVI increased, and SS was discharged from the final settling tank to increase the T-P concentration in the final effluent.

Table 4. Performance of K plant

RUN	Primary Effluent					Final Effluent				
	BOD	SS	T-N	T-P	PO_4-P	BOD	SS	T-N	T-P	PO_4-P
1	77.5	109.7	22.6	4.54	3.09	5.8	3.9	14.1	0.22	0.13
						9.6	7.5	15.4	1.00	0.76
2	47.3	68.2	18.3	2.86	2.03	6.8	3.9	10.2	0.34	0.19
						5.9	3.5	12.0	0.98	0.72
3	43.1	47.0	18.5	2.05	1.41	3.9	3.1	13.1	0.16	0.19
						8.0	4.9	13.1	0.74	0.58
4	51.7	61.4	22.4	2.69	1.99	6.3	3.7	15.4	0.35	0.27
						14.3	5.6	17.0	0.83	0.65
5	57.0	61.7	22.8	2.75	2.04	7.8	3.6	16.1	0.29	0.16
						5.1	3.2	17.8	0.53	0.28
6	59.3	65.2	23.8	2.78	2.17	4.6	7.0	18.8	0.45	0.21
						6.8	6.1	19.4	0.39	0.16
7	72.3	97.7	25.9	3.55	2.52	4.2	5.7	19.1	0.41	0.14
						6.2	10.8	19.8	0.52	0.15

Note: For the final effluent, the upper row refers to the anaerobic-aerobic process, and the lower row to the conventional activated sludge process.

Table 4 also shows the performance of the K plant for each run. Phosphorus removal was successful at the K plant; the T-P concentration in the final effluent was kept below 0.5 mg/l at a probability of 85%.

3-2. PHOSPHORUS CONTENT IN SLUDGE

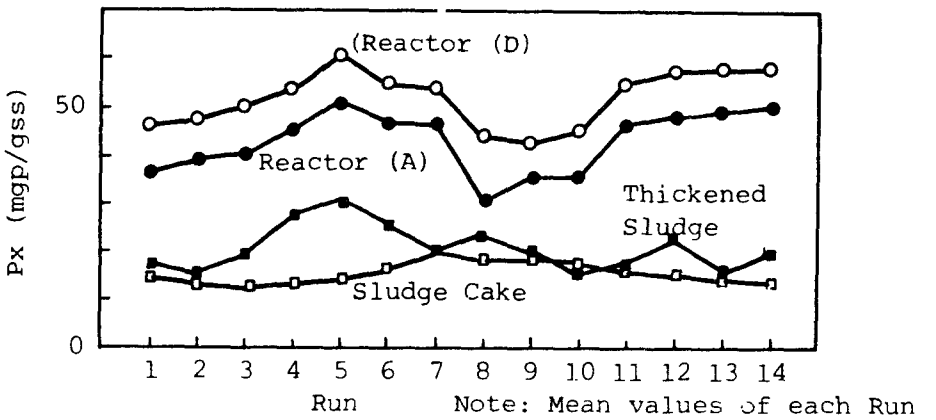


Figure 5. Phosphorus Content of Each Run

Figure 5 shows the transition of phosphorus content (Px) in the sludge by run. The phosphorus content in the activated sludge in the biological reactor increased steadily after the start of the experiment; in Run 5, the phosphorus content peaked at 60 mg-P/g-SS, and in tne subsequent runs, it fell. From Run 10 on, the phosphorus content started rising again, but leveled off from Run 12 on.

It was found that the phosphorus content in the thickened sludge changed nearly in proportion to that in the activated sludge. In the S plant, excess sludge was mixed with the sludge (containing 7 mg-P/g-SS on the average) from the primary settling tank at a volumetric ratio of 1 : 1 to 1 : 3. As phosphorus was liberated in the thickening tank, the phosphorus content in the thickened sludge became less than in the activated sludge. In Run 5, however, the phosphorus content in the thickened sludge rose to 30.9 mg-P/g-SS. While the phosphorus content in the dewatered cake can be regarded nearly equal to that in the digested sludge, it was observed that up to Run 7, there was a time lag between the change in phosphorus content in dewatered cake and that in the thickened sludge. In Run 7 and thereafter, the phosphorus content tended to fall. Except in Run 7 when thickened sludge was directly dewatered without passing through the anaerobic digestion tank, the maximum phosphorus content in the dewatered cake was 18.8 mg-P/g-SS.

At the K plant, on the other hand, the phosphorus content in the excess sludge was very stable, and remained in the range of 30 to 35 mg-P/g-SS throughout the entire period of the tests.

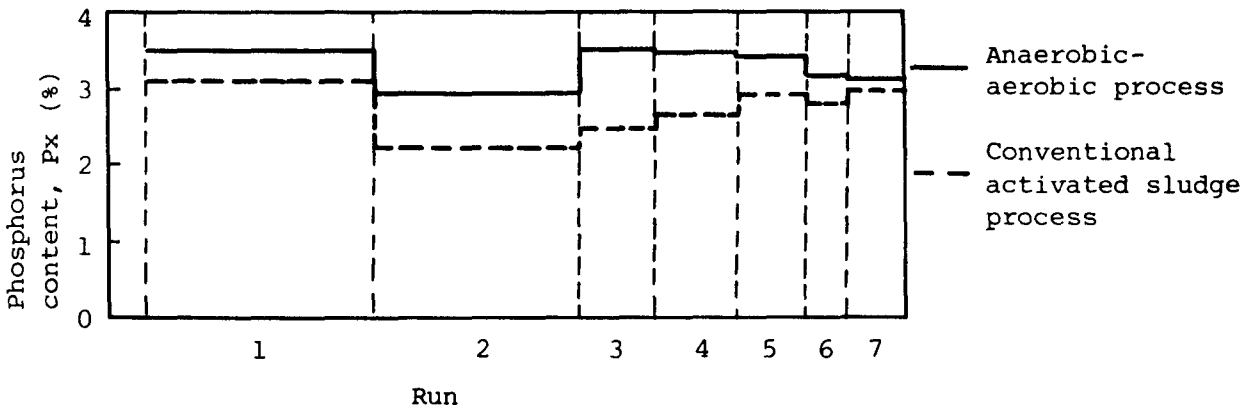


Figure 6. Transition of phosphorus content in K plant sludge

3-3. CHANGES IN PHOSPHORUS CONCENTRATION OF SLUDGE TREATMENT PROCESS RETURN FLOW

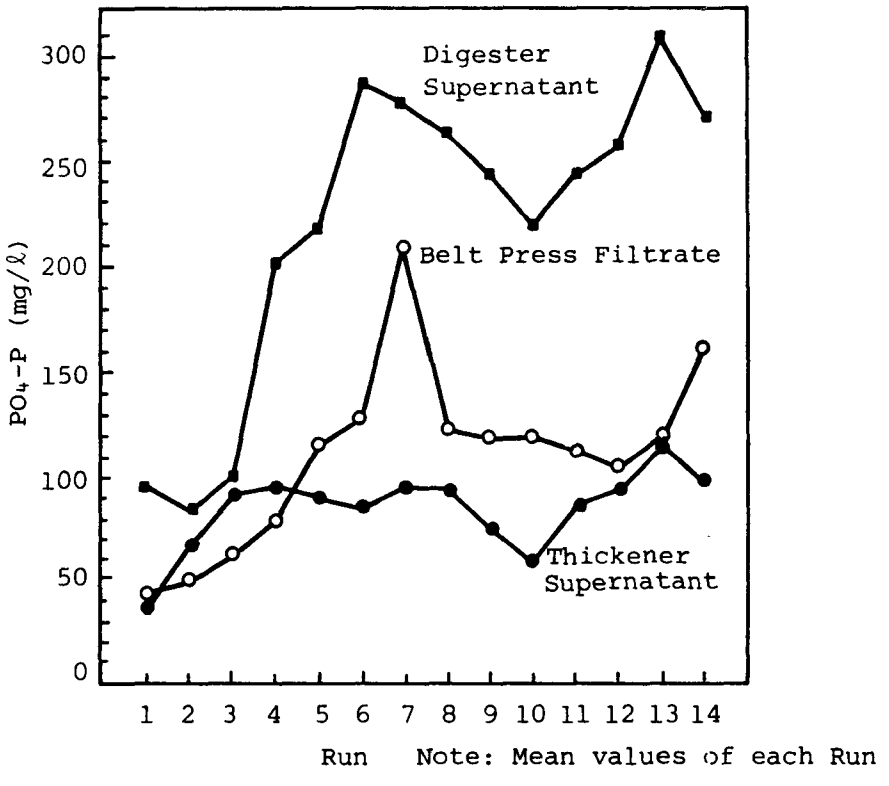


Figure 7. PO_4-P Concentration of Return Flows

Figure 7 shows the changes in the PO_4-P concentration in the sludge treatment process return flow, that is, in the thickener supernatant the digester supernatant and the belt press filtrate, in relation to the individual runs. After the start of the experiment, the PO_4-P concentration in the return flow increased drastically confirming out that the liberation of phosphorus in the sludge treatment process was very substantial. The digester supernatant in particular, indicated a pronounced increase in PO_4-P concentration.

The PO_4-P concentration in the belt press filtrate peaked in Run 7 because the thickened sludge was dewatered directly. At the K plant, the sludge treatment process was not equipped with anaerobic digestion tanks, and the phosphorus load carried in by the return flow was stable.

3-4. PHOSPHORUS MASS BALANCE IN PHOSPHORUS REMOVAL PROCESS

In order to study the behavior of phosphorus in the anaerobic-aerobic wastewater treatment and sludge treatment processes, the T-P balance was calculated for each run according to the following conditions.

(1) Calculation of flow balance based on actual measurements of final effluent, primary settling tank sludge volume, excess sludge volume, the volume of sludge fed to dewatering machine, septic tank sludge volume fed from the outside of the plant, volume of dewatered cake, and filtrate quantity.

(2) Calculation of T-P concentration at each nodal point according to the following formula.

$$Cpi = Cssi \times Px + Cpo\text{-}pi$$

where Cpi: T-P concentration at point i (mg/l)
Cssi: SS concentration at point i (g/l)
Px: Phosphorus content in SS (mg-P/g-SS)
Cpo-pi: PO_4-P concentration at point i (mg/l)

The T-P concentration did not include soluble polyphosphate, water quality analysis showed that the greater part of the soluble phosphate was orthophosphate, and that soluble polyphosphate was quite little.

For the final effluent, Px in the above formula was set at 10 mg-P/g-SS based on the final effluent SS concentration and SS-induced phosphorus concentration in the conventional activated sludge process operation at the S plant.

Px in activated sludge is increased by the introduction of the biological phosphorus removal process. In fact, Px can be higher than 1%. As the final effluent is passed through rapid sand filters, its SS usually is as low as 2 mg/l on the average. The ratio of the SS-induced phosphorus contribution to the total phosphorus concentration is smaller in comparison with the PO_4-P concentration. For this reason, Px in activated sludge was taken as 2 mg/l for mass balance calculations.

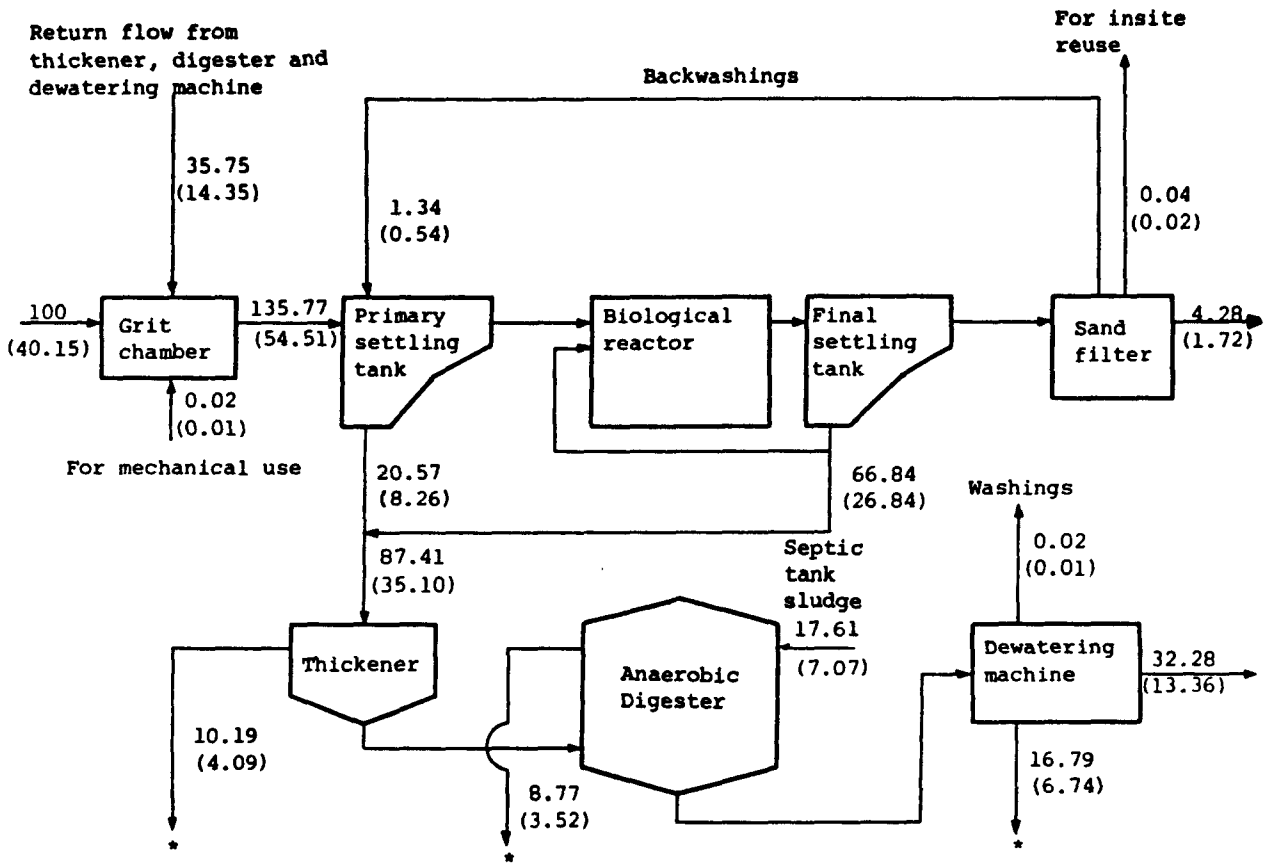


Figure 8. Mass balance of phosphorus when the return flow phosphorus load is low (S plant, Run 2)

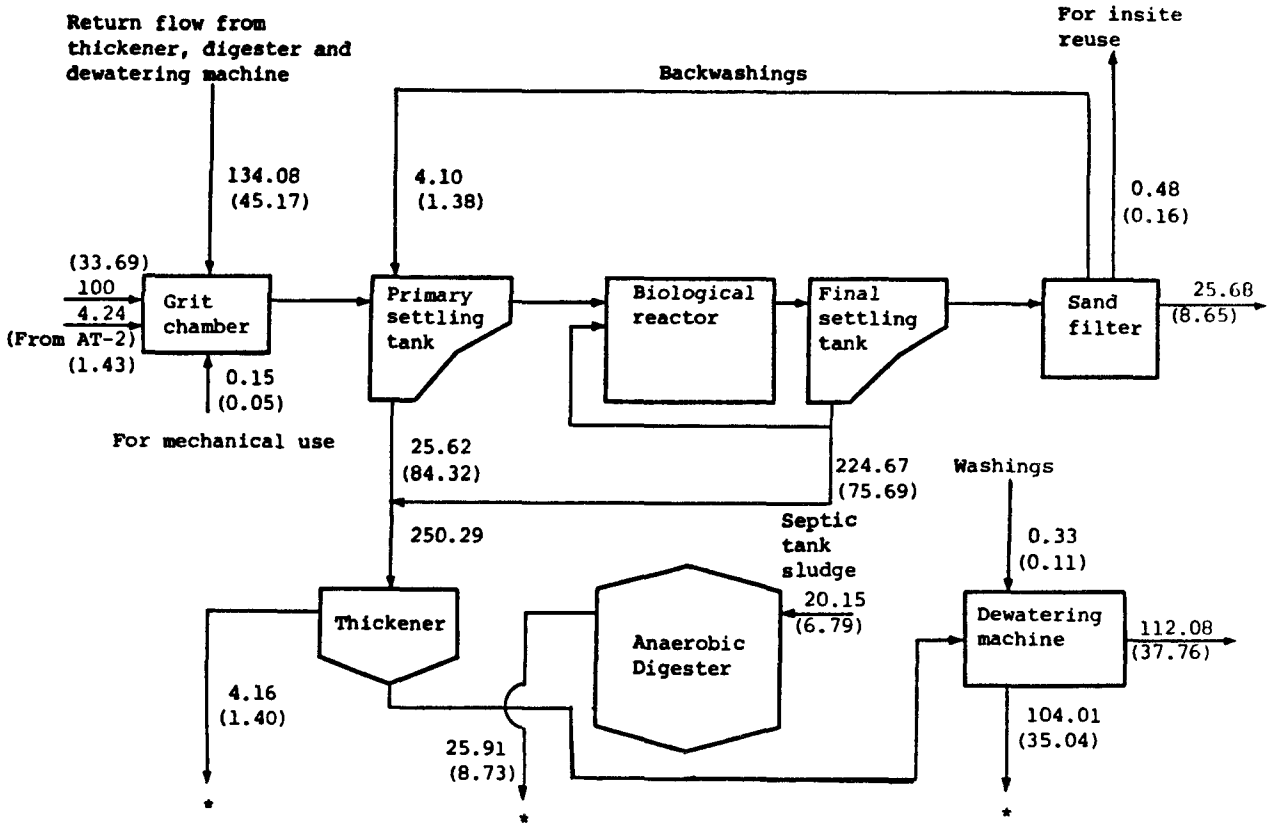


Figure 9. Mass balance of phosphorus when the return flow phosphorus load is large (S plant, Run 7)

Under the conditions specified above, the T-P balance in Runs 1 through 14 was calculated. Figures 8 and 9 show the mass phosphorus balance in Runs 2 and 7 which recorded the minimum and maximum T-P loadings in the return flow from the sludge treatment process, respectively. The data in Figures 8 and 9 is given with the T-P load in the raw influent taken as 100. The values in parentheses denote the calculated values of T-P load.

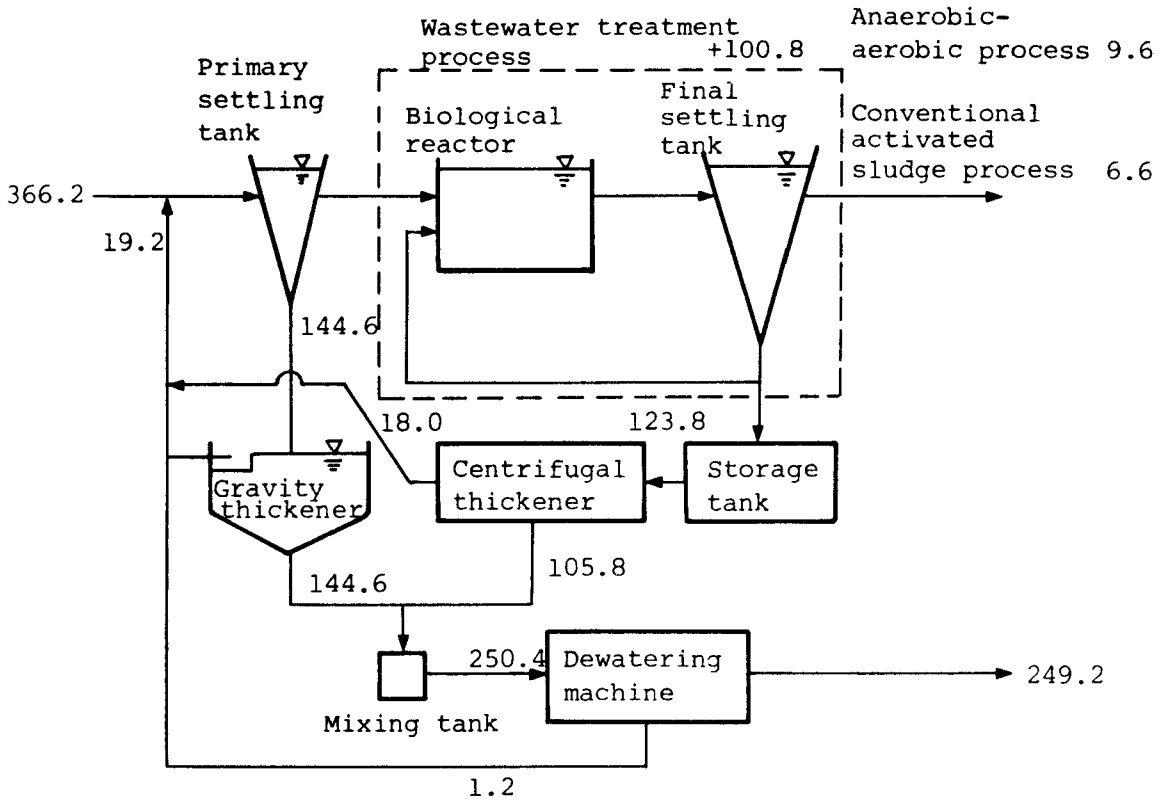


Figure 10. Phosphorus mass balance in K plant (Unit: kg/day)

Figure 10 shows the phosphorus mass balance calculated for the K plant in the same way as above. Figure 10 refers to Run 7. As phosphorus was fixed partly by centrifugal thickening and partly by coagulation with lime and ferric chloride, the phosphorus load of the return flow from the sludge treatment process was only 5 percent of the phosphorus load of the raw influent to the plant. The phosphorus concentration in the final effluent was vert small, accordingly.

3-5. ACCUMULATION OF PHOSPHORUS IN THE PROCESSES

The amount of phosphorus trapped in the wastewater and sludge treatment processes changes with fluctuation in sludge volume and phosphorus concentration in the respective processes. Figure 11 shows the change in in-process accumulation of phosphorus for each run, determined according to the phosphorus mass balance calculated in the foregoing.

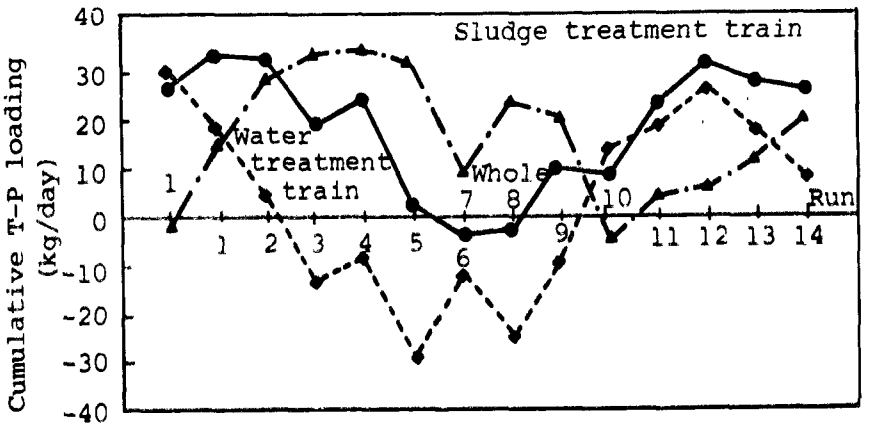


Figure 11. Transition of cumulative phosphorus load in the process

The amount of phosphorus in the plant as a whole increased steadily till Run 5. But, entering Run 6, the accumulated amount of phosphorus became almost constant. In Runs 7 and 8, the phosphorus accumulated in the processes was carried away. In Runs 9 and thereafter, and particularly from Run 10 onward when the capacity of biological reactor was doubled, the accumulation of phosphorus became conspicuous. In Run 7, thickened sludge was directly dewatered without digestion, and the phosphorus accumulation in the sludge treatment system decreased, working upon the entire system to liberate the accumulated phosphorus.

In Run 6, a charge of sludge aerobically digested in the other system into the biological reactor impaired phosphorus removal in the wastewater treatment process. This was accompanied by the reduction of phosphorus content, Px, in the sludge. As a result, the phosphorus accumulation in the wastewater treatment process became negative as shown in Figure 11. In Run 8, the air flow was reduced by about 20% as compared with the other runs operated in Mode I. As a result, the NOx-N concentration in the final effluent declined, and the phosphorus accumulation became negative in the final settling tank.

3-6. PHOSPHORUS LOAD BY RETURN FLOW

Figure 12 shows, for each run, T-P load by return flow and its breakdown.

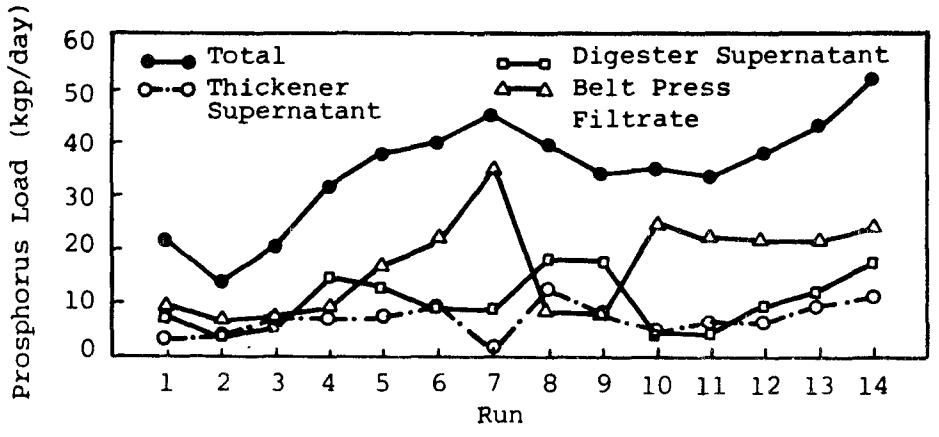


Figure 12. Return flow phosphorus load of each run

The phosphorus load by return flow increased with the time of run. It was found that the filtrate was the major culprit for the increase of phosphorus load. This was particularly the case with Run 7 in which the contribution of filtrate accounted for no more than 75% of the total phosphorus load by return flow. Figure 13 shows the phosphorus load of return flow with the T-P load of raw influent taken as 100. It also shows the T-P load of final effluent.

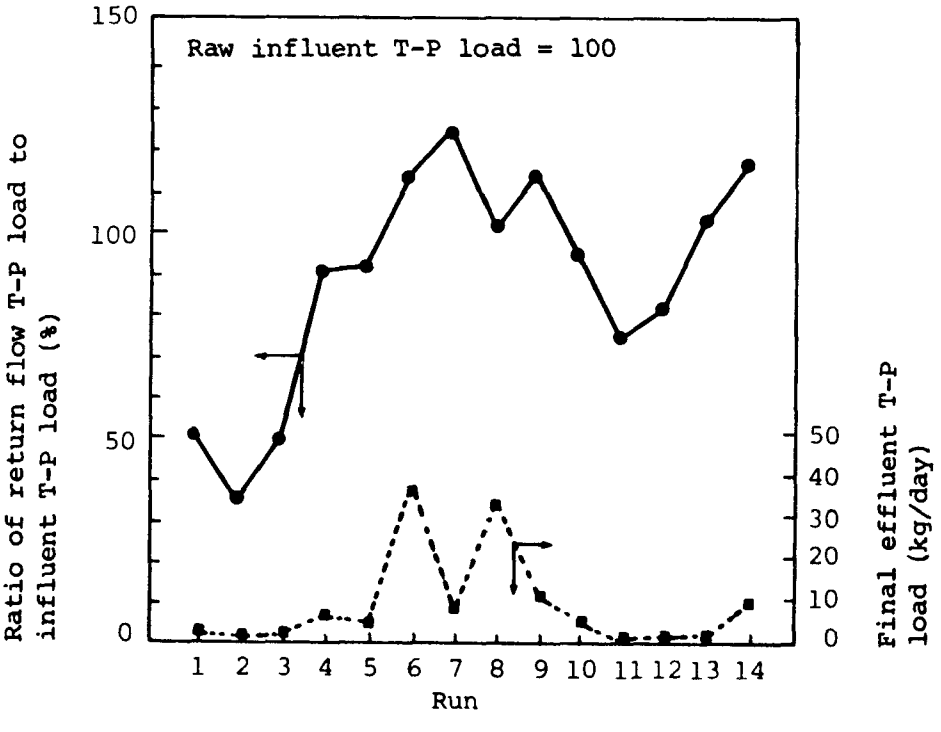


Figure 13. Transition of phosphorus load in return flow

From Figure 13, it is found that in Runs 4 and thereafter, the return flow phosphorus load exceeded 100 percent (raw influent phosphorus load), and that the final effluent phosphorus load increased with increase in the return flow phosphorus load. In Run 7 when sludge was thickened and dewatered, the filtrate increased the phosphorus load of the return flow to as high as 134 percent of raw influent phosphorus load.

For a wastewater processing rate of 9,000 m^3/day and a final effluent T-P concentration of 0.5 mg/l., the final effluent phosphorus load is 4.5 kg/day. Thus, if it is desired to limit the final effluent T-P concentration to about 0.5 mg/l, Figure 13 dictates that the phosphorus load of the return flow from the sludge treatment process should be kept below about 50 percent of the phosphorus load of the raw influent.

3-7. QUANTITY OF PHOSPHORUS DISCHARGED BY THE DEWATERED CAKE FROM THE PROCESS

Figure 14 shows the quantity of phosphorus removed from the wastewater treatment process and that removed together with dewatered cake. The quantity of phosphorus removed from the wastewater treatment process is the difference between the T-P load in the raw influent and that in the effluent.

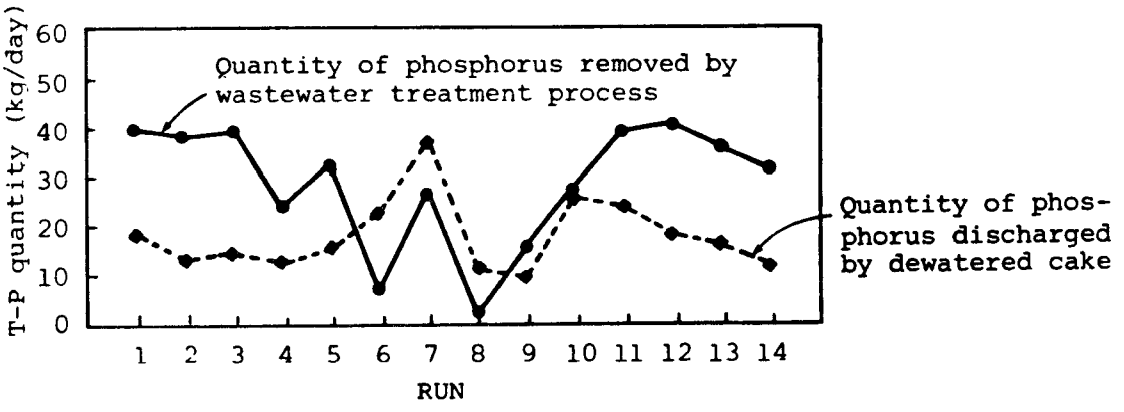


Figure 14. Quantity of phosphorus removed by wastewater treatment process and discharged by dewatered cake

As shown in Figure 14, the rate of phosphorus removal from the process together with the dewatered cake increased after the start of the experiment. This was because the phosphorus content in the dewatered cake increased.

In Runs 6 through 8, the amount of phosphorus discharged by the dewatered cake exceeded the phosphorus removed by the wastewater treatment process. This was due to the fact that the phosphorus removal efficiency rate of the wastewater treatment process in Runs 6 and 8 was not very good. In Run 7, the thickened sludge was directly dewatered, which resulted in an increase in the amount of phosphorus discharged by the dewatered cake from the process.

3-8. EFFECT OF CHANGES IN INFLUENT LOAD UPON PHOSPHORUS REMOVAL

For the purpose of investigating the response of the biological phosphorus removal process to changes in influent load, a series of tests were conducted for 24 consecutive hours in Run 7 when sludge was directly dewatered after thickening. During the 24-hour period, the inflow into the biological reactor was relatively stable at 376 to 400 m^3/hr. It should be noted that the dewatering was carried out for 5 hours from 9:00 a.m. to 2:00 p.m. using the belt press filters and that the gravity thickener returned overflow to the wastewater treatment process for about 6.5 hours from 7:00 p.m. to 1:30 a.m. Samples were taken every 2 hours to measure PO_4-P. The result is shown in Figure 15.

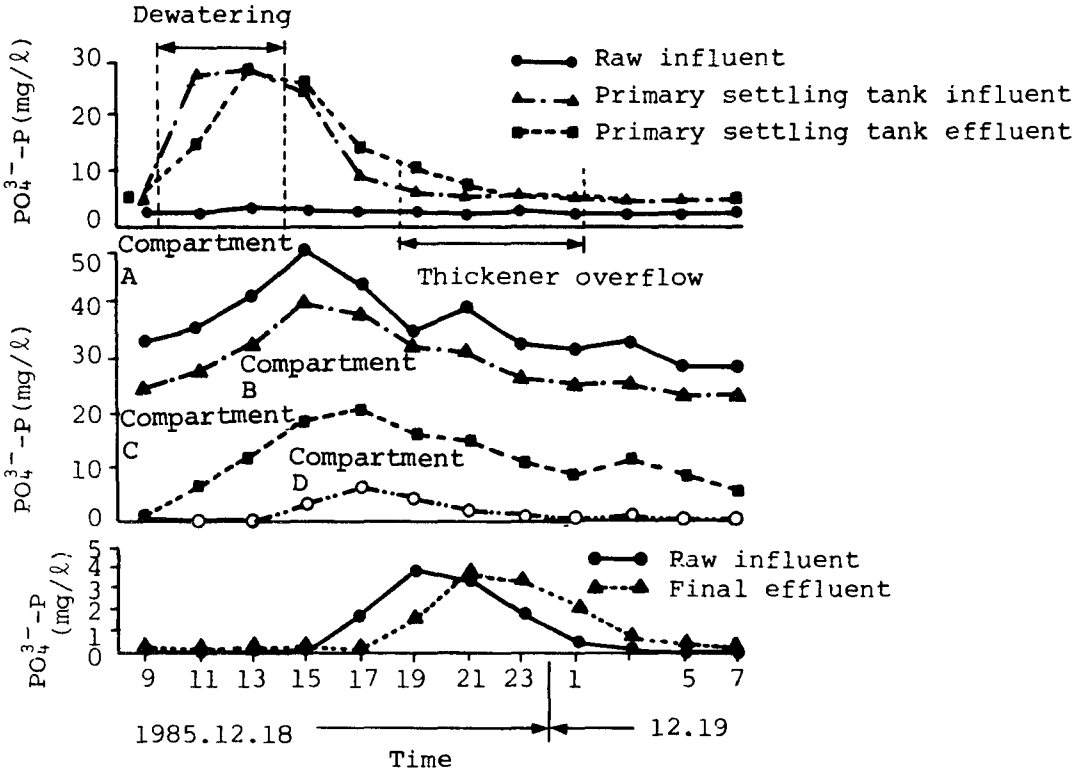


Figure 15. Changes of PO_4-P concentration

According to Figure 15, the phosphorus load in the influent to the primary settling tank increased the longer the dewatering machine operated, and the effluent from the primary settling tank responded to this effect a little later. The maximum PO_4-P concentration recorded was 29 mg/l.

The PO_4-P concentration in each compartment of the biological reactor peaked at around 3:00 p.m. to 5:00 p.m. of the day measurement was started. In the aerobic tank, the height of the peak fell because phosphorus was reabsorbed, but even in the D compartment at the downstream end of the aerobic tank, unabsorbed phosphorus still remained. Even in the effluent from the final settling tank and the final effluent from the rapid sand filter, the peaking effect of the filtrate return flow persisted, increasing up the phosphorus concentration in the 24-hour composite sample of final effluent.

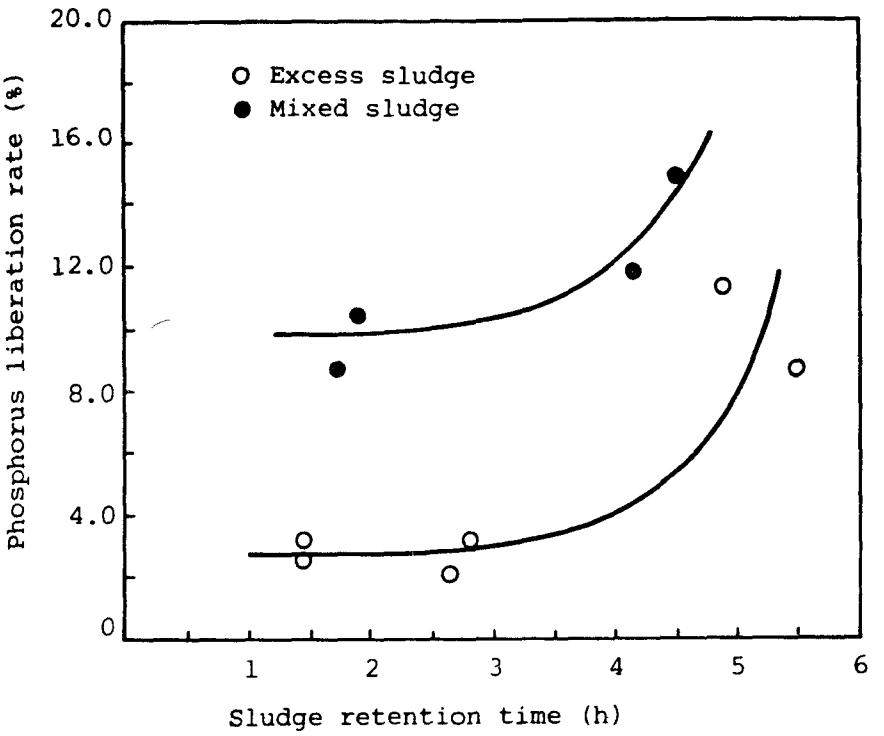
From the above findings, it is concluded that in the biological phosphorus removal process, it is very important not only to reduce the phosphorus load of the return flow from the sludge treatment process, but also to smooth out the phosphorus load with respect to time.

4. SLUDGE TREATMENT IN BIOLOGICAL PHOSPHORUS REMOVAL PROCESS

4-1. SLUDGE THICKENING

In the biological phosphorus removal process, the excess sludge containing phosphorus liberates phosphorus if it is retained in the gravity thickener for a long time. In the case of the mixed thickening process in which excess sludge and the sludge from the primary settling tank are mixed for treatment, the mixed sludge is liable to become anaerobic. In addition, the sludge from the primary settling tank contains substrate that can be used for the liberation of phosphorus. All these combine to encourage the liberation of phosphorus.

Figure 16 shows the relationship between the thickener retention time and phosphorus liberation observed at the K plant.



Note: Phosphorus liberation rate =

$$\frac{\text{Thickener overflow T-Pf + Thickened T-Pf - Exces sludge T-Pf}}{\text{Excess sludge T-P}} \times 100$$

Figure 16. Relationship between Phosphorus liberation rate, solids loading and sludge retention time

Figure 16 shows that the phosphorus liberation rate increases sharply when the retention time exceeds 3 hours, for both of excess sludge alone and mixed sludge. The phosphorus content in the excess sludge used for the test was 3.4 percent. About 40 percent of total phosphorus in the sludge was polyphosphate. Theoretically, therefore, the ratio of phosphorus liberated into the return flow to the wastewater treatment process can be said to reach a maximum value of 40 percent.

Figure 16 shows that, in case of gravity thickening, the mixed sludge liberates about 10 percent of phoshorus even when the retention time is less than 3 hrs. This is about a quarter of the amount of phosphorus that can be liberated. When the retention time is 5 hrs., the quantity of phosphorus liberated is increased to 50 percent of the amount of phosphorus that can be liberated.

The above findings negate the application of gravity thickening to mixed sludge in the biological phosphorus removal process. At the K plant, it was decided to employ a separate thickening method in which excess sludge is thickened mechanically by means of a centrifuge while the sludge from the primary settling tank is thickened in a gravity tank. As demonstrated in Figure 10, the phosphorus loading by the return flow was reduced dramatically.

4-2. FIXATION OF PHOSPHORUS IN THE SLUDGE

The main requirement in the biological phosphorus removal process is to prevent the excess phosphorus taken in by the activated sludge from being redissolved, and to discharge it from the process in the form of dewatered cake. In other words, the total phosphorus in the process increases unless the difference between the influent phosphorus load and the tolerable phosphorus load of the final effluent is completely removed in the form of dewatered cake. If the total phosphorus load in the process exceeds the biologically fixable level, the phosphorus removal efficiency will decline.

For this reason, at the S plant, we tried injecting PAC (Poly Aluminum Chloride) into the excess sludge and the sludge removed from the primary settling tank just upstream of the gravity thickener, for fixation of phosphorus in the sludge. The PAC injection rate was set at 0.15 to 0.20 mol-Al/g-SS according to the results of a laboratory test.

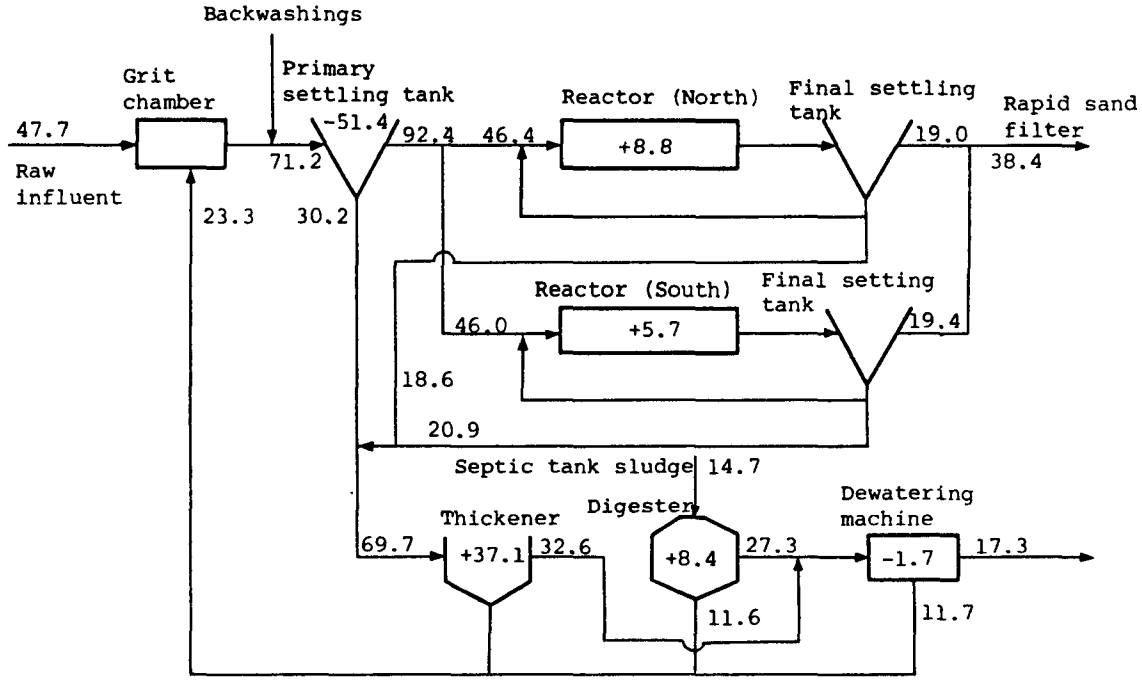


Figure 17. T-P balance before PAC injection (S plant)

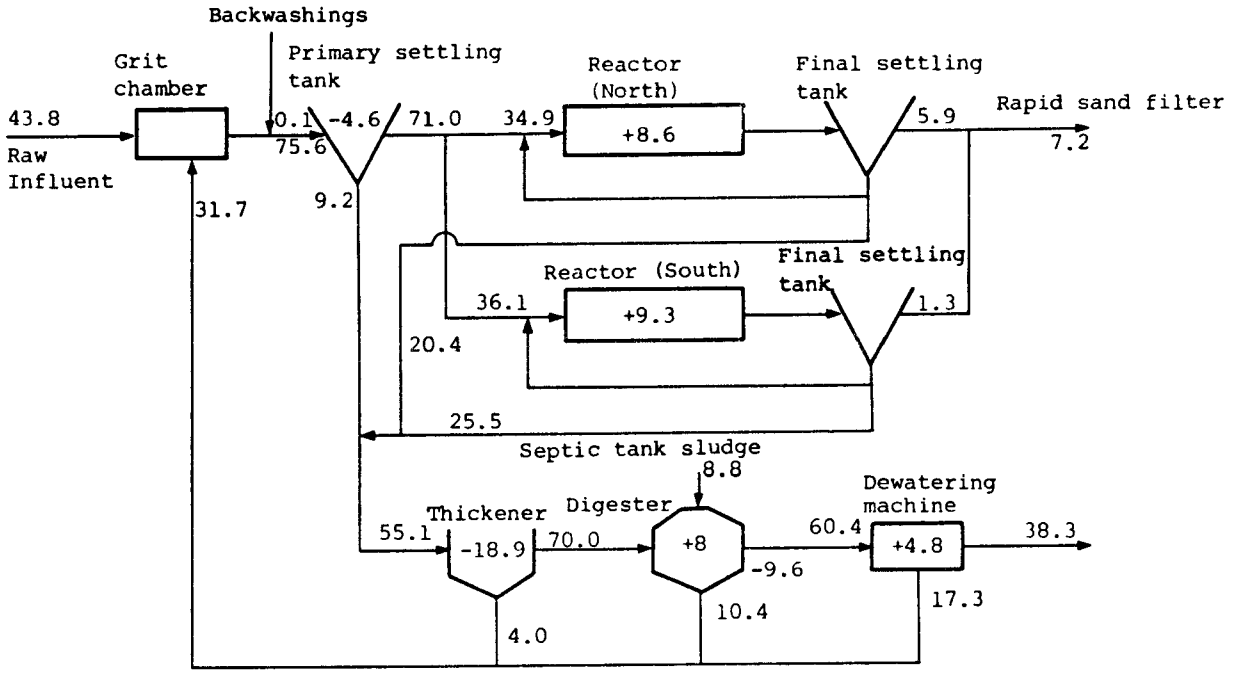


Figure 18. T-P balance after PAC injection (S plant)

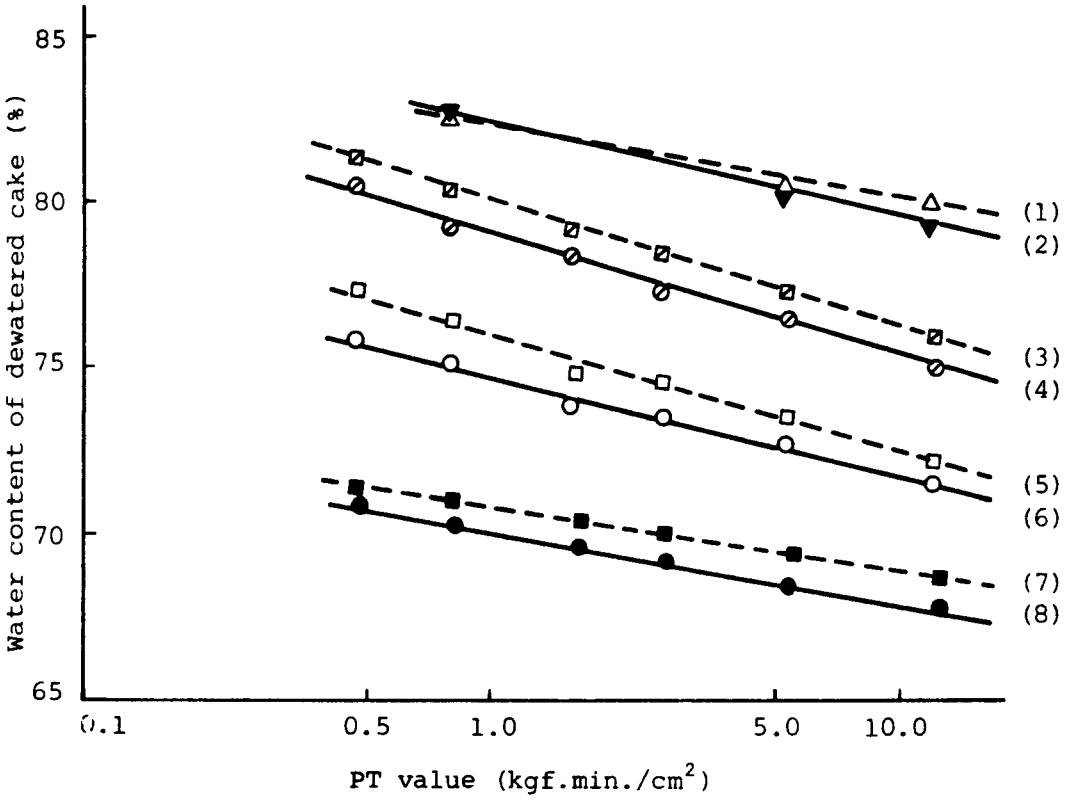
The mass phosphorus balance before and after PAC injection is given in Figures 17 and 18. Although the phosphorus load of raw influent was 40 to 50 kg/day, the phosphorus load of liquor entering the biological reactor was increased to 92.4 kg/day before PAC injection. This was due to the return flow from the sludge treatment process. The phosphorus load of the final effluent was 38.4 kg/day, and the phosphorus removal rate was a mere 20 percent. After PAC injection, the phosphorus load to the biological reactor fell to 71.0 kg/day, and that of the final effluent was also decreased to 7.2 kg/day, increasing the phosphorus removal rate to 84 percent.

In the runs before and after PAC injection, the influent phosphorus load was 44.2 kg/day (4.23 mg-P/l. x 10,450 m^3/day). It was necessary to remove the phosphorus at a rate more than 33.8 kg/day in the form of dewatered cake in order to check the T-P concentration of the final effluent below 1 mg/l. The sludge volume generated during the test period was 1,675 kg/day, and the phosphorus content in the dewatered sludge cake had to be greater than 2.02 percent. After PAC injection, the phosphorus content in dewatered cake was 2.20 percent in the case of Figure 18, meeting the above requirement. In Figures 17 and 18, there is no substantial difference in the phosphorus load carried in from the sludge treatment process. This is because, while the sample from the wastewater treatment process was a composite one, that from the return flow was a spot sample, which did not always represent every 24-hour run. The dewatering of sludge was conducted only during the day, and the 24-hour mean value should have been far less than the values shown in Figures 17 and 18.

4-3. DEWATERING OF SLUDGE

As discussed in 4-1 and 4-2, the separate thickening method is favorable where phosphorus is not fixed in the sludge. We studied the dewaterability of sludge. First, we tested excess sludge and mixed sludge by making use of a batch type belt press filter. The sludge was conditioned using two kinds of polymer coagulants, independently (single injection method) or in combination (double injection method).

The moisture content of the sludge cake was measured with the PT value (integration of compression pressure, P kgf/cm^2, and effective compression time, T min.) taken as an indicator. The measurements obtained are given in Figure 19, which shows that mixed sludge shows a higher dewaterability than excess sludge irrespective of whether the single or double injection method is used.



No.	Symbol	SS ratio of primary settling tank sludge and excess sludge	Sludge concentration, (g/l)	VS (%)	Cation injection rate (%)	Anion injection rate (%)	Injection rate (%)	Remarks
(1)	△		12.8				0.47	Conventional activated sludge process excess sludge
(2)	▼		11.8	74.5			0.42	Anaerobic-aerobic process excess sludge
(3)	▨		12.8		3.1	0.78		Conventional activated sludge process excess sludge
(4)	⊘		13.2	70.8	3.0	0.76		Anaerobic-aerobic process excess sludge
(5)	□	1.02 : 1.00	13.6				0.22	Conventional activated sludge process mixed sludge
(6)	○	1.11 : 1.00	13.0				0.23	Anaerobic-aerobic process mixed sludge
(7)	■	1.04 : 1.00	13.5		3.7	0.59		Conventional activated sludge process mixed sludge
(8)	●	1.11 : 1.00	13.0		3.8	0.62		Anaerobic-aerobic process mixed sludge

(Characteristics of sludge used for test, and chemical injection conditions)

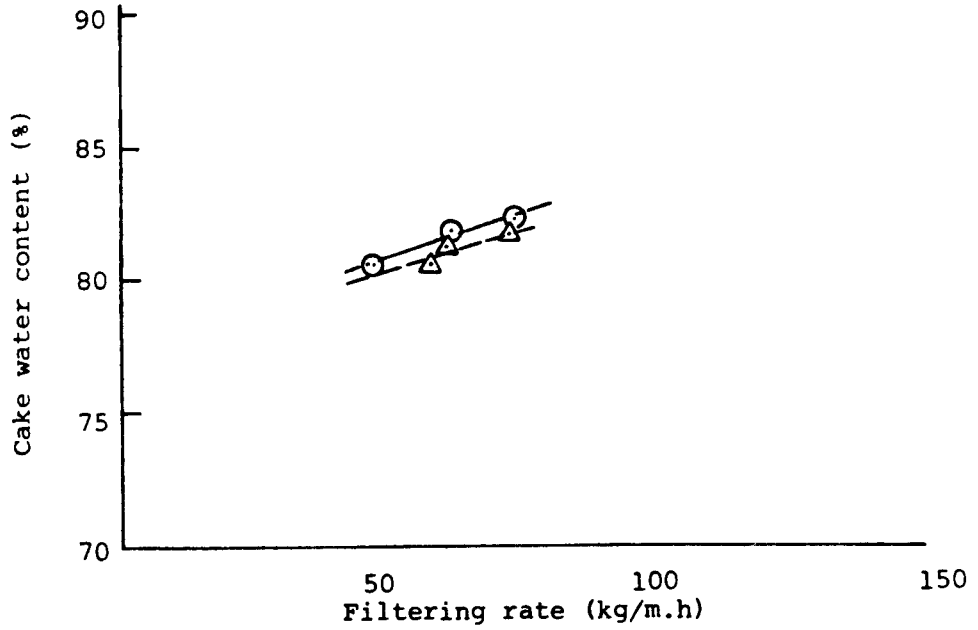
Figure 19. Dewaterability of sludge subjected to filtration through batch type belt press filter

Specifically, the dewatered cake from the mixed sludge is 7 to 9 percent lower in water content than that from excess sludge.

The dewaterability of sludge was also studied with respect to the anaerobic-aerobic process and conventional activated sludge process. When the dewatered sludge characteristics of both processes were compared, it was found that the anaerobic-aerobically treated sludge had cake with an equal or less moisture content in both the case of excess sludge and mixed sludge. This may have been caused because the sludge produced in a anaerobic-aerobic process charged was less in VSS.

Then, we temporarily installed a test belt press filter at the plant, and conducted a dewatering test for excess sludge while operating the belt press filter continuously. The belt press filter was operated at a smaller filtration rate of 50 to 110 kg/m.h than the standard rate as it had to handle excess sludge. The results are summarized in Figure 20.

Symbol	Type of sludge	Sludge SS (g/l)	Sludge VSS (%)	Polymer injection rate (% to SS)
△	Conventional activated sludge process	12.4 ∿ 14.8	75.6 ∿ 76.8	0.54 ∿ 0.65
○	Anaerobic-aerobic process	11.8 ∿ 12.5	73.5 ∿ 74.5	0.48 ∿ 0.50



Note: High-pressure equipment not used

Figure 20. Filtering rate vs. cake water content (Polymer single injection method)

Figure 20 suggests that in order to attain a moisture content of 80 percent for excess sludge cake, the filtration rate be set at about 40 kg/m.h for the single injection method.

From Figure 19, it is found that the PT value corresponding to an excess sludge cake moisture content of 80 percent by the batch type dewatering machine is about 10 kgf.min./cm^2. For this PT value, a cake water content of 72 to 73 percent can be achieved with the mixed sludge. We could not conduct a continuous dewatering test on the mixed sludge, but our theoretical calculations show that the filtration rate will be about 130 kg/m.h., or about 3 times faster than with the excess sludge.

Although high polymer coagulant is not so effective in fixing phosphorus, rapid thickening by centrifuge, followed by dewatering through a belt press filter, can minimize the release of phosphorus. According to a case study in T plant where excess sludge are thickened by centrifuge and primary settling tank sludge are done by gravity thickener (separate thickening method), then injected with polymer coagulant, and dewatered by belt press filters, it was concluded that the phosphorus load of the return flow is only 14 percent of the raw influent phosphorus load.

All told, the combination of separate thickening and mixed sludge dewatering is favorable for sludge treatment in the biological phosphorus removal process.

5. CONCLUSION

The field investigations of the biological phosphorus removal process at the Shinjiko Tobu Purification Center and the Kase Sewage Treatment Plant have disclosed the following.

(1) In the anaerobic-aerobic biological phosphorus removal process operated under the following conditions, the T-P concentration of the final effluent from the final settling tank could be reduced to less than 1 mg/l. When the T-P concentration of raw influent was in the range of 4 to 7 mg/l. (S plant)
Water temperature: 14°C or higher
Biological reactor retention time: 5.5 to 11.9 hrs. (actual retention time incl. return sludge: 3.9 to 8.4 hrs.)
MLSS concentration: 2,400 to 4,000 mg/l.

(2) When the phosphorus load was increased by the return flow from the sludge treatment process, the T-P concentration of the influent to the biological reactor occasionally increased to raise the T-P concentration of the final settling tank effluent in excess of 1 mg/l. For the purpose of operating the anaerobic-aerobic process for stable and efficient phosphorus removal, it is necessary to reduce the phosphorus load of the return flow to less than 50 percent of that of the raw influent. (S plant)

(3) By injecting PAC (poly aluminum chloride) into the sludge charged into the sludge thickener at 0.15 to 0.20 mol-Al/g-SS, the return load of phosphorus from the sludge treatment process could be reduced.

(4) When the T-P concentration of the raw influent was 2.1 to 4.5 mg/l, that of the final settling tank effluent could be reduced to less than about 0.5 mg/l. When the K plant was operated under the following conditions:
water temperature, 15°C or higher; Biological reactor retention time, 5.0 to 6.8 hrs.; MLSS, 2,000 to 2,700 mg/l.; the phosphorus load of the return flow from the sludge treatment process, less than 10 percent of the phosphorus load of raw influent.

(5) The sludge procuced by the biological phosphorus removal process should preferably be subjected to a separate thickening process in which the primary settling tank sludge and the excess sludge are thickened separately. It is also preferred to condition and mix the separately thickened sludge for dewatering.

6. ACKNOWLEDGMENTS

In conclusion, we would like express our profound thanks to Mr. Hideo Esumi, Ms. Junko Ishihara and other water quality control officers of the Shimane Prefectural Government, and Mr. Minoru Furusawa, Mr. Yasuo Sone and Tadashi Kawasaki, Mr. Toshihiro Tanaka and other officers who have been most generous in assisting us in our investigations.

CURRENT PRACTICES OF BIOLOGICAL NITROGEN REMOVAL PROCESSES IN JAPAN

by

Kazuaki Sato
Chief, Advanced Waste Treatment Section,
Water Quality Control Division,
Public Works Research Institute,
Ministry of Construction

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

Prepared for Presentation at:
The 11th United States/Japan Conference
on
Sewage Treatment Technology

October, 1987
Tokyo, Japan

ABSTRACT

Currently, there are more than 600 municipal wastewater treatment plants operating in Japan. In about twenty of these treatment plants, advanced wastewater treatment processes are applied to produce clearer effluent from those plants using the standard activated sludge mode. Nitrogen and phosphorus removing technologies are also being applied to a limited number of wastewater treatment plants to control entrification problems in lakes. This paper introduces four demonstration experiments which have been recently conducted for nitrogen control of wastewater in Japan. These experiments are listed below.

(1) Improvement of the nitrified liquor recycling nitrification-denitrification process at the Konan-Chubu treatment plant of Biwako regional sewerage.
(2) Upgrading of the nitrification-endogenous denitrification process in Hamamatsu City.
(3) Demonstration study on biological nitrogen and phosphorus removal using a step inflow process at the Shinjiko-Tobu wastewater treatment plant.
(4) Performance of the A_2O process at the Toba wastewater treatment plant of Kyoto City.

C O N T E N T S

1. VIEWPOINTS OF NITROGEN CONTROL IN WASTEWATER IN JAPAN 268

2. HISTORY OF APPLICATION OF NITROGEN REMOVAL TECHNOLOGY 270

3. BIOLOGICAL NITROGEN REMOVAL PRACTICES 271

3.1 Improvement of the Nitrified Liquor Recycling Nitrification-Denitrification Method 271

3.2 Improvement of the Nitrification-Denitrification Process Using Endogenous Respiration for Denitrification in Hamamatsu City 276

3.3 Demonstration Study on Biological Nitrogen and Phosphorus Removal Using a Step Inflow Process at the Shinjiko-Tobu Wastewater Treatment Plant 281

3.4 Performance of the A_2O Process at the Toba WasteWater Treatment Plant of Kyoto City 285

1. VIEWPOINTS OF NITROGEN CONTROL IN WASTEWATER IN JAPAN

Japan has set environmental quality standards to serve as the target for water quality in rivers, lakes, and sea areas. The effluent control for various effluents has been conducted for the purpose of achieving these environmental standards. The effluent control of treated water from public sewage systems aims at the same goal, and currently it is executed for BOD and COD, when the treated water is discharged into rivers and sea areas. Recently, new environmental standards for nitrogen and phosphorus have been set for lakes, followed by the enforcement of nitrogen and phosphorus effluent standards for firms discharging effluents into lakes. Thus, with respect to nitrogen removal from treated wastewater, the control provides the only juridical ground for preventing the eutrophication of lakes. Presently, municipal wastewater treatment plants, where nitrogen removal is practiced, all discharge their treated water into closed water areas. A list of these treatment plants are shown in Table 1.

However, it is expected that there will be more cases in Japan in which the removal of nitrogen from, or the nitrification of, treated wastewater will be required. One such case is that of treated wastewater discharged into water channels for agriculture or into rivers with low dilution capacity, often used for agriculture. In Japan, the greatest amount of agricultural water is used to irrigate paddy fields, where irrigation water with high nitrogen content, especially high ammonia-nitrogen concentration, seems to have an adverse effect on paddy rice cultivation. Sewage systems have been spreading to small and middle sized cities in Japan, and this is expected to result in more cases of treated wastewater going into agricultural water channels, thus creating a strong demand for nitrogen removal. For instance, a wastewater treatment facility in a parking area along a highway which discharges its effluent into an agricultural water channel was required to meet the treatment target of 5 mg/ℓ ammonia-nitrogen. Where treated wastewater is re-used as agricultural water, treated wastewater which has a concentration of less than 10 mg/ℓ total nitrogen, and is subjected to nitrification, is used.

Another possible case is when the percentage of treated wastewater in the river discharge is high and the river water downstream is supplied as service water. In such a case, the service water supplier demands a lower ammonia-nitrogen concentration of the treated wastewater in order to ensure stable operation of the water purification plant. The Yodo River, which has many densely populated catchment areas and serves as a source of service water for Osaka, is a typical example.[1)]

Ammonia-nitrogen in river water is finally nitrified to NO_3-nitrogen to consume dissolved oxygen in the river water. In Japan, however, the necessity to oxidize nitrogen in treated wastewater is not generally recognized for the purpose to maintain dissolved oxygen in rivers, where the flowing time is relatively short and the flow velocity is high resulting in a high coefficient of re-aeration.

Table 1 List of WTPs where N-P removal process is applied

WTP's name	Effluent quality (mg/ℓ)	Design flow capacity (m^3/day)	Treatment mode	Operation start year
Biwako R.S. Kosei	BOD 5 COD 10 SS 6 T-N 10 T-P 0.5	5000	BNDR Coagulant addition Filtration	1984
Biwako R.S. Konanchubu	BOD 5 COD 10 SS 6 T-N 10 T-P 0.5	14000	BNDR Coagulant addition Filtration	1983
Ohmihachiman City Okinoshima	T-N 10 T-P 0.5	200	Oxidation ditch Coagulant addition Filtration	1982
Kasumigaura-Kohoku R.S. Kasumigaura	BOD 6 COD 11 SS 6 T-N 15 T-P 0.5	30000	BNDR Coagulant addition Filtration	1983
Kasumigaura-Suigoh R.S. Itako	T-N 20 T-P 1	5400	BNDR Coagulant addition	1985
Nikkoh City Yumoto	T-N 10 T-P 1	3250	Oxidation ditch Coagulant addition	1985
Hamamatsu City Hitomigaoka	T-N 5 T-P 1	1620	Modified A_2O process Filtration	1983
Hamamatsu City Kotoh	T-N 5 T-P 1	1980	BNDE Coagulant addition Filtration	1982

R.S.: Regional Sewerage System
BNDR: Biological Nitrification-Denitrification Process with nitrified liquor recycling
BNDE: Biological Nitrification-Denitrification Process using Endogeneous Respiration

2. HISTORY OF APPLICATION OF NITROGEN REMOVAL TECHNOLOGY

Since the actual state of advanced wastewater treatment technology in the United States was introduced at the 1st United States/Japan Conference on Sewage Treatment Technology in 1971, Japan, including the Public Works Research Institute, commenced the development of advanced wastewater treatment technology. The technology developed for nitrogen removal at that time mainly included physico-chemical methods such as ammonia stripping, selective ion exchange and break-point chlorination. The first oil crisis in 1973 gave an occasion to re-examine the development of these techniques from the viewpoint of resources and energy conservation, with a resultant gradual shift towards the development of biological nitrogen removal methods.

Biological nitrogen removal was put to practice in early days in the field of night soil treatment in Japan, and practical facilities have been operating since the early 1970's. In the field of sewage treatment, a treatment facility in a highway service area was completed where two-stage type biological nitrification and sand filtration were combined for advanced treatment. This facility was designed to meet the target water qualities of 5 mg/ℓ BOD, 5 mg/ℓ SS and 5 mg/ℓ NH_4-N for the treated water, which was discharged into an agricultural water channel. At the start of the service, the operation of this facility at low load resulted in lowered pH, runoff of fine flocks and lowered transparency of the treated water. The Public Works Research Institute, Ministry of Construction, remodelled this facility to a one-stage, combined BOD oxidation nitrification-denitrification process type (nitrified liquor recycling method) in 1977, and conducted a demonstration study for approximately two years. Figure 1 gives a flow chart of the remodeled process, in which methanol was added in the late stage of the biological nitrification-denitrification process with nitrified liquor recycling to remove the residual NO_3-N. The performance of the remodeled facility was very satisfactory and met the target water qualitites.

In 1979, Shiga Prefecture promulgated the so called "Ordinance for Lake Biwa's Eutrophication Control," followed by another eutrophication control ordinance set in 1981 by Ibaraki Prefecture, where Kasumigaura Lake is located. These two ordinances urged authorities to practice nitrogen and phosphorus removal at municipal wastewater treatment plants. The Konan-Chubu treatment plant of the Lake Biwa Regional Sewarage System, for which construction work was started at the end of 1978, was originally designed as a facility based on the standard activated sludge process. However, because

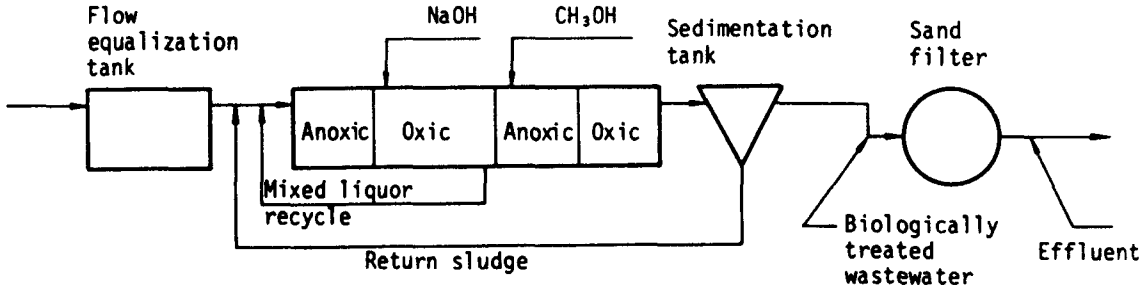


Fig. 1 Flow chart of the wastewater treatment plant at Nasu parking area

of the eutrophication control ordinance required to enforce the control of effluent from wastewater treatment plants at 20 mg/ℓ nitrogen from 1980, the target water quality of the effluent was set at 10 mg/ℓ. This led to the re-construction of the standard activated sludge facility to a combined BOD oxidation nitrification-denitrification with a nitrified liquor recycling type facility during the course of its construction. The re-constructed Konan-Chubu wastewater treatment plant, which began service in April 1982, is a first full-scale municipal wastewater treatment plant with a nitrogen and phosphorus removal function.

This nitrogen removal process based on the nitrified liquor recycling method which was subjected to a demonstration study at the Kasumigaura-Kohoku wastewater treatment plant. In addition, currently four municipal wastewater treatment plants, which discharge their effluents into lakes, adopt this process for full operation, as shown in Table 1. The performance of the nitrified liquor recycling method was reported at the 8th, 9th and 10th United States/Japan Conferences on Sewage Treatment Technology.

The above-mentioned eutrophication control ordinances are still valid in the form of more stringent prefectural effluent standards, in response to the national control of nitrogen and phosphorus effluents discharged lakes, which was the outcome of a partial amendment to the Water Pollution Control Law of 1985.

Meanwhile, Hamamatsu City which has a brackish lake, Lake Hamana, set target water qualities of total nitrogen and total phosphorus for treated sewage water at 5 mg/ℓ and 1 mg/ℓ, respectively, in order to conserve the water quality of Lake Hamana and made efforts to develop unique nitrogen and phosphorus removal methods. Two relatively small scale sewage treatment plants which discharge their effluents into the lake adopt a modified A_2O method and a method where a coagulant is added in the nitrification-endogenous denitrification method for nitrogen and phosphorus removal. The performance of these facilities was partly introduced at the 10th United States/Japan Conference on Sewage Treatment Technology, and since then the process has been further improved.

3. BIOLOGICAL NITROGEN REMOVAL PRACTICES

3-1 Improvement of the Nitrified Liquor Recycling Nitrification-Denitrification Method

The nitrified liquor recycling nitrification-denitrification method has been technically improved from the viewpoint of saving energy, and concerning the following two points: (1) the introduction of airlift circulation and (2) study on the rotation control effect of the mechanical mixer in the denitrification tank. These improvements and adjustments were conducted in order to improve the circulation facility at the Konan-Chubu wastewater treatment plant of the Shiga Prefectural Regional Sewerage System. The study was carried out by the Research and Technology Development Division of the Japan Sewage Works Agency.

(1) Examination of the multistage airlift circulation type nitrification-denitrification process

The nitrified liquor recycling process has been successfully practiced in Japan as a stable nitrogen removal process. The following items can be further improved:

① Theoretically, a higher circulation ratio R can achieve a nitrogen removal rate closer to 100%, but practically speaking, a too high circulation ratio results in increasing oxygen supply to the denitrification tank, which can inhibit the denitrification reaction. Therefore, the upper limit of R in practice is 200%, and it is difficult to get a nitrogen removal rate exceeding 80%.

R: Circulation ratio (%) = (Nitrified liquor recycling rate + Return sludge rate)/Inflow rate x 100

② The volume of the mixed liquor equivalent to 100 to 200% of the inflow water volume has to be returned with a circulation pump from the end of the nitrification tank to the forefront of the denitrification tank. This requires a pumping facility for circulation and its running cost. Several examples of operation of the nitrified liquor recycling nitrification-denitrification process suggest that the power consumption for a circulation pump equals approximately 10% of the power necessary for the water treatment facility of 0.4 - 0.7 kWh/m^3, as shown in Figure 2.

A multistage airlift circulation type nitrification-denitrification process was studied to improve these disadvantages. A flow chart of this process is shown in Figure 3.

This system is basically a multistaged process to n stages of the nitrified liquor recycling nitrification-denitrification, and inflowing sewage is supplied to each stage in division. It also provides the utilization of the airlift effect in the nitrification tank for nitrified liquor recirculation between the nitrification tank and the denitrification tank at each stage. This allows the circulation of the nitrified liquor to the denitrification tank without a special circulation line. Figure 4 is a schematic drawing of this circulation process.

Presently, this multistage airlift circulation type process is being tested at a pilot plant equipped with nitrification and denitrification tanks with a 900 capacity in total and three stages of 150 capacity for each tank. An outline of the results up to now is shown below:

① The utilization of the airlift effect allows satisfactory circulation of the nitrified liquor at a more than 200-300% (ratio to inflowing rate) level between the denitrification tank and the nitrification tank. The quantity of the circulation is dependent upon several factors including the aeration rate in the nitrification tank, the water depth of the airlift equipment, the size, shape, and location of the opening between the denitrification tank and nitrification tank, and the intensity of mixing in the denitrification tank.

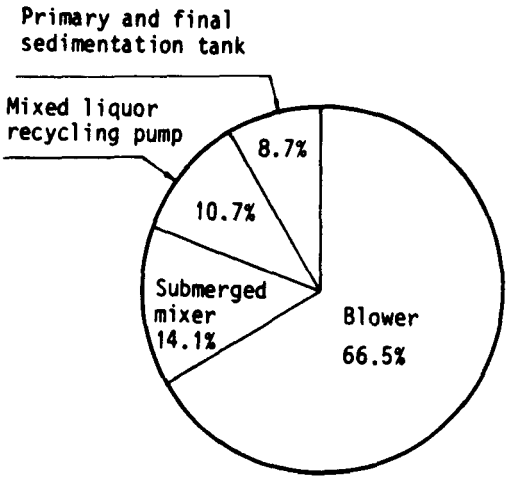


Fig. 2 Example of power consumption ratio among facilities in a biological nitrification-denitrification process with nitrified liquor recycling

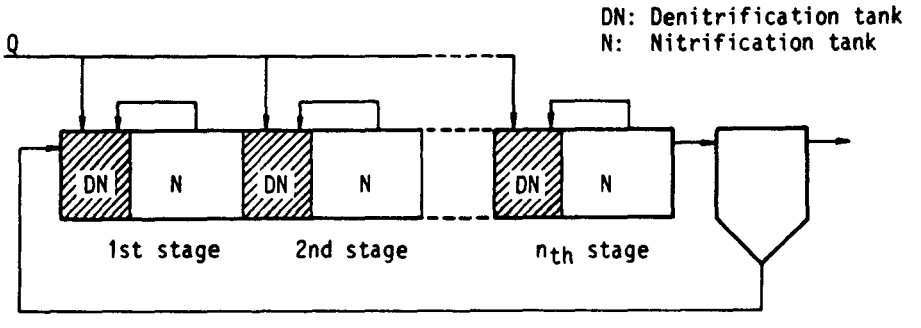


Fig. 3 Flow chart of multi-stage airlift circulation type nitrification-denitritication process

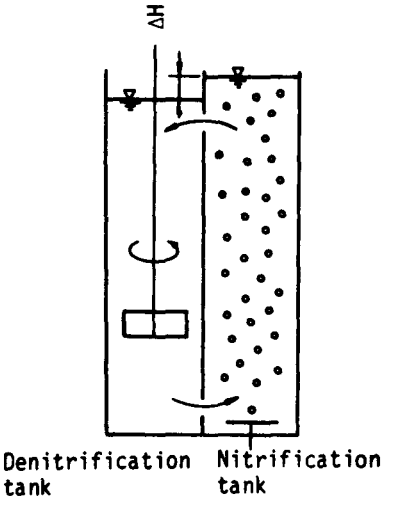


Fig. 4 The scheme of nitrified liquor recycling process by airlift effect

② A treatment experiment for actual sewage resulted in 7.5 mg/ℓ of total nitrogen, 80% of total nitrogen removal rate, and 94.2% of Kjeldahl-N removal rate for the treated water in the conditions of 14.4°C water temperature of the inflowing water, 12.4 hour detention time of nitrification-denitrification tank, and 37.6 mg/ℓ total nitrogen concentration of the inflowing water.

In response to the results of the pilot plant study for this multistage airlift circulation type process, the process is planned to be fully operated at the Konan-Chubu wastewater treatment plant.

(2) Study of the rotation control effect in the mixing facility of the denitrification tank

This study was carried out to save energy consumed by the mechanical mixer set in the denitrification tank in the aeration tank facility at the Konan-Chubu wastewater treatment plant. This treatment plant runs 8 trains of water treatment facilities where nitrification/denitrification is conducted by the nitrified liquor recycling process. A tank in one train with a 3825 m^3 capacity (W 9.0 x D 5.0 x L 85.0) is divided into four sections by partition walls, and the first two sections are anoxic tanks for denitrification. A submerged mechanical mixer is equipped on the bottom of the denitrification tank. The specifications of the mixer are shown in Table 2. Because the concentration and properties of the activated sludge were expected to change due to the quality of inflowing water and the treatment conditions when the mixer was installed in the denitrification tank, the capacity of the mixer was concluded with a fairly great allowance for safety. In addition, the mechanical mixer was originally developed as aeration equipment, and thus it can be changed to an aeration facility whenever necessary.

In this study, a method for energy saving was examined by controlling the rotation speed of the mixer through a frequency converter. The frequency converter decreased the current frequency 60 Hz to 50 Hz, 40 Hz, 30 Hz and 20 Hz to measure the concentration distribution of the mixed liquor in the denitrification tank and then to obtain the optimum degree of mixing for energy saving. Table 3 shows the necessary power in various experimental conditions and Figure 5 summarizes the vertical concentration distribution patterns of the mixed liquor in the denitrification tank.

Table 2 Specifications of the mixer installed at denitrification tank

Mixer type	Submerged mechanical aerator-mixer
Related power	3.7 kW
Impeller diameter	ϕ1150
Impeller rotation rate	193 rpm (60 Hz)
Aeration rate	3.5 Nm^3/min.

Table 3 Power control by frequency converter

Frequency	Power consumption per day	Mixing power density
60 Hz	650.9 kWh/8 mixers	12.1 W/m^3
50 Hz	443.2 kWh/8 mixers	8.1 W/m^3
40 Hz	258.9 kWh/8 mixers	4.8 W/m^3
30 Hz	142.0 kWh/8 mixers	2.6 W/m^3
20 Hz	80.9 kWh/8 mixers	1.5 W/m^3

Cf. The mixing power density is calculated on the basis of the mixing volume of 280 m^3 per mixer.

Figure 5 indicates that the sludge in the denitrification tank distributed homogenously in the 40-60 Hz condition. The formation of a sludge blanket was observed directly under the water surface at 30 Hz. The vertical distribution of sludge concentration at 20 Hz was unhomogenous, with a tendency of slightly lower concentration at the upper range. Furthermore, the sludge blanket was also formed 40-50 cm under the water surface.

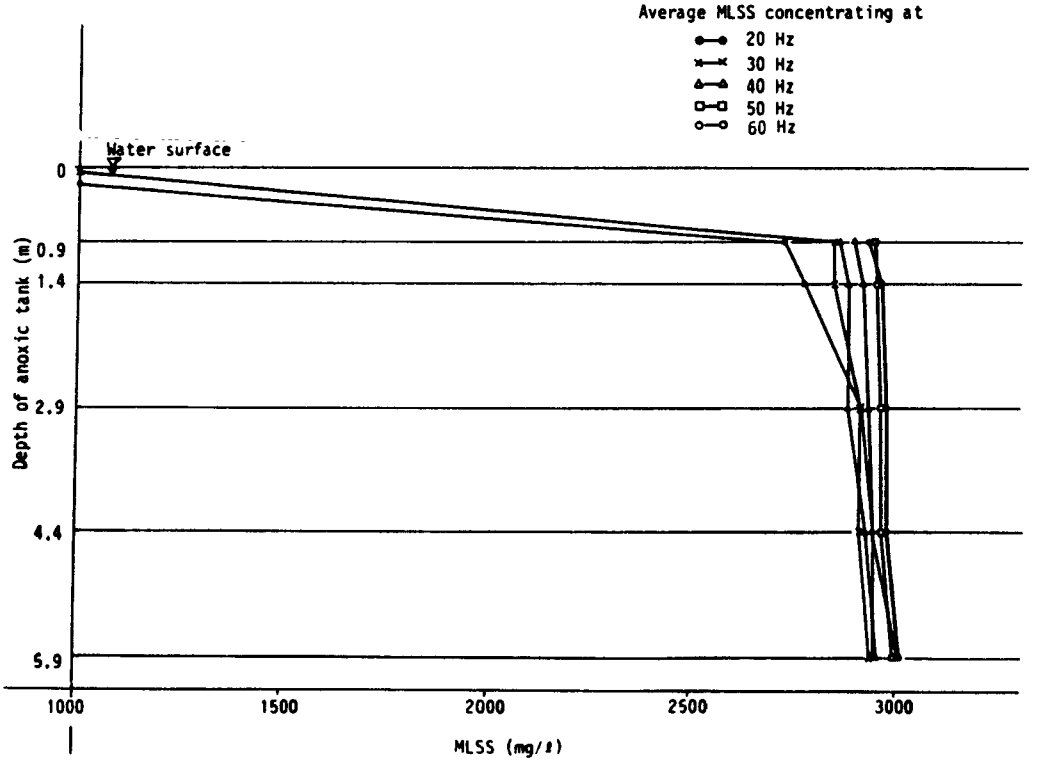


Fig. 5 The result of MLSS distribution measurement under the frequency converter control

Thus, the best operation, of the mixer in terms of energy conservation, in the denitrification tank is considered to lie around 30 Hz frequency. However, taking account of the possible operation at a higher MLSS concentration during winter, the operation control of the mixer was set at 30 Hz in summer and 40 Hz in winter. The unit mixing power in this case is 2.6 W/m^3 in summer and 4.8 W/m^3 in winter.

3-2 Improvement of the Nitrification-Denitrification Process Using Endogenous Respiration for Denitrification in Hamamatsu City

Two relatively small scale sewage treatment plants, the Koto wastewater treatment plant and the Hitomigaoka wastewater treatment plant, which treat sewage from residential areas along Lake Hamana, were originally designed and constructed to have an extended aeration process followed by tertiary treatment processes, including chemical coagulation, a separate nitrification-denitrification process, sand filtration and activated carbon absorption. This is because the treatment target was set at 5 mg/ℓ BOD, 10 mg /ℓ COD, 3 mg/ℓ SS, 5 mg/ℓ T-N, and 1 mg/ℓ T-P in order to conserve the water quality of Lake Hamana. Nevertheless, because of the troublesome and expensive operation and maintenance costs required for the effective utilization of the secondary treatment facility, with its surplus capacity owing to the original design for the extended aeration process, it was modified for biological nitrogen and phosphorus removal. The treatment performance of the facility was carefully monitored and the operational conditions of the treatment flow were revised and improved by stages so that the above-mentioned water quality targets were met.

The Koto wastewater treatment plant planned to attempt a biological nitrogen removal process called the nitrification-denitrification process using endogenous respiration for denitrification, and to cope with the phosphorus issue by adding aluminium sulfate into the aeration tank. The reason for selecting the biological nitrogen removal process, where an endogenous denitrification zone was combined, was because the compliance with the target water quality 5 mg/ℓ T-N required the application of a new nitrogen removal process which could provide a nitrogen removal rate higher than that of the nitrified liquor recycling process. The Koto wastewater treatment plant commenced an operational study of the nitrification-endogenous denitrification process in 1985. This report introduces the two years of the processes' performance.

The Koto wastewater treatment plant is a wastewater treatment plant which treats approximately 1000 m^3 per day of domestic waste water for some 4000 people. The sewage runs into a grit chamber, passes through a flow equalization tank, and flows into an aeration tank. The remodeled aeration tank is divided into five sections with parting strips (tanks A through E). The first stage is for BOD oxidation and nitrification, the middle stage is for endogenous denitrification, and the last E tank is for re-aeration. The nitrification tank and the re-aeration tank are aerated with disk diffusers, while the denitrification tank is equipped with an upward jet type mechanical mixer with a power of 1.5 kW at the bottom.

Figure 6 shows an operation flow during the test period. Initially, three aerobic tanks and one denitrification tank were operated to achieve perfect nitrification. The repair of the diffuser equipment in May 1986 proved that a sufficient amount of aeration for perfect nitrification was ensured. Then, another denitrification tank was added (2 tanks in total) in December 1986, and the operation mode was changed to II.

The operational conditions during the experimental period are summarized in Table 4. The BOD and water temperature of the inflowing water and the detention time of nitrification-denitrification tank from April 1986 to March 1987 are shown in Figure 7. As seen in this figure, the detention time of the nitrification-denitrification tank at this treatment facility is about 20 hours in the summer and 25 hours in the winter. The BOD of the inflowing water is approximately 150 mg/ℓ and 250 mg/ℓ in summer and winter, respectively.

Figure 8 illustrates the state of nitrogen removal during the experimental period. In the winter of 1985, the denitrification was degraded, which was followed by imperfect nitrification resulting in a T-N concentration higher than 10 mg/ℓ of the treated water. However, during the winter of 1986, the repair of the diffuser equipment and alteration of the operation mode resulted in the stabilization of the total nitrogen concentration of the treated water in the range between 5 mg/ℓ and 10 mg/ℓ. Figure 9 shows the behaviors of nitrogen and alkalinity in this process. Nitrification took place in the initial stage aerobic tanks A and B, and NO_3-N was removed in the denitrification tanks C and D. Nitrification and denitrification were accompanied with the consumption of alkalinity in the nitrification tanks, while alkalinity was produced in the denitrification tanks. A series of these changes in the water quality is considered to be typical water quality behavior in this process.

The results of this operational experiment have proved the significance of DO control in the nitrification tank and imperfect denitrification in the late denitrification stages when DO is too high. This seems to be a result of the supply of DO from the nitrification tank to the denitrification tank and the consumption of organic substances attaching to the activated sludge which serves as a carbon source for the endogenous denitrification. Therefore, to solve this problem, the DO in the nitrification tank B is controlled at about 1 mg/ℓ.

Another problem is the lowered alkalinity in the nitrification tank. It has been confirmed that the control of DO concentration in the nitrification tank at a lower level partly leads to denitrification in the nitrification tank with a resultant supply of alkalinity.

Thus, in this process the control of the DO concentration in the nitrification tank at a low level can promote the denitrification in the late stages and reduce the alkalinity consumption. This operational experiment has proved that the control of the DO concentration in the nitrification tank is a key for the successful operation of this process.

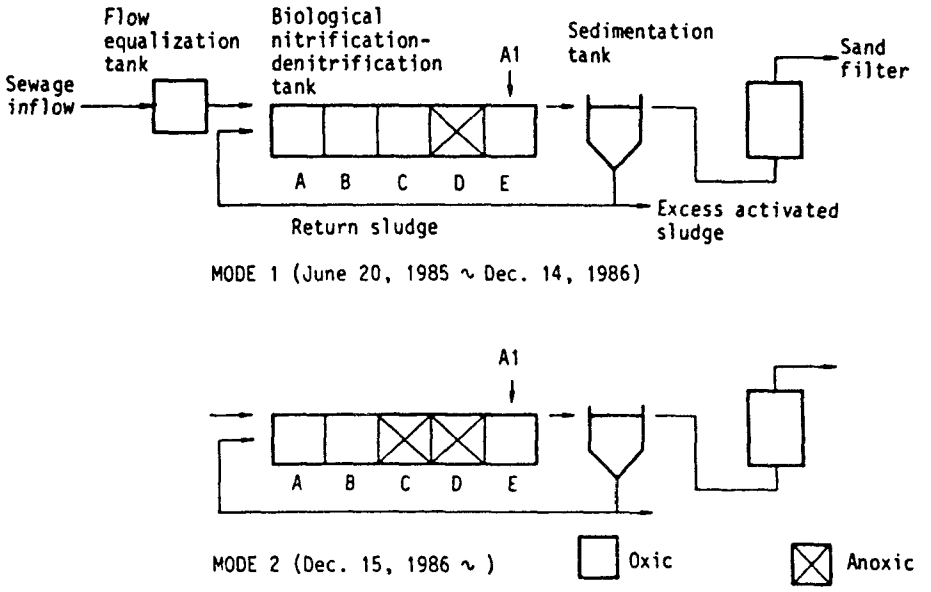


Fig. 6 Flow chart of demonstration experiment on nitrogen removal in Kotoh WTP

Table 4 Experimental

Period	1985.6 ∿1986.3	1986.4	5	6	7
Flow rate (m^3/day)	1080	1013.5	1089.5	1161.3	1289.5
Return sludge ratio (%)	81	82.9	77.1	72.3	65.1
Detention time of nitrification-denitrification tank	21.3	22.7	21.1	19.8	17.9
Aeration/flow rate	14.9	-	-	-	14.0
WAS (m^3/day)	18.8	16.2	19.0	17.7	17.7
SRT (day)	23.0	28.1	21.2	22.0	18.1
SAT (day)		16.9	12.7	13.2	10.9
BOD loading (kg/kg·day)		0.14	0.12	0.10	0.14
T-N loading (kg/kg·day)		0.021	0.021	0.027	0.029
Anoxic-oxic condition	MODE 1				

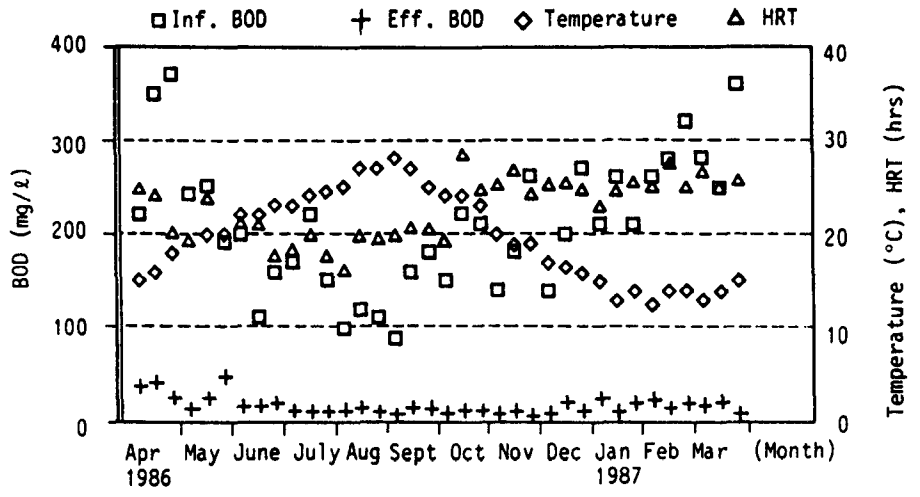


Fig. 7 Annual change of BOD and temperature of inflow sewage and the detention time of nitrification-denitrification tank

condition at Kotoh WTP

8	9	10	11	12		1987.1	2	3
1202.0	1112.8	975.8	947.8	936.4	983.0	926.8	919.9	1008.1
69.9	75.5	86.1	88.6	89.7	85.5	90.6	104.5	89.5
19.2	20.7	23.6	24.3	24.6	23.4	24.8	25.1	23.0
15.1	16.3	18.6	18.4	18.9	20.9	21.0	19.5	18.0
18.5	17.5	17.7	18.3	14.3	10.6	14.2	16.4	19.2
17.6	21.6	20.7	21.8	29.2	37.6	31.9	31.8	22.7
10.6	13.0	12.4	13.1	17.5	15.0	12.8	12.7	9.1
0.094	0.096	0.11	0.094	0.084	0.20	0.15	0.18	0.23
0.026	0.026	0.021	0.025	0.019	0.035	0.032	0.028	0.037
					MODE 2			

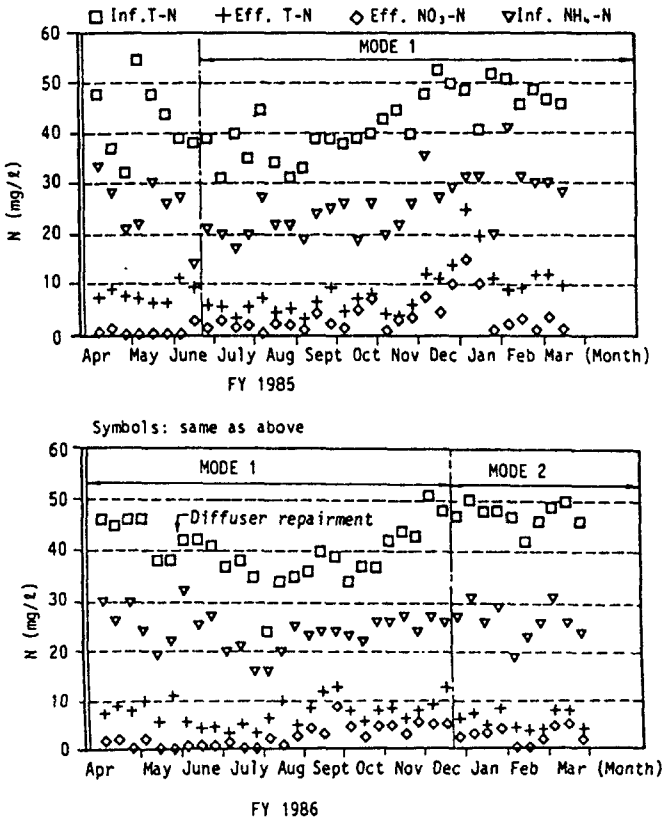


Fig. 8 Nitrogen removal condition through fiscal 1985 to 1986

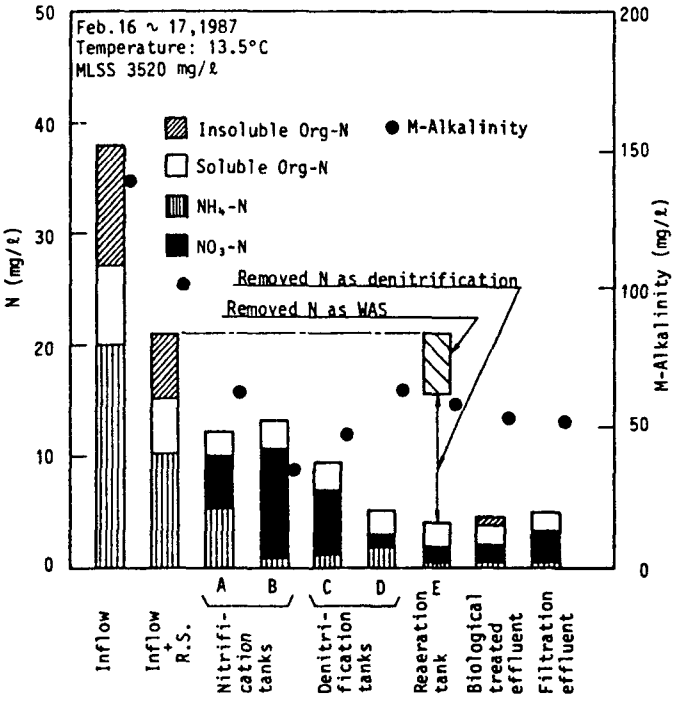


Fig. 9 The change of nitrogen compounds and alkalinity in nitrification-denitrification process

3-3 Demonstration Study on Biological Nitrogen and Phosphorus Removal Using a Step Inflow Process at the Shinjiko-Tobu Wastewater Treatment Plant

The Shinjiko-Tobu wastewater treatment plant, which discharges treated water through a river to a closed brackish water area, Lake Nakaumi, is expected to require nitrogen and phosphorus removal in the near future. For this reason, Shimane Prefecture carried out a demonstration test on biological nitrogen and phosphorus removal using water treatment trains in a new extension to the plant. This demonstration experiment was conducted by the Research and Technology Development Division, Japan Sewage Works Agency.

The experiment, commenced in June 1986, using two trains of the water treatment facility -- the south train and the north train -- for simultaneous biological removal of nitrogen and phosphorus in two types of modes, as shown in Figure 10. The north train adopted a modified nitrification endogenous denitrification process in which the primary effluent was supplied stepwise to the denitrification tank without the circulation of nitrified liquor. The south train was operated as a control in which nitrogen was treated with a removal process based on the nitrification/endogenous denitrification principle. Table 5 summarizes the operational modes of both trains during the test period. Various types of operation modes were selected up until September mainly for the purpose of perfect nitrification in the aeration tank, and the tests after Run 7 were made for full scale study. During this time span, the step rate in the north train was changed from 10% to 40%. Additionally, in the tests after Run 10, PAC (poly aluminium chloride) was added to the supernatant in the sludge thickening tank for the purpose of cutting the phosphorus load in the sidestreams from the sludge treatment system. Table 6 summarizes the operational conditions of both trains. The detention time of the nitrification-denitrification tank was 16-19 hours for both trains.

Figure 11 shows the NH_4-N concentrations of the inflowing water to the aeration tank, and the nitrogen forms in the treated water and their concentrations during the experimental period. A comparison between the test train and the control train during the experimental period after Run 7 suggests that the concentration of the residual NO_4-N concentration in the test train is about 5 mg/ℓ lower than that in the control train. In other words, denitrification was not sufficient within the detention time of 4-5 hours in the endogenous denitrification process of this time, and a technique in which a carbon source is added through step supply to increase the denitrification rate in the denitrification tank can bring about a good effect to some extent. Furthermore, the step supply may leave NH_4-N contained in the inflowing sewage water in the treated water, but the result of this study shows no residual NH_4-N even in the condition of 40% step supply. With respect to this point, a simple calculation based on the nitrification rate of 2.27 mgN/g - MLSS/hr measured in December confirmed that the nitrification ability in the re-aeration tank in the last stage was great enough to nitrify

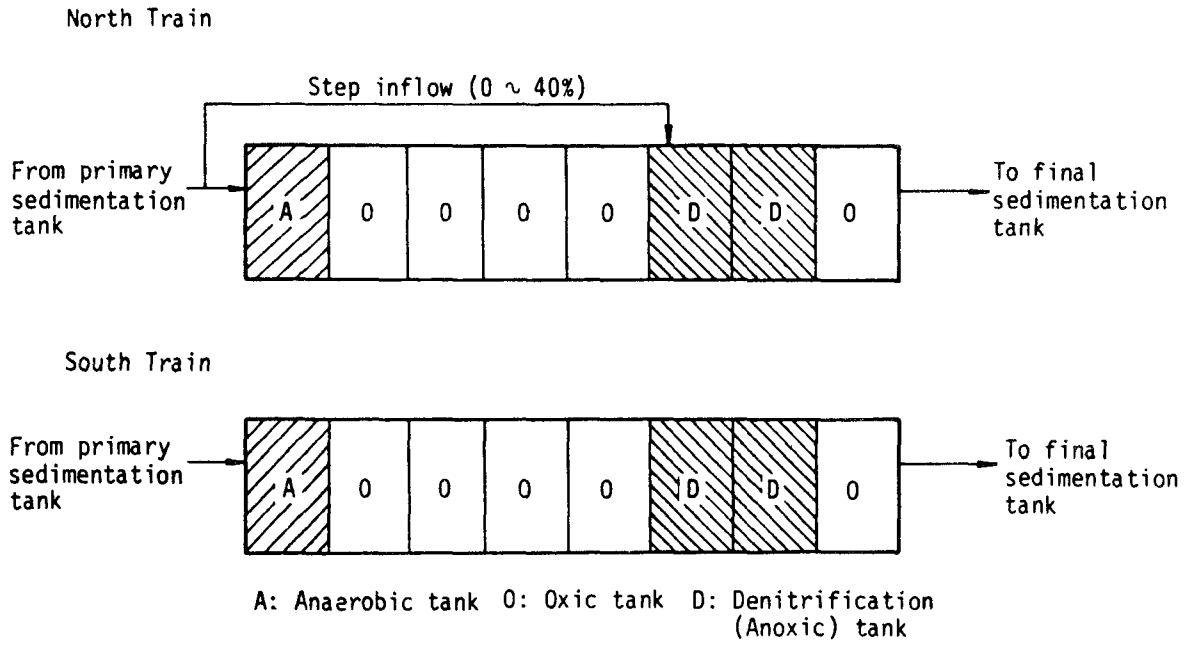


Fig. 10 Flow chart of demonstration experiment in Shinjiko-Tobu WTP

Table 5 Experimental condition of the demonstration experiment

Run	Period	North train		South train		PAC dosing
		Aeration mode	Step ratio	Aeration mode	Step ratio	
	1986					
B 1	6/1 ∿ 6/10	A0000000	0	A0000000	-	-
B 2	6/11 ∿ 6/20	AA000000	0	AA000000	-	-
B 3	6/21 ∿ 7/8	A0000DD0	0	AA000000	-	-
B 4	7/9 ∿ 7/18	A0000DD0	0	00000000	-	-
B 5	7/19 ∿ 7/18	00000000	0	00000000	-	-
B 6	7/29 ∿ 9/16	A0000DD0	0	000D00D0	-	-
B 7	9/17 ∿ 9/30	A0000DD0	10(10.3)	A0000DD0	-	-
B 8	10/1 ∿ 10/14	A0000DD0	20(19.8)	A0000DD0	-	-
B 9	10/15 ∿ 10/30	AA000DD0	30(29.6)	A0000DD0	-	-
B10	10/31 ∿ 12/2	AA000DD0	40(38.9)	AA000DD0	-	0.16 mmol/g-SS
B11	12/3 ∿ 12/9	AA000DD0	40(36.3)	AA000DD0	-	0.21 mmol/g-SS
B12	12/10 ∿ 12/26	A0000DD0	40(38.2)	AA000DD0	-	0.15 mmol/g-SS

Note 1: Aeration mode shows the aeration condition for 8 cells of the aeration tank.

A = Anaerobic O = Oxic D = Denitrification (Anoxic)

Note 2: Step ratios 10, 20, 30, 40% are design values. The values in ( ) are record ones.

Table 6 Experimental condition for each run

Run	Aeration tank Temperature (°C)	Inflow rate (m^3/day)	Return sludge rate (m^3/day)	Aeration tank detention time (hours)	Aeration tank MLSS (mg/ℓ)	Aeration tank DO (mg/ℓ)
North						
B 1	21.5	5,472	2,599	18.9	2,544	2.3
B 2	22.5	5,458	3,237	19.0	2,542	3.9
B 3	22.5	6,001	5,677	17.3	2,742	5.5
B 4	22.9	6,948	5,558	14.9	2,550	4.5
B 5	24.2	6,393	3,152	16.2	1,863	5.4
B 6	26.0	5,677	2,844	18.3	1,785	6.4
B 7	24.6	5,576	2,805	18.6	1,399	5.3
B 8	23.3	5,498	2,722	18.9	1,752	6.9
B 9	21.7	5,251	2,520	19.7	1,767	7.2
B10	19.7	5,270	2,667	19.7	2,382	5.8
B11	18.1	5,507	2,594	18.8	2,555	6.8
B12	17.7	5,539	2,620	18.7	2,689	5.7
South						
B 1	21.5	5,504	2,614	18.8	2,125	3.7
B 2	22.4	5,476	3,247	18.9	2,381	2.8
B 3	22.5	6,008	5,684	17.3	2,900	6.3
B 4	23.0	5,935	5,548	15.0	3,167	5.0
B 5	24.3	6,388	4,037	16.2	2,631	1.6
B 6	26.0	5,679	2,845	18.3	1,695	6.0
B 7	24.4	5,220	2,652	19.9	1,582	6.5
B 8	23.2	5,454	2,645	19.0	2,050	7.1
B 9	21.6	5,238	2,504	19.8	1,793	7.2
B10	19.5	5,256	2,644	19.7	2,416	4.5
B11	18.0	5,507	2,599	18.8	2,475	3.9
B12	17.5	5,514	2,603	18.8	2,887	4.3

Note 1: The values are average ones for earn run.
Note 2: The detention time of aeration tank is based on inflow rate excluding return sluge flow rate.
Note 3: The radus of MLSS and DO are measured at the end of aeration tank.

the NH_4-N concentration added by the step supply. Thus, the favorable effect of higher nitrogen removal rate through the step supply was proved within the range of this experimental condition.

Figure 12 shows the results of phosphorus treatment. The phosphorus concentration in the treated water decreased after round Run 10 in the test train. This seems to be due to the effect of the decrease of phosphorus load from the sludge treatment system which was achieved by adding a coagulant (PAC) to the supernatant in the sludge thickening tank.

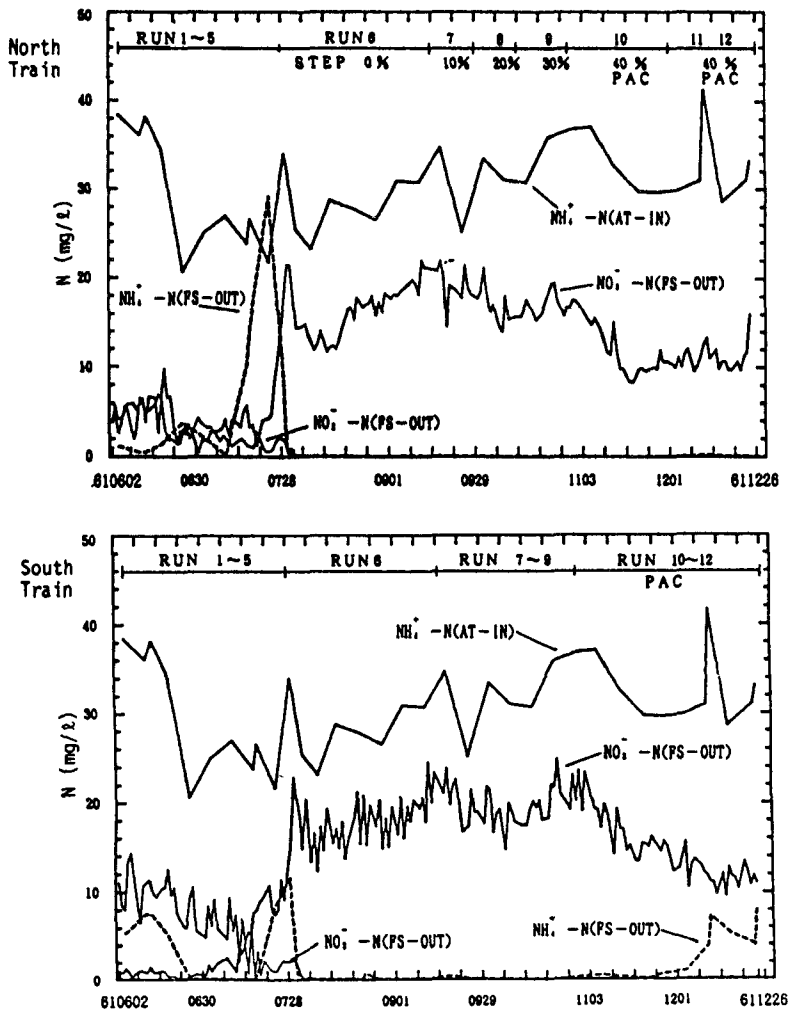


Fig. 11 Date of nitrogen removal

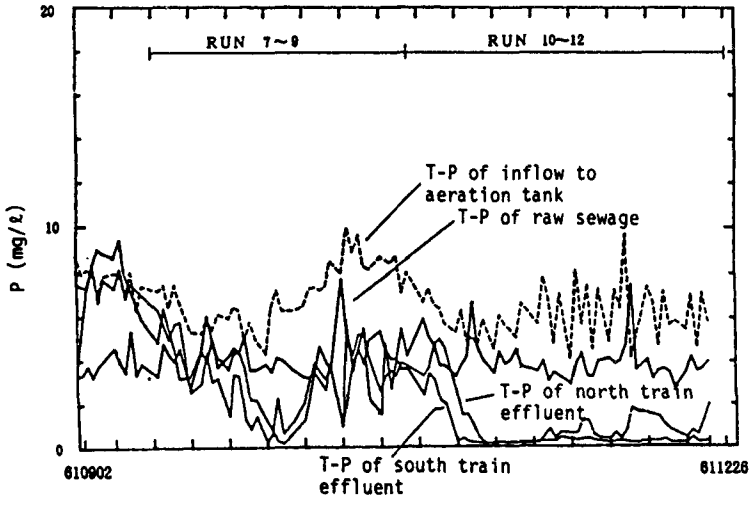


Fig. 12 Data of phosphorus removal

3-4 Performance of the A_2O Process at the Toba Wastewater Treatment Plant of Kyoto City

Kyoto City remodelled a part of an aeration tank of the activated sludge treatment process in the Toba wastewater treatment plant from November 1983 to March 1985, to conduct a demonstration study on a simultaneous biological nitrogen and phosphorus removal process. This study was reported in detail at the 2nd WPCF/JSWA Joint Technical Seminar on Sewage Treatment Technology.[2] Here, an outline of the study is introduced. The study was carried out in an aeration tank which had been remodelled for the A2O (Anaerobic-Anoxic-Oxic) flow sheet process, as shown in Figure 13. The sewage inflow rate into the test facility was approximately 16,000 m^3/day, and the detention time of the biological treatment tank relative to the inflow rate was 10 hours. In this study, an adjoining treatment line (treatment capacity of 60,000 m^3/day) using a standard activated sludge process, and the same detention time as in the test train, was used as a control. The study was designed so that the inflow rate was kept constant with no diurnal fluctuation in the first half of the test period and the inflow rate during the daytime was three time higher than that during the night-time in the second half.

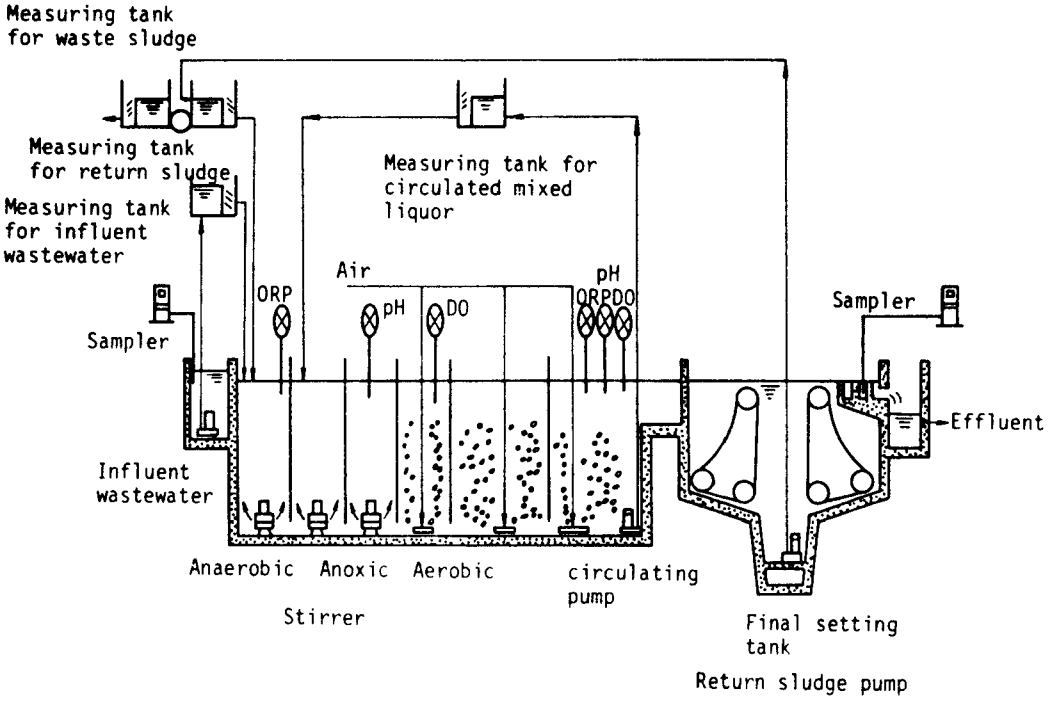


Fig. 13 Scheme of demonstration experiment facilities

Table 7 summarizes the experimental conditions during the test period. The results on water qualitites are given in Table 8.

With respect to nitrogen removal, the T-N of the treated water in the test train was 9.1 mg/ℓ on average under the constant flow rate condition and 10.1 mg/ℓ on average under the variable flow rate condition, or 64% and 55% of removal rate, respectively. In comparison with the control train, the T-N of the treated water decreased to about 1/2 under the constant flow rate condition, and to about 2/3 under the variable flow rate condition. The less decrease under the variable flow rate condition is assumed to be because of the nitrification operation in the control line, which was done in order to control the water qualities in the Yodo River system, where the treated water was discharged, during the drought season.

Table 7 Operating conditions of demonstration experiment

Item \ Section / Period / Sample	Constant flow					
	1		2		3	
	'83 12.1 ∿ 28		'84 1.9 ∿ 3.31		4.1 ∿ 6.10	
	Experiment	Control	Experiment	Control	Experiment	Control
Water temp (°C)	17.6	17.2	14.9	14.5	20.6	19.9
Inf. flow (m^3/day)	15,620	53,670	15,650	55,650	15,310	56,850
Return sludge ratio (%)	49	40	61	33	48	36
Circulating ratio (%)	96		102		92	
Waste sludge ratio (V/V%)	1.0	0.9	1.4	1.3	1.0	1.4
Air Supply: G/L (m^3/m^3)	10.7	5.4	8.0	4.7	7.3	4.8
Air Supply: per BOD removed (m^3/kg)	149	79.6	98.3	57.5	85.6	57.2
Retention Time: Anaerobic T. (hr)	1.3		1.3		1.3	
Retention Time: Anoxic T. (hr)	2.7		2.6		2.7	
Retention Time: Aerobic T. (hr)	6.0		6.0		6.1	
Retention Time: Total (hr)	10.0	10.2	9.9	10.6	10.1	10.8
Settling time (hr)	4.4	4.5	4.4	4.7	4.5	4.7
Overflow rate (m^3/m^3 • day)	21.7	21.6	21.7	20.7	21.3	20.4
MLSS (mg/ℓ)	1,570	920	1,560	920	1,810	880
MLVSS/MLSS (%)	80.9	82.6	82.1	82.6	79.0	79.5
SVI (-)	206	299	272	275	263	267
SRT (day)	8.9	6.6	9.6	6.0	10.3	5.9
BOD - SS loading (kg/kg • day)	0.12	0.20	0.41	0.21	0.12	0.24
RSSS (mg/ℓ)	5,850	4,760	4,360	4,590	6,430	4,330
P content in sludge (%)	2.9	1.9	2.3	2.0	3.2	2.4
N content in sludge (%)	8.5	6.6	8.4	8.7	7.9	7.8
MLDO (mg/ℓ)	5.8		3.7		3.5	
ORP in Anaerobic t. (mV)	-115		-68		-200	

As to phosphorus removal, the mean T-P of the treated water was 0.7 mg/ℓ under the constant flow rate condition, and 0.5 mg/ℓ under the variable flow rate condition. The removal rate was 74% and 80%, respectively, and thus it was confirmed that the phosphorus removal rate in the test train was fairly high compared with that in the control train. However, test 4 suggested a fairly high phosphorus level in the treated water in the test train. This appears to be due to the inhibition of biological phosphorus removal phenomenon caused by inflow of rainwater during the rainy season.

Table 9 presents comparative figures of electric power consumption between the test train and the control train in this demonstration study on the A_2O process. The table shows that the electric power consumption was 0.247 and 0.146 kWh/m³ of the treated water in the test train (for the simultaneous nitrogen and phosphorus removal process) and the control train, respectively, with a difference of 0.101 kWh/m³.

in Toba wastewater treatment plant

		Fluctuant flow					
4		5		6		7	
6.11 ∿ 7.3		7.4 ∿ 9.10		10.1 ∿ 12.17		'85 1.4 ∿ 3.29	
Experiment	Control	Experiment	Control	Experiment	Control	Experiment	Control
23.8	23.5	25.7	25.7	20.8	20.0	15.3	14.6
16,090	65,010	16,220	64,280	16,190	58,150	16,150	63,280
31	31	20	31	36	35	36	37
91		97		99		98	
1.0	1.2	1.0	0.8	0.9	1.3	0.9	0.9
6.6	4.3	5.7	3.8	6.6	5.8	6.9	6.0
117	77.3	97.2	65.2	83.9	74.6	86.5	74.8
1.3		1.3		1.3		1.3	
6.6		2.5		2.5		2.5	
5.8		5.8		5.8		5.8	
9.7	9.8	9.6	9.8	9.6	11.5	9.6	9.9
4.2	4.3	4.2	4.3	4.2	4.9	4.2	4.4
22.3	22.5	22.5	22.3	22.5	20.2	22.4	22.4
1,930	920	1,100	970	1,500	1,030	1,550	1,600
77.2	77.4	75.5	76.3	77.3	78.6	76.1	76.3
223	145	150	136	138	178	156	106
15.6	14.1	8.3	9.9	8.2	8.2	8.8	9.6
0.08	0.15	0.14	0.16	0.14	0.20	0.14	0.12
8,940	4,020	8,440	5,170	6,460	4,690	11,500	7,110
3.0	2.3	2.9	2.0	3.2	2.4	2.8	1.8
7.3	7.5	7.0	6.7	7.5	7.7	7.8	7.8
4.9		4.0		3.0		4.9	
-235		-233		-208		-168	

Table 8 Results of water

Section		Constant flow								
		1			2			3		
Period		'83 12.1 ∿ 12.28			'84 1.9 ∿ 3.31			4.1 ∿ 6.10		
Sample			Effluent			Effluent			Effluent	
Item		Inf.	E.	C.	Inf.	E.	C.	Inf.	E.	C.
BOD (mg/ℓ)	T-	78.2	12.9 (83.5)	14.5 (81.5)	87.8	13.8 (84.3)	8.0 (90.9)	89.1	7.6 (91.5)	11.3 (87.5)
	C-		6.5 (91.7)	10.5 (86.6)		6.5 (92.6)	6.1 (93.1)		3.9 (95.6)	5.3 (94.1)
	S-	38.0	1.4 (96.3)	2.2 (94.2)	41.6	1.5 (96.3)	2.4 (94.2)	42.5	1.5 (96.5)	2.7 (93.6)
SS (mg/ℓ)		64	14 (78.1)	18 (71.9)	63	9 (85.7)	5 (92.1)	58	4 (93.1)	4 (93.2)
COD (mg/ℓ)	T-	68.9	21.3 (69.1)	27.0 (60.8)	73.2	21.0 (71.3)	24.0 (67.2)	72.7	16.9 (76.8)	23.6 (67.7)
	S-	46.0	14.6 (68.3)	17.4 (62.2)	48.9	16.2 (66.9)	21.2 (56.6)	46.1	14.3 (69.0)	20.3 (56.0)
TOC (mg/ℓ)	T-		-	-		-	-		-	-
	S-		-	-		-	-		-	-
N (mg/ℓ)	T-	27.0	8.3 (69.3)	20.2 (25.2)	26.9	10.0 (62.8)	19.6 (27.1)	24.7	8.9 (64.0)	19.4 (21.5)
	TKN	26.4	3.7 (86.0)	19.2 (27.3)	26.0	4.7 (81.9)	18.7 (28.1)	24.5	2.2 (91.0)	18.4 (25.2)
	NH_4-	17.5	1.1	16.0	17.3	2.4	16.9	15.8	0.6	16.1
	NO_2-	0.4	0.2	0.2	0.4	0.9	0.2	0.1	0.2	0.2
	NO_3-	0.2	4.4	0.8	0.5	4.4	0.7	0.0	6.4	0.8
P (mg/ℓ)	T-	2.7	0.8 (70.4)	0.8 (70.4)	2.8	1.1 (60.7)	1.3 (53.6)	3.0	0.3 (90.0)	1.3 (56.7)
	S-	1.9	0.3	0.5	1.8	0.9	1.2	2.0	0.2	1.2
	Ort-	1.7	0.3	0.4	1.5	0.8	1.1	1.7	0.1	1.1
pH (-)		8.5	7.1	7.3	8.7	7.0	7.3	8.3	7.1	7.3
Alkalinity (mg/ℓ)		148	74	141	143	75	137	140	64	127
S - P in Anaerobic T.(mg/ℓ)			8.1			3.1			10.7	

qualtity analysis

Fluctuant flow											
4			5			6			7		
6.11 ∿ 7.3			7.4 ∿ 9.10			10.1 ∿ 10.27			'85 1.4 ∿ 3.29		
	Effluent			Effluent			Effluent			Effluent	
Inf.	E.	C.	Inf.	E.	C.	Inf.	E.	C.	Inf.	E.	C.
58.4	3.3 (94.3) 2.1 (96.4)	6.3 (89.2) 2.8 (95.2)	60.8	3.4 (94.4) 2.2 (96.4)	4.8 (92.1) 2.5 (95.9)	81.8	6.7 (91.8) 3.2 (96.1)	9.2 (88.8) 4.1 (95.0)	83.7	10.7 (87.2) 4.0 (95.2)	9.2 (89.0) 3.5 (95.8)
22.7	1.0 (95.6)	1.4 (93.8)	22.6	1.1 (95.2)	1.3 (94.1)	29.8	1.2 (96.0)	1.5 (95.0)	35.4 64	1.1 (96.9)	1.4 (96.0)
37	2 (94.6)	2 (94.6)	42	2 (95.1)	2 (95.2)	57	3 (94.7)	3 (94.7)	70.6	4 (93.8)	3 (96.0)
61.2 38.1	12.3 (79.9) 10.9 (71.4)	14.6 (76.1) 13.0 (65.9)	57.8 37.1	12.3 (78.7) 11.3 (69.5)	12.1 (79.1) 10.8 (70.9)	70.8 44.3	15.8 (77.7) 14.2 (67.9)	17.2 (75.7) 15.4 (65.2)	45.4 55.7	17.1 (75.8) 14.7 (68.3)	16.0 (77.3) 13.8 (69.6)
	- -	- -	51.3 30.9	14.5 (71.7) 12.8 (58.6)	16.2 (68.4) 11.7 (62.1)	61.0 39.8	14.5 (76.2) 13.1 (67.1)	17.3 (71.6) 15.0 (62.3)	29.3 26.2	11.5 (79.4) 8.7 (70.3)	11.6 (79.2) 9.1 (68.9)
17.5 17.0 10.2 0.2 0.3	6.9 (60.6) 1.4 (91.8) 0.0 0.0 5.5	12.7 (27.4) 7.4 (56.5) 5.9 0.6 4.7	17.7 17.6 11.0 0.1 0.0	7.8 (55.9) 1.9 (89.2) 0.5 0.0 5.9	11.3 (36.2) 4.5 (74.4) 2.9 0.1 6.7	23.7 23.5 16.2 0.1 0.1	11.2 (52.7) 4.5 (80.9) 3.0 0.2 6.5	16.0 (32.5) 10.6 (54.9) 8.8 0.3 5.1	25.5 17.8 0.3 0.4 2.7	11.0 (58.0) 3.6 (85.9) 1.7 0.7 6.7	16.8 (35.9) 5.4 (78.8) 3.5 0.1 11.3
1.9 1.2 1.0	1.5 (21.1) 1.4 1.4	0.9 (52.7) 0.8 0.8	2.0 1.2 1.0	0.5 (75.0) 0.5 0.4	1.1 (45.0) 1.1 1.0	2.7 1.8 1.6	0.5 (81.5) 0.4 0.3	1.0 (63.0) 1.0 1.0	1.8 1.5 8.0 131	0.7 (74.1) 0.5 0.5	1.4 (48.1) 1.3 1.2
8.2	7.2	7.2	8.1	7.2	7.1	8.3	7.1	7.2		6.8	6.6
109	58	70	115	59	66	136	72	93		52	43
				7.1			7.7			9.5	

Note: E. = Experimental process
C. = Control process removal ratio (%) is shown in parentheses.

Table 9 Energy consumption in biological process

(Unit: kWh/m³)

Facilities	Experimental process	Control process
Blowers	0.155	0.134
Sludge return pumps Waste sludge pumps Collectors	0.074	0.012
Circulating pumps		-
Stirrers	0.018	-
Total	0.247	0.146

REFERENCES

1) Tamai, Y., "Water Quality Control in the Yodo River System," Proceedings of the 24th JSWA Annual Technical Conference International Session, Japan Sewage Works Association, 1987.

2) Sasai, S., "Simultaneous Removal of Nitrogen and Phosphorus in the Anaerobic Aerobic Biological Process," Proceeding of the 2nd WPCF/JSWA Joint Technical Seminar on Sewage Treatment Technology, 1986.

CHARACTERIZATION OF VARIOUS DIFFUSERS AND ITS APPLICATION

by

Kazunori Hosokawa
Director, Construction Division
Sewage Works Bureau
City of Yokohama

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

Prepared for Presentation at:
11th United States/Japan Conference
on
Sewage Treatment Technology

October 1987
Tokyo

Abstract

Aeration is not only an important component of the activated sludge process which directly affects clarification efficiency, but because it consumes 40 to 50 percent of the electric power used at sewage treatment plants, its weight in maintenance management is a large one.

For this reason, a variety of diffusers have been developed over the years. Such diffusion characteristics as bubble size, pore size, and dry air flow rate are widely used in their design specifications. The correlation between these characteristics and oxygen transfer efficiency (OTE), however, is not clear. As a result, there is a lack of basic information needed for the efficient design of diffusers.

This situation led us to investigate the relationship between diffuser characteristics and OTE, using 16 diffusers ranging from fine-bubble to coarse-bubble diffusers.

Three levels of experiments were involved: (1) a study of diffuser characteristics using small-sized test tanks, (2) experiments using 5 m (W) × 5 m (L) × 5.8 m (H) tanks, and (3) on-site operational experiments.

Our findings were as follows:

OTE for fine-bubble diffusers was 20 to 30 percent while that for coarse-bubble diffusers was 9 to 12 percent. The total surface area of bubbles per air flow rate of 1 liter displayed the highest correlation with OTE. The experiments demonstrated an optimum pore size for OTE and power consumption efficiency, and revealed the relationship between OTE, diffuser placement density, and diffuser distribution.

We also discovered that there is little change to the effectiveness of fine-bubble diffusers over a three-year period. The value for OTE varied more between sewage treatment plants than between diffusers, revealing that standardized procedures need to be established to accurately measure OTE.

Further, an understanding useful for diffuser design was gained in such areas as the relationship between OTE, inorganic salt, and surface active agents.

Contents

1. **Introduction** 294

2. **Installation of Diffusers in Yokohama City: Past and Present Performance of Various Diffusers** ... 295

3. **Characteristics of Oxygen Transfer** 297
   - 3-1. Oxygen Transfer Rate (OTR) 297
   - 3-2. Measurements of OTR 300
   - 3-3. Aeration Efficiency 301

4. **Evaluation of Diffuser Performance in Test Tanks** ... 303
   - 4-1. Outline of Experiments 303
   - 4-2. Results of Experiments I and II 307
   - 4-3. Results of Experiment III 314
   - 4-4. Relationship between Diffuser Characteristics and OTE 321
   - 4-5. Power Consumption Efficiency of Each Type of Diffuser 326

5. **On-site Study** 329
   - 5-1. Results of On-site Study 330
   - 5-2. Changes in Diffuser Performance over Time 337

6. **Conclusion** 341

7. **Afterword** 343

References 344

1. Introduction

Since the activated sludge process was first used for the treatment of sewage and organic wastewater around the turn of the century, various types of diffusers have been developed and installed at sewage treatment plants.

Sewage treatment first began in Japan in 1922. Presently there are 631 sewage treatment plants in operation throughout the country as of March 1986. Over 90 percent of them use an activated sludge process incorporating variations such as step aeration. Diffusers, rather than mechanical aeration devices, are by and large used for aeration. Spiral-flow aeration using plate or tube diffusers is the principal aeration method employed.

With the outbreak of the global oil crisis in the 1970s, greater emphasis was placed on energy efficiency, and sewage treatment plants faced the need of reducing electricity consumption. Blowers, in particular, needed improvement as they accounted for 40 to 50 percent of sewage treatment power consumption. Consequently, in the 1980s, engineering efforts focused on the development of a whole-floor aeration system using fine-bubble diffusers. To this day, however, the relationships between oxygen transfer efficiency and bubble size, pore size, dry pressure as well as dynamic wet pressure are not fully understood and basic data for designing diffuser performance is still insufficient.

This report summarizes our experiments dealing with 16 types of diffusers ranging from coarse-bubble diffusers to fine-bubble diffusers in terms of their bubble size, dry pressure, dynamic wet pressure and oxygen transfer efficiency.

2. Installation of Diffusers in Yokohama City: Past and Present Performance of Various Diffusers

A total of eleven sewage treatment plants started operation during the years of 1962 to 1984 in Yokohama and all of these facilities are in service at present. Complete treatment takes place at each plant, using an activated sludge process with diffusers.

The following types of diffusers, presented in chronological order, have been installed in Yokohama:

1962 to 1965: Swing-type Saran-wrapped diffusers were introduced to the three treatment plants of Chubu, Nambu and Hokubu No. 1, constructed in the early 1960s. These diffusers can be installed or dismantled easily without suspending wastewater treatment operations at the plant.

1972 to 1979: Perforated-pipe diffusers were used at the five plants of Totsuka No. 2, Kohoku, Midori, Kanagawa and Kanazawa, built in the 1970s.

A switch was made to perforated-pipe diffusers during this period because Saran-wrapped diffusers clogged frequently, depending on the quality of wastewater and especially in winter with the decline in water temperature, and this required considerable labor for chemically cleaning clogged diffusers.

In order to cope with this problem at Nambu and Hokubu No. 1 of the first three plants, a change was made to plastic-tube diffusers. However, since the Chubu plant experienced few clogging problems, Saran-wrapped diffusers continue to be used to this day.

1983 to 1984: In the most recently completed three plants of Seibu, Totsuka No. 1 and Hokubu No. 2, static mixers were adopted to avoid clogging, and ceramic or plastic plate diffusers were installed in the 10 m-deep aeration tanks. (Plate diffusers are used at the Seibu and Totsuka No. 1 plants while static mixers are installed at Hokubu No. 2.)

1985 to 1986: To reduce energy consumption, perforated-pipe diffusers were changed to plastic, ceramic, and flexible-sheath tube diffusers at the Totsuka No. 2, Midori and Kanazawa plants.

Moreover, as more energy-efficient diffusers have become commercially available, diffusers for whole-floor aeration were introduced, each time existing facilities were added or remodeled. (E.g., additional facilities have been set up at the Chubu and Hokubu No. 2 plants and diffusers at the Nambu plant have been replaced.)

An outline of diffusers installed at sewage treatment plants in Yokohama is given in Table 2-1.

Table 2-1 Various Types of Diffusers Installed at Sewage Treatment Plants in Yokohama

Plant	Start of operation	Treatment capacity (m^3/day)		Diffusers	
		Planned capacity	Present capacity		
1. Chubu	April 1962	96,300	96,300	Spiral-flow aeration Whole-floor aeration	Saran-wrapped diffuser Dome diffuser
2. Nambu	July 1965	225,000	225,000	Spiral-flow aeration Whole-floor aeration	Plastic-tube diffuser Ceramic-plate diffuser
3. Hokubu No. 1	July 1968	196,000	196,000	Spiral-flow aeration	Plastic-tube diffuser
4. Totsuka No. 2	October 1972	206,000	131,000	Spiral-flow aeration Spiral-flow aeration Spiral-flow aeration Spiral-flow aeration	Plastic-tube diffuser Coarse-bubble perforated-pipe diffuser* Ceramic-plate diffuser* Plastic-plate diffuser*
5. Kohoku	December 1972	439,000	172,500	Spiral-flow aeration Spiral-flow aeration	Coarse-bubble perforated-pipe diffuser Plastic-plate diffuser*
6. Midori	May 1977	433,000	126,000	Spiral-flow aeration Spiral-flow aeration	Plastic-tube diffuser Ceramic-plate diffuser
7. Kanagawa	March 1978	543,200	380,240	Whole-floor aeration Whole-floor aeration Whole-floor aeration Spiral-flow aeration Spiral-flow aeration Spiral-flow aeration Whole-floor aeration	Ceramic-plate diffuser Dome diffuser Static mixer Leaf-spring diffuser Plastic-tube diffuser Coarse-bubble perforated-pipe diffuser Dome diffuser
8. Kanazawa	October 1979	345,000	227,200	Spiral-flow aeration Spiral-flow aeration Whole-floor aeration	Flexible-sheath tube diffuser Plastic-tube diffuser Ceramic-plate diffuser
9. Seibu	March 1983	191,000	31,800	Spiral-flow aeration Spiral-flow aeration Spiral-flow aeration	Coarse-bubble perforated-pipe diffuser* Ceramic-plate diffuser* Plastic-plate diffuser*
10. Hokubu No. 2	August 1984	86,400	86,400	Spiral-flow aeration Spiral-flow aeration Whole-floor aeration	Static mixer Plastic-tube diffuser Plastic-disk diffuser
11. Totsuka No. 1	December 1984	124,000	62,000	Spiral-flow aeration	Plastic-plate diffuser*

Note: Tank depth is approximately 5 m except for those marked with an asterisk which are approximately 10 m.

3. Characteristics of Oxygen Transfer

A diffuser's performance is evaluated generally in terms of its oxygen transfer capacity in the following ways:

(a) Oxygen transfer rate (OTR) determined by the rate that oxygen in the bubbles generated by diffusers dissolves into water.
(b) OTR per energy unit consumed.
(c) OTR per initial and operations cost.

The characteristics of oxygen transfer are summarized in the following sections.

3-1. Oxygen Transfer Rate (OTR)

(1) OTR in clean water

OTR is determined in terms of the amount of oxygen dissolving into water from bubbles per unit of time. In calculating this rate, the following equation from the two-film theory proposed by Lewis and Whitman[1] in 1923 to 24 is generally applied.

$$N_A = V \times dc/dt \times 10^{-3} = K_La \cdot V(C_s - C) \times 10^{-3} \quad (1)$$

Where:

N_A: OTR from bubbles into water (kgO_2/hr)
V: Tank volume (m^3)
dc/dt: Rate of dissolved oxygen (DO) change ($mgO_2/\ell \cdot hr$)
K_La: Overall capacity coefficient of oxygen transfer (1/hr)
C_s: DO saturation concentration (mg/ℓ)
C: DO concentration in the tank (mg/ℓ)

It should be noted that K_L of K_La is the overall oxygen transfer coefficient expressed in m/hr, and a is the gas/liquid interfacial area per tank volume (m^2/m^3 = 1/m). Moreover, the ratio of the amount of oxygen dissolved into water from diffuser bubbles to that in the air supplied to the tank when $C=0$ in Equation (1) is called oxygen transfer efficiency (OTE). This is given by the following equation and indicates the performance of each diffuser:

$$\eta = \frac{K_La \cdot V \cdot Cs \times 10^{-3}}{Gs \cdot m} \times 100 \quad (2)$$

Where:

η: Oxygen transfer efficiency (OTE) (%)
G_s: Air flow rate (Nm^3/hr)
m: Density of oxygen in air flow (kgO_2/Nm^3)

(2) OTR in mixed liquor

In clean water, the concentration of dissolved oxygen (DO) changes only when oxygen is released from bubbles into water. In mixed liquor, however, oxygen is consumed by activated sludge. Thus the OTR is given by the following equation:

$$N_A = V\cdot(dc/dt + Rr) \times 10^{-3} = K_La\cdot V(C_s - C) \times 10^{-3} \quad (3)$$

Where:

Rr: Respiration rate of mixed liquor ($mgO_2/\ell\cdot hr$)

In mixed liquor, K_La is determined assuming that DO is constant ($dc/dt = 0$).

(3) OTR in process water

If tap water or other clean water can be used in the evaluation of diffuser performance, we can determine K_La, C_s and OTE of each diffuser and compare them under identical conditions. This, however, is not practical because the use of tap water entails considerable cost, especially when an experiment is conducted on a large scale or when an actual aeration tank is used. In such a case, therefore, overall capacity coefficient of oxygen transfer K_La' and DO saturation concentration C_s' of test water are compared with those values of clean water using the following equation:

$$K_La' = \alpha\cdot K_La \quad (4)$$

$$C_s' = \beta\cdot C_s \quad (5)$$

Where:

α: The ratio of K_La of test water to that of clean water
β: The ratio of C_s in test water to that in clean water

(4) Temperature compensation for K_La

K_La varies depending on water temperature. We have no means to adjust water temperature in a field test using a large-size tank or an actual sewage treatment plant. Thus, in our investigations, we adopted the following generally-used equation in order to compensate for the temperature differentials of K_La.

$$K_La(20) = K_La(T) \times 1.024^{(20-T)} \quad (6)$$

Where:

$K_La(T)$: K_La when water temperature is T°C (1/hr)
$K_La(20)$: K_La when water temperature is 20°C (1/hr)

(5) Pressure compensation for C_s

C_s changes by static pressure as well as by water temperature. Diffusers are placed generally in the bottom area of tanks. However C_s is not the same since static pressure differs between bottom areas and the areas directly below water surface. Thus Oldshue[2] proposed the following formula in 1956 to determine C_s.

$$C_s = \frac{1}{2} C_s(w) \cdot \left(\frac{P_b}{760} + \frac{O_t}{21} \right) \quad (7)$$

Where:

$C_{s(w)}$: C_s at atmospheric density of oxygen (21%)
P_b: Static pressure at water depth at which diffuser is installed (mmHg)
O_t: Oxygen density in bubbles in areas directly below water surface (%)

OTE must be a known value in order to determine O_t. Since its value is generally not known, however, we calculated C_s based on an assumed value of O_t. When determining C_s using Oldshue's formula, care needs to be taken to assume a value which is most close to real O_t. According to reports by the Sewage Works Bureau of the Tokyo Metropolitan Government[3] and Kubota et al.,[4] the diffusion of oxygen takes place in a very short time in water and it is diffused uniformly throughout the tank.

In our experiments using clean water, we aerated water in the tank until DO became uniform and we adopted this concentration as C_s. On the other hand, in experiments on activated sludge in actual treatment plants, we determined C_s using Equation (7) since we could not obtain C_s in the mixed liquor.

(6) Water depth compensation for K_La

Diffusers are placed at slightly different water depths due to dissimilar placement design. Since our studies are aimed at evaluating the performance of various types of diffusers under the same conditions, we compensated for water depth differentials using Equation (8) proposed by Eckenfelder.[5]

$$K_La' = K_La(H_1/H_2)^n \quad (8)$$

Where:

K_La': K_La after water depth compensation
H_1: Standard water depth (5.8 m in our test)
H_2: Water depth from water surface to diffusers
n: 0.7

On the basis of the above factors, we examined OTR in order to compare the performance of 16 types of diffusers under the following conditions:

1. With regard to α and β, we investigated the effects of salinity and surface active agents. If they were significant, they were compensated for.

2. $K_La(20)$ (equivalent to 20°C) was used for K_La.

3. K_La' compensated for K_La in terms of standard water depth.

4. DO after aerating water in the tank for a long time was taken as C_s. In experiments at actual plants, we calculated the value of C_s using Oldshue's formula.

3-2. Measurements of OTR

(1) Unsteady-state method (in the case of using clean water or process water as test water)

a. Measurement method

DO was forced to be 0 mg/ℓ in our experiments by dosing with Na_2SO_3 for deoxygenation and with cobalt chloride as a catalyst. Test water was aerated and K_La was calculated on the basis of the rise in DO. This is called the unsteady-state method.

b. Calculation of K_La

A number of methods for calculating K_La have been proposed for the unsteady-state method, but the following formulas are generally considered to be most appropriate:

(i) Log-dificit method (LD method)

$$\ell_n(C_s - C) = -K_La \cdot t + \ell n C_s \qquad (9)$$

(ii) Moment method

This was developed by Gilbert and Libby[6] and introduced by Hashimoto et al.[7] to Japan as the moment method. K_La is determined by trial and error using the following equation as well as the first-order and second-order moment of the equation.

$$C = C_s \cdot (1 - e^{-K_La(ti+to)}) \qquad (10)$$

(iii) Non-linear regression method (NLR method)

C_s, K_La and t_0 in Equation (10) are assumed to be variables and the approximate value for each variable of Equation (10) is calculated by the method of least squares.

Both the moment and NLR methods offer the advantage that we can calculate K_La without first knowing the value of C_s. Therefore, we decided to employ these methods for calculating K_La. When we compared the residual sums of the squares of calculated and measured DO values using the three methods above, the sum of the NLR method proved to be the smallest and we therefore decided to adopt the values obtained using this method.

(2) Steady-state method (in the case of using mixed liquor as test water)

R_r and DO of the mixed liquor are measured. At the same time, after the supernatant of the mixed liquor is aerated for a long time, DO saturation concentration is measured in order to determine $C_{s(w)}$, and C_s is calculated using Equation (7). Thereafter, K_La is computed from Equation (3). O_t in Equation (7) is assumed for each diffuser. This is called the steady-state method.

(3) Measurement of oxygen density in exhaust gas to determine K_La

OTR can also be obtained by measuring oxygen density in exhaust gas from the test tank. In our experiments, we covered the top of the tank and measured the oxygen density of exhaust gas using a zirconia oxygen sensor. OTE and K_La(T) were calculated using the formulas given below by measuring oxygen density in the exhaust gas.

$$\eta = \frac{X_0 - X_1}{X_0(100 - X_1)/100} \times 100 \tag{11}$$

$$K_La(T) = \frac{\eta \cdot m \cdot G_s}{C_s(T) \cdot V} \times 10 \tag{12}$$

Where:

X_0: Oxygen density of air (%)
X_1: Oxygen density of exhaust gas (%)

3-3. Aeration Efficiency

In terms of OTR, diffusers which generate finer bubbles offer greater advantage. However, when compared with diffusers which emit coarse bubbles, pressure loss is greater with fine-bubble diffusers and, consequently they are more

disadvantageous in terms of power efficiency. Thus, using the formulas shown below, we analyzed the aeration efficiency of these two types of diffusers.

$$A.E. = \frac{N_A}{P} \tag{13}$$

$$P = \frac{K}{K-1} \cdot \frac{P_1 \cdot G_s}{6120 \cdot \eta_b} \left\{ \left(\frac{P_2}{P_1} \right)^{\frac{K-1}{K}} - 1 \right\} \tag{14}$$

Where:

A.E.: Aeration efficiency (O_2kg/kWh)
P: Theoretical power consumption (kW)
P_1: Absolute pressure at air inlet (mmAq)
P_2: Absolute pressure at air outlet (mmAq)
K: Specific heat ratio
η_b: Adiabatic efficiency

We adopted a theoretical value of power consumption in Equation (14) in anticipation of the need to compare performance using the same blower for various types of diffusers.

4. Evaluation of Diffuser Performance in Test Tanks

4-1. Outline of Experiments

(1) Experimental subject matter and test facilities

We used the three facilities outlined below in order to study the experimental subject matter.

Table 4-1 Experimental Subject Matter and Test Facilities

Experiment	I	II	III
Subject matter	1. Bubble distribution 2. Air flow rate and pressure loss 3. The effect of inorganic salt and surface active agents	1. The relationship between pore and bubble sizes 2. Gas hold-up 3. Power consumption efficiency	1. Air flow rate and OTE 2. Placement density and OTE 3. Placement distribution and OTE
Diffusers	F-1 through F-9, M-1 through M-3, C-1 and C-2	Pore size (μm) ∅100, 200, 300, 400, 600, 800, 1000, 2000, 3000, 4000	F-1 through F-10, M-1 through M-3, C-1 through C-3
Test tank	1.8 m (W) × 0.95 m (L) × 1.8 m (H)	∅0.2 m × 2.0 m (H)	Two tanks of 5 m (W) × 5 m (L) × 5.8 m (H)
Material of test facility	Vinyl chloride (transparent)	Acrylic (transparent)	Reinforced concrete
Major equipment of test facility	- Root type blower (20 m^3/hr) - DO meter, 1 unit - Bubble-size measurement system - Water manometer - Mercury manometer	- Air pump (60 ℓ/min) - DO meter, 1 unit - Bubble-size measurement system - Mercury manometer - Plate diffuser, 10 types	- Air flow control system - Root type blowers, 2 units (4N m^3/min) - DO meters, 10 units - Oxygen sensors, 2 units - Thermometers, 2 units
Test water	Tap water	Tap water	Recycled water*
Air flow rate	6 to 20 m^3/hr	4 to 20 ℓ/min	0.5 to 1.5 V.V.H.**

Note: * Recyled water is obtained by rapid filtering the effluent of the Chubu sewage treatment plant.

** Volumetric air flow rate per tank unit volume.

(2) Measurement of bubble size

Bubble size is generally determined by photographic measurement. In our experiments, however, we used an electrode sensor illustrated below to measure bubble size.

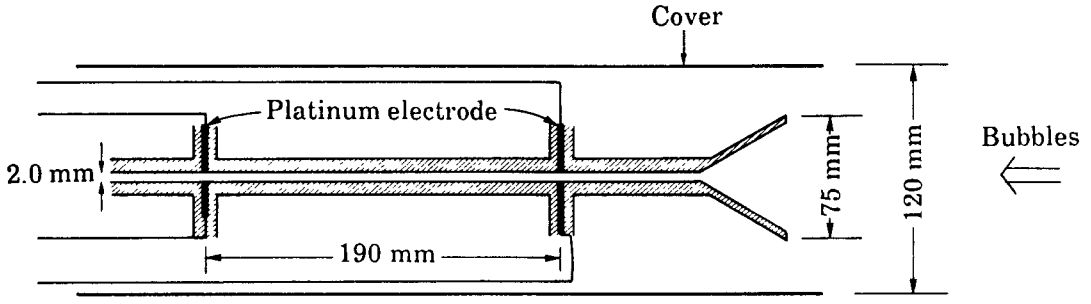


Fig. 4-1 Bubble Measurement Device

By applying a direct current to the platinum electrodes, the sensor detects differences in electrical resistance between the time when the sensor is full of water and when bubbles cross its path.

(3) Dry pressure and dynamic wet pressure

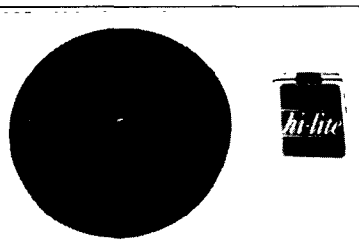
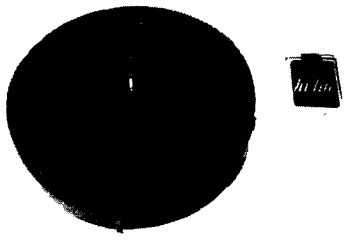
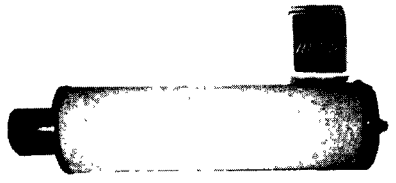
Dry pressure and dynamic wet pressure for each type of diffuser is measured in Experiment I as shown in the above Table. Dry pressure is defined as operating head loss across diffuser media in the atmosphere, and dynamic wet pressure as operating head loss for diffusers submerged in water.

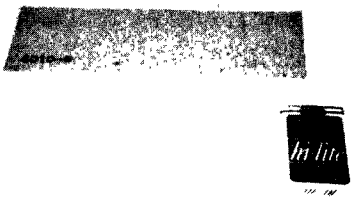
A water manometer was used to measure dry pressure while dynamic wet pressure was determined by a mercury manometer.

(4) Outline of each type of diffuser

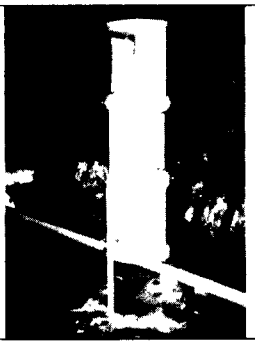
The outline of each type of diffuser used in our experiments is shown in Table 4-2. Diffusers were classified in the following manner: Fine-bubble diffusers, F-1 through F-9; coarse-bubble diffusers, C-1 to C-3; intermediate-bubble diffusers, M-1 to M-3; and submerged diffusing agitator, F-10.

Table 4-2 Outline of Each Type of Diffuser

Type	F-1	F-2
Diffuser	Dome diffuser	Flexible-sheath tube diffuser
		
Major Specifications		
a. Material	Ceramic	Vinyl chloride tube diffuser Frame: polyacetal
b. Dimensions	Ø178 mm	Ø60 mm × 600 mm
c. Normal air flow rate (ℓ/min per diffuser)	30 to 40	100 to 150
d. Nominal pore size	150 μm	1 mm (slit width)
Type	F-3	F-4
Diffuser	Disk diffuser	Tube diffuser
		
Major Specifications		
a. Material	Ceramic	Ceramic
b. Dimensions	Ø240 mm	Ø75 mm × 250 mm
c. Normal air flow rate (ℓ/min per diffuser)	20 to 100	25 to 40
d. Nominal pore size	180 μm	200 μm
Type	F-5	F-6
Diffuser	Disk diffuser	Disk diffuser
Major Specifications		
a. Material	Polyethylene resin	Methacrylic resin
b. Dimensions	Ø213 mm	Ø250 mm
c. Normal air flow rate (ℓ/min per diffuser)	25 to 100	40 to 80
d. Nominal pore size	80 to 100 μm	150 μm

Type	F-7, M-2		F-8, F-9, M-3		
Diffuser	Tube diffuser		Plate diffuser		
					
Major Specifications	F-7	M-2	F-8	F-9	M-3
a. Material	Methacrylic resin	ABS resin	Ceramic	←	←
b. Dimensions	∅75 mm × 500 mm	∅75 mm × 500 mm	300 mm × 100 mm	←	←
c. Normal air flow rate (ℓ/min per diffuser)	50 to 110	100 to 150	20 to 40	←	←
d. Nominal pore size	120 μm	300 μm	260 μm	130 μm	400 μm

Type	F-10	C-2
Diffuser	Submerged diffusing agitator	Leaf-spring diffuser
Major Specifications		
a. Material	Cast iron	Main body: ABS resin Vibrator plate: SUS 301
b. Dimensions	∅900 mm × 1,130mm	178 mm × 51 mm, 175 mm × 30 mm
c. Normal air flow rate (ℓ/min per diffuser)	1,300 to 2,000	110 to 340
d. Nominal pore size	–	–

Type	C-3	M-1, C-1	
Diffuser	Static mixer	Tube diffuser	Perforated-pipe diffuser
			
Major Specifications			
a. Material	Polypropylene	Saran-wrapped	SUS304
b. Dimensions	∅370 mm × 1,600mm	∅80 mm × 610mm	∅35 mm × 650 mm
c. Normal air flow rate (ℓ/min per diffuser)	300 to 1,000	230	300
d. Nominal pore size	–	–	4 mm

4-2. Results of Experiments I and II

(1) Characteristics of each type of diffuser

a. Air flow rate and pressure loss

A total of 14 types of diffusers were installed on the bottom of a 1.8 m-deep tank and air flow rate was changed at 6 levels in order to measure the dry pressure and dynamic wet pressure of each diffuser. The results are given in Table 4-3. From this table, we can see that coarse-bubble diffusers have lower dry pressure and dynamic wet pressure while such pressures for fine-bubble diffusers are higher. This indicates the fact that smaller pore size causes greater pressure loss. Moreover, when air flow rate was changed, fine-bubble diffusers showed a steeper rise in pressure loss as compared with that for coarse-bubble diffusers.

Table 4-3 Dry Pressure and Dynamic Wet Pressure

(mmAq)

Type	Dry pressure						Dynamic wet pressure					
	6 m³/hr	9 m³/hr	12 m³/hr	15 m³/hr	18 m³/hr	20 m³/hr	6 m³/hr	9 m³/hr	12 m³/hr	15 m³/hr	18 m³/hr	20 m³/hr
F-1	147	278	486	749	1125	1398	164	208	253	334	403	437
F-2	155	191	224	257	280	299	261	277	306	331	353	372
F-3	33	40	56	96	145	202	156	162	187	220	280	322
F-4	22	15	32	34	57	87	91	93	112	125	203	178
F-5	56	101	141	185	236	262	100	139	181	233	286	319
F-6	22	47	85	133	192	241	161	187	226	275	334	369
F-7	4	6	8	10	13	15	166	180	193	216	222	239
F-8	25	53	98	154	224	277	97	128	176	235	305	356
F-9	30	61	107	164	237	292	156	187	235	290	358	400
M-1	6	4	5	10	20	25	10	12	23	29	34	40
M-2	1	2	2	4	5	6	5	7	10	10	12	21
M-3	24	51	93	146	213	266	49	75	121	171	239	272
C-1	-3	6	-3	5	2	28	-43	-43	-43	-43	-41	-39
C-2	260	322	344	374	427	456	241	278	311	339	372	381

b. Dry and dynamic wet air flow rates

Using the measurement results of Table 4-3, we determined dry and dynamic wet air flow rates under identical pressure to compare each type of diffuser. The results are summarized in Table 4-4. This data shows that the dynamic wet pressure of fine-bubble diffusers is lower than that for other types of diffusers. It can be seen that the dynamic wet air flow rate for fine-bubble diffusers measured less than 50 mℓ/cm² per min while that for intermediate- and coarse-bubble diffusers was over 80 mℓ/cm² per min.

Table 4-4 Air Flow Rate per Effective Surface Area of Diffuser under Header Pressure of 50 mmAq

Type	Dry air flow rate (ml / cm^2 · min)	Dynamic wet air flow rate (ml / cm^2 · min)
F-1	78	35
F-2	9	0
F-3	115	14
F-4	237	42
F-5	60	34
F-6	122	17
F-7	711	1
F-8	106	44
F-9	99	20
M-1	358	198
M-2	1,066	725
M-3	109	80
C-1	-	-
C-2	15	11

c. Air flow rate and bubble size

We measured the size of bubbles generated from each type of diffuser in the following manner. First, sensors were attached to a glass tube (inner diameter: 2 mm) and a known volume of bubbles were fed into the tube. Thereafter, we drew a calibration curve of bubble volume and the time taken for bubbles to pass before the sensor based on sensor voltage changes and used it as reference. The data on the bubble size of 14 types of diffusers is given in Table 4-5.

The following relationship can be found between air flow rate and bubble size. In the case of fine-bubble diffusers, the average diameter and average volume of bubbles increase in parallel to the increase in air flow rate; these changes are not significant for coarse- and intermediate-bubble diffusers.

Moreover, we calculated the total surface area of bubbles per air flow rate of 1 ℓ to compare the characteristics of diffusers. As a consequence, it was confirmed that in the case of coarse-bubble diffusers, the total surface area of bubbles is smaller than that for fine-bubble diffusers. For instance, at a level of 9 Nm^3/hr it is 1.2 m^2 or less in the case of coarse- and intermediate-bubble diffusers while fine-bubble diffusers show greater total surface area of approximately 1.3 m^2.

Table 4-5 Relationship between Air Flow Rate and Bubble Dimensions

Type	Air flow rate (m^3/h)	Average diameter (mm)	75% diameter (mm)	Average volume ($\mu\ell$)	Total surface area (m^2/ℓ air)	Type	Air flow rate (m^3/h)	Average diameter (mm)	75% diameter (mm)	Average volume ($\mu\ell$)	Total surface area (m^2/ℓ air)
	6	4.19	4.64	42.9	1.33		6	3.83	4.18	31.6	1.49
	9	4.21	4.66	42.6	1.35		9	4.35	4.98	47.4	1.30
F-1	12	4.12	4.64	41.4	1.34	F-8	12	4.38	4.89	48.1	1.29
	15	4.49	4.98	51.4	1.27		15	4.42	4.83	48.9	1.29
	18	4.41	4.82	49.9	1.27		18	4.40	4.89	48.1	1.30
	20	4.54	5.05	54.3	1.23		20	4.42	4.87	48.5	1.30
	6	3.74	4.14	30.2	1.50		6	4.46	4.87	50.3	1.28
	9	3.87	4.34	35.5	1.40		9	4.36	4.93	48.1	1.29
F-2	12	4.00	4.47	37.1	1.40	F-9	12	4.83	5.53	66.1	1.15
	15	4.50	4.94	53.6	1.24		15	4.99	5.67	70.9	1.14
	18	4.41	4.94	48.8	1.29		18	4.52	5.07	53.1	1.25
	19.9	4.41	4.80	48.8	1.28		19.8	4.45	5.18	52.9	1.23
	6	4.30	4.87	46.4	1.30		6	4.88	5.56	71.2	1.10
	9	4.01	4.37	36.3	1.43		9	4.60	5.34	58.6	1.19
F-3	12	4.15	4.50	39.9	1.39	M-1	12	4.66	5.31	61.6	1.17
	15	4.92	5.61	69.0	1.14		15	4.37	4.98	51.6	1.23
	18	5.00	5.55	72.3	1.12		18	5.54	6.39	119.2	0.89
	20	5.06	5.60	72.9	1.13		20	4.40	5.15	53.2	1.21
	6	4.37	4.93	47.3	1.30		6	4.65	5.32	59.5	1.19
	9	4.45	4.86	49.6	1.28		9	5.00	5.79	74.9	1.10
F-4	12	4.44	4.93	52.6	1.23	M-2	12	4.48	4.81	50.4	1.28
	15	4.61	4.90	56.0	1.23		15	4.66	5.30	58.1	1.21
	18	4.56	5.08	54.4	1.24		18	4.84	5.70	67.0	1.15
	20	4.87	5.30	64.9	1.18		19.8	4.89	5.58	68.6	1.14
	6	4.30	4.82	47.2	1.28		6	4.27	4.89	45.3	1.31
	9	4.36	4.84	47.9	1.29		9	4.70	5.43	61.4	1.18
F-5	12	4.50	4.97	52.3	1.26	M-3	12	4.69	5.43	62.6	1.16
	15	4.59	5.01	54.1	1.25		15	4.79	5.44	64.1	1.17
	18	4.77	5.18	60.8	1.20		18	4.74	5.45	63.4	1.16
	20	4.50	4.95	51.2	1.27		19.5	4.57	5.10	53.8	1.25
	6	4.23	4.79	46.7	1.27		6	4.54	5.72	61.6	1.14
	9	3.76	4.37	31.2	1.48		9	4.32	5.77	59.7	1.11
F-6	12	4.26	4.56	44.4	1.33	C-1	12	4.20	5.56	54.8	1.15
	15	4.10	4.52	39.8	1.37		15	4.64	6.19	72.1	1.06
	18	4.35	5.05	48.8	1.27		18	4.69	5.63	68.1	1.10
	19.8	4.39	4.91	48.7	1.28		20	5.44	6.35	118.9	0.88
	6	3.17	3.60	19.3	1.72		6	4.55	5.52	60.4	1.16
	9	3.90	4.35	34.8	1.43		9	4.87	5.53	73.0	1.09
F-7	12	4.16	4.62	41.6	1.35	C-2	12	4.49	5.36	56.8	1.19
	15	4.14	4.62	42.2	1.33		15	4.54	5.28	57.1	1.20
	18	4.29	4.80	47.0	1.28		18	4.59	5.45	62.5	1.14
	19.8	4.35	4.89	47.5	1.29		20	4.48	5.41	59.5	1.15

Note: 75% diameter shows cumulative excess probability over 75% of bubble size distribution.

d. Pore size and bubble size (Experiment II)

Each manufacturer uses different measurement standards for indicating the nominal pore size of diffusers, making it impractical to adopt such measurements as a basis of comparison. We made, therefore, 10 disks with a variety of pore sizes which were measured beforehand. We used these disks to understand the relationship between pore size and bubble size by placing them in a 2 m-deep tank.

The data on bubble sizes measured for each pore size at an air flow rate of 12 ℓ/min is presented in Table 4-6. These figures indicate that while the average diameter and other dimensions of bubbles increase in tandem with the increase in pore size, the total surface area of bubbles per air flow rate of 1 ℓ decreases with the increase in pore size.

The distribution of bubble sizes for each pore size is shown in Fig. 4-2. From these graphs, we can see that bubble-size distribution patterns differ when pore size is 800 μm or smaller and when 1,000 μm or greater. While distribution is normal in pore sizes of 800 μm and under, it shows a uniform pattern when 1,000 μm and over. Thus it can be said that a pore size of 800 to 1,000 μm is the border between coarse bubbles and other bubbles. Moreover, coarse-bubble diffusers are characterized not only by their greater average diameters but also by the fact that the bubbles they produce burst into fine bubbles while ascending in water and bubbles in various sizes are generated in the process.

Table 4-6 Relationship between Pore Size and Bubble Dimensions

Air flow rate	Pore size (μm)	Average diameter (mm)	75% diameter (mm)	Average volume (μℓ)	Total surface area ($m^2/ℓ$·air)
12ℓ/min	100	3.36	3.69	21.5	1.69
	200	3.41	3.85	22.6	1.66
	300	3.98	4.42	35.5	1.44
	400	4.03	4.50	37.6	1.40
	600	3.65	4.02	27.6	1.56
	800	4.35	5.01	47.6	1.29
	1000	4.86	6.04	73.8	1.08
	2000	4.84	5.79	70.1	1.11
	3000	4.72	5.72	69.7	1.09
	4000	4.64	5.55	63.7	1.14

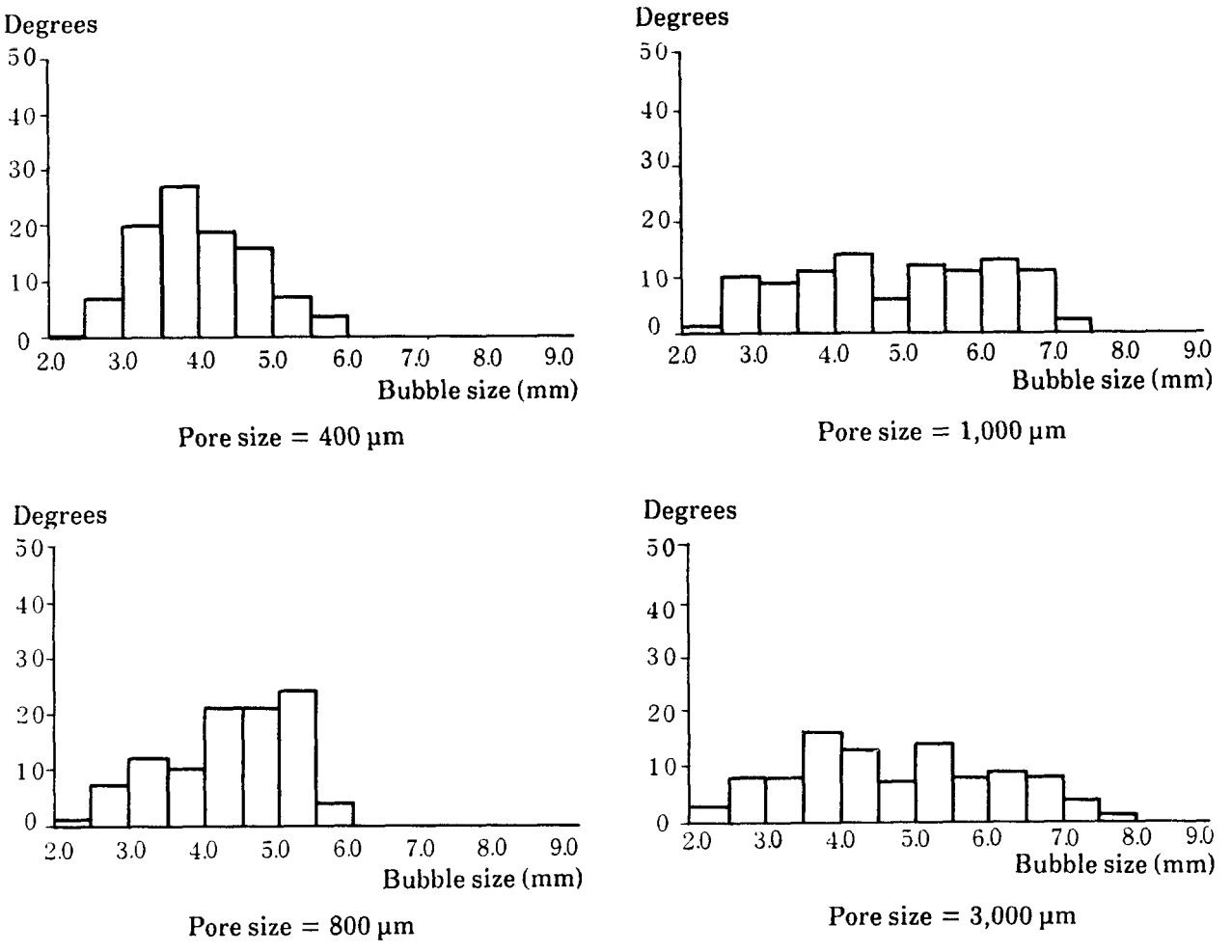


Fig. 4-2 Distribution of Bubble Size in Relation to Pore Size

(2) Effects of salinity

In order to study the effects of inorganic salt on K_La, we carried out an experiment in two parts, adding Na_2SO_3 to the effluent in one and Na_2SO_3 and NaCl in the other. The results for the 14 types of diffusers are presented in Table 4-7 and some of these are shown graphically in Fig. 4-3. In these experiments, salinity was determined in terms of electric conductivity and its regression to K_La was examined.

This Figure indicates that K_La rises in parallel with the increase in electric conductivity. The increase rates, however, are lower for coarse-bubble diffusers while fine-bubble diffusers demonstrate higher increase rates of K_La. Therefore, the effects of salinity caused by sea water and industrial wastewater on fine-bubble diffusers are considered to be much greater. Furthermore, these results suggest that bubble size and salinity could be one of the major causes for changes of the α factor in process water reported in some of the previously published papers.

Table 4-7 Regression by Salinity

Type	Na_2SO_3		Na_2SO_3+NaCl	
	B	Coefficient of determination	B	Coefficient of determination
F-1	0.095	0.825	0.112	0.958
F-2	0.152	0.898	0.121	0.995
F-3	0.163	0.970	0.102	0.982
F-4	0.130	0.936	0.095	0.991
F-5	0.113	0.929	0.083	0.963
F-6	0.148	0.869	0.112	0.987
F-7	0.106	0.957	0.108	0.992
F-8	0.107	0.952	0.067	0.969
F-9	0.207	0.897	0.089	0.977
M-1	0.109	0.762	0.035	0.828
M-2	0.071	0.722	0.036	0.907
M-3	0.073	0.946	0.020	0.833
C-1	0.071	0.887	0.027	0.802
C-2	0.068	0.513	0.040	0.931

Note: Exponential regression of K_La on electrical conductivity.
$K_La = A \cdot \exp(B \times \text{electric conductivity})$

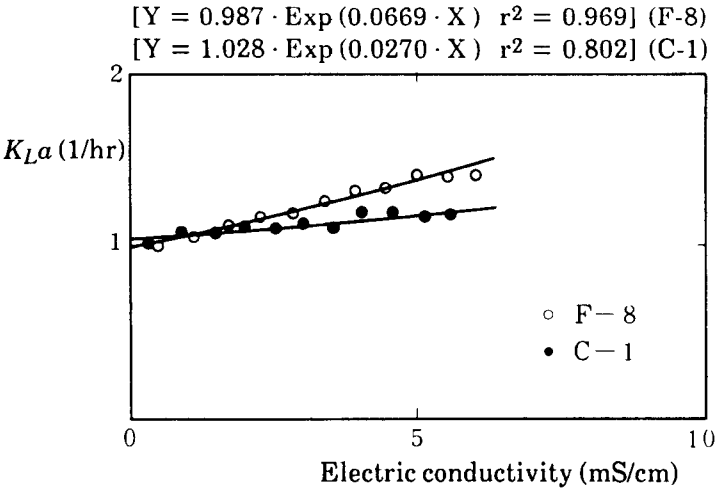


Fig. 4-3 Regression in F-8 and C-1

(3) Effects of surface active agents

Anionic and nonionic surface active agents contained in detergents are other substances considered to have effects on K_La in addition to inorganic salt. Thus we performed further experiments using these agents. Table 4-8 shows the results of these tests and samples of them are illustrated in Fig. 4-4.

K_La declines with the increase in the concentration of both anionic and nonionic agents, but it shows a steeper fall in relation to the increase in nonionic agents. Thus surface active agents present an opposite case as that for salinity since they act to decrease the α factor, and they are also considered to be one of the major causes for changes of the α factor in process water. Especially, since adequate research has not yet been made on the treatment characteristics of nonionic surface active agents, future investigations on these substances are needed as part of efforts to improve the quality of secondary effluent.

Table 4-8 Regression of Anion and Nonion Surface Active Agents

Type	Anion		Nonion	
	B	Coefficient of determination	B	Coefficient of determination
F-1	-0.024	0.383	-0.262	0.870
F-2	-0.057	0.598	-0.400	0.942
F-3	-0.040	0.564	-0.643	0.969
F-4	-0.109	0.989	-0.333	0.985
F-5	-0.102	0.943	-0.415	0.991
F-6	-0.005	0.058	-0.342	0.996
F-7	-0.029	0.599	-0.210	0.965
F-8	-0.078	0.833	-0.349	0.985
F-9	-0.066	0.784	-0.393	0.995
M-1	-0.140	0.995	-0.319	0.934
M-2	-0.123	0.992	-0.329	0.984
M-3	-0.122	0.992	-0.280	0.979
C-1	-0.079	0.989	-0.180	0.861
C-2	-0.038	0.522	-0.202	0.903

Note: Exponential regression of K_La on anion and nonion surface active agents.
$K_La = A \cdot \exp(B \times$ concentration of surface active agents)

[Y = 1.075 · Exp(−0.0776 · X) r^2 = 0.833] (Anion)
[Y = 1.089 · Exp(−0.3494 · X) r^2 = 0.985] (Nonion)

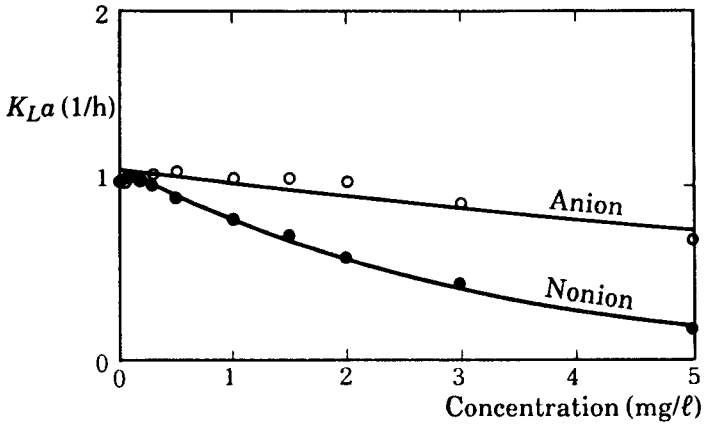


Fig. 4-4 Effects of Surface Active Agents (F-8)

(4) α and β factors

Because the above experiments were performed using clean water, K_La values measured in these experiments do not agree with the values measured at sewage treatment plants. We measured, therefore, K_La and OTE of process water and clean water, and determined the compensation coefficients, i.e., the α and β factors. Consequently, it was confirmed that α was within the range of 0.98 to 1.34 (0.86 to 1.18 when adjusted for salinity) and β 0.89 to 1.04.

As a result, salinity was found to have major effects on K_La and we compensated for salinity based on the data given in Table 4-7.

4-3. Results of Experiment III

(1) Relationship between air flow rate and OTE in various types of diffusers

On the basis of the correlation between salinity (electrical conductivity) and K_La determined in the experiments using a 1.8 m-deep tank, we corrected the results obtained in experiments using 5.8 m-deep tanks and compared K_La and OTE of each diffuser. These corrections were made by setting the reference value of electrical conductivity at 2 mS/cm.

Based on the test results, the relationship between K_La and air flow rate per diffuser unit is expressed by the following equation:

$$K_La/N = A \times (G_s/N)^B \qquad (15)$$

Where:

N: Number of diffuser units
A, B: Constants

Table 4-9 K_La and OTE of Each Type of Diffusers (Per Unit of Diffuser)

Type	Installation Layout	Gs/N Max Min	$(K_La/N)=A\times(Gs/N)^B$ A	B	Gs/N=1.0 K_La/N	η(%)	Gs/N=2.0 K_La/N	η(%)	Gs/N=5.0 K_La/N	η(%)	Gs/N=10.0 K_La/N	η(%)	Gs/N=20.0 K_La/N	η(%)	Gs/N=30.0 K_La/N	η(%)	Gs/N=40.0 K_La/N	η(%)
F-1	WFA	6 1	0.0586	0.857	0.059	30.3	0.106	27.4	0.233	24.1								
F-2	WFA	10 1	0.0553	0.979	0.055	28.3	0.109	27.9	0.267	27.3								
	SFA	15 2	0.0456	0.872	0.046	23.4	0.083	21.4	0.185	19.1								
F-3	WFA	10 1	0.0537	0.950	0.054	27.5	0.104	26.6	0.248	25.4								
F-4	WFA	4 1	0.0512	0.981	0.051	26.9	0.101	26.5	0.248	26.1								
F-5	WFA	6 1	0.0613	0.817	0.061	32.4	0.108	28.5	0.228	24.1								
F-6	WFA	10 1	0.0648	0.920	0.065	33.0	0.123	31.2	0.285	29.0								
F-7	WFA	30 3	0.0710	0.764					0.243	25.0	0.412	21.2	0.700	18.0	0.955	16.4		
	SFA	30 4	0.0400	0.876					0.164	16.9	0.301	15.5	0.552	14.2	0.787	13.5		
F-8	WFA	5 1	0.0518	1.067	0.052	28.0	0.108	29.3	0.288	31.2								
F-9	WFA	5 1	0.0623	0.895	0.062	34.0	0.116	31.6	0.263	28.7								
F-10	WFA	250 70	0.1444	0.703									1.186	30.8	1.578	27.3	1.931	25.1
M-1	SFA	45 5	0.0323	0.943					0.147	15.0	0.284	14.4	0.545	13.9	0.799	13.5	1.048	13.3
M-2	SFA	30 4	0.0358	0.916					0.156	15.7	0.294	14.8	0.555	13.9	0.805	13.5		
M-3	WFA	6 1	0.0438	0.975	0.044	23.2	0.086	22.8	0.211	22.3								
C-1	SFA	55 5	0.0154	1.116					0.093	9.3	0.202	10.1	0.437	10.9	0.687	11.5	0.947	11.9
C-2	SFA	30 10	0.0181	1.040					0.097	9.2	0.199	9.4	0.409	9.7	0.623	9.9		
C-3	WFA	110 20	0.0133	1.115									0.376	9.3	0.591	9.8	0.814	10.1

Type	Installation Layout	Gs/N Max Min	$(K_La/N)=A\times(Gs/N)^B$ A	B	Gs/N=50 K_La/N	η(%)	Gs/N=60 K_La/N	η(%)	Gs/N=70 K_La/N	η(%)	Gs/N=100 K_La/N	η(%)	Gs/N=150 K_La/N	η(%)	Gs/N=200 K_La/N	η(%)	Gs/N=250 K_La/N	η(%)
F-10	WFA	250 70	0.1444	0.703	2.259	23.5	2.568	22.2	2.862	21.2	3.678	19.1	4.891	16.9	5.988	15.6	7.005	14.56

Note: WFA–Whole-floor aeration SFA–Spiral-flow aeration

Fig. 4-5 shows the relationship between air flow rate per tank unit volume (V.V.H.) and OTE using a portion of the data. Taking an air flow rate of 5 Nm^3/hr per diffuser ($G_s/N=5$) from the data of Table 4-9 and taking the information of Fig. 4-5 into account, we found OTE for whole-floor aeration to be 20 to 30 percent for fine-bubble diffusers, 14 to 17 percent for intermediate-bubble diffusers, and 9 to 12 percent for coarse-bubble diffusers.

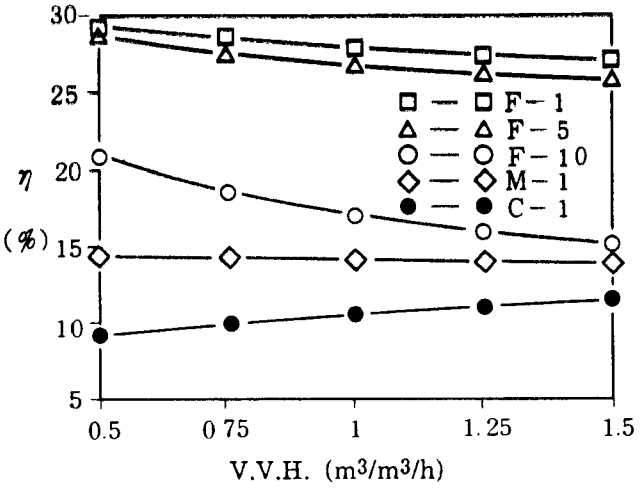


Fig. 4-5 Air Flow Rate and OTE

Moreover, OTE of fine-bubble diffusers decreased with the increase in air flow rate per unit of diffuser. On the other hand, in coarse-bubble diffusers, especially the perforated-pipe diffuser (C-1) and static mixer (C-3), higher air flow rate resulted in a gradual increase in OTE.

We found that the B value for Equation (15) was 0.703 for a submerged diffusing agitator (F-10), which is significantly lower when compared with other fine-bubble diffusers. This diffuser has a mechanism providing additional agitation to the air flow and agitation levels remain relatively unchanged even when the air flow rate is lowered. For this reason, OTE of diffuser F-10 is high in the low ranges of air flow rate.

In the case where the same type of diffusers is used, OTE also varies depending on the number of units installed. Some examples of OTE and K_La by diffuser placement density are given in Table 4-10 and Fig. 4-6.

These data indicate that if the air flow rate is the same, OTE rises in proportion to the increase in the number of diffuser units installed. Although this efficiency drops when air flow rate is increased, the rate of efficiency drop decreases more gradually when a greater number of units are in use.

Table 4-10 Examples of K_La and η in Relation to Installation Density

Type	Installation Layout N	$(K_La/N) = A \times (Gs/N)^B$		0.5 V.V.H. Gs(Nm³/m) 1.2		0.75 V.V.H. Gs(Nm³/m) 1.8		1.0 V.V.H. Gs(Nm³/m) 2.4		1.25 V.V.H. Gs(Nm³/m) 3.0		1.5 V.V.H. Gs(Nm³/m) 3.6	
		A	B	K_La20	η (%)	K_La20	η (%)	K_La20	η (%)	K_La20	η (%)	K_La20	η (%)
F-1	WFA 80	0.0574	0.943	4.19	29.8	6.14	29.1	8.05	28.6	9.94	28.3	11.80	28.0
	WFA 60	0.0571	0.925	4.08	29.3	5.94	28.4	7.74	27.8	9.52	27.4	11.27	27.0
	WFA 40	0.0516	0.915	3.56	25.3	5.15	24.4	6.71	23.8	8.22	23.4	9.72	23.0
F-7	WFA 16	0.0608	0.856	3.55	25.3	5.02	23.9	6.42	22.9	7.77	22.2	9.08	21.6
	WFA 12	0.0626	0.824	3.30	23.5	4.62	21.9	5.85	20.8	7.03	20.0	8.17	19.4
	WFA 8	0.0697	0.747	2.89	20.4	3.92	18.4	4.86	17.1	5.74	16.2	6.58	15.5
C-1	WFA 16	0.0154	1.184	1.47	10.3	2.38	11.1	3.35	11.7	4.36	12.2	5.41	12.6
	WFA 8	0.0130	1.198	1.46	10.0	2.37	10.8	3.35	11.5	4.38	12.0	5.44	12.4
	WFA 4	0.0100	1.230	1.42	9.8	2.34	10.7	3.33	11.4	4.38	12.0	5.48	12.6

Note: WFA--Whole-floor aeration

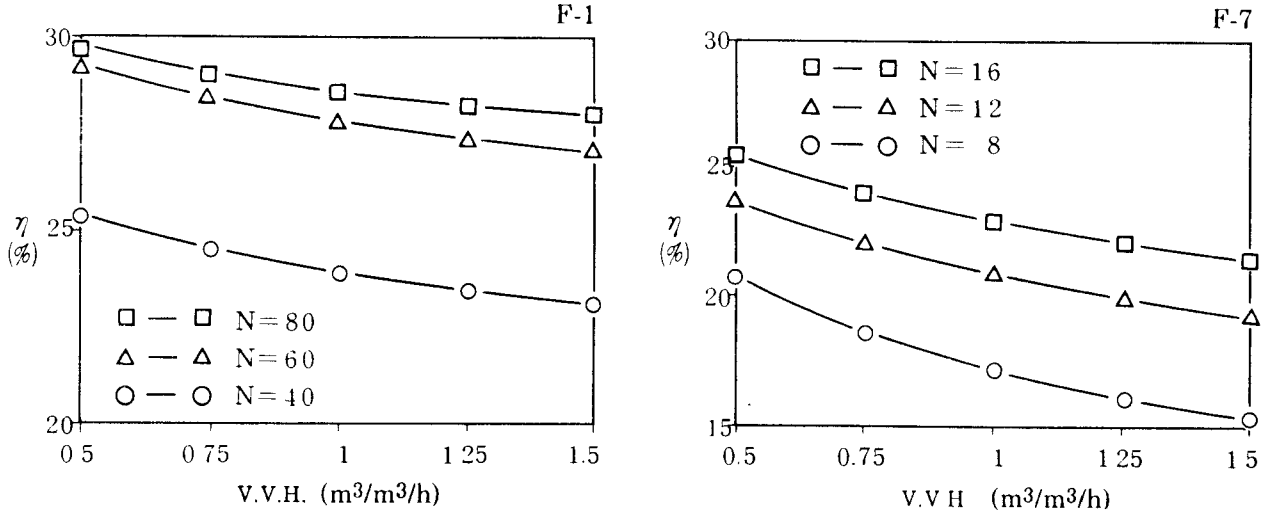


Fig. 4-6 Air Flow Rate and Oxygen Transfer Efficiency

(2) Comparison of whole-floor aeration and spiral-flow aeration

We installed diffusers for whole-floor aeration in one tank and for spiral-flow aeration in the other in order to investigate the relationship between diffuser layout and OTE. In each experiment, we used the same type of diffusers but changed the number of diffuser units each time. Some of the test results are shown in Table 4-11 and Figs. 4-7 to 4-10 comparing whole-floor aeration to spiral-flow aeration.

When comparison was made between fine-bubble diffusers and coarse-bubble diffusers at the same air flow rate per unit using the same number of diffusers, fine-bubble diffusers displayed a higher OTE ratio as compared to coarse-bubble diffusers. This demonstrates that higher efficiency can be achieved by whole-floor aeration as far as fine-bubble diffusers are concerned.

Table 4-11 Comparison of Whole-Floor Aeration and Spiral-Flow Aeration

Type	N	0.5 V.V.H. Gs(Nm^3/m) 1.2		0.75 V.V.H. Gs(Nm^3/m) 1.8		1.0 V.V.H. Gs(Nm^3/m) 2.4		1.25 V.V.H. Gs(Nm^3/m) 3.0		1.5 V.V.H. Gs(Nm^3/m) 3.6	
		η WFA / η SFA	The difference in η (%)	η WFA / η SFA	The difference in η (%)	η WFA / η SFA	The difference in η (%)	η WFA / η SFA	The difference in η (%)	η WFA / η SFA	The difference in η (%)
F-7	16	1.42	7.50	1.36	6.39	1.33	5.63	1.30	5.07	1.27	4.64
	12	1.59	8.72	1.45	8.77	1.35	5.45	1.29	4.46	1.23	3.68
	8	1.79	8.99	1.50	6.18	1.33	4.27	1.21	2.83	1.12	1.68
M-1	16	1.18	2.51	1.18	2.55	1.17	2.57	1.17	2.58	1.17	2.60
	8	1.10	1.42	1.10	1.48	1.11	1.52	1.11	1.54	1.11	1.57
	4	0.99	−0.09	1.01	0.10	1.02	0.22	1.03	0.31	1.03	0.39
C-1	16	1.11	1.02	1.12	1.19	1.13	1.32	1.13	1.43	1.14	1.52
	8	1.07	0.66	1.07	0.72	1.07	0.76	1.07	0.80	1.07	0.83
	4	0.96	−0.41	0.98	−0.22	0.99	−0.08	1.01	0.05	1.01	0.17

Note: WFA–Whole-floor aeration SFA–Spiral-flow aeration

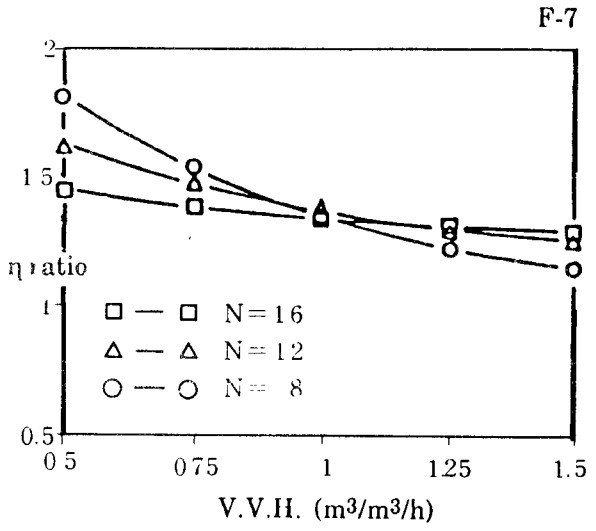


Fig. 4-7 Comparison of η ratios for WFA and SFA

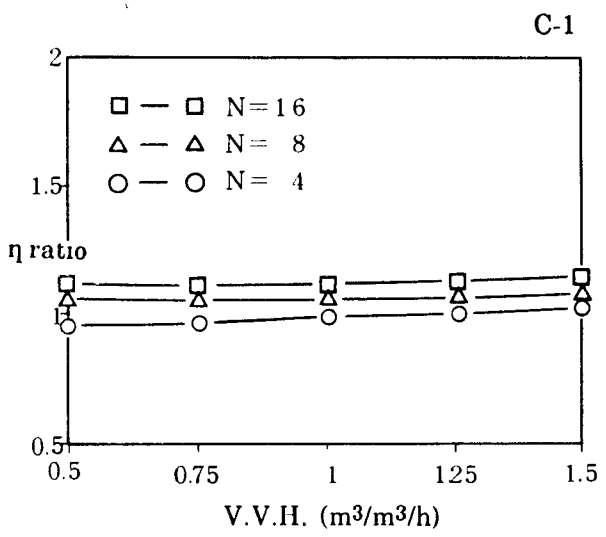


Fig. 4-8 Comparison of η ratios for WFA and SFA

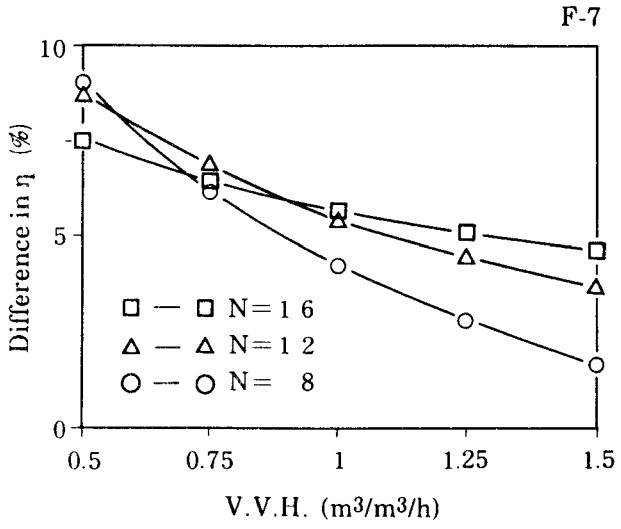


Fig. 4-9 Comparison of η differences for WFA and SFA

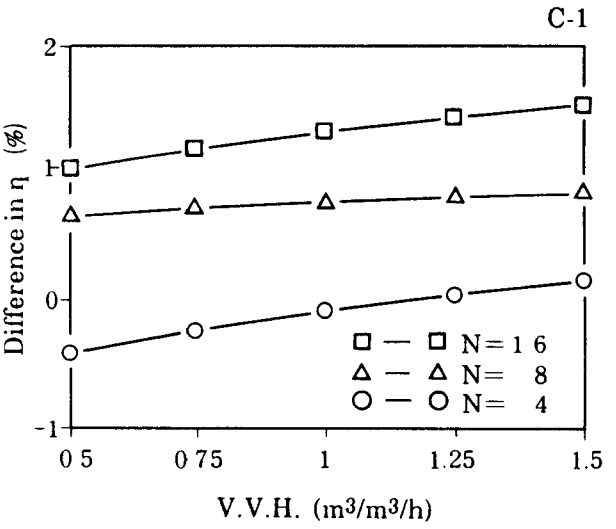


Fig. 4-10 Comparison of η differences for WFA and SFA

If whole-floor and spiral-flow aeration are compared in terms of air flow rate, this ratio decreases for fine-bubble diffusers as the air flow rate rises. At a low air flow rate, this ratio is a high 1.4 to 1.8. This is a 7.5 to 9 percent increase in terms of OTE, indicating higher efficiency for whole-floor aeration. Moreover, the ratio drops more gradually when a large number of diffusers are in use. For coarse-bubble diffusers, however, the ratio remains little changed, indicating that they are not much affected by diffuser installation layout.

(3) Unsteady-state method and exhaust-gas oxygen-detection method

The unsteady-state method is generally employed to measure the value of OTE. In our experiments, we also calculated OTE values based on the results obtained by detecting the oxygen density of exhaust gas and compared such values with those derived from the unsteady-state method. Some examples of these comparisons are illustrated in Fig. 4-11.

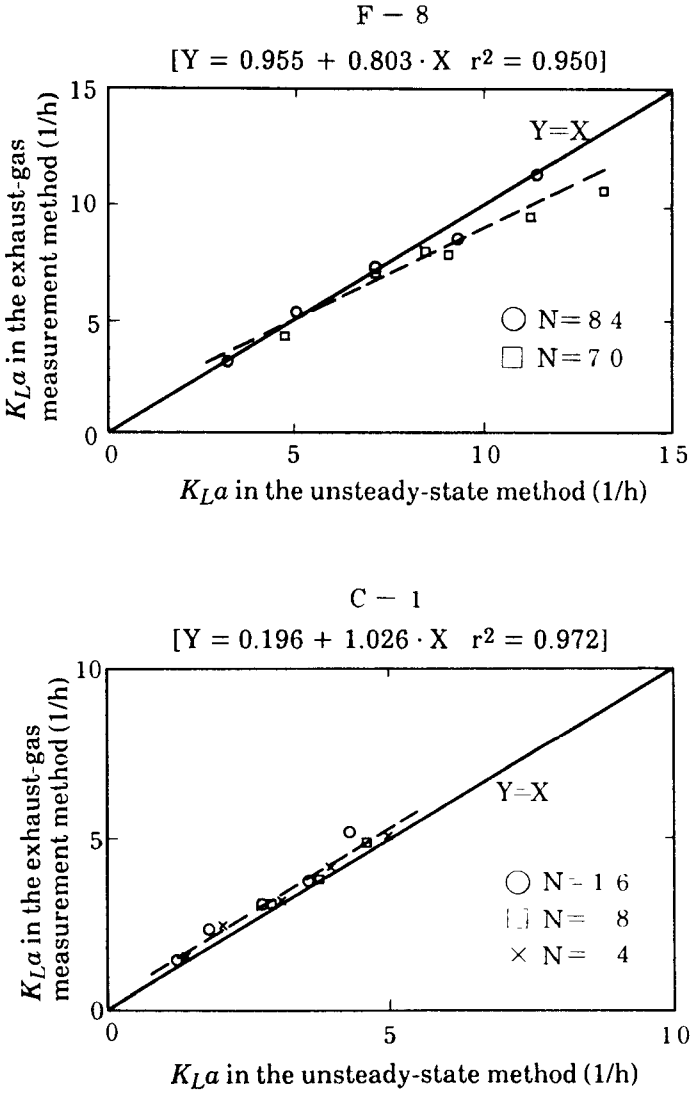


Fig. 4-11 Comparison of K_La for Exhaust-gas Measurement Method and Unsteady-state Method

These graphs show that OTE values in the exhaust-gas oxygen-detection measurement method correspond fairly closely with those in the unsteady-state method and, in light of these results, these two methods can be considered suited for practical application.

4-4. Relationship between Diffuser Characteristics and OTE

With the aim of identifying the relationship between the characteristics of each type of diffuser described in 4-2 (1) and OTE, we considered the following two aspects of this relationship using data collected in our experiments:

(1) Relationship between diffuser characteristics evidenced in experiments using a 1.8 m-deep tank and OTE obtained from experiments using 5.8 m-deep tanks

This experiment was undertaken to see if it was possible to determine OTE for 5.8 m-deep tanks from the diffuser characteristics of a 1.8 m-deep tank.

The characteristics of each type of diffuser are analyzed in terms of such bubble dimensions as average diameter, 75 percent diameter, total surface area per air flow rate of 1ℓ as well as dry pressure and dynamic wet pressure. We examined the correlation between these parameters and OTE.

Furthermore, we adopted values at 9 m^3/hr for average diameter, 75 percent diameter and total surface area as well as values of $G_S/N=5$ for OTE. The data utilized for this purpose are listed in Tables 4-9 and 4-12 while graphs derived from them are shown in Figs. 4-12 and 4-13.

Table 4-12 Diffusion Characteristics in Relation to η and K_La

Type	η (%)	K_La/N (1/h)	Average diameter (mm)	75%-diameter (mm)	Total surface area ($m^2/\ell \cdot$ air)
F-1	24.1	0.233	4.21	4 66	1.35
F-2	27.3	0.267	3.87	4.34	1.40
F-3	25.4	0.248	4 01	4.37	1.43
F-4	26.1	0.248	4.45	4.86	1.28
F-5	24.1	0.228	4.36	4.84	1.29
F-6	29.0	0.285	3.76	4.37	1.48
F-7	25.0	0.243	3.90	4.35	1.43
F-8	31.2	0 288	4.35	4.98	1.30
F-9	28.7	0.263	4.36	4.93	1.29
M-1	17.9	0.176	4.60	5.34	1.19
M-2	19.1	0.190	5.00	5.79	1.10
M-3	22.3	0.211	4.70	5.43	1.18
C-1	10.6	0.106	4.32	5.77	1.11
C-2	10.4	0.110	4.87	5.53	1.09

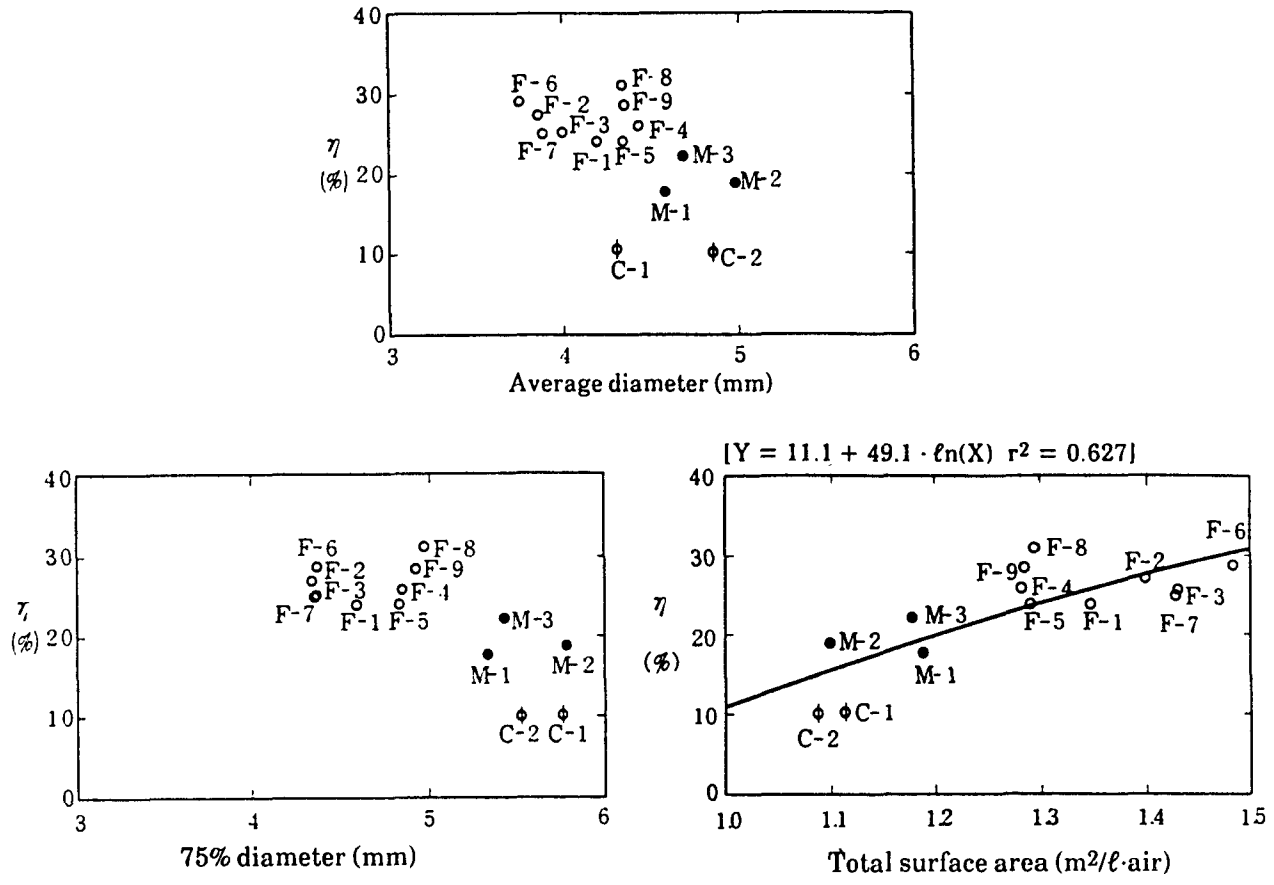


Fig. 4-12 Relation between Diffusion Characteristics and Oxygen Transfer Efficiency
(Vertical axis, experiment III data; horizontal axis, experiment I data)

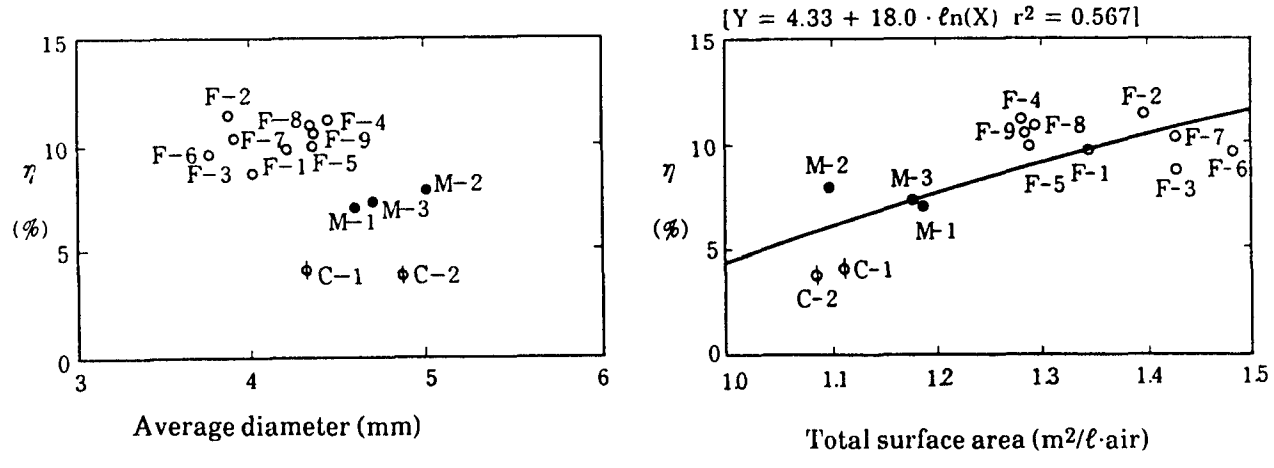


Fig. 4-13 Relation between Diffusion Characteristics and Oxygen Transfer Efficiency (Experiment I)

These graphs demonstrate to some extent that there is a relationship between OTE and the parameters of average diameter and 75 percent diameter, but the correlation with total surface area is most conspicuous. On the other hand, we could not find any clear link in our experiments between OTE and dry and dynamic wet air flow rate.

On the basis of these findings, we can estimate an approximate OTE for a given type of diffuser by measuring the volume of bubbles generated from the diffuser and computing the total surface area per air flow rate of 1ℓ.

(2) Relationship between OTE and diffuser characteristics identified in experiment II using plate diffusers

In many cases, diffuser specifications are defined in terms of pore size in relation to bubble size. Individual manufacturers, however, do not use common standards to indicate the same pore size. Thus, even when diffusers of different manufacturers indicate pore sizes, we cannot use them as the basis for comparison. Under such circumstances, we decided to use certain known pore sizes to investigate their relationship with bubble sizes.

OTE which has been obtained from experiments using plate diffusers shown in Table 4-13 and Figs. 4-14 and 4-15 serves as the basis for identifying the relationship between diffusion characteristics and OTE.

Since OTE increases as pore sizes decrease, OTE being largest at 300 μm and conversely declining when 200 μm or smaller, this provides grounds for determining the most effective pore sizes for diffusers. In addition to the correlation parameters of diffusion characteristics cited in the above section, we also checked the effects of gas hold-up and pressure. Table 4-13 contains the data used, while the results are given in Fig. 4-15.

Table 4-13 Diffusion Characteristics in Relation to η and K_La

Pore size (μm)	K_La (1/h)	η (%)	Average diameter (mm)	75% diameter (mm)	Total surface area ($m^2/\ell \cdot$ air)	Gas hold-up (%)	Pressure (mmHg)	Total surface area × Gas hold-up ($m^2/\ell \cdot$ air)
100	29 0	7.52	3.36	3 69	1 69	1.2	267	2.08×10^{-2}
200	34.1	8 59	3.41	3 85	1 66	1.9	209	$3\ 09\times10^{-2}$
300	59.4	15.13	3 98	4.42	1.44	3.5	171	$5\ 10\times10^{-2}$
400	54 8	14.17	4 03	4.50	1.40	3.1	156	4.39×10^{-2}
600	51.9	13.35	3 65	4.02	1.56	2.6	154	$4\ 10\times10^{-2}$
800	35.8	9.23	4.35	5 01	1.29	2.4	152	3.07×10^{-2}
1,000	25.0	6.43	4.86	6.04	1.08	2.1	151	2.29×10^{-2}
2,000	19 5	5.01	4.84	5.79	1.11	1.6	148	1.78×10^{-2}
3,000	24.1	6 33	4.72	5.72	1.09	1.9	151	2.03×10^{-2}
4,000	18.3	4 67	4.64	5 55	1 14	1.0	149	1.16×10^{-2}

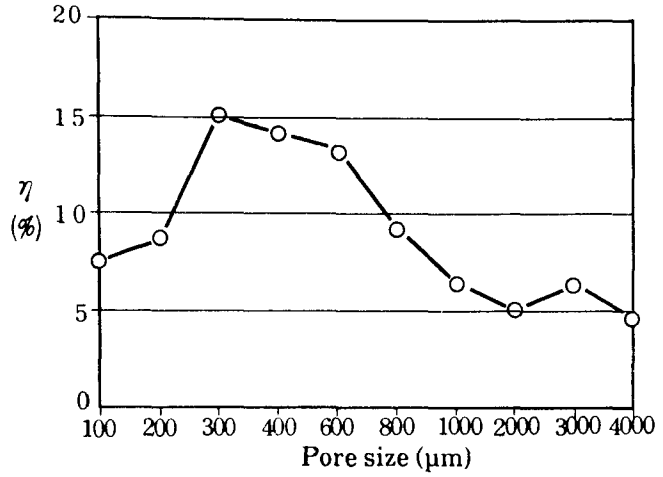
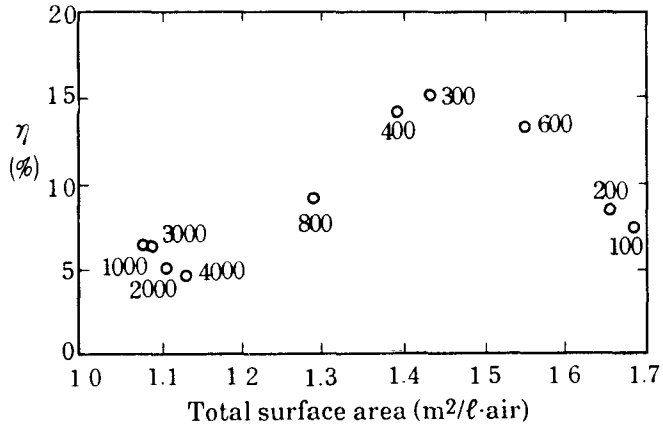


Fig. 4-14 Pore size and Oxygen Transfer Efficiency



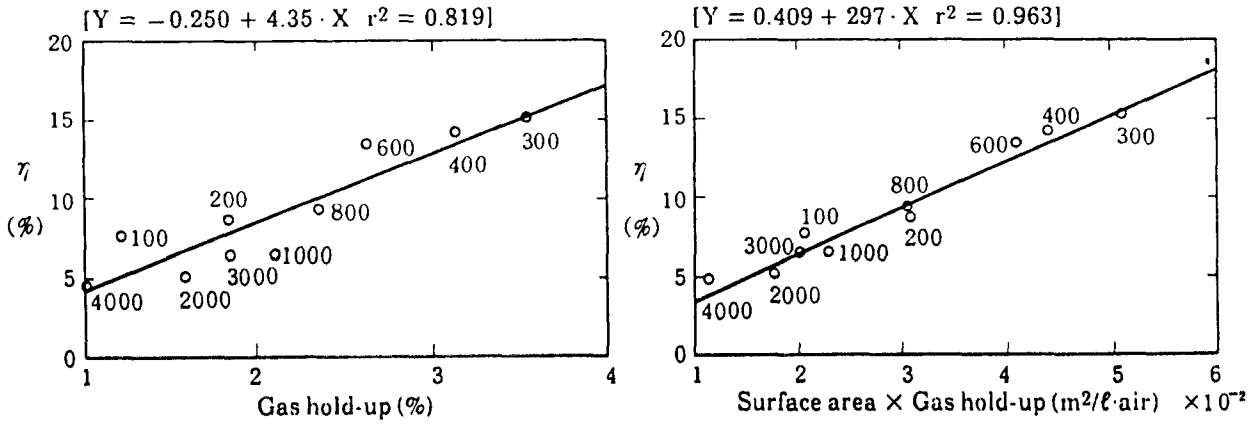


Fig. 4-15 Relationship between Diffusion Characteristics and Oxygen Transfer Efficiency (Experiment II)

These data, with the exception of the results obtained when pore size is 100 μm and 200 μm, show that the relationship between diffusion characteristics and OTE is in large part predicated on pore size. For example, if pore size is small, both total surface area and OTE are large and, conversely, they tend to be small when pore size has a larger diameter. However, this rule does not always apply in cases where pore size is 600 μm. The results shown in Fig. 4-15 have been obtained by setting air flow rate at a fixed level. Should we change air flow rate, therefore, it is possible that somewhat different results would be produced.

Fig. 4-15 shows the relationship between gas hold-up, the total surface area of bubbles, and OTE. Since gas hold-up is related to the retention time of bubbles in the tank, there is a significant correlation between gas hold-up by itself and OTE. This indicates that when considering OTE, bubble retention time cannot be overlooked.

If gas hold-up and total surface area of bubbles are considered together, their correlation with OTE is even higher than with gas hold-up alone, making it possible to nearly determine OTE from these two factors.

Fig. 4-16 shows the relationship between pressure directly below plate diffusers, and pore size. Moreover, pressure in the tank directly below plate diffusers remains more or less at a constant level when pore size is 400 μm or greater, and pressure rises in proportion to the reduction in pore size if it is under 400 μm. Fig. 4-17 shows the relationship between pore size and power consumption efficiency. In this respect, pore size in the range of 300 to 400 μm is considered to be most effective from the viewpoint of aeration efficiency.

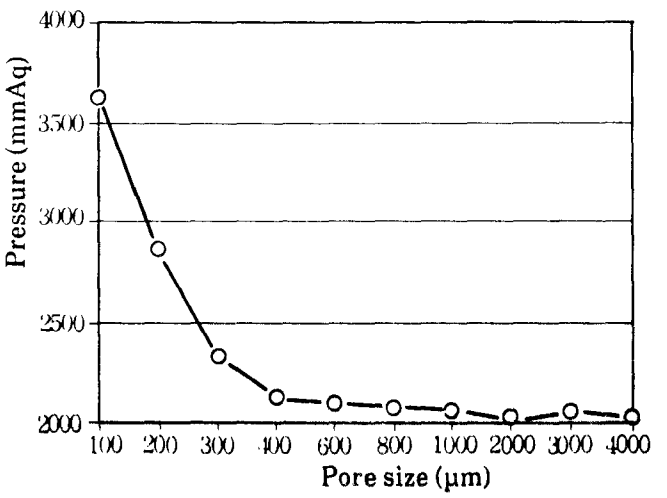


Fig. 4-16 Pore Size and Pressure

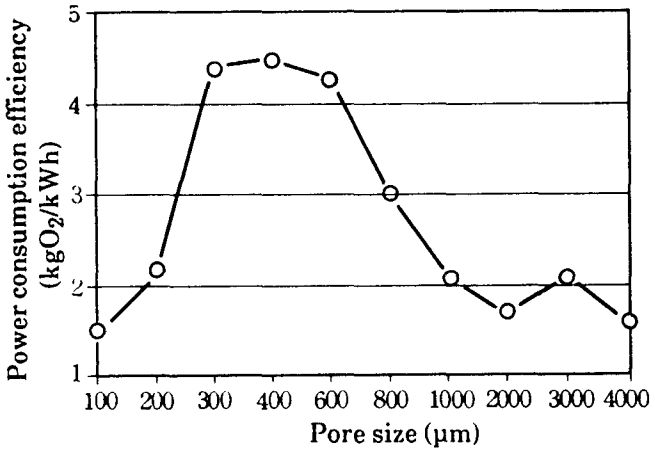


Fig. 4-17 Pore Size and Power Consumption Efficiency

4-5. Power Consumption Efficiency of Each Type of Diffuser

(1) Comparison parameters

We compared the power consumption efficiency of each type of diffuser. Power consumption efficiency was computed based on the following two sets of data:

1. Values specified in the designs for the construction of additional aeration tanks at the Kanagawa plant.
2. Values obtained for OTE in the present experiments.

- Removed amount of BOD — 13 t/day
- Required amount of oxygen — Removed amount of BOD × 1.1
- OTE — Average OTE when $G_s/N = 5$
- Head loss through the diffuser — Dynamic wet pressure when $G_s/N = 5$
- Water depth
  - Whole-floor aeration — 5,000 mmAq
  - Spiral-flow aeration — 4,500 mmAq
- Plumbing loss — 1,000 mmAq

(2) Results

The calculated results of power consumption efficiency are shown in Table 4-14.

Fine-bubble diffusers F-1 through F-9 are generally considered to be more energy efficient. In fact, they display consistently higher OTE values as compared to diffusers used until now, but dynamic wet pressure is also great since permeability is low in these diffusers.

When power consumption efficiency is expressed by electricity consumption per unit of removed BOD (kWh/kg), fine-bubble diffusers display more or less uniform efficiency at 0.25-0.32. On the other hand, the power efficiency of intermediate-bubble diffuser M-1 is 0.45, while that of coarse-bubble diffuser C-1 is 0.64. In this respect, fine-bubble diffusers are considered to have achieved much improvement in power consumption efficiency as compared with other types of diffusers.

Table 4-14 Comparison of Power Consumption Efficiency

	F-1	F-2	F-3	F-4	F-5	F-6	F-7
Bubble size	FBD	FBD	FBD	FBD	FBD	FBD	FBD
Diffuser layout	WFA	WFA	WFA	WFA	WFA	WFA	WFA
Removed amount of BOD (T/D)	13	13	13	13	13	13	13
Oxygen requirement (T/D)	14.3	14.3	14.3	14.3	14.3	14.3	14.3
Oxygen transfer efficiency (η) (%)	24.1	27.3	25.4	26.1	24.1	29.0	25.0
Oxygen supply (T/D)	59.4	52.3	56.4	54.8	59.2	49.3	57.2
Air flow rate (m^3/min)	144	127	137	133	144	120	13.9
Air flow rate per unit (m^3/min/diffuser)	5	5	5	5	5	5	5
No. of units	1726	1522	1639	1594	1722	1433	1661
Diffusion depth (mmAq)	5000	5000	5000	5000	5000	5000	5000
Air flow pressure (mmAq)	333	363	240	229	462	284	232
Plumbing loss (mmAq)	1000	1000	1000	1000	1000	1000	1000
Required pressure (mmAq)	6333	6363	6240	6229	6462	6284	6232
Required electricity (kW)	170	150	159	155	172	140	161
Electricity consumption per removed amount of BOD (kWh/BOD kg)	0.313	0.277	0.294	0.286	0.318	0.259	0.298
Required energy (kWh/O_2kg)	0.285	0.252	0.267	0.260	0.289	0.235	0.271

	F-8	F-9	M-1	M-2	M-3	C-1	C-2
Bubble size	FBD	FBD	IBD	IBD	IBD	CBD	CBD
Diffuser layout	WFA	WFA	SFA	WFA	WFA	SFA	WFA
Removed amount of BOD (T/D)	13	13	13	13	13	13	13
Oxygen requirement (T/D)	14.3	14.3	14.3	14.3	14.3	14.3	14.3
Oxygen transfer efficiency (η) (%)	31.2	28.7	15.0	19.1	22.3	10.6	10.4
Oxygen supply (T/D)	45.9	49.8	95.4	75.0	64.1	135.2	137.0
Air flow rate (m^3/min)	111	120	231	182	155	327	332
Air flow rate per unit (m^3/min/diffuser)	5	5	5	5	5	5	5
No. of units	1335	1446	2773	2181	1864	3929	3982
Diffusion depth (mmAq)	5000	5000	4500	5000	5000	4500	5000
Air flow pressure (mmAq)	519	522	40	16	483	40	341
Plumbing loss (mmAq)	1000	1000	1000	1000	1000	1000	1000
Required pressure (mmAq)	6519	6522	5540	6016	6483	5540	6341
Required electricity (kW)	134	146	244	206	187	346	392
Electricity consumption per removed amount of BOD (kWh/BOD kg)	0.248	0.269	0.451	0.380	0.345	0.639	0.724
Required energy (kWh/O_2kg)	0.226	0.245	0 410	0.345	0.314	0.581	0.658

Note: FBD: Fine-bubble diffuser
IBD: Intermediate-bubble diffuser
CBD: Coarse-bubble diffuser
WFA: Whole-floor aeration
SFA: Spiral-flow aeration

5. On-site Study

In November and December of 1983, the Sewage Works Bureau of Yokohama City changed a portion of the C-1 diffusers of the Kanagawa plant to more efficient F-1 and F-8 fine-bubble diffusers. These diffusers have since been in operation for more than three years. In the meantime, we continued our experiments using 5.8 m-deep tanks at the Chubu plant. Furthermore, we investigated the changes in performance over time of these fine-bubble diffusers which have been in use for three years using 1.8 m- and 5.8 m-deep tanks.

Based on our investigations on clean water using 1.8 m- and 5.8 m-deep tanks, we installed F-1 diffusers at the Chubu plant. Our report will include the results of the on-site study of these newly installed diffusers that were all installed for whole-floor aeration.

(1) Outline of on-site study at the Kanagawa and Chubu plants

a) Distribution of dissolved oxygen (DO)

DO was measured by collecting water at five points within four or five cross sections in the aeration tank.

b) Water current profile

Flow directions and flow rates were measured in the central area of the aeration tank for a total of 36 cross-sectional points at 6 horizontal levels and 6 vertical levels.

c) MLSS distribution

Mixed liquor was sampled at seven points within the center cross section of the aeration tank and measured for MLSS.

d) K_La and OTE

Mixed liquor was sampled at a point approximately at the center of each of the 5 cross sections used in the measurement of DO and measured for K_La (T) and OTE by the steady-state method.

Volumetric air flow rate was changed between three levels in this study.

(2) Performance test in the tanks

The fine-bubble diffusers F-1 and F-8 have been in operation for more than three years. We checked the diffusion performance of these units using the 1.8 m- and 5.8 m-deep tanks and compared it with that of new diffusers.

a) Performance test in 5.8 m-deep tanks

The performance of new diffusers and that of three-year-old diffusers were tested in 5.8 m-deep tanks. The diffusers were installed for whole-floor aeration with a placement density of 4 percent for F-1 diffusers and 4.4 percent for F-8 diffusers. Test diffusers were placed near the inlet and outlet of the aeration tank and the performance of each unit was analyzed.

b) Performance test in a 1.8 m-deep tank

The performance of new diffusers and that of the diffusers in service for three years were tested in a 1.8 m-deep tank under the same conditions as described in Table 4-1.

5-1. Results of On-site Study

(1) DO distribution

DO distribution at the two plants is shown in Tables 5-1 and 5-2 and Fig. 5-1. At each plant, DO distribution rises in the direction of water flow in the aeration tank. An air flow rate of 0.4 to 0.6 V. V. H. is needed to maintain DO at 2 mg/ℓ or higher at the outlet in our experiments. The two plants under study use a combined system and DO distribution at the five cross sections is highly uniform for both plants. The maximum difference of DO in the same cross-sectional plane was 0.4 mg/ℓ.

Table 5-1 DO Distribution at Kanagawa Plant (Diffuser F-1)

(mg/ℓ)

Air flow rate (V.V.H.)	Item	Test tank				
		1	2	3	4	5
0.4	Maximum	0.25	0.40	1.35	2.40	2.35
	Minimum	0.18	0.30	1.10	2.30	2.30
	Mean	0.22	0.37	1.21	2.36	2.33
	Max.–Min. (Max. minus Min.)	0.07	0.10	0.25	0.10	0.05
0.6	Maximum	0.80	1.65	3.85	5.10	5.20
	Minimum	0.75	1.55	3.65	5.00	5.00
	Mean	0.77	1.59	3.73	5.06	5.11
	Max.–Min. (Max. minus Min.)	0.05	0.10	0.20	0.10	0.20
0.8	Maximum	2.70	3.95	5.50	6.30	6.35
	Minimum	2.60	3.85	5.40	6.20	6.10
	Mean	2.66	3.92	5.45	6.27	6.23
	Max.–Min. (Max. minus Min.)	0.10	0.10	0.10	0.10	0.25

Note: Air flow rate (V.V.H.)–air m^3/tank m^3/hr

Table 5-2 DO Distribution at Chubu Plant (Diffuser F-1)

(mg/ℓ)

Air flow rate (V.V.H.)	Item	Test tank			
		1	2	3	4
0.4	Maximum	1.20	1.90	2.50	2.80
	Minimum	1.00	1.70	2.30	2.60
	Mean	1.08	1.79	2.37	2.73
	Max.–Min. (Max. minus Min.)	0.20	0.20	0.20	0.20
0.6	Maximum	2.20	2.60	3.00	3.40
	Minimum	2.00	2.50	2.85	3.30
	Mean	2.08	2.55	2.92	3.37
	Max.–Min. (Max. minus Min.)	0.20	0.10	0.15	0.10
0.8	Maximum	2.50	3.20	4.20	4.80
	Minimum	2.30	3.10	4.00	4.70
	Mean	2.38	3.16	4.08	4.74
	Max.–Min. (Max. minus Min.)	0.20	0.10	0.20	0.10

Note: Air flow rate (V.V.H.)–air m^3/tank m^3/hr

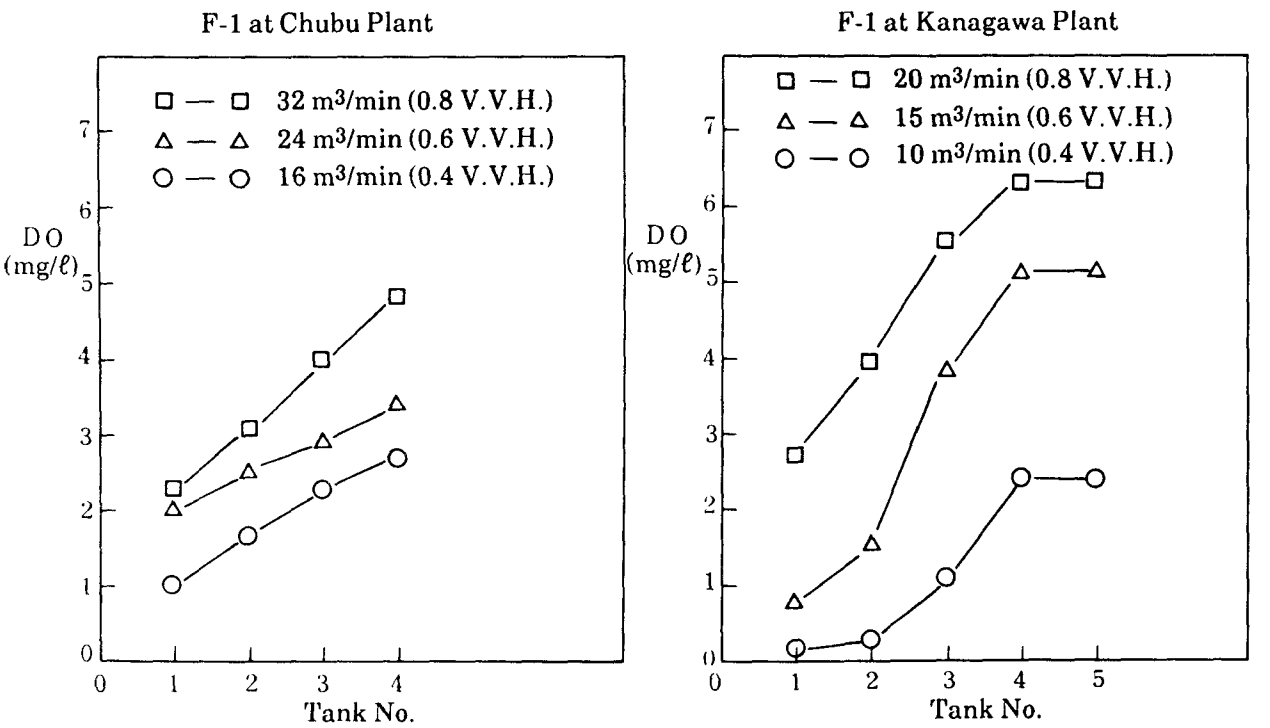
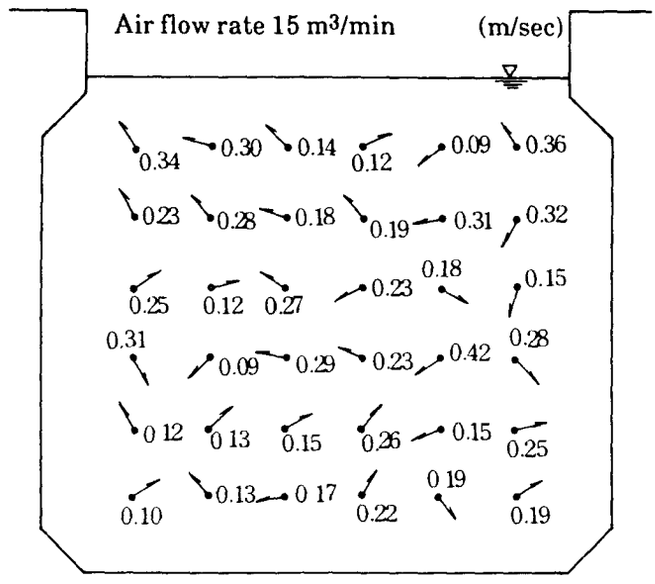


Fig. 5-1 DO Distribution in On-site Study

(2) Water current profile

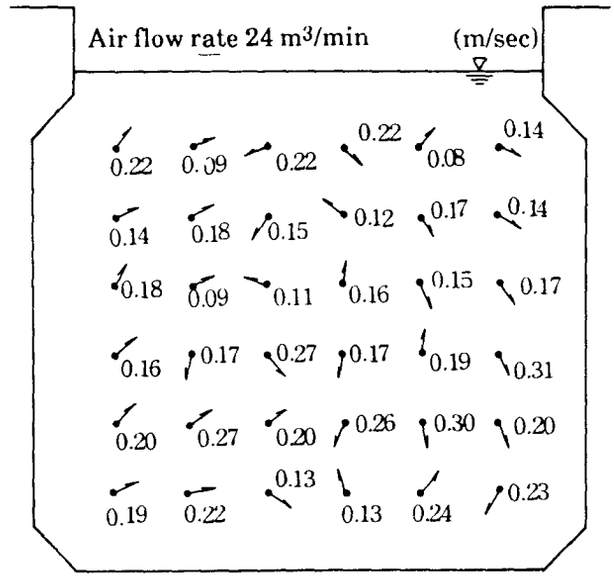
The distribution of flow directions and flow rates at the respective plants is indicated in Fig. 5-2. When air flow rate is increased, average flow rates also rise at each plant, but individual plants show different values for flow rate increase. This indicates that the flow rate is influenced by tank configuration and diffuser installation layout in addition to air flow rate.

F-1 at Kanagawa Plant



(0.6 V. V. H)

F-1 at Chubu Plant



(0.6 V. V. H)

Fig. 5-2 Water Current Profile at Plants

(3) MLSS distribution

As shown in Table 5-3, each plant displays extremely uniform MLSS distribution. Coefficient of variation of MLSS were within 5 percent, and no sediment accumulation was observed on the bottom of the tanks.

Table 5-3 MLSS Distribution

Plant		Kanagawa		Chubu
Diffuser		F-1	F-8	F-1
Air flow rate (V.V.H.)	Mixed liquor sampling point	MLSS (mg/ℓ)	MLSS (mg/ℓ)	MLSS (mg/ℓ)
0.4	1	1440	1110	1470
	2	1540	1060	1400
	3	1480	1110	1530
	4	1460	1120	1470
	5	1420	1090	1530
	6	1460	1160	1400
	7	1560	1100	1470
	x	1480	1110	1470
	Δx	140	100	130
	$CV(\%)$	3.5	2.7	3.6
0.6	1	1420	1160	1530
	2	1460	1160	1470
	3	1400	1150	1470
	4	1480	1170	1530
	5	1540	1180	1530
	6	1420	1130	1600
	7	1540	1150	1470
	x	1470	1160	1510
	Δx	140	50	130
	$CV(\%)$	3.9	1.4	3.2
0.8	1	1540	1140	1530
	2	1520	1170	1470
	3	1520	1140	1600
	4	1540	1130	1530
	5	1560	1120	1600
	6	1540	1160	1670
	7	1500	1150	1600
	x	1530	1140	1570
	Δx	60	50	200
	$CV(\%)$	1.3	1.5	4.2

(4) Distribution of R_r, K_La and OTE

Table 5-4 summarizes the distribution of DO, R_r, K_La and OTE in each plant. Fig. 5-3 indicates that R_r distribution decreases in the direction of flow for the two plants. F-1 and F-8 diffusers showed almost identical R_r distribution at the Kanagawa plant.

Table 5-4 Distribution of DO, R_r, K_La (20), and OTE (20)

Kanagawa Plant						Kanagawa Plant						Chubu Plant					
F - 8						F - 1						F - 1					
V.V.H.	Tank No.	DO (mg/ℓ)	Rr (mg/ℓ/hr)	K_La (20) (1/h)	η (20) (%)	V.V.H.	Tank No.	DO (mg/ℓ)	Rr (mg/ℓ/hr)	K_La (20) (1/h)	η (20) (%)	V.V.H.	Tank No.	DO (mg/ℓ)	Rr (mg/ℓ/hr)	K_La (20) (1/h)	η (20) (%)
0.4	1	0.16	25.6	2.53	23.6	0.4	1	0.22	20.8	2.42	24.1	0.4	1	1.08	44.9	4.95	48.2
	2	0.40	23.9	2.38	22.5		2	0.37	19.1	2.40	23.9		2	1.79	33.4	3.93	39.3
	3	1.16	19.9	2.23	20.8		3	1.21	17.0	2.47	24.6		3	2.37	29.7	3.69	35.9
	4	2.28	18.8	2.24	21.4		4	2.36	17.5	2.64	26.2		4	2.73	27.4	3.58	34.9
	5	2.24	18.3	2.27	21.0		5	2.33	17.5	2.64	26.2						
0.6	1	0.30	24.5	2.39	15.2	0.6	1	0.77	36.0	3.84	25.4	0.6	1	2.08	36.4	4.42	28.7
	2	1.24	20.3	2.21	13.9		2	1.59	23.0	2.65	17.6		2	2.55	29.7	3.78	24.6
	3	3.02	19.3	2.46	15.8		3	3.73	20.3	2.93	19.4		3	2.92	29.4	3.92	25.4
	4	4.56	18.0	2.74	17.8		4	5.06	19.3	3.22	21.3		4	3.37	27.4	3.90	25.3
	5	4.42	18.2	2.70	17.5		5	5.11	18.5	3.13	20.7						

K_La showed the same tendency as R_r at the Chubu plant which was not the case for the Kanagawa plant. (Refer to Fig. 5-4.)

At the Kanagawa plant, OTE distribution tends to increase in the direction of the flow for both F-1 and F-8 diffusers when OTE is 20 percent or thereabout. On the other hand, OTE distribution declines in the flow direction at the Chubu plant and OTE is 30 to 40 percent at an air flow rate of 0.4 V.V.H. but it drops to almost 20 percent when air flow rate is changed.

In this study, we found that each plant has different patterns of R_r, K_La, and OTE distribution. In particular, the difference in OTE was more significant between plants rather than between diffuser types.

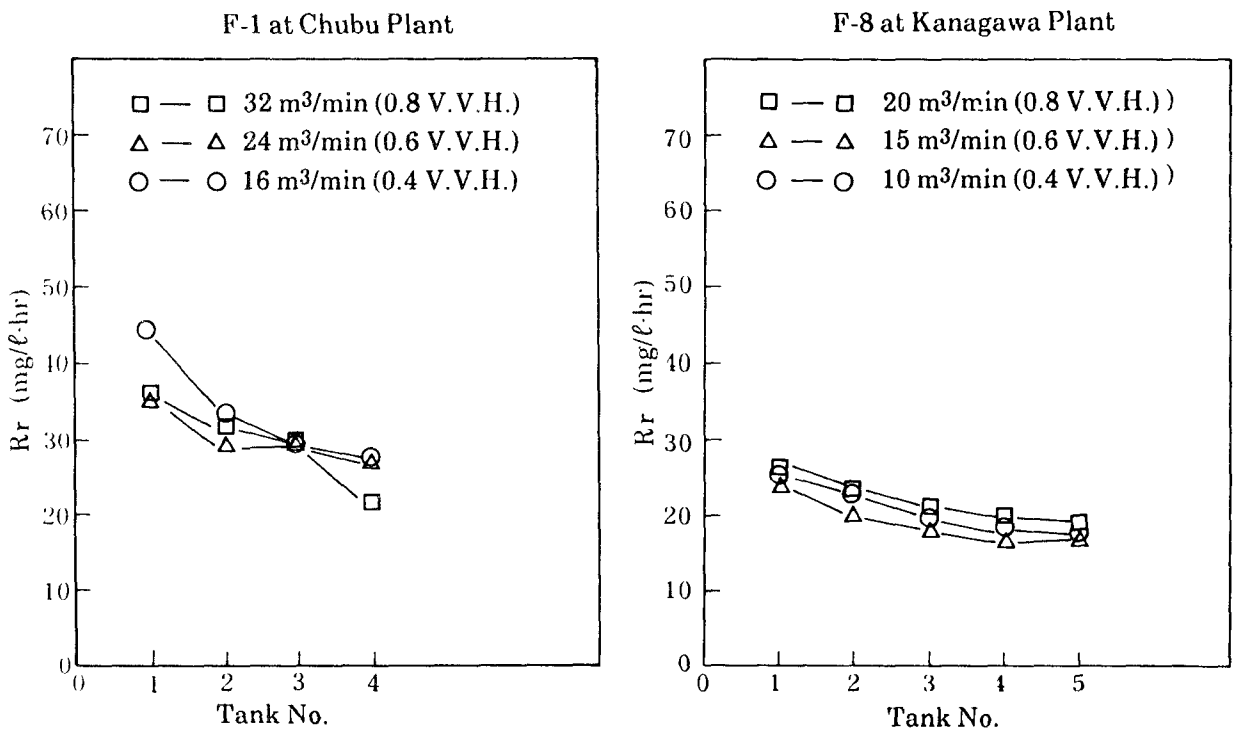


Fig. 5-3 *Rr* Distribution in On-site Study

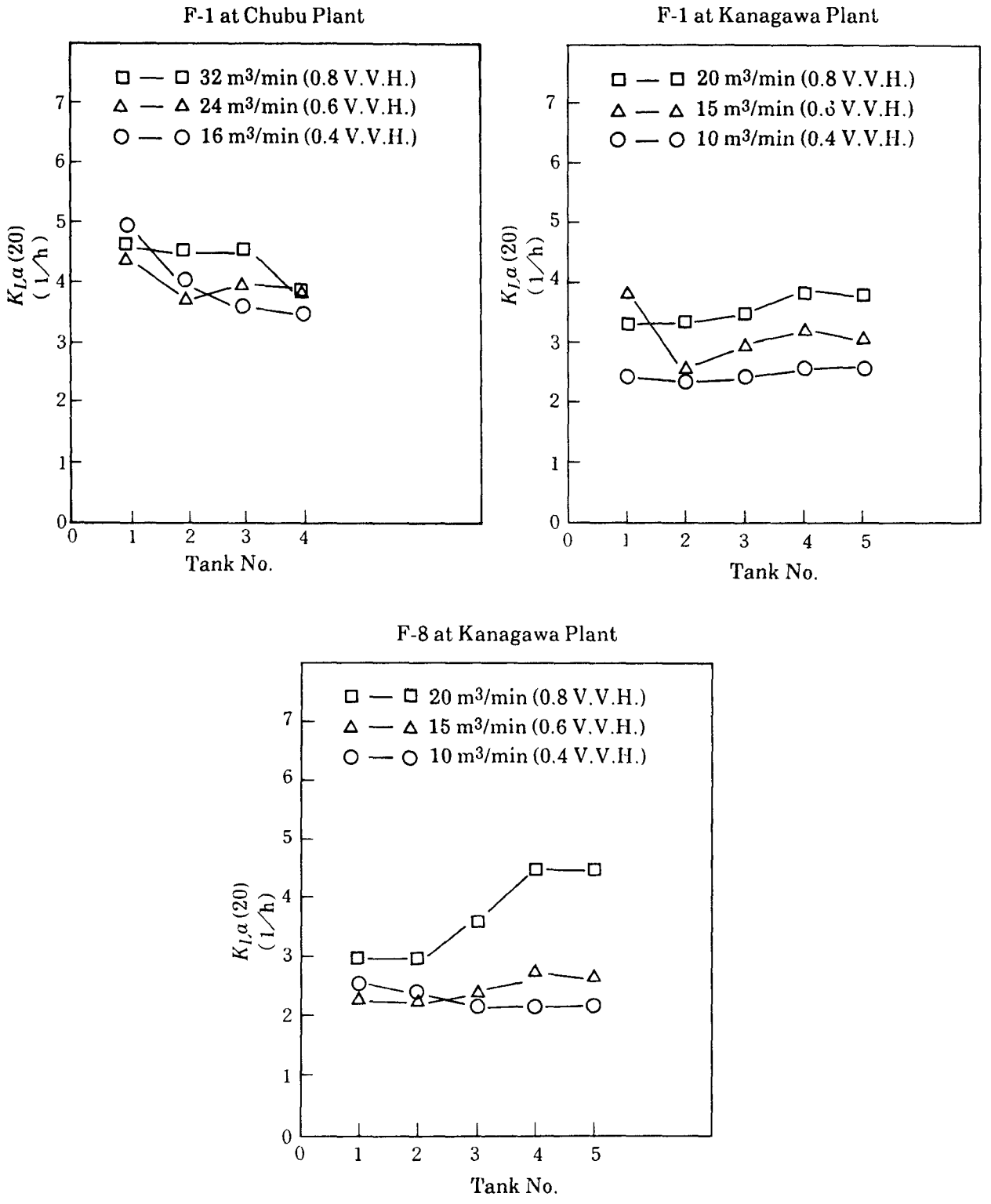


Fig. 5-4 K_La Distribution in On-site Study

One of the reasons for this difference is believed to be caused by R_r which is used to calculate K_La and OTE. R_r was determined by the sewage testing method (published by Japan Sewage Works Association). However, the levels of DO generally applied to the calculation of R_r tend to be higher than those actually measured in the aeration tank. Thus, it can be assumed that there is a gap between the theoretical values of R_r and the R_r values occurring in the aeration tank. Based on the results obtained in the present study, the relationship between R_r and OTE is displayed in Fig. 5-5. The graph shows that R_r increases in proportion with the rise in OTE, indicating a correlation between these two parameters, although there are some variances.

The difference in diffusion performance between the plants can be attributed to the effects of R_r. Therefore, we will need to take this possibility into account when measuring OTE in future on-site studies as well as the applicability of the exhaust-gas oxygen-detection method.

5-2. Changes in Diffuser Performance over Time

We compared the performance of the F-1 and F-8 diffusers in operation for three years with the data on new diffusers in respect of the following parameters:

a. Effects of air flow rate on K_La and OTE

b. Changes in dry pressure and dynamic wet pressure over time

c. Bubble generation from the inlet to outlet of aeration tanks

d. Clogging

(1) Effects of air flow rate on K_La and OTE in 5.8 m-deep tanks

The effects of air flow rate on K_La and OTE are tabulated in Table 5-5. In the case of F-1 diffusers three years in operation, OTE is 75 to 85 percent at the inlet and 90 percent at the outlet when this value is set at 100 percent for new diffusers. On the other hand, OTE for F-8 diffusers with three years of service is virtually unchaged at 90 to 106 percent as compared to new diffusers.

We tested F-1 diffusers with three years of use again after an acid cleaning in which diffusers were immersed in 2N HCl for two hours, and then brushed and washed with water. OTE returned to virtually the same level as new diffusers at 93 to 119 percent, proving that the performance of diffusers can be renewed by cleaning.

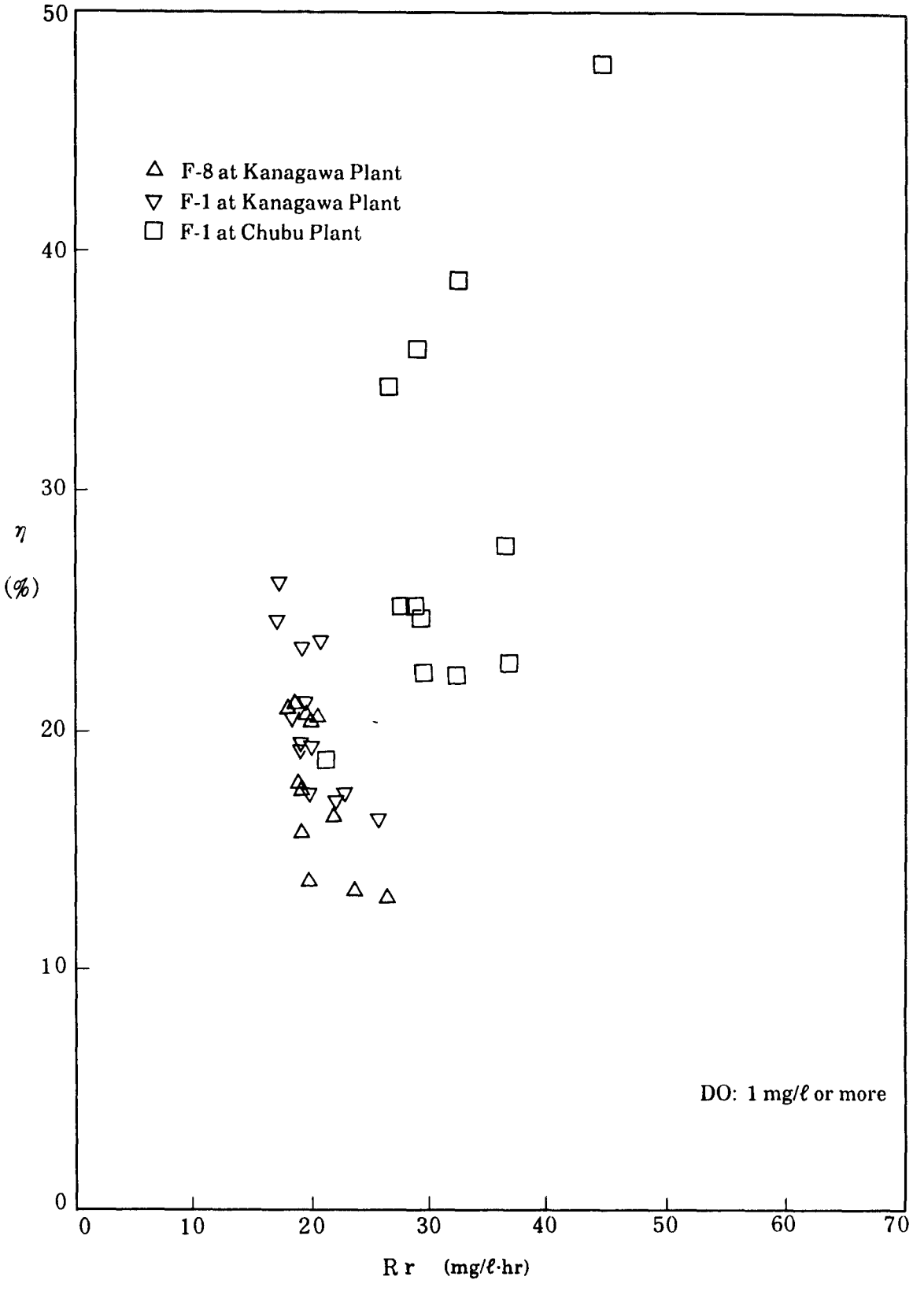


Fig. 5-5 Oxygen Transfer Efficiency and Rr

Table 5-5 Comparison of New, Cleaned and In-Service Diffusers

Type	Installation layout N	$(K_La/N)=A\times(Gs/N)^B$		0.5 V.V.H. Gs(Nm³/m) 1.2	0.75 V.V.H. Gs(Nm³/m) 1.8	1.0 V.V.H. Gs(Nm³/m) 2.4	1.25 V.V.H. Gs(Nm³/m) 3.0	1.5 V.V.H. Gs(Nm³/m) 3.6
		A	B	η (%)	η (%)	η (%)	η (%)	η (%)
F-1	New diffusers 40	0.0516	0.915	25.3	24.4	23.8	23.4	23.0
	No.1 tank 40	0.0448	0.842	21.2 (83.7)	19.9 (81.3)	19.0 (79.6)	18.3 (78.3)	17.8 (77.3)
	No.5 tank 40	0.0459	0.911	22.6 (89.2)	21.8 (89.0)	21.2 (89.0)	20.8 (88.8)	20.5 (88.8)
	No.1 tank, cleaned units 40	0.0476	0.920	23.7 (93.6)	22.9 (93.8)	22.4 (94.0)	22.0 (94.1)	21.7 (94.2)
	No.5 tank, cleaned units 40	0.0635	0.837	30.1 (118.9)	28.1 (115.1)	26.8 (112.6)	25.9 (110.6)	25.1 (109.1)
F-8	New units 49	0.0532	0.969	27.9	27.5	27.3	27.1	26.9
	No.1 tank 49	0.0598	0.845	28.9 (103.7)	27.2 (98.6)	26.0 (95.2)	25.1 (92.6)	24.4 (90.5)
	No.5 tank 49	0.0624	0.795	29.6 (106.0)	27.2 (98.8)	25.6 (94.0)	24.5 (90.4)	23.6 (87.6)

Note: Figures in parentheses are the percentages of OTE compared to that of new diffusers.

(2) Dry pressure and dynamic wet pressure in a 1.8 m-deep tank

Dry pressure and dynamic wet pressure measured in a 1.8 m-deep tank are given in Fig. 5-6. Dry pressure decreased significantly in the direction from the inlet to the outlet for F-1 diffusers with no orifices, indicating clogging taking place on the inlet side.

As for diffusers with orifices, dry pressure remained almost unchanged for both F-1 and F-8 products. This shows that it is necessary to install some sort of pressure control unit such as orifices, to prevent the air flow rate from changing by clogging.

(3) Bubble generation

The results of a bubble study is presented in Table 5-6. In terms of bubble volume, F-1 diffusers three years in operation generated bubbles with larger volume when compared with new diffusers from the inlet to the outlet. The bubble volume of F-8 diffusers with three years' service was virtually the same as that of new diffusers.

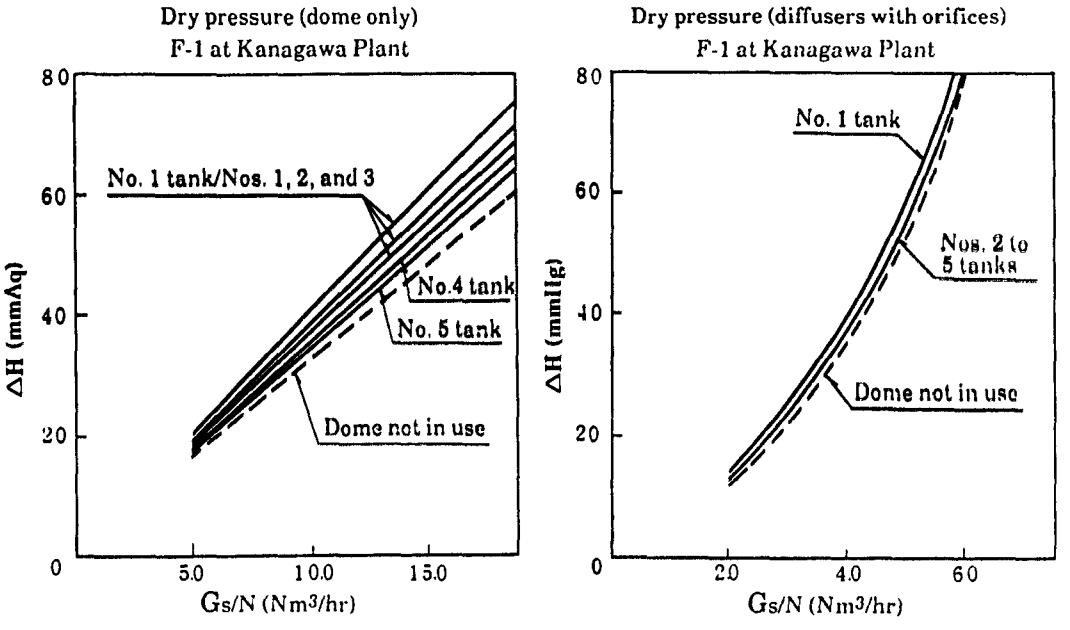


Fig. 5-6 Dry Pressure and Dynamic Wet Pressure

Table 5-6 Bubble Volume from Used Diffusers

Tank No.	Type	Air flow rate (m^3/hr)	Average volume ($\mu\ell$)	Diameter (mm)
1	F-1	9	77.72	5.29
5	F-1	9	61.10	4.89
1	F-8	9	42.71	4.34
5	F-8	9	38.54	4.19

(4) Clogging examined by disassembling diffuser units

Diffuser units were broken down in the vertical direction and the status of clogging was observed by eye. There was no extensive clogging in the outer section of the unit for both F-1 and F-8 diffusers. Sediments clogging the inner section from air blow were more noticeable.

To sum up, some clogging and pressure loss increase were recognized for diffusers in use for three years. Since the same diffusers were used for various tests of dry and dynamic wet pressure, however, clogging status could have changed during the experiments and this could have caused some inconsistency in our data. In future investigations, we will need to conduct various tests under the same clogging conditions. On the other hand, if clogging can be eliminated by a number of pressure tests, it will not present any problems for aeration tank operations. Since few reports have been published on the assessment of diffuser clogging, we will need to study and establish a clogging assessment method in the future.

6. Conclusion

In our investigations, the relationship of diffusion characteristics and OTE was surveyed using a total of 16 types of diffusers grouped into fine-bubble, intermediate and coarse-bubble diffusers. All diffusers were newly supplied from manufacturers' warehouses, but in two instances, diffusers already in operation were also tested.

(1) Relationship between diffusion characteristics and OTE (Results of experiments in 1.8 m- and 5.8 m-deep tanks)

a. Dynamic wet air flow rate and the total surface area of bubbles per air flow rate of 1ℓ are two effective criteria for classifying diffusers into fine-bubble and coarse-bubble groups.

   Dynamic wet air flow rate under a pressure of 50 mmAq was 50 mℓ/cm^2 per min or less in fine-bubble diffusers and 80 mℓ/cm^2 per min or more in intermediate- and coarse-bubble diffusers. The total surface area of bubbles per air flow rate of 1ℓ was 1.3 m^2 or greater for fine-bubble diffusers, and 1.2 m^2 or less for intermediate- and coarse-bubble diffusers.

b. On the basis of classification made for our study, OTE was 20 to 30 percent for fine-bubble diffusers, 14 to 17 percent for intermediate-bubble diffusers and 9 to 12 percent for coarse-bubble diffusers. Fine-bubble diffusers also proved to be superior in terms of power consumption efficiency.

c. OTE increased with the reduction in pore size. However, if pore size was 200 μm or less, OTE also declined. This relationship as well as that for power consumption efficiency indicates the existence of an optimum range of pore sizes for diffusers within the limits of this investigation.

d. Close correlation was confirmed between the product obtained by multiplying the total surface area of bubbles per air flow rate of 1ℓ by gas hold-up and OTE. Thus, besides bubble size, OTE is also affected by the retention time of bubbles in the tank.

e. In experiments using 1.8 m- and 5.8 m-deep tanks, OTE and the total surface area of bubbles showed a similar relationship. Therefore, we can infer OTE of a 5.8 m-deep tank by measuring the total surface area of bubbles per air flow rate of 1ℓ in a 1.8 m-deep tank.

f. In the present experiments, OTE was measured by the unsteady-state method. Since measurement by the exhaust-gas oxygen-detection method shows similar results, it can also be used for the determination of OTE.

g. Fine-bubble diffusers display bubble size distribution that is more or less normal. In the case of coarse-bubble diffusers, bubble sizes are distributed almost uniformly, because bubbles burst while ascending in the water tank.

h. Salinity effects OTE. OTE rises in proportion to the increase in salinity. This effect is more conspicuous for fine-bubble diffusers. In contrast, OTE declines with the increasing concentration of surface active agents, with nonionic agents inflicting the greatest impact on OTE.

(2) Relationship between installation layout of diffusers and OTE

a. For fine-bubble diffusers, OTE tends to rise with the increase in installation density. Conversely, OTE declines with the increase in air flow rate, but this takes place at a slower pace if diffusers are installed in greater density.

b. In contrast, coarse-bubble diffusers have little impact on OTE even if installation density is increased. OTE shows a tendency to rise, however, in response to an increased air flow rate.

c. Regardless of which type of diffuser was used, OTE showed consistently higher values for whole-floor aeration over spiral-flow aeration. This tendency was most noticeable for fine-bubble diffusers at low air flow rates; OTE for whole-floor aeration proved to be 1.4 to 1.8 times that of spiral-flow aeration and their OTE difference was 7.5 to 9 percent, clearly pointing out the advantage of whole-floor aeration. In the case of coarse-bubble diffusers, little difference was seen in OTE for the two aeration methods, and the placement of diffusers had little influence on this result.

(3) On-site study of two types of fine-bubble diffusers in whole-floor aeration

a. DO and MLSS distribution at an air flow rate of 0.4 V.V.H. was uniform, proving that mixed liquor is agitated evenly in the tank. On the other hand, differences were observed in the water current profile and OTE between the plants. Thus the effects of tank configuration need to be considered in addition to the air flow rate.

b. We investigated changes in diffusion performance over time on diffusers which have been in operation for three years. OTE dropped in diffusers supplied by one manufacturer. Although OTE changed for the diffusers of other manufacturers, its decrease was far less significant. Tests on air flow rate produced similar results showing performance differences between manufacturers. However, the causes of these differences are unclear, pointing out the need to further study methods for assessing clogging.

7. Afterword

At present, such diffusion characteristics as pore size and dry pressure, etc., are widely used in the design specifications of diffusers. In our investigation, however, the total surface area of bubbles per air flow rate of 1ℓ displayed the highest correlation with OTE. Moreover, the product obtained by multiplying gas hold-up by the total surface area of bubbles per air flow rate of 1ℓ displayed an extremely high correlation with K_La because the retention time of bubbles in the tank is included in gas hold-up. We believe that we can make a more accurate assessment of diffuser performance by developing an assessment method incorporating these new diffusion characteristics.

In our studies, we also found that there is an optimum range of bubble sizes and pore sizes to maximize diffuser performance. In particular, it was confirmed that fine-bubble diffusers are effected by inorganic salt and surface active agents.

Since fine-bubble diffusers demonstrate superior characteristics in power consumption efficiency, we plan to use such diffusers at our plants in Yokohama from now on. At the same time, data collected in our investigations will serve as a reference when selecting diffusers. In this connection, we hope that our data will be conductive to the development of new types of diffusers and will prove to be instrumental in establishing a method under which diffusion performance is assessed from a broader perspective.

*

In closing, I would like to acknowledge all those who assisted in the preparation of this paper.

References

1. W. K. Lewis and W. C. Whitman, *Ind. Eng. Chem.* 16 (1924): 1215.

2. J. Oldshue, *Biological Treatment of Sewage and Industrial Waste* 1 (New York: Reinhold Pub. Corp., 1956).

3. H. Fujii, K. Terasawa, and T. Takeshima, *Oxygen Transfer in Deep Aeration Tanks* (Tokyo Metropolitan Government Sewage Works Bureau, 1979).

4. H.Kubota, S. Hoshino, and T. Kasakura, "Oxygen Transfer in Activated Sludge Basin—a Proposal of Reasonable Measure for Diffuser Performance," *J. Japan Sewage Works Assoc.* 16, no. 184 (1979).

5. W. W. Eckenfelder, Jr., and D. J. O'connor, *Biological Waste Treatment* (Pergamon Press, 1961).

6. R. G. Gilbert and D. Libby, "Field Testing for Oxygen Transfer Efficiency in a Full-Scale Deep Tank," *Proceedings 31st Industrial Conference* (Purdue University, 1976).

7. S. Hashimoto, "A Theoretical Approach to Calculating the Overall Capacity Coefficient of Oxygen Transfer in Sewage Treatment," *Water Purification and Liquid Wastes Treatment*, no. 6 (1970).

PROGRESS IN THE RESEARCH ON APPLICATION OF BIOTECHNOLOGY ON WASTEWATER TREATMENT

by

Shigeru Andoh
Director, River-Basin Sewerage Division
Department of Sewerage and Sewage Purification
Ministry of Construction

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

Prepared for Presentation at:
11th United States/Japan Conference
on
Sewage Treatment Technology

October, 1987
Tokyo, Japan

ABSTRACT

A five year research project called the "Development of New Wastewater Treatment Systems Employing Biotechnology" was started by the Ministry of Construction in 1985. This project, commonly known as "Biofocus W.T.", intends to develop new wastewater treatment systems with the following features to be achieved through the use of biotechnology.

a. Lower energy requirements and lower O & M costs
b. Smaller land requirements for wastewater treatment facilities
c. Better effluent quality
d. Recovery of valuable resources

The areas of research and development for the attainment of these objectives follow:

a. A study for the development of a microorganism bank
b. A study of the application of genetic engineering on microorganisms for wastewater treatment
c. A study of immobilization methods
d. A study of bioreactors for wastewater treatment
e. A study of bioreactors for sludge treatment
f. A study of biosensors
g. A study of solids/liquid separation units
h. A study for systematizing the developed techniques

The outline and the present situation of Biofocus W.T. are summarized in this paper.

C O N T E N T S

1. PREFACE 348

2. THE OUTLINE OF BIOFOCUS W.T. 348
2.1 The Research Organization for Biofocus W.T. 348
2.2 The Research and Development Plan for Biofocus W.T. 348
2.2.1 The Study for the Development of a Microorganism Bank 348
2.2.2 The Study of the Application of Genetic Engineering on Microorganisms for Wastewater Treatment 349
2.2.3 The Study of Immobilization Methods 350
2.2.4 The Study of Bioreactors for Wastewater Treatment 351
2.2.5 The Study of Bioreactors for Sludge Treatment 352
2.2.6 The Study of Biosensors 353
2.2.7 The Study of Solids/Liquid Separation Units 354
2.2.8 The Study on the Systematization of the Developed Techniques 354

3. THE RESEARCH SITUATION OF BIOFOCUS W.T. 355
3.1 The Study of the Microorganism Bank 355
3.2 Development of a Multi-Stage Reversing-Flow Bioreactor: MRB 356
3.3 The Development of Bioreactors for Nitrogen Removal 360
3.4 The Study on Sludge Components 361
3.5 Development of Biosensors 361
3.6 The Development of Self-Sustaining Energy Systems 363

4. POSTSCRIPT 364

1. PREFACE

The Ministry of Construction of the Government of Japan started "The Development of New Wastewater Treatment Systems Employing Biotechnology" (commonly called Biofocus W.T.) in fiscal year 1985 as one of the five-year comprehensive research and development projects.

By broadly applying biotechnology, which is the technology that utilizes the life maintenance functions of biological organisms, to wastewater treatment, Biofocus W.T. is primarily intended to solve the present and future problems in wastewater treatment listed below.

a. Lower energy requirements and lower O & M costs
b. Smaller land requirements for wastewater treatment facilities
c. Better effluent quality
d. Recover of valuable resources

The executing bodies of this project are the Public Works Research Institute and the Building Research Institute of the Ministry of Construction. The former is mainly in charge of matters related to publicly owned wastewater treatment plants (POWTP) while the latter oversees matters related to household wastewater treatment tanks. This paper presents mainly the outline and development situation of the area being handled by the Public Works Research Institute.

2. THE OUTLINE OF BIOFOCUS W.T.

2.1 The Research Organization for Biofocus W.T.

Biofocus W.T. is being carried out by two research institutes of the Ministry of Construction, a research contract with the Japan Sewage Works Agency, and through joint studies with private companies. The Committee for the Development of New Wastewater Treatment Systems Employing Biotechnology is reviewing this project.

2.2 The Research and Development Plan for Biofocus W.T.

Table 1 shows the research and development plan for Biofocus W.T. As indicated in this table, the project covers a wide range of research subjects including immobilization of microorganisms, separation, classification and cultivation of microorganisms, genetic engineering of microorganisms, the development of bioreactors and biosensors for wastewater and sludge treatment, and the development of new wastewater treatment systems.

2.2.1 The Study for the Development of a Microorganism Bank (Refer to 3.1)

The purpose of this study is to separate and classify the microorganisms effective in treating wastewater and sludge, to study the substrate metabolism characteristics, preservation method and mass cultivation method of microorganisms, and to clarify methods for integrating the achievements of this research in the form of a microorganism bank for wastewater treatment. The following microorganisms are currently being studied.

Table 1 Research and development plan for new wastewater treatment systems utilizing biotechnology

I. The study for the development of a microorganism bank
II. The study of the application of genetic engineering on microorganisms for wastewater tratment
III. The study of immobilization methods
IV. The study of bioreactors for wastewater treatment
V. The study of bioreactors for sludge treatment
VI. The study of biosensors
VII. The study of solids/liquid separation units
VIII. The study for systematizing the developed techniques

a. Nitrifiers
b. Sulfur oxidizing bacteria
c. Photometabolic bacteria
d. Resistant bacteria to heavy metals
e. Bacteria degrading refractory matters
f. Filamentous bacteria
g. Sulfate reducing bacteria
h. Flock inducing bacteria
i. Denitrifiers
j. Phosphorus accumulating bacteria
k. Methane bacteria
l. Solids-liquefying bacteria
m. Others

2.2.2 The Study of the Application of Genetic Engineering on Microorganisms for Wastewater Treatment

Though recombinant DNA has become the hub of biotechnology in recent years, the application of microorganisms with recombinant DNA in an open system, such as in wastewater treatment, is prohibited, due to the view of dangerous biohazards. However, the following study is being carried out in order to reveal the characteristics of microorganisms related to wastewater treatment by utilizing the technology developed through the research on recombinant DNA at the gene level, and also to prepare for the case where the use of microorganisms with recombinant DNA may be permitted in open systems.

a. The conjugation of genes and the stability of genetic information in wastewater and/or sludge treatment processes

Where some strains with plasmid contain useful genetic information which will be applied to wastewater treatment, factors governing the stability of

the genetic information are being reviewed, including the possibility of the occurrence of the mobilization and conjugation of the plasmid to other strains.

b. An analysis of the genetic information of bacteria which decompose refractory materials

Using bacteria (unidentified strains), degrading polyvinyl alcohol, dichlorobenzen, chlorophenol, etc., the genetic information involved in the biodegradation will be abstracted and analyzed.

c. The application of genetic engineering to wastewater and sludge treatment

By utilizing genetic information successfully obtained from the analysis mentioned in b , bacteria having new characters will be bred by genetic engineering and attempts will be made to put them to practical use. The new characters will be a higher multiplication rate than existing degrading bacteria, the additional ability to decompose many kinds of refractory materials, an increase in degrading concentration, and so forth.

2.2.3 The Study of Immobilization Methods

The immobilization of microorganisms related to wastewater and sludge treatment is effective for the following purposes.

a. To increase the concentration of microorganisms in the reactor and to reduce the volume of treatment facilities.

b. To hold a large amount of specific microorganisms required for removing specific materials.

Because of the above, the following studies are being conducted with respect to the entrapping method, binding method and self-immobilization method.

a. The entrapping method

Applicability of entrapping agents such as polyvinyl alcohol-boric acid, epoxy-based photo polymerizing resins, epoxy plus various polymerizer, polyethylene glycol, polyacrylamide and -calaginan to wastewater treatment are being examined.

The entrapping method has little advantage in removing ordinary BOD components, because the cost is relatively high and microorganisms attached to the surface of the entrapping media and suspended in the reactor are sufficient for the purpose. The entrapping method seems to be advantageous for the immobilization of special microorganisms such as nitrifiers. Items for review are; 1) the partition coefficient and rate of penetration of specific substrates within immobilizing agents, 2) the recovery activity of immobilized microorganisms after entrapping, 3) the physical properties (specific gravity, strength, etc.) of the entrapping media, and 4) the production and disposal cost.

b. The binding method

The attachment media for the binding method being reviewed are zeolite, silica sand, polypropylene, activated carbon, oyster shell, artificial aggregates and so forth. Items for review are; 1) the binding properties of microorganisms, 2) the physical properties of attachment media, 3) the regeneration method for attachment media, 4) methods for eliminating the clogging of attachment media in a fixed bed, and 5) the production and disposal costs. Evaluation of attachment media for the binding method relys greatly upon the characteristics of the bioreactor to be used, and thus the final evaluation must include the reactor.

c. Self-immobilization method

The self-immobilization method, in which the immobilization is made by the microorganisms themselves without using other materials, is an ideal immobilization method. With respect to the self-immobilization method, the clarification of the self-immobilizing mechanism is being made by using three types of reactors: The upflow anaerobic sludge blanket reactor process (UASB), the multi-stage reversing-flow bioreactor (MRB explained later) and the aerobic upflow sludge blanket reactor process (AUSB). A sludge blanket with good flocculation characteristics is formed in UASB and AUSB. In the future, the method of the stably held blanket will be reviewed. With respect to the MRB, the blanket is formed by granule sludge. The mechanisms and control of self-granulation are scheduled to be clarified.

2.2.4 The Study of Bioreactors for Wastewater Treatment

The Bioreactors for wastewater treatment to be developed in Biofocus W.T. are for removing organic substances, nitrogen, nitrogen and phosphorus simultaneously, and for removing trace substances.

a. The reactor for removing organic substances

1) Aerobic reactors

Two types of aerobic reactors, fixed bed and fluidized bed types, are being studied, and reduction in treatment time is now considered to be very promising. The future problems are 1) how measures are to be taken against suspended solids, 2) the reduction in fluidizing energy, 3) how to improve oxygen transfer efficiency, 4) developing a clog prevention method, and 5) finding a solution for the various hydraulic problems when increasing the scale.

2) Anaerobic reactors

For the anaerobic reactors as a whole, studies have been quite successful for enhancing BOD and SS treatment efficiency and for reducing equipment size. A current important theme is the removal of hydrogen sulfide remaining in treated water for which physical, chemical and biochemical methods are being reviewed.

3) Multi-stage reversing-flow bioreactor (MRB, refer to 3.2)

This is a completely new kind of reactor and, as such, basic design factors have to be reviewed, such as BOD-volumetric loading, the amount of oxygen supply, the intensity of agitation, hydraulic retention time, solids retention time, etc. It is also necessary to review the reactor shape when increasing the scale.

b. Bioreactors for nitrogen removal (Refer to 3.3)

The purpose of this research is to establish an entrapping method for nitrifiers and denitrifiers and to develop an efficient bioreactor and a system for removing nitrogen by using the established entrapping method through the following studies.

1) Treatment performance of immobilized nitrifiers and immobilized denitrifiers

Stability of long-term treatment is reviewed by the bench scale experiments using real wastewater.

2) Reactor types

Operating conditions and configuration (hydraulic conditions) of reactors are reviewed by using immobilized nitrifiers and immobilized denitrifiers.

3) Reactor system

To be reviewed is how to make up the system to include the reactors for nitrification and denitrification, reviewed in 2).

c. Bioreactors for simultaneous removal of nitrogen and phosphorus

The purpose of this study is to develop bioreactors for simultaneous removal of nitrogen and phosphorus by immobilized microorganisms. In addition, algae and photometabolic bacteria are also employed.

d. Bioreactors for removing trace substances

Bioreactors utilizing entrapping and binding methods for removing trace substances are being developed. The types of treatments being reviewed are in the second treatment stage and in the post-treatment stage of the activated sludge system.

2.2.5 The Study of Bioreactors for Sludge Treatment

The research is being performed to enhance the efficiency of anaerobic digestion, to remove heavy metals from sludge using microorganisms for accelerating the effective utilization of sludge, to treat sidestreams by photometabolic bacteria for treating sidestreams and recovering protein, and to recover valuable resources from sludge.

a. Upgrading the efficiency of anaerobic digestion

An attempt is being made to shorten the period of anaerobic digestion, increase the percentage of solid decomposition, and recover more digested gas, through the use of immobilization and sludge liquefication techniques. The binding method using ceramic attachment and so forth is mainly applied to immobilizing microorganisms. The methods being reviewed now for increasing the percentage of solids decomposition are the pretreatment methods, such as the addition of liquefying enzymes, heat treatment, mechanical crushing, and acid and alkali treatment.

b. Removal of heavy metals from sludge by bacteria leaching

In this study, the bacteria leaching technique practiced in recovering metals from ores by oxidizing sulfur and reducing pH by the action of iron bacteria and sulfur oxidizing bacteria is applied to wastewater sludge in order to remove its heavy metals. The behavior of heavy metals in bacteria leaching and the effect of treated sludge on dewatability and effective utilization are being reviewed.

c. The treatment of sidestreams by photometabolic bacteria

The feasibility of this is being studied by using purple nonsulfur bacteria.

d. The recovery of valuable resources from sludge (Refer to 3.4)

In order to study the recoverability of valuable resources, detailed component analysis is being made for various wastewater sludge. As a consequence of the high proportion of cellulose in primary sludge, basic experiments for recovering alcohol from carbohydrate (cellulose) from the sludge are being performed.

2.2.6 The Study of Biosensors (Refer to 3.5)

The purpose of this study is to develop various practical biosensors which use vital reactions for improving environmental measurement and process control in wastewater treatment.

a. BOD sensor

Substraite utilization mechanisms within vital reaction elements are being theoretically and experimentally reviewed in order to establish the optimum design conditions for vital reaction elements. Microorganisms are being retrieved to increase the sensitivity of a BOD sensor and to improve the reliability of measured values in low concentration ranges. The vital reaction elements for BOD sensor are improved. Also, measuring systems, including supporting equipment, to be used in laboratories are being developed and improved, and their further application to outdoor measurement are being studied.

b. Ammonia sensor and organic acid sensor

The development and improvement of ammonia and organic acid sensors, with field tests, are being conducted.

c. New useful biosensors

Based on the results of a questionnaire survey conducted last year, the sensors for water quality that will become necessary in the future, are being reviewed to determine the development themes and policies. Moreover, efforts will be made for reviewing, developing and improving the principles of biosensors having high feasibility and usefulness such as an odor sensor and a poison sensor.

2.2.7 The Study on Solids/Liquid Separation Units

The purpose of this study is to develop separators of solids for the primary and final stage of bioreactors to enhance the treatment efficiency by combining the separators with the bioreactors in which high-density microorganisms will be retained by means of immobilization technology.

a. Measuring methods for grain size distribution

Methods for measuring the grain size distribution of particles in wastewater, before and after the separation of solids, and for evaluating the grain size separating functions in the separation of solids are being reviewed. Though electrical resistance methods and light scattering methods are available for quickly measuring grain size distributions, their results differ greatly from measurements using sieves and filter papers as the division method for components by grain size. A similar situation has also existed in the image processing method. Therefore, the measuring conditions of each measuring method will be reexamined and reviewed.

b. Grain size distribution of solids in raw wastewater

Grain size distribution of solids components contained in raw wastewater is being surveyed. In addition, biodegradation of organic matter by grain size will be studied to clarify the grain size to be removed.

c. Solids separators

For a solids separator to be used in the primary stage of a bioreactor, several methods, such as filtration, floatation and the granule sludge blanket process are reviewed. As for the granulation methods, both the bioflocculation method and the chemical flocculation will be reviewed. Membrane separation is being reviewed for the final-stage.

2.2.8 The Study on the Systematization of the Developed Techniques

The aim of this study is to systematize the techniques developed in the above-mentioned studies and to propose new wastewater treatment systems.

a. New wastewater treatment systems required

Some examples of new wastewater treatment systems that will be required in the future are summarized below.

1) An energy-saving type (self-sustaining) wastewater treatment system (Refer to 3.6)

2) An area-saving type of wastewater treatment system

3) A nitrogen-removing type of wastewater treatment system

4) A simultaneous nitrogen and phosphorus removing type of wastewater treatment system

5) A small wastewater flow system

6) A specific substance removing type system

7) A wastewater treatment system with less sludge treatment

8) A wastewater treatment system utilizing light

9) A wastewater treatment system which recovers valuable resources

10) Other new wastewater treatment systems

b. Evaluation of combined processes

The unit processes to be developed in the present project are a solids separator, bioreactors for wastewater treatment and sludge treatment, and biosensors. Methods for efficiently combining these unit processes and sensors will be reviewed here.

c. Proposing new wastewater treatment systems

Definitive proposals will be made for several systems among the new wastewater treatment systems indicated in Paragraph "a" based upon the results of laboratory tests and pilot plant experiments. And the proposed systems will be reviewed from the viewpoints of the degree of completeness, their biotechnological performance, and their originality and usefulness.

3. THE RESEARCH SITUATION OF BIOFOCUS W.T.

Biofocus W.T.'s research situation will be outlined below with respect to its several subjects.

3.1 The Study of the Microorganism Bank

The purpose of this study is to make technical studies for the purpose of storing various microorganisms related to wastewater treatment and to establish "a microorganism bank related to wastewater treatment" for preparing and storing a data base classified from an engineering viewpoint. The

conventional complicated microbiological classification method, is not easy, from an engineering point of view, to review certain bacteria. For instance, the bacteria involved in the biological treatment of sulfide in wastewater and sludge are classified into many genera and categories such as *Thiobacillus thiooxidans*, *Beggiatoa sp*, and *Chromatiaceae sp*. And even microbiologically unidentified bacteria will be included. From the engineering viewpoint, an easy-to-use data bank may be created by classifying all of these bacteria into only one sulfur oxidizing bacteria group and by preparing a data base based upon items of concern only to wastewater treatment engineers and researchers, such as "environments in which bacteria are able to grow" and "the degree of sulfur oxidizing ability that can be expected". For this purpose, the following surveys and research has become necessary:

a. A review of more efficient classifying methods
b. Determination of data required for classification and the method of necessary experiments

With respect to a., a rough classification, as shown in Table 2, was made in fiscal year 1986. It will be revised further while carrying out the actual research in future. As for b., the details of a data base using nitrifiers as the model and its standard experimental flow has been prepared. Additional data bases will be prepared sequentially in the future for individual bacteria, such as bacteria resistant to heavy metals, sulfur oxidizing bacteria, filamentous bacteria, and bacteria which decompose refractory matters, etc.

3.2 Development of a Multi-Stage Reversing-Flow Bioreactor: MRB

An MRB is a bioreactor for wastewater treatment, newly developed by the Public Works Research Institute, and is a very promising wastewater treatment method for utilizing biological principles different from conventional wastewater treatment processes.

Figure 1 shows a conceptional diagram of an MRR. Wastewater flowing in the upper part of the first downflow aeration vessel (AV), is aerated while flowing down. Solids in the influent are moved to the next upflow biological reaction vessel (BRV) without being retained therein. No aeration is performed and wastewater is moved upward while gently mixing in the BRV. The cross sectional area of the highest portion is enlarged to three times that of the lower portion, and the upflow velocity drops to one-third. Thereafter, AV and BRV continue in the same manner, and effluent is discharged from the final BRV. An independent solids separator nor the returning of sludge is required. In the BRV, solids in water with a settling velocity higher than the upflow velocity are retained there and its concentration gradually increases. At the same time, the solids are floccoulated by the gentle agitation due to upflow, by which self-granulated sludge is easily formed.

The granule sludge has a spherical shape with the diameter of 2 to 10 mm and can withstand being pinched with the fingers. But the sludge spheres may be broken if an excessive shearing force is applied by a pump or the like. It is possible to store them inside tap water for a month at normal

Table 2 Classifications for the microorganism bank (draft)

Subject	Classification of treatment contents
Wastewater treatment	Removal of biodegradable organic substances
	Removal of refractory organic substances
	Removal of refractory, hazardrous and synthetic organic matters
	Oxidation of nitrogen
	Removal of nitrogen
	Oxidation of sulfur
	Reduction of sulfur
	Removal of phosphorus
	Removal of Fe and Mn
	Removal of trace heavy metals
	Removal of dissolved inorganics
	Floatation separation of SS
	Sedimentation of SS
	Removal of SS by liquefaction
	Odor removal
	Color removal
	Algae removal
	Bacteria removal
Sludge treatment	Sludge liquefaction
	Gasification of sludge
	Removal of heavy metals in sludge
	Composting
	Production of valuable substances from sludge resources

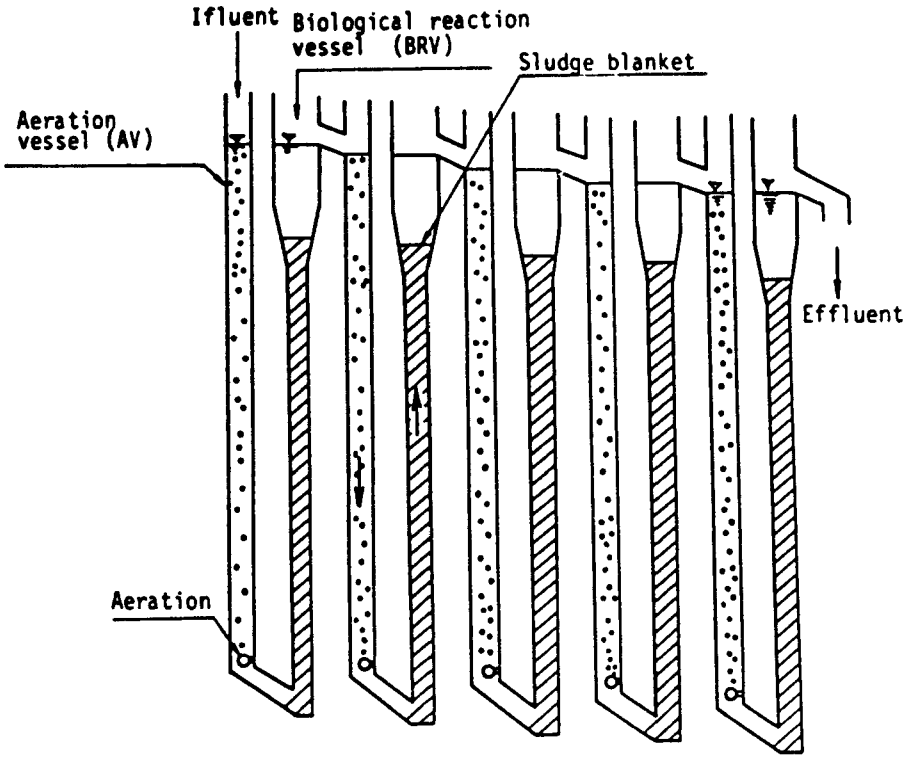


Fig. 1 Flow sheet of MRB reactor (Vertical cross section)

temperature. The inside of the sphere is comprised of completely dark sludge, and thus anaerobic microorganisms, including sulfate reducing bacteria, seem to live there. The surface is mostly covered with a white thin film.

According to optical microscope observations, this film contains *Beggiatoa*, a sort of filamentous bacteria, as the dominant species. *Beggiatoa* is an aerobic bacteria and has the ability to utilize sulfide and often contains many sulfur particles in its cells. Its film looks white, probably because of these sulfur particles. The formation mechanism of granule sludge is unknown at present but, as a hypothesis, sulfur compounds can be considered to participate. In MRB, the oxygen supply and biological reaction are made at different places and thus, in BRV, dissolved oxygen is suddenly consumed and the inside of sludge is always kept anaerobic. Also, sulfide is formed therein by the action of sulfate reducing bacteria. The flock surface has the opportunity to be in contact with oxygen, and then *Beggiatoa*, an aerobic bacteria capable of utilizing sulfide, will multiply there and cover the flock surface.

The MRB reactor was installed in the Kohoku Pilot Plant Facility of the Public Works Research Institute in the Kasumigaura Purification Center, Ibaraki Prefecture, and its steady-state operation started in December 1986 by using settled wastewater. Specifications of the reactor are indicated in Table 3. The water quality of influent and effluent during the steady-state operation is shown in Table 4. Although the influent was weak, the treatment results were found to be very good. At that time MLSS concentration in the

Table 3 Design and operation conditions of MRB reactor with five stages

Total volume: 213 ℓ, BRV volume: 122 ℓ, Flow rate: 1,120 ℓ/day	
Up flow velocity at BRV	: 10 cm/min. (for lower part)
	3 cm/min. (for upper part)
BOD-volumetric ratio	: 0.42 kg-BOD/m^3·day (total volume)
	0.73 kg-BOD/m^3·day (BRV volume)
Wastewater detention time:	4.5 hours based on total volume

Table 4 Treatment performance of MRB

	Water temperature (°C)	BOD (mg/ℓ)	D-BOD (mg/ℓ)	SS (mg/ℓ)	Sulfide (mg/ℓ)
Influent	13∿16	80	50	30	3.9
Effluent	9∿12	14	9	4	0

Table 5 Sample materials and immobilization methods

	Sample materials	Immobilization methods
Nitrifiers	Five kinds of epoxy-based resins	Normal temperature immobilization method, low temperature immobilization method
	Ten kinds of polyethylene glycol(PEG)-based resins	Photo polymerization method, anaerobic polymerization method, potassium persulfate method
	Combination of two kinds of epoxy-based resins and cation exchange resin	Combined immobilization method
Denitrifiers	Epoxy-based resin	Normal temperature immobilization method, low temperature immobilization method

sludge blanket of each BRV was 21,000, 14,600, 8,900, 5,900 and 14,100 mg/ℓ respectively starting from the upstream side, which indicates that sludge with a very high concentration is retained though the influent was weak.

The MRB reactor has the potential for achieving the following items:

a. Sludge with a high concentration can be retained even if wastewater is weak, so that a decrease in treatment time and a decrease in the amount of sludge generated can be expected.

b. Most microorganisms in sludge are anaerobic ones so that there is a possibility of reducing the amount of energy required for aeration. Also, no energy is required for the circulation of water and sludge.

c. Settlability of granule sludge is very good, and SS capturing ability by a series of sludge blankets is high, and thus no settling basin is required.

d. Though anaerobic microorganisms participate in this treatment method, the final upflow reactor process can be operated under a complete aerobic state, and thus the effluent quality is almost equivalent to that of the activated sludge process.

In the future, it will be necessary to study the characteristics of MRB in more detail and to establish its design and its control parameters.

3.3 The Development of Bioreactors for Nitrogen Removal

This study aims at developing bioreactors for efficient and stable nitrogen removal by applying the entrapping method to nitrifiers and denitrifiers. Thus, the development of more efficient and practical entrapping methods and a survey of the reactor performance are being performed in parallel. With respect to the immobilizing method, various comparative studies were made for the materials and immobilizing methods as shown in Table 5. As a result, PEG (polyethylene glycol)-based resins were found to be promising. Some of the PEG-based resins had a considerably high partition coefficient of NH_4. That is, NH_4 ions easily permeated the inside of resins, and this seemed to be suitable for the immobilization of nitrifiers. Nitrifiers were then immobilized using the PEG-based resins, and it was found that the residual activity immediately after immobilization was about 5% based on the rate of respiration and most nitrifiers lost their activity at the time of immobilization. However, pellets, immobilized by proper immobilization agent, indicated high nitrification activity about one month after being cultivated in artificial wastewater containing ammonia. The respiration rate at that time was about 1,200 mg-O_2/ℓ gel·hr (influent, NH_4-N 20 mg/ℓ). In the case of pellets using an epoxy-based resin, no sufficient activity occured because of the restriction imposed by the diffusion velocity of NO^{-2} ions or NO^{-3} ions. From this, it was confirmed again that the diffusion velocity of substraites creates a major problem in the entrapping method. The treatment performance of denitrifiers immobilized by epoxy-based resins indicated an activity lower than that of nitrifiers. Therefore, it is now necessary to seek other materials that can further increase the degree of denitrification activity.

3.4 The Study on Sludge Components

Wastewater sludge is now being utilized effectively as compost for greening and farming, building materials in the form of smelted slag and so forth. And there is a good possibility of creating better products of higher value by the application of biotechnology. For this, it is necessary to study and research the following:

a. An accurate and precise measurement for the qualitative properties of wastewater sludge

b. The proper selection of materials to be recovered, and to review the technical feasibility of their recovery

c. Review of the market value and the economy of the products to be created

Now, mainly the qualitative properties of sludge are being studied. The compounds of about 19 items have been analyzed, mainly with respect to carbohydrate, protein and lipid from primary sludge, excess sludge, and supernatant from anaerobic sludge digestion. The results are summarized below.

a. The carbohydrates in primary sludge contains mostly polysaccharide with a high proportion of cellulose.

b. The carbohydrates in excess sludge has a high proportion of monosaccharide and a low proportion of cellulose.

c. The carbohydrates in anaerobic digested supernatant has a high proportion of refractory sugar, such as lignin.

d. The lipids in primary sludge has a high proportion of free fatty acid.

From the results of the component analysis of wastewater sludge, primary sludge was found to contain a very large amount of cellulose, and so basic experiments were conducted to produce alcohol from the carbohydrates in primary sludge. In the saccharification process, cellulase, β-amylase, and glucoamylase were used, while bread yeast was adopted in the alcohol fermentation process. Ethanol concentration obtained in the alcohol fermentation process was very low: 0.1%. However, after the distillation of alcohol fermented liquid, the alcohol obtained had an ethanol concentration of 15.3%. This yield of 15.3% alcohol was equivalent to 1 mℓ of alcohol per 1 gram of dry sludge.

3.5 Development of Biosensors

Various kinds of sensors are being used for monitoring the water quality of public water bodies and at wastewater treatment facilities. These are physical and electrochemical devices, and their sensitivity, accuracy and stability are not always satisfactory at the present time for measuring particular substances or biochemical characteristics. The biosensors to be developed in this study are based on the properties of microorganisms react-

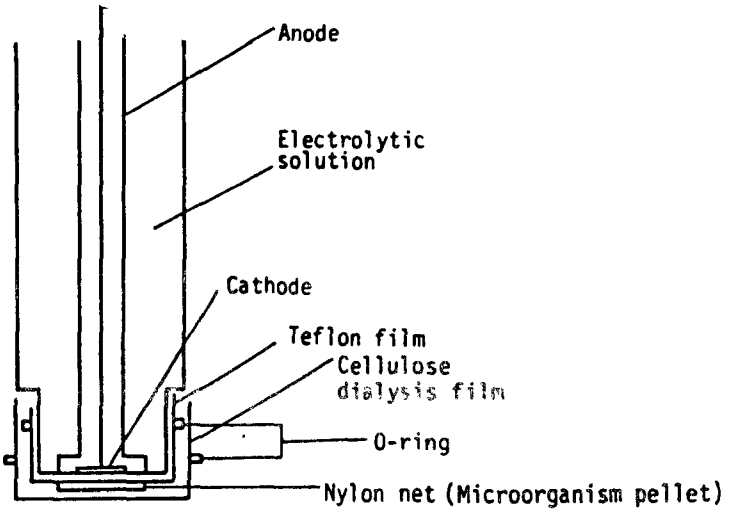


Fig. 2 Construction of detecting portion of sensor

ing to particular substances or to overall environmental conditions, that is, using the response of microorganisms to monitor water quality.

In Biofocus W.T., a BOD sensor, an ammonia sensor and an organic acid sensor are now being developed. Only the development of the BOD sensor will be presented here. A BOD sensor has a film attached to the detecting section of the dissolved oxygen meter, on which aerobic microorganisms are immobilized. In this sensor, aerobic microorganisms consume the dissolved oxygen in response to the concentration of organic substances causing BOD, and thus the BOD concentration is obtained from the concentration of consumed oxygen. The construction of its detecting section is shown in Fig. 2. The microorganisms to be employed in this sensor should have a large oxygen consumption in response to the concentration of organic substances, should respond quickly, and should be able to quickly return to its original state after the supply of organic matter stops.

After evaluating various kinds of microorganisms, the following seven kinds were thought to be promising.

a. *Sacharomyces cereuisiae*

b. *Trichosporan cutaneum*

c. *Candida Tropycalis*

d. Soil bacteria (groups)

e. Wastewater bacteria (groups)

f. Activated sludge bacteria (groups)

g. Activated sludge bacteria (groups): Acclimated sludge in a laboratory

When these microorganisms were immobilized, the activity of the microorganisms had to be retained for a long period of time. The microorganisms stated above and the various immobilizing agents were combined, and their residual activity measured. As a result, the following three combinations were found to be promising:

a. T. cutaneum + Gelatin Glutaric aldehyde-treated film

b. T. cutaneum + Acetyl cellulose-adsorbed film

c. Soil bacteria (groups) + Entrapped film by polyvinyl alcohol

The BOD sensor, based mainly on the microorganisms and immobilization methods selected above, will be produced for trial and put in to practical use in the future.

3.6 The Development of Self-Sustaining Energy Systems

In Biofocus W.T., research efforts are being made to develop several new wastewater treatment systems suited to the previously stated purposes, one of which is the self sustaining energy system.

The conceptional diagram of this system is shown in Fig. 3.

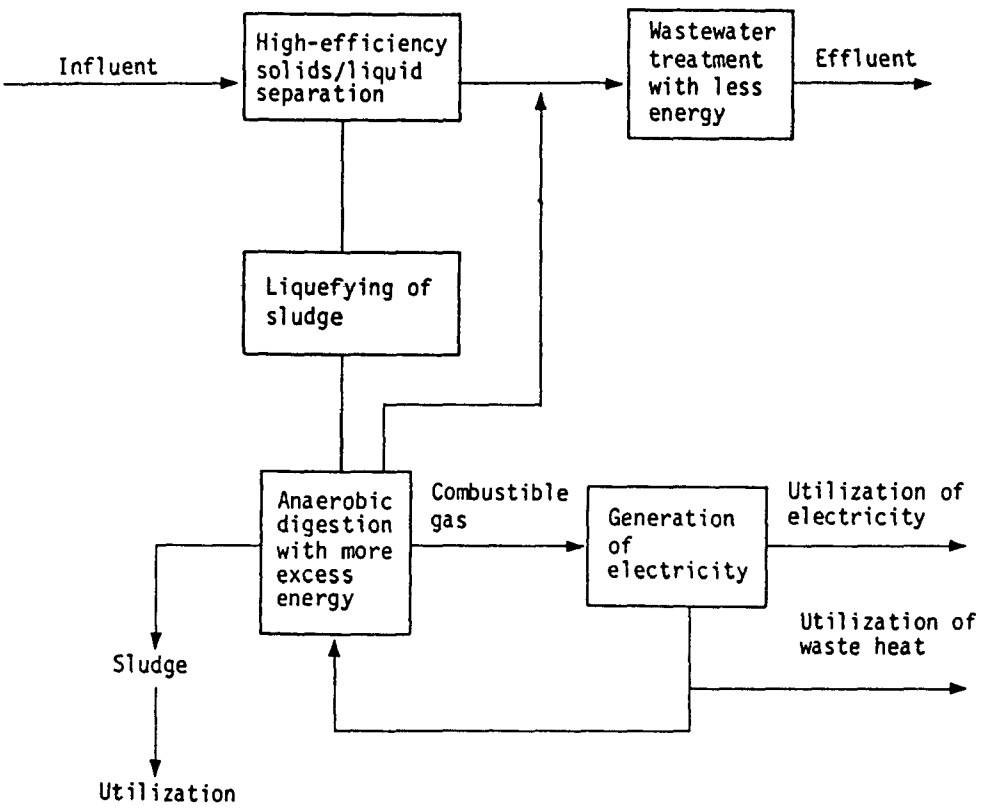


Fig. 3 Self-energy sustaining wastewater treatment system

The total amount of electric power being consumed in wastewater treatment plants in Japan is about three billion kWh per year; This is about 0.54% of the total amount of power consumed in Japan (FY 1983). This value will increase in the future along with the increase in the sewered population and the self-sustaining energy system was proposed mainly to solve this problem. In this system, most of the suspended solids in the influent are removed and loading to secondary treatment is reduced and soluble components are mainly handled there, so that most loading to sludge treatment are the solids in the influent (primary sludge). By reducing influent loading and adopting an energy-saving type of bioreactor in secondary treatment, energy consumption in secondary treatment can be greatly reduced.

On the other hand, in sludge digestion, feed solids have a larger amount of primary sludge which has a higher potential for generating digestion gas than that of excess sludge. Also, an energy-creating type of bioreactor is employed, by which a greater amount of energy can be recovered. A key point to the system's development is the development of a highly efficient solids/liquid separation unit, as well as bioreactors.

4. POSTSCRIPT

Biofocus W.T. is now in its third year of the five-year project, and many of the research and development activities stated above are now in progress. Some of these activities will be fully researched to the point where their achievements may soon be applied in the field by the end of the project, and will greatly contribute to the progress of wastewater works. However, some of these may not be able to reach this level by the end of the project, but will play an important role in fostering new seeds for the technological renovation of wastewater works in the future.

THE NATION'S WATER QUALITY: INTO THE 21st CENTURY

by

Robert J. Blanco
Director
Municipal Facilities Division
Office of Water
U.S. Environmental Protection Agency
Washington, D.C. 20460

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

Prepared for Presentation at:

Eleventh United States/Japan Conference
on Sewage Treatment Technology
Tokyo, Japan

October 12-14, 1987

INTRODUCTION

I am deeply honored to be here with you today. Together we share the responsibility of protecting the health and welfare of the citizens of our countries by preserving and enhancing our nations' water resources. For the United States, one successful tool in our arsenal over the past 15 years has been national water quality legislation, and today I would like to inform you of where we have been with our water quality program, to acquaint you with the Water Quality Act of 1987 and how the Environmental Protection Agency is implementing several of its key provisions, and to propose to you what I believe this new legislation means as we take our water quality campaign through the close of the 20th century and into the 21st.

I would like to begin by providing background information on key events leading up to the Water Quality Act of 1987, including a discussion of the development of water quality programs in the United States.

I will then present specific information about the major provisions of the Water Quality Act. This legislation, passed by the United States Congress in February of this year, consists of amendments to a law which was first passed in 1956. It modifies, rather than replaces, the existing water pollution control legislation. I will also address the general requirements of the Act; but in many cases the details pertaining to how EPA will implement the Act are still pending.

Finally, I will focus on the provision requiring the development of State revolving loan fund programs. The establishment of alternative financing mechanisms at the State level is an issue of particular interest to me. I believe it will prove to be extremely important, not only as a continuing source of revenue for the construction of municipal wastewater treatment facilities, but also as a future means to satisfy national water quality program needs in general.

Background

The U.S. Congress first passed pollution control legislation in 1948, and between 1948 and 1970, eight pollution control laws were enacted which were intended to protect the integrity of the Nation's water resources. Although these early pollution control laws were well intentioned, there were several problems which severely limited their effectiveness.

One problem was that regulation was based solely on in-stream water quality standards. Where a discharger was causing a water quality problem, the burden of proof for enforcement rested solely with the enforcing agency. This meant that extensive water quality monitoring was necessary before any enforcement action could be initiated. At the time, the Federal Government did not have adequate resources to effectively regulate pollutant discharges this way.

A second problem was that authorized funding for constructing municipal wastewater facilities fell far short of needs. Initially, Federal funding was limited to 30% of the total project cost - with a maximum allowable project cost of $250,000. This limitation prevented Federal funds from assisting in major urban areas with large needs and more serious water quality problems.

A third problem involved Federal authority extending only to interstate navigable waters. This meant that the Federal Government had no regulatory powers over a large percentage of the Nation's waters.

A fourth problem, rapid population growth and trends toward urbanization and industrialization limited the effectiveness of early water pollution control legislation. During the 1950's and 1960's, these factors contributed to significantly greater quantities of pollutants being discharged nationwide.

Public support for pollution control legislation increased substantially during the 1960's, and contributed directly to environmental protection as a major political issue in the United States.

By 1972, it was clear that existing legislation was inadequate. Many waterbodies were severely polluted, fish kills were common, accelerated eutrophication was recognized as a problem, and numerous waterbodies were unusable for recreation.

Congress reacted to public pressures by enacting a sweeping re-write of all existing water pollution control legislation. The Federal Water

Federal Water Pollution Control Act Amendments of 1972 made significant changes in the way water pollution was regulated in the United States.

- o It provided for a stronger Federal role, including the extension of regulatory authority to intrastate as well as interstate waters;

- o It called for a combination of water quality-based and effluent-based regulation of pollution, including a national effluent permits program to be administered by EPA, and

- o It created a multi-billion dollar construction grants program to assist municipalities with construction of wastewater treatment works.

The 1972 legislation's major goal was to restore all navigable waters to "fishable and swimmable" condition by 1983. This was to be accomplished by using a two-tiered approach to pollution control. First, the law required that all point source discharges meet minimum treatment requirements such as, technology-based limits. This was described as "best practicable technology" for industrial discharges and secondary treatment as the equivalent for municipal discharges. Second, where technology-based effluent limits were not sufficient to meet stream water quality standards, more stringent water quality-based effluent limits were required.

To ensure that dischargers would meet their effluent requirements, the law provided for a national permits program, the National Pollutant Discharge Elimination System. This program was charged with developing a permit for each discharge, including specific limits on the quantity and/or concentration of each regulated pollutant. EPA was directed to develop the needed numerical effluent limits.

To help municipalities meet the new construction requirements, the law significantly expanded the Federal construction grants program which had been operational since 1956. It included an authorization of $18 billion over 3 years for treatment works construction - or roughly 9 times the total that had been provided over the previous 15 years. Almost any type of wastewater treatment construction project qualified for 75% Federal funding under the program, including the treatment plant itself, sewer rehabilitation, storm sewers, interceptor and collector sewers, control of infiltration and inflow, combined sewer overflows and purchase of land for land treatment systems.

Further legislation passed in 1977 transferred partial authority for the administration of the construction grants program to State governments. This law also made available funds for small community projects, encouraged the development of innovative and alternative technologies, allowed waivers to the secondary treatment requirement to be awarded to qualifying ocean-dumping communities, and authorized an additional $22.5 billion for construction grants over 5 years.

The most significant feature of the 1977 legislation, however, was that it was a first step toward decreasing Federal responsibilities and increasing State responsibilities for day-to-day management of the overall water program.

Increased State responsibilities continued as a result of legislation passed in 1981. This law transferred additional grants program management responsibility to the States, and required that EPA assume a role of program oversight and monitoring. The 1981 law also reduced the annual level of Federal grant funding, targeted funds to current treatment needs and restricted Federal monies to certain types of projects, such as, only treatment, new interceptors and infiltration and inflow correction.

The new Water Quality Act of 1987 made important mid course corrections to existing pollution control legislation and was passed in order to build upon progress achieved since 1972 which include:

- the Construction Grants Program which has assisted in the completion of improvement projects at over 4,600 sewage treatment plants;
- expenditures at the Federal, state and local level which have allowed an increase of population served by secondary treatment by more than 50 million people since 1972;
- State governments successfully assigning designated use and water quality criteria to the waterbodies of the United States;
- more than 60,000 point source dischargers were issued permits;
- substantial improvements in water quality management capability and expertise occurring at the State and local levels of government, and
- water quality improvements despite substantial population growth and industrial development. Although most waterbodies have maintained the same water quality since 1972, over 4 times more stream miles have been improved than have been degraded.

These accomplishments are significant, but major water pollution control problems remain to be solved. The Water Quality Act of 1987 will address such issues as:

- developing innovative methods for financing of municipal facility construction,
- controlling nonpoint source pollution,
- providing adequate environmental protection for estuaries,

- controlling municipal and industrial stormwater discharges,
- identification and control of toxic pollutants, and
- establishing programs, including regulation, for sewage sludge management.

Major Provisions of the Water Quality Act of 1987

Having detailed some history of water pollution control legislation in the United States, I would now like to move on to a discussion of the major features of the Water Quality Act of 1987, and how this legislation will guide our water quality programs into the 21st century. The topics I will cover will include how the Act addresses nonpoint source pollution, the National Estuaries Program, stormwater discharges, toxics, sewage sludge management, the phase-out of the Construction Grants Program and the phase-in of State revolving loan funds.

As I mentioned earlier, I would like to focus on the provision which authorizes Federal capitalization grants for State revolving loan fund programs. Because I will be going over this feature of the Act in greater detail, I will defer the discussion of it until after I have briefly described the other major provisions of the Act.

Nonpoint Source Pollution

Nonpoint source pollution is a major, largely unchecked source of pollution in the United States. In recent years the evidence has been mounting that control of nonpoint sources will require intensive sustained effort from all levels of government. For example, in 1986, thirty-three out of the fifty States reported to EPA that nonpoint source pollution is a major water quality problem. We have also determined that for those waters known to be not supporting their designated uses, such as recreation, swimming or fishing, nonpoint source pollution is the most frequent cause of water quality standards violation.

Management of this problem is complicated by the fact that nonpoint source pollution can assume many forms. Although by definition all nonpoint source pollution is generated through the processes of rainfall and stormwater runoff, each of the different sources, such as agricultural, silvicultural, urban, mining and construction, yields nonpoint source runoff with different chemical constituents and different control requirements.

The most widespread source of nonpoint source pollution is agricultural activities. A 1985 assessment found that out of 165,000 river miles, where uses are impaired or threatened by nonpoint sources, agricultural activity was the primary cause of nonpoint source pollution for 106,000 or 64% of the impacted river miles (Figure 1). This agricultural component of the nonpoint source problem is currently being addressed in most of the States by voluntary programs which provide education and training to farmers. These programs have not been entirely successful, however, especially in

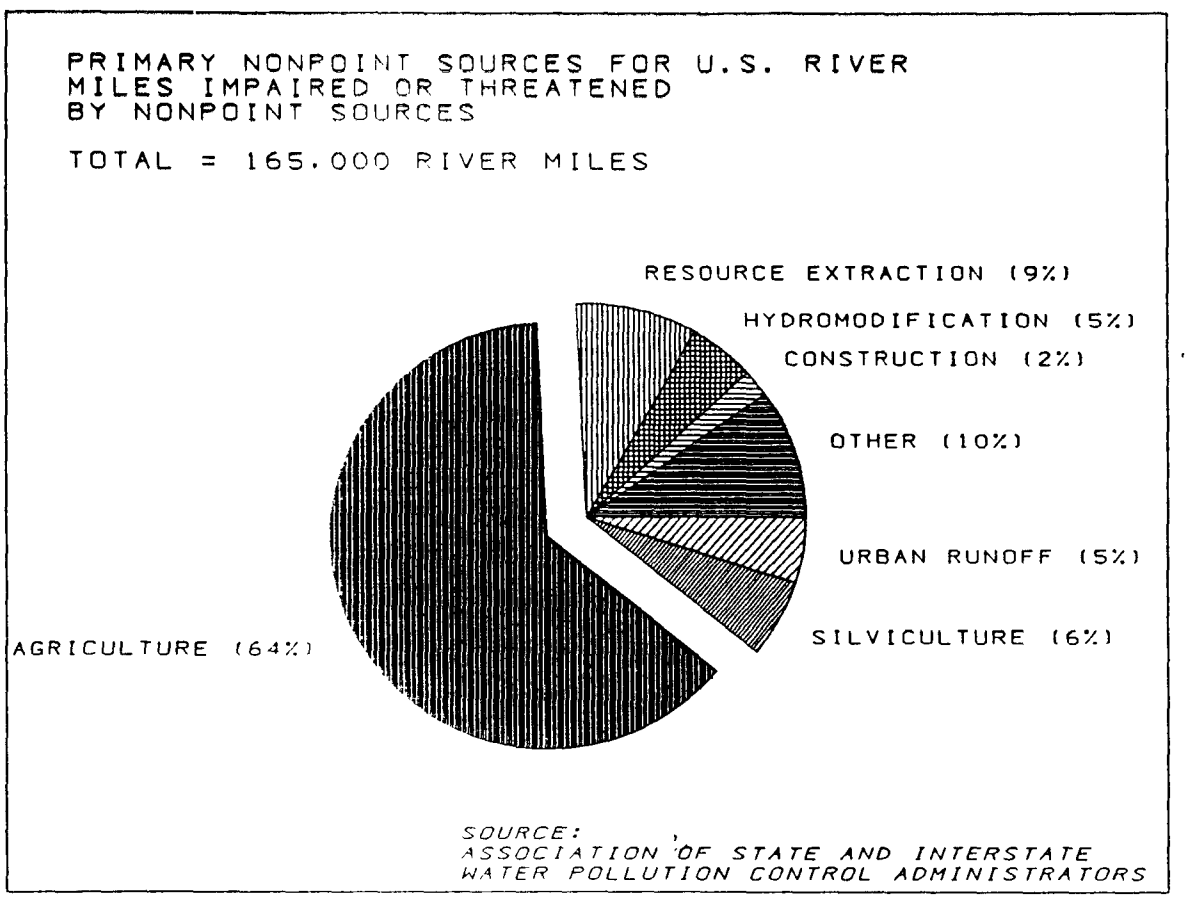


Figure 1. Primary Nonpoint Sources for Impacted Rivers.

cases where controlling nonpoint source pollution is not in the economic self-interest of the farmer.

Although nonpoint source impacts are found throughout the United States, EPA has learned that each case is unique, and that site-specific control plans which reflect local conditions are key to effective abatement. In other words, the point source control strategy of applying uniform technological controls for each class of dischargers is not appropriate for control of nonpoint sources.

As is also true for point sources, costs for control of nonpoint sources are incurred both at the State level, where planning and management programs must be established, and at the local level, where necessary control technologies must be implemented. Unfortunately, expenditures to control nonpoint source pollution at each of these levels of government has been inadequate to date. As a result, State and local programs which specifically address nonpoint sources and provide appropriate technical and financial assistance are often lacking.

The Water Quality Act of 1987 takes a significant first step toward ensuring more effective control of this major water quality problem. The Act addresses problems resulting from nonpoint source pollution by requiring the States to establish nonpoint source pollution management programs. The basis for each program will be an assessment report, to be prepared by the States, which characterizes the extent of problems caused by nonpoint source pollution, identifies the specific nonpoint sources causing the problems, and identifies the mechanism by which each problem is to be solved. After completion of the assessment report, each State is required to establish a nonpoint source pollution management program that will:

- identify measures necessary to control each nonpoint source pollution problem;
- identify the State program which will implement each measure;
- certify that existing State laws provided adequate authority to implement the program;
- identify sources of funding; and
- specify a schedule for implementation.

The Act provides four sources of funds to support the implementation of nonpoint source management programs. First, the Act authorizes $400 million over 4 years for use as grant funds. Second, under certain conditions, States will also be able to issue loans from their revolving loan funds to public agencies, institutions, corporations or individuals to implement nonpoint source management programs. Third, one percent of each State's construction grant allocation or $100,000, whichever is greater, will be provided for nonpoint source management program development. Fourth, each governor has discretionary authority to use up to 20% of his

municipal wastewater treatment construction grant allocation for nonpoint source program implementation or other purposes.

Thus, although these authorizations are small in proportion to the magnitude of the problem and certainly do not match what is authorized for the control of point sources, the Act does take significant strides by giving the States the flexibility to use limited Federal funds to address their most serious water pollution control problems first, and by ensuring more effective management of nonpoint source problems in the United States.

National Estuaries Program

The Act also provides assistance to the National Estuaries Program which was established to promote the management of estuaries of national significance. This provision recognizes the existing estuaries program and authorizes the EPA to expand the program where necessary.

For each estuary included in the program, EPA is responsible for convening a management conference whose function is to develop a comprehensive estuary management plan. The conference consist of representatives from state, interstate or regional agencies, Federal agencies, local governments, affected industries, educational institutions, and the general public. Although State governors can nominate any estuary for the program, the EPA Administrator is authorized to decide which estuaries to include. Funding for the program is authorized at a level of $12 million per year over 5 years.

Stormwater Discharges

Another major feature of the Act establishes requirements and deadlines for the regulation of stormwater discharges. Such regulation is important because stormwater constitutes a significant source of pollution which in many areas of the United States results in serious surface and groundwater quality problems.

Stormwater discharges may contain sediment, organics, heavy metals, inorganics, nutrients, petroleum products, and pesticides. Heavy metals and inorganics are of particular concern because they can accumulate in fish tissue and sediment and thus cause serious long-term impacts on human health and aquatic life.

The regulatory approach Congress has developed relieves many stormwater dischargers of the obligation to obtain a permit until 1992. This will allow available resources to be focused on only the most serious stormwater discharge problems.

The following schedule for developing necessary regulations and issuing permits is included in the law:

- Within 2 years, EPA is to develop regulations concerning permit requirements for industrial and municipal sources serving more than 500,000 people - the so-called large municipal discharges. Within 3 years, applications from industrial and large municipal sources must be filed, and within 4 years, permits are to be issued for these sources.

- Also within 4 years, EPA is to establish regulations for municipal sources serving between 100,000 to 500,000 people - the so-called medium municipal sources. Within five years, applications from the medium municipal sources must be filed and within 6 years, permits are to be issued.

Although meeting this schedule may impose substantial economic impacts on many cities and towns across the country, the water quality benefits will, hopefully, exceed the costs of controlling the stormwater discharges.

Toxics

The identification and control of toxic pollutants in surface water discharges is another major remaining water quality problem. Toxic chemicals can have severe, sometimes irreversible, impacts on human health and the aquatic environment. The control of toxics is complicated by the fact that over 60,000 commercial chemical substances are currently in use in the United States.

Although many individual States now recognize that toxic chemicals pose a serious public health and water quality problem, monitoring of toxics in surface waters is still quite limited. Unfortunately, in many areas of the country, we just do not have the data we need to assess the extent of toxics contamination. Judging the extent of the toxics problem is further complicated by the fact that at present we have a limited scientific basis on which to develop water quality criteria and for many toxic chemicals, no criteria currently exist.

EPA has worked with the States to develop a national surface water toxics control program. A driving force which has helped to shape the program has been that controlling water pollution beyond the technology-based provisions of the Clean Water Act requires an integrated strategy consisting of both biological and chemical methods to address toxic and nonconventional pollutants from point sources. EPA's overall policy in this regard is to ensure a degree of national consistency while preserving sufficient flexibility to deal with specific problems.

The Water Quality Act of 1987 includes requirements for the identification and control of toxic pollutants and provides ambitious goals for the existing surface water toxics control program. The Act's requirements place emphasis on control of the 126 priority pollutants, and place States on an accelerated schedule to achieve compliance. For discussion purposes, it is convenient to separate the Act's requirements into 4 major steps.

Step One: The States are to submit lists of waterbodies to EPA by Feb. 4, 1987. The first of these, a so-called "long list," is to identify all waters where application of technology-based effluent limits is not expected to be sufficient to meet water quality standards. The second list, a so-called "short list," is to identify all waters where application of technology-based effluent limits is not expected to be sufficient to meet water quality standards primarily because of criteria violations resulting from the discharge of priority toxic pollutants from point sources.

Step Two: For each waterbody included on the lists, the States are required to identify all point sources believed to be impairing water quality by discharging toxics. For each of these discharges, the States are also required to determine the amount of each toxic pollutant being discharged.

Step Three: For each waterbody included on the lists, the States are also required to develop an individual toxics control strategy which will impose the necessary toxics effluent limitations on each point source which is impairing water quality by discharging toxics. When combined with existing controls on point and nonpoint sources, the toxics control strategies are intended to ensure that all water quality standards will be met not later than 3 years after the strategies are established.

Step Four: As States review and revise their expiring water quality standards, they are to include criteria for the priority pollutants for which EPA has published criteria. Where EPA has not published criteria, the States are to add numeric criteria based on biological monitoring and assessment techniques. EPA is to publish guidance on using biological monitoring and assessment methods in setting water quality standards.

Several other EPA activities are also being planned in support of these Water Quality Act requirements. One such activity will be to review and assess the adequacy of existing State toxics programs. The assessments will consider all significant aspects, including the completeness of current State water quality standards for toxics, procedures for identifying waterbodies with toxics problems, appropriateness of the States' waste-load allocation procedures, and the ability of State personnel to develop

and implement controls on toxic pollutants. In addition, EPA will assist the States in developing a plan that lists actions to be taken to strengthen their toxic control programs and ensure compliance with the requirements of the Water Quality Act. This plan will be based on findings from the toxics program assessment.

To assist with the development of toxics control strategies at municipal sewage treatment plants, EPA is considering several activities that would yield new and valuable information on control of toxic chemicals. One project would be to further evaluate the toxics-removal capability of existing sewage treatment technologies. Another would be to develop new techniques to enhance the toxics-removal capability of existing systems. The information generated from these efforts could then be used to develop a guidance document or manual to assist municipalities in developing a total system strategy for dealing with toxics problems. The document would include information on options to control the discharge of toxic waste to sewage treatment plants, control techniques and a protocol on how to isolate and identify the toxics problem.

Sewage Sludge Management

Managing municipal sewage sludge is another very important issue addressed by the Water Quality Act. Nearly 7 million dry metric tons of raw municipal sludge are generated annually by the nation's 15,000 sewage treatment plants. Management of this sewage sludge continues to be a difficult problem in the United States, particularly for a number of cities with limited use or disposal methods.

A major barrier to effective management of sludge has been the fragmented and uncoordinated regulatory structure at both the Federal and State levels of government. Because provisions in numerous different major Federal laws have been passed dealing with sludge management, current sludge regulations have not been effectively coordinated.

A complicating factor is that the constituents of sewage sludge are quite variable and thus there are sludge composition differences both between treatment plants and within treatment plants over time. This means that some sludges may be completely harmless while others may be contaminated by high levels of toxic chemicals.

The new sewage sludge provisions of the Water Quality Act requires EPA to undertake a number of sludge-related activities, including some which are already underway.

First, the Water Quality Act requires the identification and regulation of toxic pollutants present in sewage sludge. This regulation will establish numerical limits and specify acceptable management practices, or other alternative standards where necessary, for sewage sludge containing the identified pollutants of concern. EPA anticipates proposing its initial round of technical regulations by late 1987.

Second the Act requires EPA to include requirements implementing the new regulation in permits issued to publicly-owned treatment works or other treatment works treating domestic sewage. These permits are to be issued thru the National Pollutant Discharge Elimination System program, other permit programs authorized under appropriate provisions of applicable Federal laws, or thru State permit programs approved by the EPA Administrator.

Third, the Act requires EPA to issue procedures for approval of State sludge management programs within a fixed time frame. These procedures will include minimum requirements for approvable State programs.

Fourth, compliance with the new sewage sludge regulation is required no later than 1 year after publication, or 2 years if construction is required.

Fifth, the Act authorizes $5 million to conduct scientific studies, demonstration projects, and public information projects designed to promote safe beneficial management or use of sewage sludge. Examples of how this money may be utilized, if appropriated, include supporting the establishment of demonstration projects to improve acceptance of beneficial use practices, supporting long-term existing project evaluations which document beneficial uses and any environmental or public health problems associated with such projects, and working with other Federal agencies and the States in distributing information to farmers, mining companies, and others on beneficial uses of sludge.

Although the requirements of the law with regard to sludge management are relatively clear-cut, there are several key implementation issues which still need to be addressed. For instance, what will be the impact of the new sludge regulations on communities which are currently making beneficial use of their sludge? Also, should permits be written so as to hold the generator responsible for the ultimate disposal of the sludge? EPA will be working toward developing answers for these questions so that national implementation of the Water Quality Act can proceed smoothly and effectively.

Implementation of the State Revolving Fund Program

Earlier, I mentioned that a major feature of the Water Quality Act of 1987 is the State Water Pollution Control Revolving Fund. I would now like to spend some time discussing the details of this program and how it will be implemented in place of the Construction Grants Program.

The basic concept of the State Revolving Fund allows the Federal Government to award initial grant money to the States for use as capital in instituting a water pollution control revolving loan fund. These capitalization grants will be awarded over a period of several years. Each State will use its revolving fund primarily to make loans for local construction projects. The repayments of principal and interest from these loans will be used to replenish the fund. During the implementation of the State Revolving Fund, the Federal Construction Grants Program will be phased-out.

There are three fundamental differences between the Construction Grants Program and the State Revolving Fund Program that I would like to point out. First, under the State Revolving Fund Program, financial assistance for construction is in the form of up to 100 percent loans rather than partial grants; second, the fund is renewable; and third, the fund is administered by each State rather than by the Federal Government.

State administration of the fund is a very significant feature. For several years now, there has been a general trend in water pollution abatement away from centrally organized government and toward greater State government control. This movement began in 1977 with the delegation of certain program management functions to the States under the Clean Water Act even though ultimate control over funding still rested at the Federal level.

The phase-out of the Construction Grants Program and the establishment of the State Revolving Fund Program under the Water Quality Act marks the advent of a new phase of the Federal-State relationship in wastewater treatment works construction.

In the future, the States will have the lead role in distributing funds for construction projects. The Environmental Protection Agency's role will be to oversee the implementation of the new program to assure that environmental objectives and the goals of the Act are achieved.

Funding

Through the new Water Quality Act, the United States Congress has authorized the use of 18 billion dollars nationwide for assistance in

construction of wastewater treatment plants from 1986 through 1994. Federal construction grants of up to 9.6 billion dollars can be awarded until the end of 1990, at which time the Construction Grants Program will cease to exist. A portion of these funds may be converted by the States for use in their new revolving loan programs.

Beginning in 1989, the Federal Government will award capitalization grants to the States for instituting their revolving funds. Up to 8.4 billion dollars will be awarded to the States until the end of 1994. After that time, the States will have to rely on repayments and interest from the loan program and non-Federal sources of money for continuation of their municipal treatment works construction programs.

From 1987 through 1990 there will be a transition from the Construction Grants Program to the State Revolving Fund Program. During this time, the States will have some flexibility in determining how much of their annual allotment of Federal money they will use to capitalize their revolving funds. In 1987 and 1988, all Federal monies are authorized for the Construction Grants Program, and a State is not required to apply any of its allotment to its revolving fund. In each of these years, a State may choose to transfer a portion of its construction grants allotment to its revolving fund.

In 1989 and 1990, a state must use at least half of its annual allotment for capitalization of its revolving fund. The remaining half is authorized for use as construction grants, but a State has the option of transferring up to 100 percent of its annual allotment of funds to its revolving fund.

During the implementation of the State Revolving Fund Program, the Federal Government will make capitalization grants to the States in quarterly installments according to a payment schedule agreed upon by the Environmental Protection Agency and the individual States. For each Federal capitalization grant awarded, a State must agree to contribute matching funds in an amount not less than 20 percent of the Federal contribution.

After the initial capitalization of the fund by the Federal Government and the States, money for the revolving fund will come from non-Federal sources, including repayments of principal and interest on loans, proceeds from investment of the capitalization grants, and State contributions in excess of the minimum 20 percent match.

Use of Funds

When a State applies for a Federal capitalization grant to establish a revolving loan fund, it must agree to use the funds in accordance with the Water Quality Act. The State must agree to use the capitalization grant plus State match money expeditiously. The State must also agree to enter into binding commitments to provide financial assistance from the revolving fund in an amount equal to each quarterly grant payment, plus State match,

within one year after receipt of the payment.

The primary purpose of the State Revolving Fund Program is to provide States with a source of capital for making loans to municipalities for construction of wastewater treatment works projects. A State can make loans to municipalities at interest rates ranging from zero up to the prevailing market rate, and determination of interest rates charged is entirely up to the State. Repayment of the principal and interest on loans is required at least annually beginning not later than one year after the date of completion of construction. Loans must be fully repaid within 20 years after completion of construction.

In addition to making loans for new construction, a State can use its revolving fund for refinancing local debt incurred for the purpose of constructing publicly owned treatment works. This feature is designed to encourage municipalities to seek financing from a local lender prior to the availability of a loan from the State Revolving Fund. The municipality can thus proceed with needed construction and still have the opportunity to receive financing from the revolving fund at a later date at or below the market interest rate.

Monies from a State's revolving fund can also be used for eligible programs and projects other than treatment works projects, as I will discuss shortly, but certain restrictions apply to these other uses.

While the State Revolving Fund is intended to be used primarily for making loans, not all of the monies in the fund are strictly limited to that use. For instance, a State may choose to increase the power of its revolving fund by using the resources from the fund for investment purposes and depositing the proceeds back into the fund. A State may also use a specified percentage of all capitalization grants for covering administrative costs. In addition, the Water Quality Act continues a requirement of the Construction Grants Program that each State set aside a specified portion of its annual allotment of Federal funds for water quality management planning. Under the new Act, this set aside includes estuaries and nonpoint source management.

Eligible Projects

As I mentioned a moment ago, there are some projects and programs other than treatment works projects which are eligible to receive funds from the State Revolving Fund.

A State may use its revolving fund for providing financial assistance to any municipality, intermunicipal, interstate or State agency for construction of publicly owned treatment works. This means that State revolving fund monies can be used to make loans for construction of facilities for treatment of municipal sewage and industrial wastes. Also eligible for loans from State revolving funds are projects that are designed to correct water quality problems created by overflow of combined stormwater and

sanitary sewer systems if these combined systems are considered to be causing priority water quality problems.

The types of construction projects I have just mentioned are currently eligible for grant assistance, so in that respect the Revolving Fund Program is no different from the Construction Grants Program. But, in addition to funding these types of treatment works projects, a State may use its revolving fund to make loans to projects that treat stormwater runoff. Assistance for stormwater projects is not allowed under the Construction Grants Program.

Earlier, when I spoke about some of the major features of the Water Quality Act, I discussed sections of the Act that require States to develop and implement nonpoint source management programs, including groundwater protection, and estuaries conservation and management plans. The Act allows States to use revolving fund monies, within certain restrictions, to provide assistance for these programs in addition to the funds that must be reserved for water quality management planning.

Funds for implementation of a nonpoint source program can be provided to State agencies for research, planning, enforcement, demonstration projects and other aspects of a comprehensive management program. A State can provide assistance to other State agencies or public or private organizations for technical work in developing an estuaries conservation and management plan and to State agencies for implementing an estuaries program.

Restrictions on Funding

The Water Quality Act places restrictions on how and when certain types of funds from the State Revolving Fund can be used. The Act requires that all funds in the State Revolving Fund as a result of capitalization grants will first be used to assure maintenance of progress towards compliance with enforceable deadlines, goals and requirements of the Clean Water Act.

The municipal compliance deadline of July 1, 1988 requires that all municipal dischargers to United States waters be able to demonstrate that they are making progress toward compliance with the enforceable requirements of the Clean Water Act. According to the legislative history of the Clean Water Act, progress toward compliance with enforceable deadlines, goals and requirements of the Act may be assured through a funding commitment or through establishment of an enforceable compliance schedule. Enforceable requirements will have been fulfilled when all non-compliant facilities are on enforceable schedules, or have had enforcement actions filed against them, or have received funding commitments.

The enforceable requirements restriction on the use of funds from the State Revolving Fund effectively means that only after all treatment works in a State are in compliance or on an enforceable schedule can the State use its revolving fund monies resulting from capitalization grants for any other treatment works projects or for other eligible programs or projects.

Because the enforceable requirements restriction is such an important factor in the use of the State Revolving Fund, a key issue has been whether to retain the current Construction Grants regulatory definition of enforceable requirements, or to narrow or broaden the scope of the interpretation in accordance with the new Act. The broadest interpretation would allow States to include all potential sources of water quality degradation. The advantage of retaining the current, somewhat flexible, regulatory definition is that the Environmental Protection Agency and States have had experience with it, and it reinforces the Congressional intent of the Clean Water Act.

Another issue deals with determining the portion of funds in the State Revolving Fund to which the enforceable requirements restriction applies. At a minimum, the portion of funds "as a result of capitalization grants" includes the Federal capitalization grant and 20 percent State match, and all principal and interest repayments on the capitalization grants plus State match. Some or all of the proceeds from investment of these funds may also be subject to this restriction. Funds as a result of capitalization grants do not include State money in excess of the 20 percent match or any repayments of principal and interest on that money.

The Federal-State Relationship

As I discussed earlier, implementation of the State Revolving Fund Program and the phase-out of the Construction Grants Program are moving the United States into a new phase of the Federal/State relationship in water pollution control.

State Responsibilities

The Water Quality Act declares that water pollution control revolving loan funds shall be administered by an instrument of the State subject to the requirements of the Act. This means that each State has a general responsibility for administering its revolving fund, and must take on certain specific responsibilities in carrying out its administrative duties.

As I have already explained, the State is responsible for jointly establishing a payment schedule with the Federal Government, contributing 20 percent matching funds, committing funds expeditiously, reserving funds for water quality management planning, and ensuring that enforceable requirements are fulfilled before assistance is awarded for other purposes. All of these responsibilities are included as conditions in the State's grant agreement with the Federal Government. A State, as a Federal grant recipient, must also agree to follow established accounting and auditing procedures and must require loan recipients to follow certain similar procedures.

The Water Quality Act also requires each State to conduct an environmental review on any project constructed with funds directly made available by capitalization grants. The Act gives the Environmental Protection Agency the authority to specify additional grant conditions, and the Agency intends to require each State to assure that all treatment works constructed with funds from a State's revolving fund will be subject to environmental review.

Among the most important State responsibilities is a requirement that each State submit a plan detailing how it intends to use its revolving fund, and that it submit an annual report describing how the fund actually operated. These documents, and other records, will form the basis for an annual review of the State revolving fund program by the Environmental Protection Agency.

An intended use plan must be submitted by a State as a part of its application for a capitalization grant. For each annual capitalization grant, the plan must include a list of the projects that are eligible for assistance from the State's revolving fund. These projects must also appear somewhere on the State's Construction Grants priority list, which ranks projects in priority order according to their expected water quality and public health benefits. The intended use plan must also include a list of any nonpoint source and estuary projects eligible for assistance from the revolving fund, but these projects do not appear on the priority list.

The plan must also describe the long and short term goals of the

State's revolving fund program, and provide information on activities to be supported by the program, such as a description of the project categories and the terms of financial assistance. The State must describe how its revolving fund program will fulfill the requirements of the Act related to timing and use of funds and must provide information on the criteria it will use for distribution of funds. Before submitting the final intended use plan to the Environmental Protection Agency, the State must provide opportunity for public comment and review.

Each State must submit an annual report at the end of the fiscal year which updates the intended use plan and details the dates, amounts, recipients and conditions of loans and other types of financial assistance from the State's revolving fund. This requirement only applies as long as the State is still receiving Federal capitalization grants or is using the portion of funds resulting from capitalization grants. Limited information may be required thereafter.

The Federal Role

As I have stated, the water pollution control revolving fund concept is intended to be a State administered program, but the Federal Government will maintain an oversight role during its implementation. During the period when Federal capitalization grants are being awarded and used, the Agency will have an active role in the administration of each State's revolving fund at the program level. The Agency will not have a role in project level management, but may review selected projects for the purpose of evaluating program performance. The Agency will also be responsible for collecting information to be used in evaluating program results. When all capitalization grants or projects funded by capitalization grants have been closed out, the Federal role will be reduced to the point where the Environmental Protection Agency will only be responsible for limited information collection activities.

During the period of active Federal involvement in the State Revolving Fund Program, the Environmental Protection Agency is responsible for program level review of each State's compliance with its capitalization grant agreement and the overall requirements of the program. The Agency will review annually each State's intended use plan and annual report according to terms specified in the capitalization grant agreement.

In addition to its administrative responsibilities, the Agency has limited power to take remedial actions against the States. The Agency cannot levy penalties against States, but it can withhold or recover funds in certain situations. If the Agency finds that a State has failed to satisfy the terms of its capitalization grant agreement or other requirements of the State Revolving Fund Program, the Agency will notify the State and suggest actions to be taken to correct the non-compliance problem. The Agency can withhold payment to the State's revolving fund if corrective action is not initiated within 60 days. Also, if the Agency finds that capitalization grant funds, or funds as a result of the capitalization grant, were subjected to waste, fraud or abuse, it may recover grant funds or program management funds from the State.

The Agency's information collection activities will occur primarily at the program level through the State intended use plan and annual report. The Agency will also be responsible for collecting a limited amount of project-level information from the States, which will be used to assess program progress.

Direct project-level involvement by the Agency will be limited to the review of selected projects at the time of review of the annual report to assure program compliance, and review of projects at any time as part of an

investigation into allegations of waste, fraud, or abuse of a State program grant.

Once the State Revolving Fund Program was established by the Water Quality Act, the first responsibility of the Environmental Protection Agency was to develop guidelines for implementation of the program. The Agency has encountered several issues during the process of developing these guidelines.

One question has concerned the manner in which the Agency would issue the program implementation guidelines. The State Revolving Fund Program could be implemented through regulations, through guidance, or through some combination of the two. The Agency has taken the position that the program should be implemented primarily through guidance, with a limited set of regulations to deal with complex and ill-defined issues. Although regulations provide a firmer interpretation of the complexities of the legislation than guidance, they also require more time to develop. Because States have the option of instituting a revolving fund in the 1987 fiscal year, it is important that guidelines be established quickly. Also, because the revolving fund is to be a State run program, the use of guidance allows the States greater flexibility in implementing the program than regulations do.

Another issue concerns how the Agency plans to oversee maintenance of progress of projects toward compliance with enforceable deadlines, goals, and requirements of the Clean Water Act. The Agency has made a preliminary recommendation that States submit a list of all permitted facilities and their compliance status. The list would be submitted prior to the first year that projects which do not have enforceable requirements are to be funded. If any projects on the list are in non-compliance, they must be funded during the term of the capitalization grant.

A third issue has addressed the extent to which the Agency should be involved in ensuring compliance with project environmental review requirements of the National Environmental Policy Act. The Agency's preferred approach is to allow States to adopt regulations and procedures adequate to fulfill environmental review requirements and to apply them to all projects funded from the revolving fund. The State regulations and procedures would be subject to Agency approval, and compliance with them would be reviewed annually. This approach would minimize Federal involvement at the project level with a State's revolving fund program.

Summary

I introduced this discussion of the State Revolving Fund by emphasizing the increased authority of the State role and the less active role of the Environmental Protection Agency in operating the municipal wastewater facilities construction program. It is clearly the intent of the Water Quality Act that States administer the State Revolving Fund Program, and it is being carefully implemented with that in mind. Although the roles of Federal and State governments are shifting, this does not mean that the States and the Federal Government will act as discrete and non-communica-

tive entities in solving municipal water pollution control problems. The Federal and State partnership in municipal treatment works construction must continue if we are to achieve the national goals of the Clean Water Act. That partnership will remain unyielding under the new Water Quality Act through the next decade and into the next century to come.

Public Involvement in Environmental Programs

I have stressed the point that the relationship between the Federal and State governments in water pollution control must be a strong one. But the relationship extends beyond that - to all levels of the American public from local governments to public interest groups to private citizens. Through the democratic process, the American public has the power and responsibility to guide the Nation's environmental policy, of which it is both benefactor and beneficiary. Citizens influence environmental legislation by voicing their opinions to their representatives in State legislatures as well as in Congress. They influence implementation of environmental programs by openly debating the issues in public forums as individuals or groups representing diverse interests.

The citizens of the United States are responsible for the environmental legislation and programs that have helped the Nation establish its high environmental standards. That public involvement, confidence, and trust must continue so that the United States can move forward in reaching for these environmental values. To ensure the integrity of our environmental ethic, citizens must be educated in environmental issues and programs, and must continue to have the opportunity to express their opinions in the public arena.

Closing Remarks

Over the past 15 years, we in the United States have worked very hard to improve the quality of our rivers, lakes, bays, and estuaries. Our lawmakers have enacted tough and aggressive legislation with ambitious goals to spearhead our water cleanup efforts. The Environmental Protection Agency and the States have worked diligently to implement these laws. We have established water quality criteria and standards for our waterbodies based on the objectives of the laws. We have established maximum allowable levels of pollutant discharges to meet the standards we set and have issued permits to industries and municipalities that discharge pollutants to our waters. We have required industries and municipalities to meet their effluent limitations and have taken enforcement actions and assessed penalties against them when necessary. To assist municipalities in meeting their wastewater treatment needs, we have provided Federal construction grants.

Municipal wastewater treatment has been a focal point of our clean-up efforts. The Construction Grants Program has been a massive effort, providing about 50 billion dollars in Federal assistance to build thousands of municipal treatment works projects. We have coordinated the grants program with a comprehensive permit and enforcement program requiring all municipal dischargers to achieve compliance with their permitted discharge limits, and requiring secondary levels of treatment or better. The majority of the larger and more significant dischargers are now in compliance.

We have achieved much success through our efforts. We read and hear about numerous success stories of near-dead waterbodies that have been resurrected in the past fifteen years. Our waters have become more aesthetically pleasing. The number of waterbodies available for recreational opportunities, such as fishing and swimming, has been increasing. Improved water quality has also increased the availability of waters for other beneficial uses, such as drinking water supplies and commercial uses, at greater convenience and less cost.

We recognize and appreciate the great strides we have made in improving water quality in the United States, but there is little time to revel in our accomplishments. Water quality problems continue in our Nation, and we need to move on to the next phase of our clean-up efforts. We still need to address a severe nonpoint source pollution problem, and although we have achieved respectable control of conventional pollutants, we have a lot of work to do in the toxics area.

The U.S. Congress enacted the new Water Quality Act to usher in the next era in water pollution control. The new law has many salient features. It addresses the nonpoint source and toxics problems, as well as other

issues which in the past may not have received the attention they deserve, such as estuaries conservation, sewage sludge management and treatment of stormwater discharges. Perhaps the most prominent aspect of the Water Quality Act is the revolving loan fund program which will continue the Nation's efforts in municipal wastewater treatment works construction. This time around, the program will be administered by the States, with limited Federal funding support.

The current Water Quality Act has traveled a rocky road in Congress to reach its current position. The merits of this Act have been hotly debated by our Nation's lawmakers, and I think the Nation has benefitted from the adversity this law has faced. Although it was a long time in coming, the Water Quality Act is a strong law and it will likely serve the Nation's water quality needs well.

Many new and complex issues emerge through the Water Quality Act, and the Environmental Protection Agency has a full agenda ahead of it if the Act's many objectives are to be realized. We at the Agency see these objectives not as problems to solve, but as challenges to meet, and we are freshly motivated to rise to the occasion.

To meet them, we must contemplate and accept our new role, and work to support the States in their new one. This is especially true of the revolving loan fund program. We are depending on our States to manage and implement their revolving fund programs with efficiency and integrity to achieve the greatest possible water quality benefits, and we have every reason to believe they will.

There is enormous environmental benefit to be gained from this program, and the States have responded very favorably to it. In this new era of increasing Federal and State cooperation, the States will be able to plan their water quality improvement projects based on their knowledge of the financial system within which they must live. States will have maximum flexibility to design their water pollution control programs to meet their individual needs, while the Environmental Protection Agency's role will be one of oversight, ensuring that Congressional intent with the Water Quality Act is being carried out properly and effectively.

As we enter this new phase of the water quality improvement effort in the U.S., we are looking forward with renewed energy and commitment to the tremendous challenges before us. We know that further successes in protecting and enhancing our precious water resources are within sight.

SLUDGE MANAGEMENT AND ENERGY PRODUCTION AT LOS ANGELES COUNTY SANITATION DISTRICTS

by

James F. Stahl
Robert W. Horvath
Los Angeles County Sanitation Districts
Whittier, California 90607, U.S.A.

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

Prepared for Presentation at:

Eleventh United States/Japan Conference
on Sewage Treatment Technology
Tokyo, Japan

October 12-14, 1987

SLUDGE MANAGEMENT AND ENERGY PRODUCTION AT LOS ANGELES COUNTY SANITATION DISTRICTS

by

James F. Stahl
Robert W. Horvath
Los Angeles County Sanitation Districts
Whittier, California 90607, U.S.A.

1. INTRODUCTION

The County Sanitation Districts of Los Angeles County operate a regional sewerage system. A network of wastewater collection, treatment, reclamation and disposal facilities serve all or portions of 76 cities with a combined population of approximately 4 million people. The service area generally includes those areas of the county outside the City of Los Angeles, which maintains a separate system of about equal size. This paper describes the Sanitation Districts' sewage sludge management operations, with particular emphasis on the energy production aspects. Districts' facilities described below are located as shown on Figure 1.

The Districts' sewerage system collects and treats approximately 22 m^3/s (500 mgd) of combined residential, industrial and commercial wastewaters. The largest treatment plant is the Joint Water Pollution Control Plant (JWPCP), located in the City of Carson, where approximately 15.8 m^3/s (360 mgd) of sewage receives advanced primary and partial secondary treatment prior to disposal via deep ocean outfalls. There are also five inland water reclamation plants with a combined capacity of approximately 6.6 m^3/s (150 mgd) which provide tertiary treatment and produce effluent suitable for unrestricted recreational contact and a variety of other uses including agricultural and landscape irrigation, industrial reuse, and groundwater replenishment. Each of these five inland plants intercept only a part of the flow at each location, with the remainder flowing to the JWPCP. Solids from these plants are also returned to the sewer and carried by sewage to the JWPCP, where they are separated and processed in centralized sludge processing facilities serving the six plant system.

In outlying areas of the County, the Districts operate five smaller treatment plants ranging in size from 0.01 m^3/s (0.2 mgd) to 0.33 m^3/s (7.5 mgd). Two of these plants (Saugus and Valencia) are also interconnected for solids handling, while sludge from the smallest is truck-hauled to the main sewerage system described previously.

The Districts also operate disposal facilities for municipal solid waste. Four active landfills receive 20,000 metric tons (22,000 U.S. tons) per day of solid waste. The largest, at Puente Hills, is the disposal site for a significant portion of dewatered sludge from the JWPCP.

2. OVERVIEW OF SLUDGE MANAGEMENT FACILITIES

At the JWPCP, all wastewater first undergoes primary settling, aided by addition of anionic polymers, which results in over 80% removal of suspended solids. About 60% of the primary effluent receives further treatment by pure oxygen activated sludge. See Figure 2 for a schematic diagram of the JWPCP and Figure 3 for an aerial view of the plant. The waste activated sludge is thickened by dissolved air flotation. All sludge is anaerobically digested and dewatered by centrifuges. A portion of the dewatered sludge is composted and sold to a private company which produces bagged soil amendment products. The remainder is hauled to Puente Hills Landfill where it is co-disposed with municipal refuse. In the future, most of the sludge which is now landfilled will be processed by Carver-Greenfield dehydration and incineration, with ash disposal to landfill. Details of the sludge processing, energy recovery, and air pollution control facilities are described in the remainder of this paper.

The Districts' four treatment plants in the desert area of the county have their own sludge management facilities. The 0.22 m^3/s (5 mgd) Saugus Water Reclamation Plant (W.R.P.) and the 0.33 m^3/s (7.5 mgd) Valencia W.R.P. were recently interconnected to form a new small regional system. Sludge from the Saugus W.R.P. is transported by force main to the Valencia W.R.P. where the combined sludges are anaerobically digested. The digested sludge is land-spread for agricultural use at the Peter Pitchess Honor Rancho, a detention facility operated by the County Sheriff's Department. In the coming year, construction of a filter press dewatering facility will be completed so that sludge may also be landfilled when needed. The Lancaster and Palmdale Water Reclamation Plants are two similar facilities in the high desert which employ primary settling and secondary treatment by oxidation ponds. Sludge from the primary tanks is anaerobically digested, air dried in shallow drying beds, and stockpiled for eventual give-away as a soil amendment.

3. ANAEROBIC DIGESTION AND GAS UTILIZATION AT JWPCP

3.1 Digestion System

All primary and thickened waste activated sludge produced at the JWPCP is anaerobically digested. The purposes of the digestion system include reduction of the amount of solids to be disposed, stabilization, pathogen reduction, and the production of gas for energy recovery. Electrical production from digester gas normally meets the entire power needs at the JWPCP via the Total Energy Facilities (described below). Digester gas is also used to fuel reciprocating engines for two stations pumping primary

effluent to the ocean and to the secondary treatment facilities. Thus, reliability of gas production is critical to operations at the JWPCP.

There are two relatively independent digestion systems at the plant. Digestion System No. 1 consists of 24 rectangular digesters placed into service between 1950-69, each with a capacity of 2800 m^3 (100,000 ft^3), plus one recently constructed circular digester with a capacity of 14,000 m^3 (500,000 ft^3). Digestion System No. 2 consists of 12 circular digesters placed into service between 1971-83, each with a capacity of 14,000 m^3 (500,000 ft^3). Maximum capacity is thus 250,000 m^3 (8,900,000 ft^3). In practice, there are generally one or more digesters out of service for cleaning, and only 90-95% of the operating volume is considered active because of the buildup of inert solids between cleanings. The active volume provides a detention time of 14-18 days.

The digesters are continually mixed by gas recirculation through multiple internal draft tubes. They are heated by direct steam injection into the sludge as it rises through a draft tube, and operated in the mesophilic temperature range at about 35° C (95° F). The digesters are automatically fed on an incremental basis throughout each day; for example, the circular digesters sequentially receive an equal volume feeding of sludge about ten times per day. All digesters have a fixed roof, with gravity overflows to maintain a constant level, and a gas space of about 1 m (3 ft).

Solids loading to the digestion system is approximately 550 metric tons (600 U.S. tons) per day. Of this total, about 74% is volatile, and the digesters destroy 50% of the volatile solids (or 37% of the total solids). Digester gas production is about 9,000 Nm^3/h (8 million scfd), with a higher heating value of 24 MJ/m^3 (640 BTU/scf).

3.2 Total Energy Facility

Prior to construction of the Total Energy Facility (TEF) in 1985, digester gas was used to operate reciprocating engine generator sets, reciprocating engine pump drives, and fire tube boilers for digester heating at the JWPCP. About 2 MW of power were generated onsite with the balance purchased from Southern California Edison Company to supply JWPCP's needs. Approximately half of the JWPCP gas production was beneficially used onsite, with the remainder sold to an adjacent oil refinery at a nominal charge.

Upon startup of partial secondary treatment in 1983, electrical demand increased from 5 MW to 11 MW. The TEF was designed to serve this power need and the heating needs of the JWPCP using a centralized, highly efficient cogeneration plant. TEF is a combined cycle power plant employing both gas and steam turbines with digester gas as the fuel.

The facility uses up to 90 percent of the entire digester gas production and produces 15 MW of electricity and 16,000 kg/h (35,000 lb/h) of 240 kPa (35 psia) extraction steam, while producing less than 20 percent

of the previous emission contaminants, principally oxides of nitrogen. Digester gas meets the needs for all plant power, digester heating, and primary effluent pumping, and a surplus of power is now generated for sale to Southern California Edison Company.

Figure 4 is a process flow diagram for the TEF. Gas is drawn from the digesters which are operated at 5 kPa (0.7 psi) above atmospheric pressure. The gas is treated in venturi scrubbers for particulate removal. Although there is currently no chemical treatment of the digester gas, hydrogen sulfide concentration in the digester gas is controlled by ferrous chloride addition to the digesters in order to meet sulfur emission standards. This method is capable of reducing H_2S concentration in the gas from several thousand ppm to an average of about 150 ppm. The Districts' research staff is also investigating ways to directly treat the gas for additional H_2S removal if sulfur emissions must be further reduced. Also under investigation is the need and means to remove heavier hydrocarbons which can cause fouling of compressor valves and turbine fuel nozzles.

The treated gas is compressed to 2240 kPa (325 psia) by three-stage variable capacity reciprocating compressors, and is super-heated for delivery to the gas turbines. The gas turbines are three 7.5 MW Solar MARS combustion turbines, two of which normally operate with the third as standby. Each can combust up to 3500 Nm^3/h (2200 scfm) of digester gas with a higher heating value of 24 MJ/m^3 (640 BTU/cf). Natural gas can also be used as a supplemental fuel, but normally is not needed to operate the turbines to their thermal capacity due to an adequate quantity of digester gas. The turbines retard NO_x formation by maintaining very short residence times at the temperatures that NO_x is formed.

The gas turbines exhaust at 400°C (760°F) to dual pressure stainless steel steam generators, which act as boilers. High and low pressure steam are generated at 3000 kPa (430 psia) and 280 kPa (40 psia), respectively. The steam expands through the steam turbine to generate up to 4 MW of power (Figure 5). The steam turbine is a condensing type with heat rejected to cooling water, which can be used to heat digesters via spiral heat exchangers. Use of this low level heat can increase the cycle fuel use efficiency to over 60 percent, compared to 30-42% for conventional power plants. The steam turbine also includes an extraction port which can supply steam for digester heating and building heating and cooling. Construction of the Total Energy Facility was completed in 1986 at a cost of $45 million.

3.3 Pumping Facilities

Ten percent of JWPCP's digester gas is used at two major pump stations at the JWPCP. The Primary Effluent Pump Station consists of five pumps each with a nominal capacity of 5.4 m^3/s (125 mgd) at 10 m (33 ft) of head, providing capacity to pump the entire plant flow to the ocean outfalls about 10 km (6 mi) distant. Since the inception of partial secondary treatment, only part of this capacity is normally in use for pumping primary effluent to the ocean. The Secondary Influent Pump Station pumps

the remaining primary effluent, about 60% of the plant flow, to the secondary treatment facilities. It consists of three pumps with a nominal capacity of 5.94 m^3/s (135 mgd) at 9.5 m (31 ft) of head.

All eight pumps are vertical, mixed-flow pumps driven through right angle gears by Ingersol-Rand 10 cylinder, naturally aspirated, four cycle engines rated nominally at 820 kW. The diurnal flow results in variable engine speeds and loadings ranging from 200 rpm and 185 kW to a maximum of 360 rpm and 750 kW, with an average power output of 2.0 MW for the total of five engines in normal operation. The engines are fueled by untreated digester gas. Propane and natural gas are available as a standby fuel, but have not been used for the past 20 years.

In recent years, the Air Quality Management District which regulates the Los Angeles air basin has adopted new rules pertaining to stationary engines which limit oxides of nitrogen (NO_x) and carbon monoxide (CO) emissions. In order to achieve compliance, the Districts tested two different emission control systems. The first was selective catalytic reduction (SCR) with ammonia addition applied to the engine exhaust, using a base metal oxide in the exhaust reactor. The second system consisted of engine modifications which reduce the production of NO_x, using an electronic device to control spark angle plus direct air injection to the power cylinder intake valves. After lengthy research involving field testing of full scale units for up to one year, the air injection/ electronic spark advance (AI/ESA) system was selected on the basis of significantly lower capital and operational costs. The major component of the SCR system costs about $65,000 per engine whereas the major component of the AI/ESA costs $12,000. In addition, the SCR system requires ammonia costing about $10,000 per year.

Prior to installation of control equipment, the average total NO_x and CO emissions from the engines were 177 kg of NO_x/day and 215 kg of CO per day per MW of power output. The stack emissions after employing the AI/ESA technology were 39 kg of NO_x/day and 21.5 kg of CO/day per MW of power output, which shows that the NO_x was reduced by 78% and the CO was reduced by 90% on the basis of one MW of power output. The NO_x and CO reductions are in compliance with the local air pollution regulatory agency limits for rich burn engines.

4. JWPCP SLUDGE DEWATERING, DISPOSAL AND REUSE

4.1 Dewatering

Sludge dewatering at the JWPCP has undergone major changes in the past decade. Beginning in the 1950s, digested sludge was dewatered, using horizontal scroll-type centrifuges. These centrifuges produced dry cake with no chemical addition, but recovered only about one-third of the feed solids, with the remainder being discharged to the plant effluent. In order to improve effluent quality, basket centrifuges were added in 1977 to dewater the centrate from the scroll centrifuges. This two-stage, series operation was eventually changed to a parallel system with the basket

centrifuges dewatering most of the digested sludge. The basket centrifuges have been able to capture nearly 95% of feed solids and produce a cake of 23% total solids when fed digested primary sludge with a polymer dose of 2 g/kg (4 lb/ton).

Research was initiated in the late 1970s to replace the old scroll centrifuges and to accommodate additional sludge production from secondary treatment, which was placed into service in late 1983. After full scale testing of various scroll centrifuges and belt filter presses, medium speed scroll centrifuges with automatic, variable speed backdrives were selected to expand the dewatering station. These centrifuges were judged most effective and flexible for dewatering a blend of digested primary and waste activated sludges, for feed sludges which were tested in the range of 25-50% waste activated sludge (W.A.S.) solids by weight.

The new centrifuges have variable speed backdrives which control the scroll differential to allow the centrifuge to maintain a desired amount of sludge in the bowl, maximizing sludge dryness, and to sense changes in solids loadings due to inlet sludge variation. A microprocessor adjusts the scroll speed to maintain near optimal operation as loadings change. Additionally, the scrolls have received ceramic hard facing. The ceramic facing is projected to extend the life of high wear areas of the scroll by many times, in comparison to metallic hard facing.

The new centrifuge installation contains twenty machines with a bowl diameter of 90 cm (36 inch), each designed for a feed rate of 7.9 l/s (125 gpm). The feed rate was selected at less than the manufacturer's nominal capacity in order to maximize cake solids and minimize polymer dosage. Recent performance of the centrifuges has been as follows. Digested sludge, containing about 75% primary and 25% W.A.S. by weight, is fed at an average concentration of 2.5% solids. Cake solids are above 23%, and solids recovery of about 95% is achieved, using a polymer dosage of approximately 3 g/kg (6 lb/ton). The centrifuges have demonstrated the ability to perform at above the design hydraulic feed rate which, under normal circumstances, has allowed the dewatering of all sludge without using the older, basket centrifuges. This reduces power usage and also reduces maintenance, since the basket centrifuges have relatively complex drive mechanisms. The automatic backdrive system of the scroll centrifuges is credited with providing the flexibility to adjust to variations in feed rate, feed concentration, and feed quality.

Dewatered sludge production from the JWPCP for 1986 averaged, on a wet basis, 1320 metric tons (1450 U.S. tons) per day. On a dry solids basis, this equals 310 metric tons (340 U.S. tons) per day. The Districts currently have two methods for disposal of this sludge (see Figure 6): approximately 25% is composted and sold for production of soil amendments, and 75% is co-disposed with refuse at the Districts-owned Puente Hills Landfill. In 1989, a third method, Carver-Greenfield dehydration/thermal processing, will also come into use.

4.2 Composting

Composting is the preferred method for sludge disposal at the JWPCP. It produces a product for beneficial reuse, uses little energy, and is lowest in cost. However, its capacity is limited because of land availability, and because it represents a significant odor source at the JWPCP site, which is surrounded by high density residential and commercial development. Therefore, the Districts restrict composting to the interior of the site, and limit the amount of sludge on the fields, to avoid an odor nuisance.

The Districts employ a two-step windrowing process which relies on the use of amendments, two sizes of windrows, and two sizes of composter machines (Figure 7). The two-step process was developed from research conducted at the JWPCP to optimize production and achieve a high level of pathogen inactivation. Windrows are constructed from a combination of dewatered sludge cake and amendments. Among the amendments used in the past or present are finished compost (recycle), sawdust (wood shavings), and rice hulls; rice hulls are no longer used because they resulted in higher odor production. Sludge cake and recycled compost or sawdust are mixed to achieve an initial total solids content of at least 40%, which assures that composting activity will begin rapidly.

In Step 1, windrows are formed by dumping loads of cake and amendments into continuous rows which are 4.3 m (14 ft.) wide, 1.2-1.5 m (4-5 ft) high, and spaced about 7 m (24 ft) apart. Windrows are mixed by a composting machine which straddles the windrow and turns the material over via a high speed rotating drum with flails. Step 1 lasts three to four weeks during favorable weather, during which each windrow is turned 3-5 times per week. Step 1 promotes faster drying and partial pathogen inactivation.

In Step 2, several windrows (normally three) are pushed together to form one larger windrow about 5.5 m (18 ft) wide and 1.8-2.1 m (6-7 ft) high. The larger windrows are capable of maintaining higher temperatures for longer periods of time, maximizing the pathogen destruction. After several weeks, during which the material is turned by a larger composting machine, the compost is released to Kellogg Supply Co. Inc., a private company. Temperature and coliform content are monitored during composting to assure that guidelines for pathogen quality are met. Kellogg Supply uses the finished compost as a base material for a variety of products such as soil conditioners, which are marketed in bagged form in the southwestern United States. The Districts receive a percentage of Kellogg's gross sales, which partially offsets the cost of operations. More importantly, Kellogg has maintained a highly successful marketing program for decades which assures a continuous outlet for all of the compost that the Districts can produce.

4.3 Landfill Disposal

As noted above, approximately three-fourths of the dewatered sludge operation began in 1977 when the basket centrifuges became fully operational and composting operations could no longer dispose of the total

dewatered sludge production. Puente Hills Landfill is owned and operated by the Districts. The landfill is not part of the wastewater treatment system, but is managed by the Solid Waste Department within the Districts. The JWPCP is considered a customer of the landfill, which must continue to serve primarily as a refuse disposal facility.

Sludge is hauled by truck to the landfill. Sludge hauling must be coordinated with the arrival of refuse at the landfill, since the two must be mixed for disposal. Typically, between 9 to 12 trucks [20.4 metric tons (22.5 U. S. tons) payload] are used for hauling sludge to the landfill. During normal dry weather operations, trucks require approximately 2 hours to make the round trip to and from the landfill.

The ability to haul sludge to the landfill is controlled by the number of trucks available, traffic conditions, hours of landfill operation, and the availability of solid waste from other sources which is used to absorb liquids in the sludge. Also, during the wet season, there are major constraints for sludge disposal due to rain. During heavy rain, traffic safety considerations can limit hauling. For several days after a moderate rain, the Puente Hills Landfill may lack the capacity to take all the sludge that can be dewatered.

A key element of this disposal system is the provision for storage of dewatered sludge cake. Dewatering takes place for 24 hours every day, whereas the landfill is only open during daylight hours, Monday through Saturday. Also, since 1983, the permit governing the landfill has contained a limit on both the daily and weekly tonnage of total waste material than can be disposed at the site. Since there is a shortage of landfill space in the Los Angeles area, this limit has often caused the landfill to close down early, sometimes as early as mid-day. In order to account for the limited disposal hours, as well as for periods when wet weather can restrict landfill operations, the JWPCP has twelve storage silos for dewatered sludge, with a combined capacity of 6000 metric tons (6600 U.S. tons) on a wet weight basis.

At the current daily sludge production of about 1360 metric tons (1500 U.S. tons) per day, the storage silos have a capacity to hold about four days of production, when normal dry weather overnight and weekend storage requirements are taken into account. Since both landfill and composting operations can be severely impacted by wet weather, down times of one or two days duration are not unusual during winter months. The current effluent requirements set by the state and EPA in 1984 recognize that limited sludge disposal capacity can affect effluent quality. During the winter months, effluent limits are set at 120 mg/l suspended solids and 180 mg/l B.O.D., whereas during the rest of the year these limits are 90 mg/l and 120 mg/l, respectively. The permit also allows discharge of digested sludge in excess of disposal capacity to the effluent as a result of rainstorms, although the JWPCP has never used this provision.

The Districts have recognized the need to develop year-round, reliable sludge disposal capacity that will not be affected by wet weather, as well as the need to rely less heavily on landfill disposal in view of dwindling

landfill space. Thus, a third method of sludge disposal, dehydration and thermal processing, is being constructed.

4.4 Landfill Gas Energy Recovery

The Puente Hills Landfill receives 67,000 metric tons (72,000 U.S. tons) of waste per week, which includes the sludge from the JWPCP. Decomposition of the waste in the landfill produces gas much like an anaerobic sludge digester, although the gas is typically lower in methane content (37-47%). Both the sludge and refuse in the landfill represent a source of recoverable energy.

Landfill gas must be collected as a safety measure to prevent migration to surrounding areas, and for odor and air pollution control (Figure 8). The Puente Hills Landfill contains an extensive system of vertical wells and horizontal trenches filled with crushed rock and subsequently buried as the landfill is constructed. With the necessity for a large investment to collect the gas, energy recovery is naturally attractive.

A number of uses for landfill gas were investigated by the Districts, including sale to the gas utility, conversion to methane, electrical production, etc. Purification and sale to a gas utility had been practiced at another Districts' landfill for many years, but this process suffers when air is drawn in with the gas, as is often the case for gas wells at the periphery of the landfill, which must draw enough volume to prevent migration off the site. Production of electricity has been selected as the most suitable of the alternatives.

Initially, gas turbines and reciprocating engines were considered as prime movers for electrical generation. Turbines were chosen because of lower NO_X production, and a better ability to burn lower caloric content gas, such as that drawn from near the face of the landfill. Beginning in late 1983, the Districts operated two different types of gas turbines at the Puente Hills Landfill to utilize approximately 20% of the collected gas for electrical power production; the remaining gas was flared at the adjacent flaring station. Both gas turbines have successfully demonstrated that gas turbines can reliably generate power from untreated landfill gas. Total net power production from this installation was approximately 2.8 MW. These turbines were taken out of service with the advent of a new project to use all of the gas, but will be used in the future when excess gas is available.

In order to utilize all of the gas produced at the landfill, the Puente Hills Energy Recovery from Gas (PERG) project was developed. In studying alternatives for this project, it was concluded that the Rankine cycle, the most prevalent technology for electrical power generation in the United States, was the most cost-effective system in the range contemplated for the Puente Hills Landfill. The Rankine cycle consists of combusting a fuel in a boiler to generate high pressure steam. The high pressure steam is then expanded in a steam turbine which drives a generator for electrical power production. The Rankine cycle was selected on the basis of: low air

emissions, low construction cost, ease of operation, and high efficiency. However, for smaller landfill gas-to-energy projects, the other technology are also considered viable alternatives.

The PERG project was designed for a gas flow rate of 28,000 Nm^3/h (17,500 scfm). A simplified process diagram is shown in Figure 9, and Figure 10 is an aerial view of the facility. The facility consists of two Zurn steam generators, each firing 50% of the landfill gas. The units each produce 80,000 kg/hr (175,000 lb/hr) of 9,400 kPa (1,350 psig), 540°C (1000°F) steam. The steam is used to drive a Fuji steam turbine-generator and initially produce 43.5 MW of electricity. Operation of the facility requires approximately 3.3 MW of the generated power, leaving a net of 40.2 MW of power available for export to Southern California Edison. The facility is designed to utilize future landfill gas flows to generate up to 50 MW of electricity gross with a net output of approximately 46.5 MW.

The contractor for this project was Schneider, Inc., whose evaluated bid was lower than others from Foster Wheeler, and Mitsui & Co. (U.S.A.). An unusual feature of this project, for the Districts, was the use of a single contractor on a "turnkey" basis to take responsibility for the design, construction, and financing of the project. Considering that the unused gas at the landfill represented a substantial stream of revenue, the turnkey approach offered valuable advantages of faster completion as well as avoidance of initial capital outlay. The Districts will make lease payments to the contractor which will be more than offset by the revenue from sale of electricity. The total cost of the facility, equivalent to $28 million in 1985 dollars, will be paid off by 60 monthly payments of approximately $726,000, while the revenue from sale of electricity will be about $2,000,000 per month. The excess revenue will be used for operation and maintenance expenses and to support other solid waste management projects.

The Districts have been required to use "Best Available Control Technology" for NO_x control as defined by the South Coast Air Quality Management District. This includes a derated boiler design, flue gas recirculation, and low excess oxygen burner operation. As a result, the overall emissions from the PERG project are substantially lower than the previous emissions with most of the gas being flared.

In addition, the Sanitation Districts, in cooperation with the South Coast Air Quality Management District, will run a test program utilizing thermal $DeNO_x$ for the reduction of oxides of nitrogen. If the test program is successful, this additional control device will be added to the facility to further reduce the NO_x emissions. The results of this testing could set the standard for control technology for similar facilities.

Startup of the facility took place in late 1986. Electrical output for the first months of operation has exceeded the projected values. Thus, the project is proving to be a very cost effective and environmentally sound method to control gas emissions and recover energy from the landfill.

4.5 Carver-Greenfield Sludge Dehydration/Energy Recovery Project

Increasing sludge production from improved wastewater treatment, decreasing landfill capacity, and the need to develop more reliable wet weather disposal capacity led the Districts to seek an additional means of sludge disposal. After participating in a regional sludge management study with the neighboring City of Los Angeles and Orange County Sanitation Districts, Carver-Greenfield dehydration followed by combustion with energy recovery was selected as part of the disposal plan for JWPCP sludge. Construction of this system is now in progress.

The Carver-Greenfield dehydration system will be used to convert the digested, dewatered sludge to a dry powder. Figure 11 is a simplified diagram of the process. The powder, containing about 95% solids, will be burned in circulating fluidized bed boilers to produce steam. About half of the steam will be needed for the dehydration process, with the excess being used for electrical power generation. Thus, the overall process will be a net energy producing resource recovery facility.

The system will include three dehydration trains, each with a capacity to handle up to 110 dry metric tons (120 dry U.S. tons) per day of dewatered sludge. It is planned that, during most of the year, two trains will be in service, with the balance of the sludge being disposed by composting and landfilling. During the wet season, for perhaps 20 percent of the year, all three trains would be placed into service, providing a disposal capacity of 330 dry metric tons per day (360 U.S. tons).

The multiple effect evaporation portion of the system is much like several sludge processing plants in Japan. Dewatered sludge is first mixed with light oil to fluidize the sludge, which is then fed to a four effect evaporation system. Fluidized sludge and steam travel counter-currently through four evaporative effects, which consist mainly of heat exchangers and vapor chambers (see Figure 12). Steam from the fluidized bed boilers is fed to the fourth stage (first effect). Water driven off from the sludge in the fourth stage (first effect) flashes to steam which is used in the third stage, and so on through the first stage (fourth effect). Since each effect is under increasing vacuum, the successively lower temperature steam can be used in the adjacent step to continue to drive off water. By using four effects, the heat requirement is only about one-third to one-half of the amount that would be needed for a single-step conventional dryer.

With nearly all of the water evaporated, the sludge solids remain fluidized in the oil. The next phase is the fluidizing oil recovery process, consisting of centrifugation, hydroextraction, and thermal oil stripping (Figure 13). This phase is designed to recover virtually all of the fluidizing oil, leaving only about one percent oil in the dehydrated product. The system differs from some Japanese installations which press out the oil, leaving a relatively high oil content in the product that is fed to the combustion process. The Districts also use a lighter, more volatile fluidizing oil compared to the heavy oil used in Japan. The sophisticated oil recovery system reduces the makeup oil to a minimum, and

also reduces the overall air emissions compared to a system in which large amounts of complex oil are also burned. Since air quality regulations for Los Angeles require new sources of pollution to completely offset any net increase in emissions, reduction of emissions by oil recovery is an important factor.

The product of the evaporation process is an extremely fine, dry powder. The powder is pneumatically conveyed to three circulating fluid bed combustors, designed by Babcock and Wilcox Company (see Figure 14 for a schematic diagram of the combustion system). The circulating fluidized combustors maintain a large sand and limestone bed in relatively high velocity circulation around the furnace section, resulting in extremely good mixing, high carbon burnout and high heat transfer to the boiler waterwalls. The limestone sorbent was selected to reduce sulfur oxide emissions as described below.

Ash from combustion will be truck-hauled to Puente Hills Landfill, 56 km (35 miles) away. The ash has been determined to be a nonhazardous waste, allowing it to be disposed at a facility which handles normal refuse. The ash to be hauled includes the noncombustible solids from the sludge, some bed material from the combustors, and products from the air pollution control systems. Thus, the weight of the ash could be between one-half and two-thirds of the weight of dry power combusted. However, when compared to disposal of dewatered sludge, the tonnage remaining to be landfilled is reduced by a factor of five. The ash has a high calcium/metal oxide content, and may thus be usable in a secondary market such as cement manufacturing.

Steam is generated from the combustors at 446°C (835°F) and 6200 kPa (900 psia). It is fed to two Fuji Electric Company turbines where the steam is expanded and reduced to pressures needed to feed the Carver-Greenfield process and for extraction feedwater heating. The remainder of the steam passes to the condenser. A backpressure turbine exhausts steam for sole use in the evaporators, and the condensing turbine has a controlled extraction port to provide constant pressure steam for the oil recovery process.

Air emission controls are a major part of the overall system. Oxides of nitrogen are controlled to a large extent by maintaining low combustion temperatures of about 840°C (155°F), below the threshold of significant NO_x formation. Careful control of secondary and tertiary combustion air addition also limits NO_x formation. The relatively low amount of excess air also reduces the size of ductwork, emission control equipment, and fan horsepower. The boiler supplier has guaranteed uncontrolled NO_x emissions to be less than 135 parts per million by volume. NO_x emissions will be further reduced by non-catalytic, selective reduction via ammonia injection into certain zones of the boilers. Hydrogen addition may also be needed to catalyze the reaction because of the low operating temperatures.

Sulfur emissions are initially controlled by the limestone circulating in the combustors. The manufacturer has guaranteed 90% capture of sulfur in the bed. The flue gas passes to a spray dryer which atomizes lime

slurry, and mixes it with flue gas components to convert the remaining sulfur to calcium sulfate. Water from the lime slurry is completely evaporated by the heat of the flue gas, and the resultant mixture passes to the baghouse primarily in a dry form. The heavy particulate-laden stream coats the bags, achieving final SO_2 and particulate capture.

The overall construction costs of the evaporation, combustion, air pollution and support facilities are approximately $150 million. Net operating costs considering power production are estimated to be $10 million per year. Net power production at maximum capacity is expected to be approximately 2 MW. Construction has been phased, with the boiler contract issued in February 1985, on-site construction beginning in April 1986, and completion of all facilities anticipated by late 1989.

5. DESERT FACILITIES

5.1 Joint Treatment Facilities

The two small communities of Saugus and Valencia were originally served by independent sewer systems and water reclamation plants. In recent years, it became apparent that flow at the Saugus Water Reclamation Plant (W.R.P) would exceed the 0.22 m^3s (5.0 mgd) design capacity, but the site allowed no room for further expansion. Therefore, the two communities entered into an agreement to divert excess flow from Saugus to the 0.33 m^3/s (7.5 mgd) Valencia W.R.P, and also to establish joint sludge management facilities at the Valencia W.R.P. Sludge from Saugus is transported by force main to Valencia for digestion, dewatering if necessary, and disposal. Currently, digested sludge is landspread for agricultural utilization, or hauled to a manhole in the sewer system leading to the Joint Water Pollution Control Plant. Later this year, a filter press dewatering facility will be completed. Excess sludge which cannot be landspread (especially during wet weather) will be dewatered to at least 50% total solids, allowing disposal at the nearest landfill permitted to receive sludge.

5.2 Agricultural Utilization

Liquid digested sludge is land applied at the Peter Pitchess Honor Rancho, a correctional facility operated by the Los Angeles County Sheriff's Department. Approximately 70 hectares (170 acres) of land are used for growing fodder crops such as alfalfa or oats. Application rates are controlled to meet the nitrogen needs of the crops, which are up to 390 kg/year of available N per crop. Sludge application rate is approximately 18 metric tons/hectare. There are also limits on the amount of cadmium, zinc, copper, nickel, and lead that may be applied. Groundwater, approximately 8 m (25 ft) below the surface, is monitored as well.

Sludge is hauled to the site 5 km (3 miles) from the Valencia W.R.P. in 19 m^3 (5000 gal) tank trucks. The same trucks spread the sludge on the field, which is tilled the same day to incorporate the sludge in the soil. Although the treatment plant will soon have the ability to dewater sludge

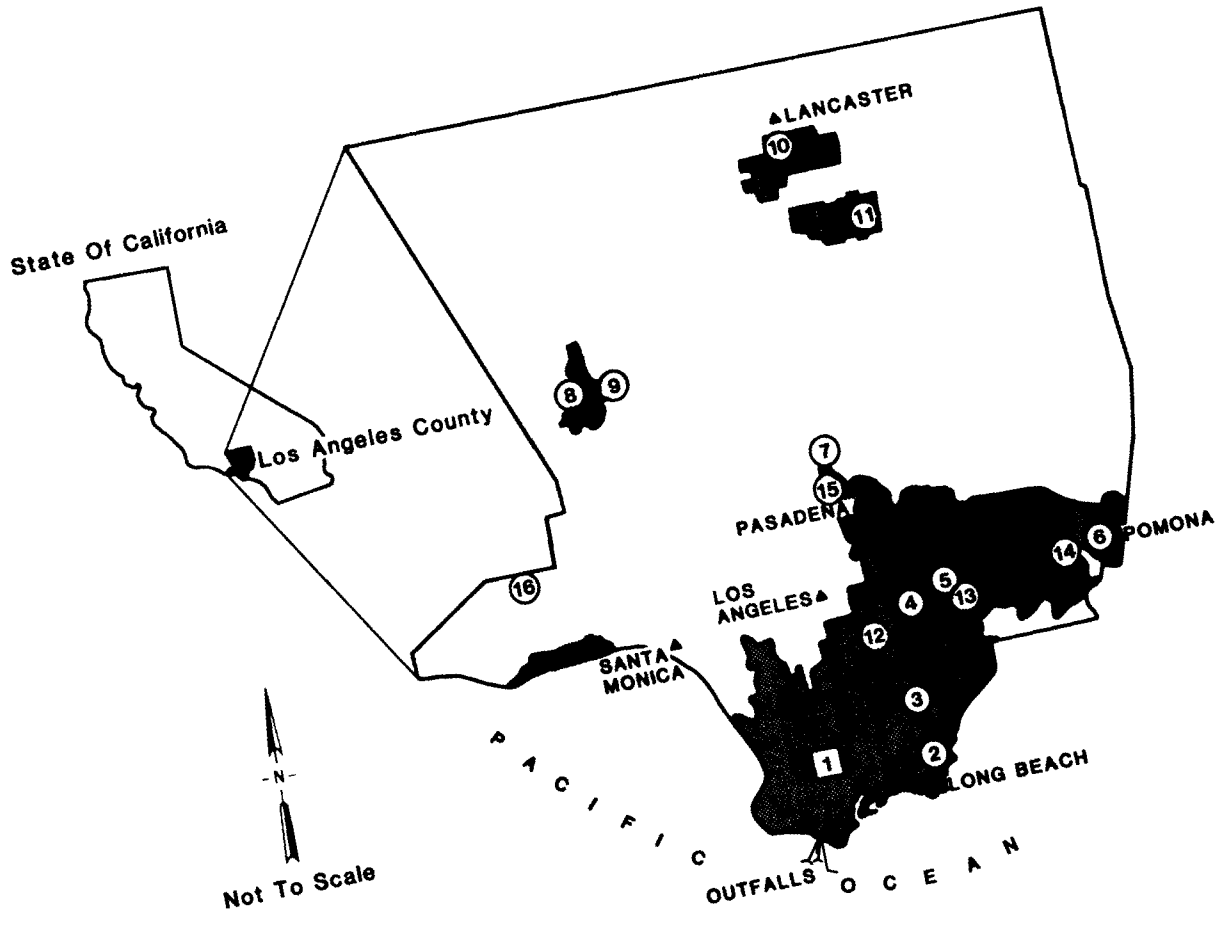
for landfill disposal, the landspreading operation will continue to be favored because of low cost, little energy demand, no need for chemicals and the desirability of beneficial agricultural reuse as opposed to filling of valuable landfill space.

5.3 Power Generation from Digester Gas

Prior to the recent expansion of the Valencia W.R.P., digester gas was simply flared on site. The plant expansion and the institution of combined sludge processing for the Saugus and Valencia W.R.P.s made power production from digester gas practical for even this relatively small plant. The sludges from the two plants are anaerobically digested at Valencia, producing 250 Nm^3/h (225,000 scfd) of gas. The gas is used onsite for cogeneration of power and steam by a 500 kW ebulliently-cooled, reciprocating engine-generator set.

An ebulliently-cooled engine operates very close to the boiling temperature of the cooling water. As shown in Figure 15, steam is produced in a separator from the rejected heat (530 kJ/s) of the engine jacket water and the exhaust gases. The steam is used to heat the digesters, which had formerly been heated using purchased natural gas.

Since the new filter press dewatering operation will represent a considerable energy demand, operation will be shifted to off-peak hours to lower the maximum demand. Utilizing gas storage, the generator will also produce more power during the day to reduce peak demand for purchased power. In summary, the total cost of power will be considerably reduced, and the need to purchase natural gas for digester heating will be eliminated. The cost for this installation, including a hydrogen sulfide removal system for the digester gas, was $600,000.



LEGEND

SANITATION DISTRICTS OF LOS ANGELES COUNTY

1. JOINT WATER POLLUTION CONTROL PLANT
2. LONG BEACH WATER RECLAMATION PLANT
3. LOS COYOTES WATER RECLAMATION PLANT
4. WHITTIER NARROWS WATER RECLAMATION PLANT
5. SAN JOSE CREEK WATER RECLAMATION PLANT
6. POMONA WATER RECLAMATION PLANT
7. LA CANADA WATER RECLAMATION PLANT
8. VALENCIA WATER RECLAMATION PLANT
9. SAUGUS WATER RECLAMATION PLANT
10. LANCASTER WATER RECLAMATION PLANT
11. PALMDALE WATER RECLAMATION PLANT
12. SOUTH GATE TRANSFER STATION
13. PUENTE HILLS LANDFILL
14. SPADRA LANDFILL
15. SCHOLL CANYON LANDFILL
16. CALABASAS LANDFILL

FIGURE 1 FACILITIES WITHIN THE SANITATION DISTRICTS OF LOS ANGELES COUNTY.

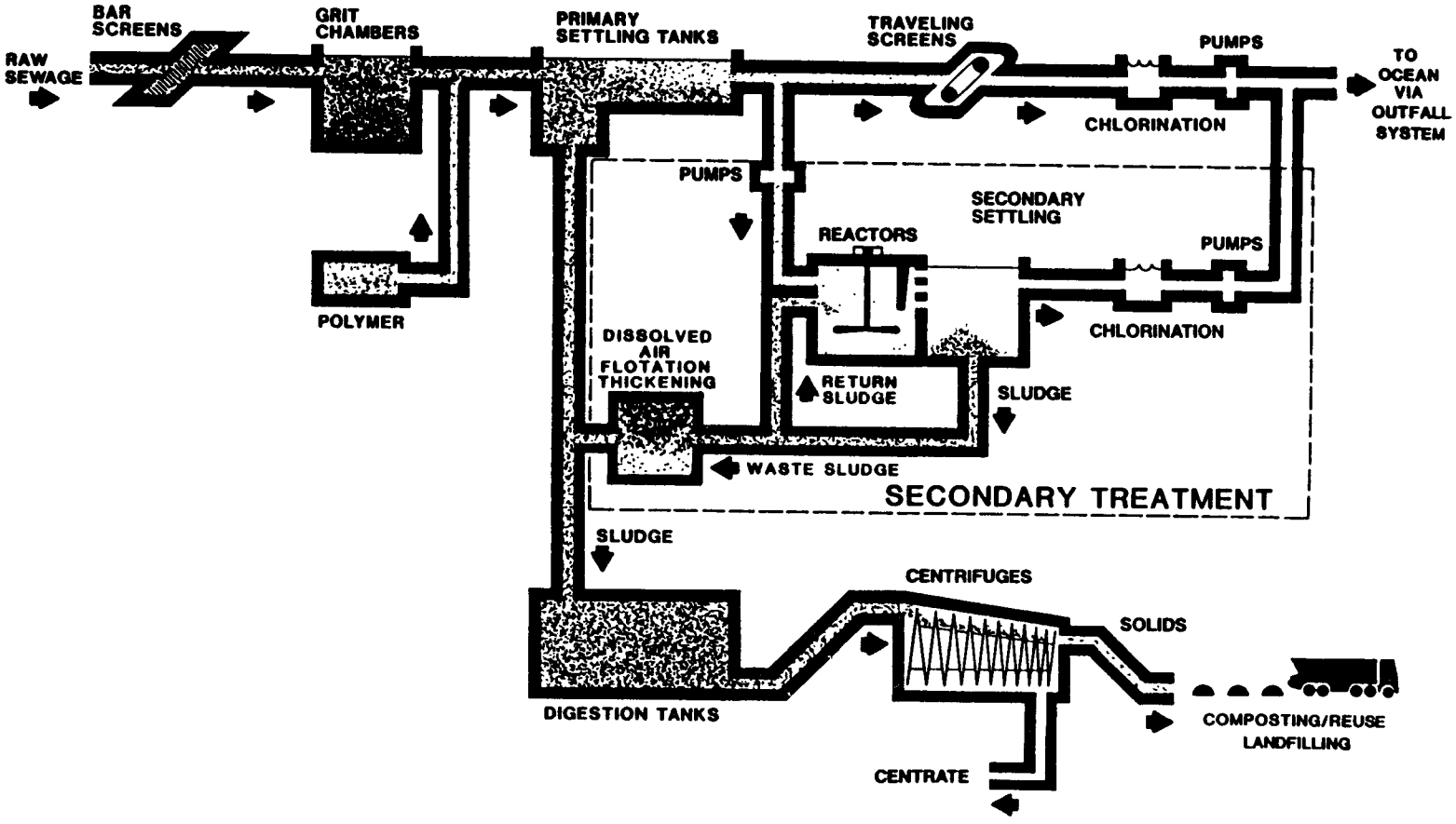


FIGURE 2 JOINT WATER POLLUTION CONTROL PLANT PROCESS FLOW DIAGRAM

FIGURE 3 AERIAL VIEW OF THE JO'NT WATER POLLUTION CONTROL PLANT

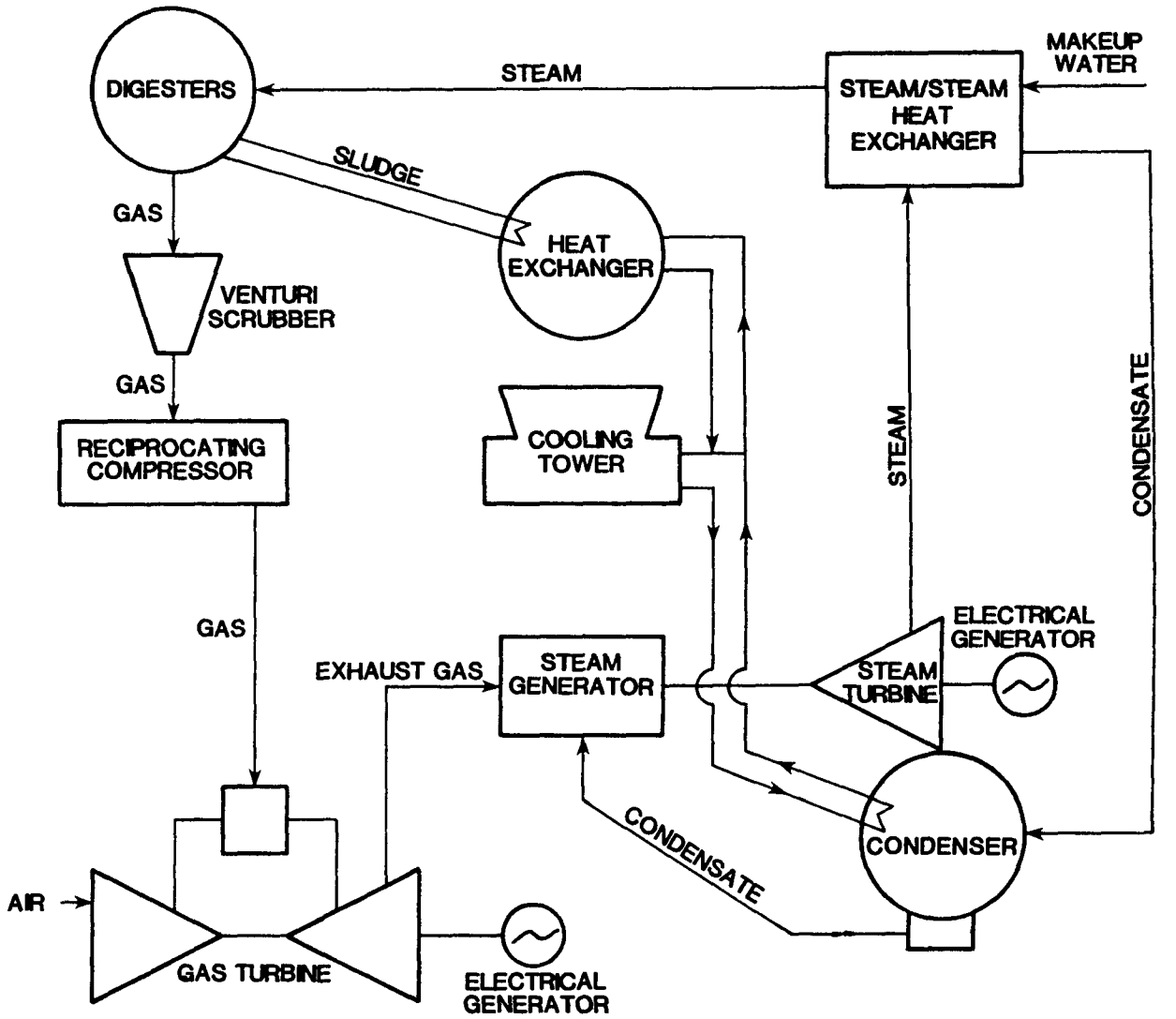


FIGURE 4 TOTAL ENERGY FACILITY SCHEMATIC DIAGRAM

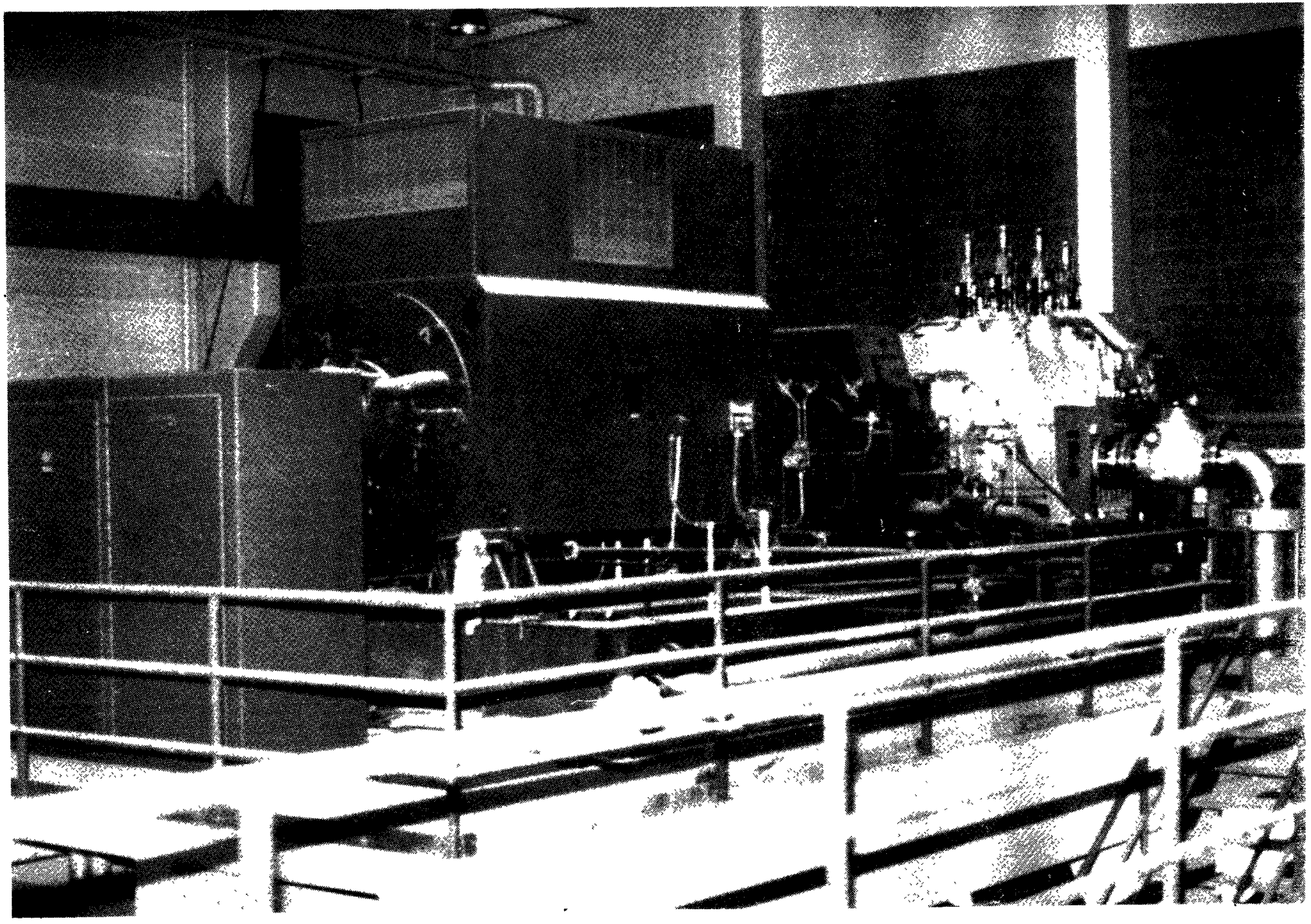


FIGURE 5 STEAM TURBINE (AT REAR) AND ELECTRICAL GENERATOR AT T.E.F.

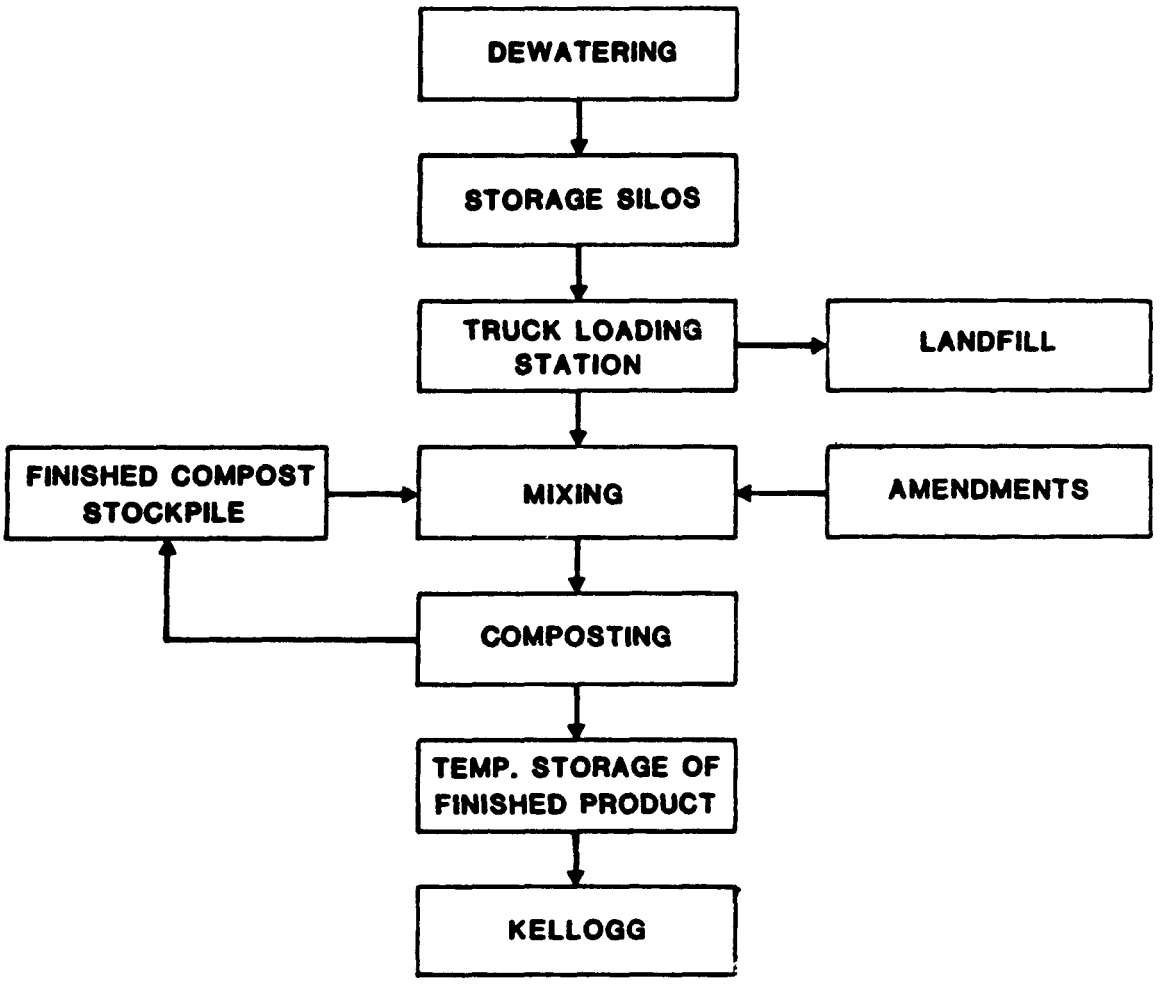
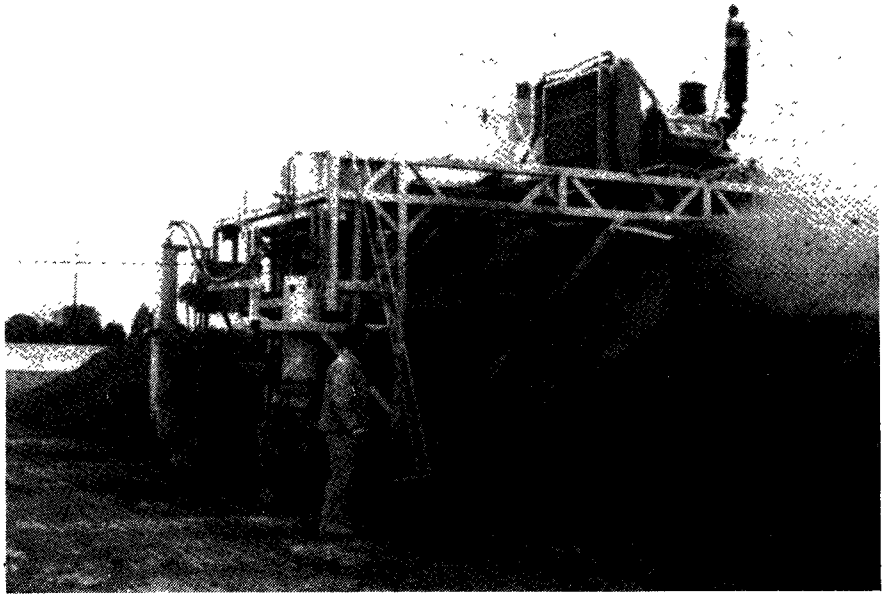


FIGURE 6. DIAGRAM OF THE SOLIDS PROCESSING OPERATION AT THE JWPCP.

STEP 1 SMALL WINDROWS



STEP 2 LARGE WINDROWS

FIGURE 7 TWO-STEP WINDROW COMPOSTING AT JWPCP

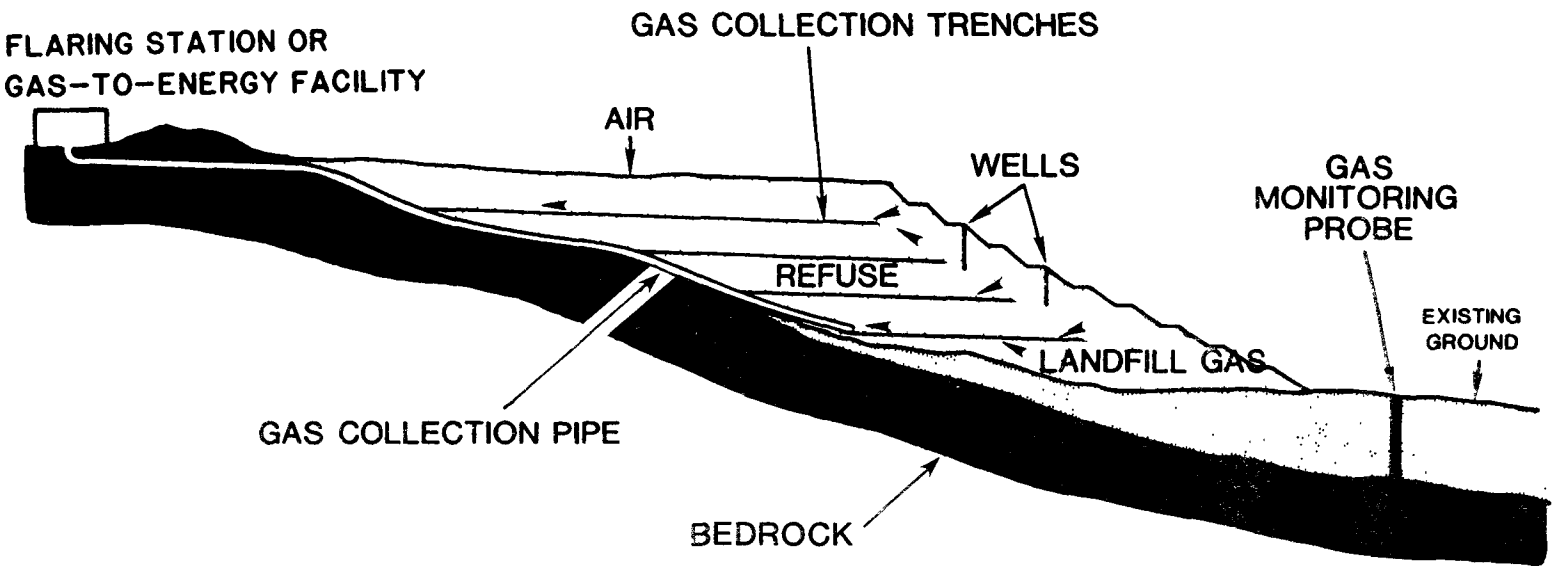


FIGURE 8 LANDFILL GAS COLLECTION AND CONTROL.

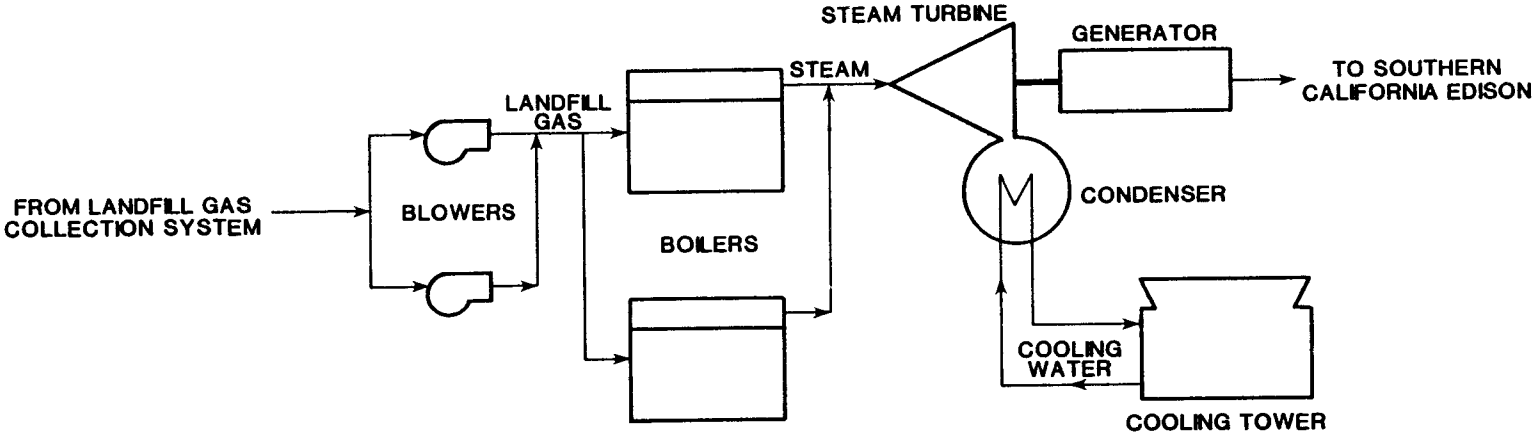


FIGURE 9 PUENTE HILLS ENERGY RECOVERY FROM GAS (PERG)
SIMPLIFIED PROCESS DIAGRAM

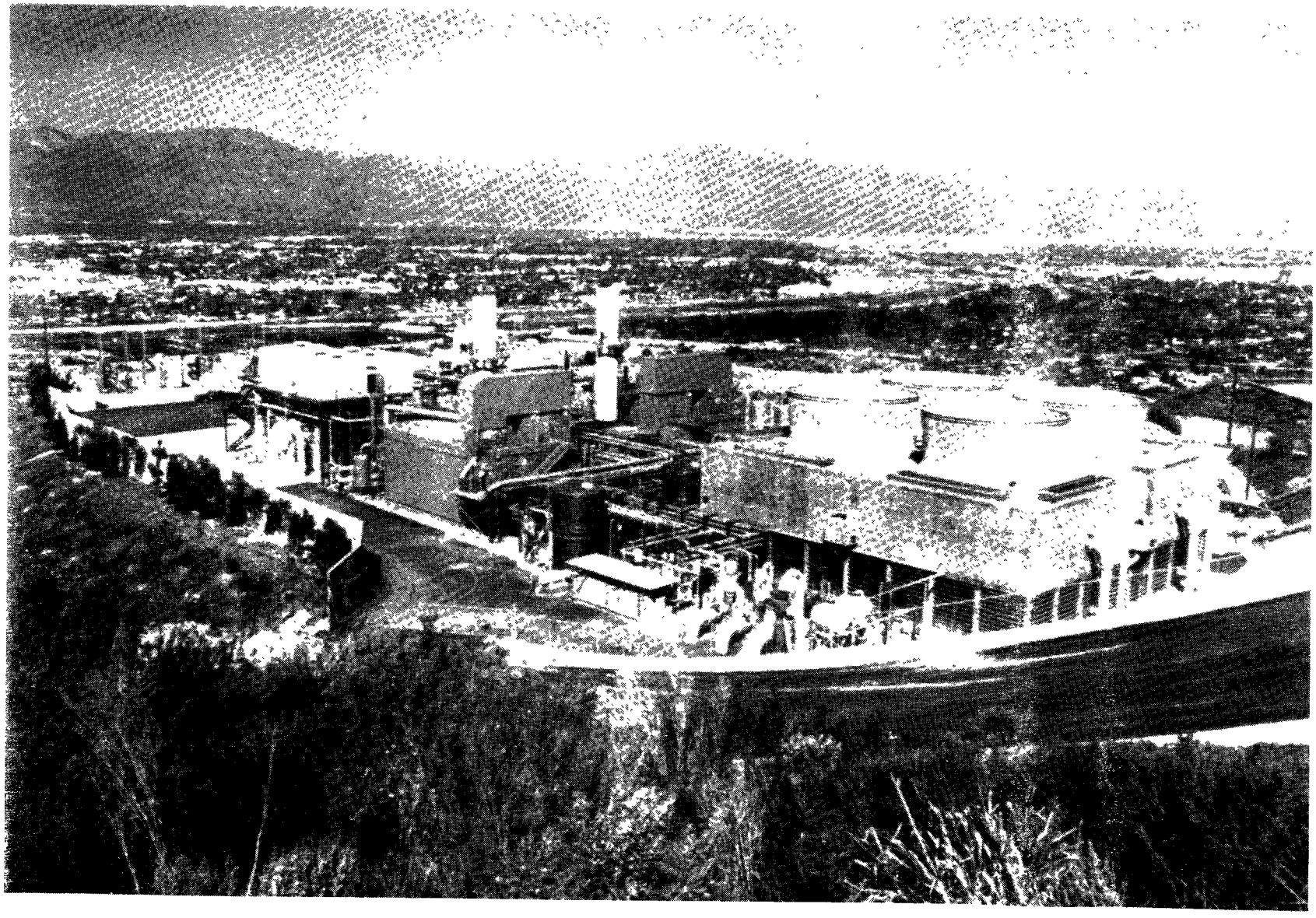


FIGURE 10 AERIAL VIEW OF P.E.R.G.

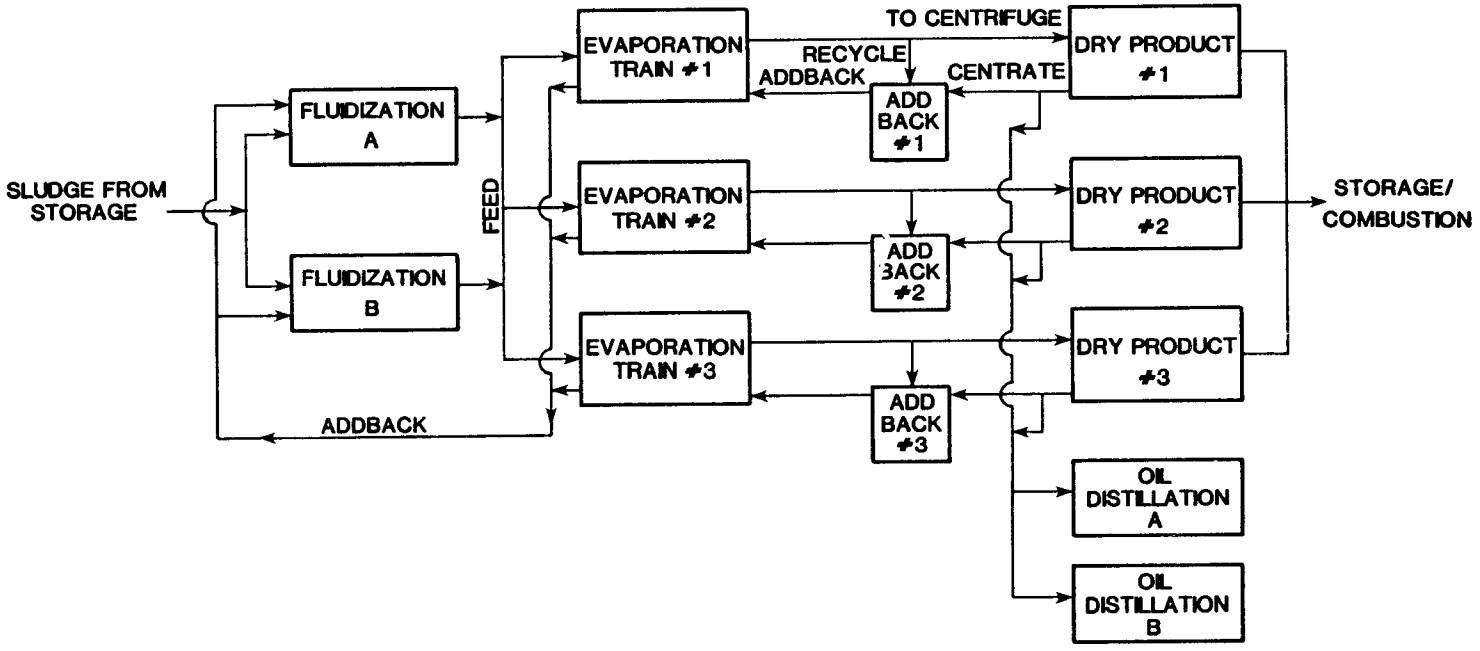


FIGURE 11 CARVER-GREENFIELD SLUDGE PROCESSING - SIMPLIFIED PROCESS DIAGRAM

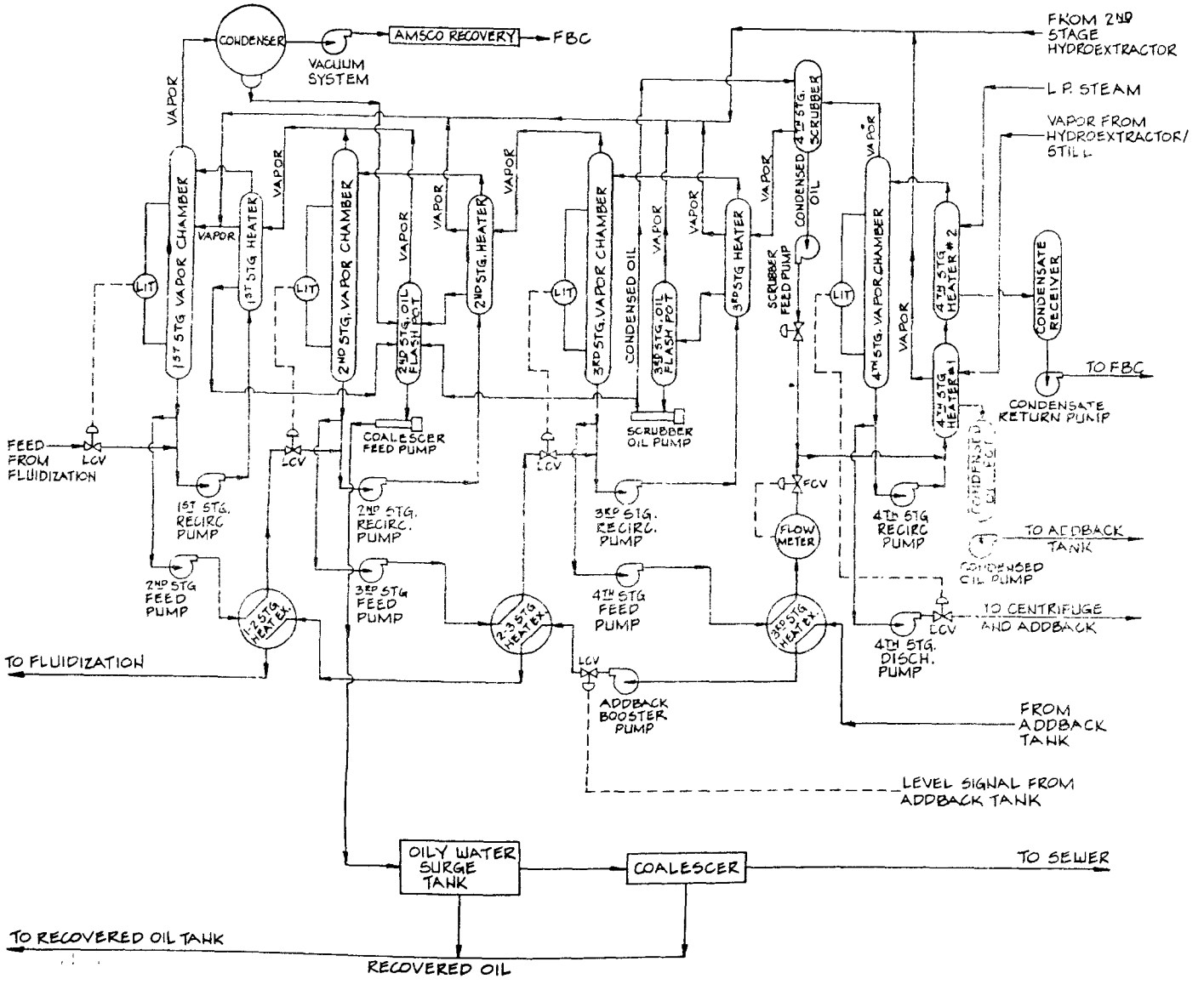


FIGURE 12 CARVER-GREENFIELD FACILITY DEHYDRATION TRAIN

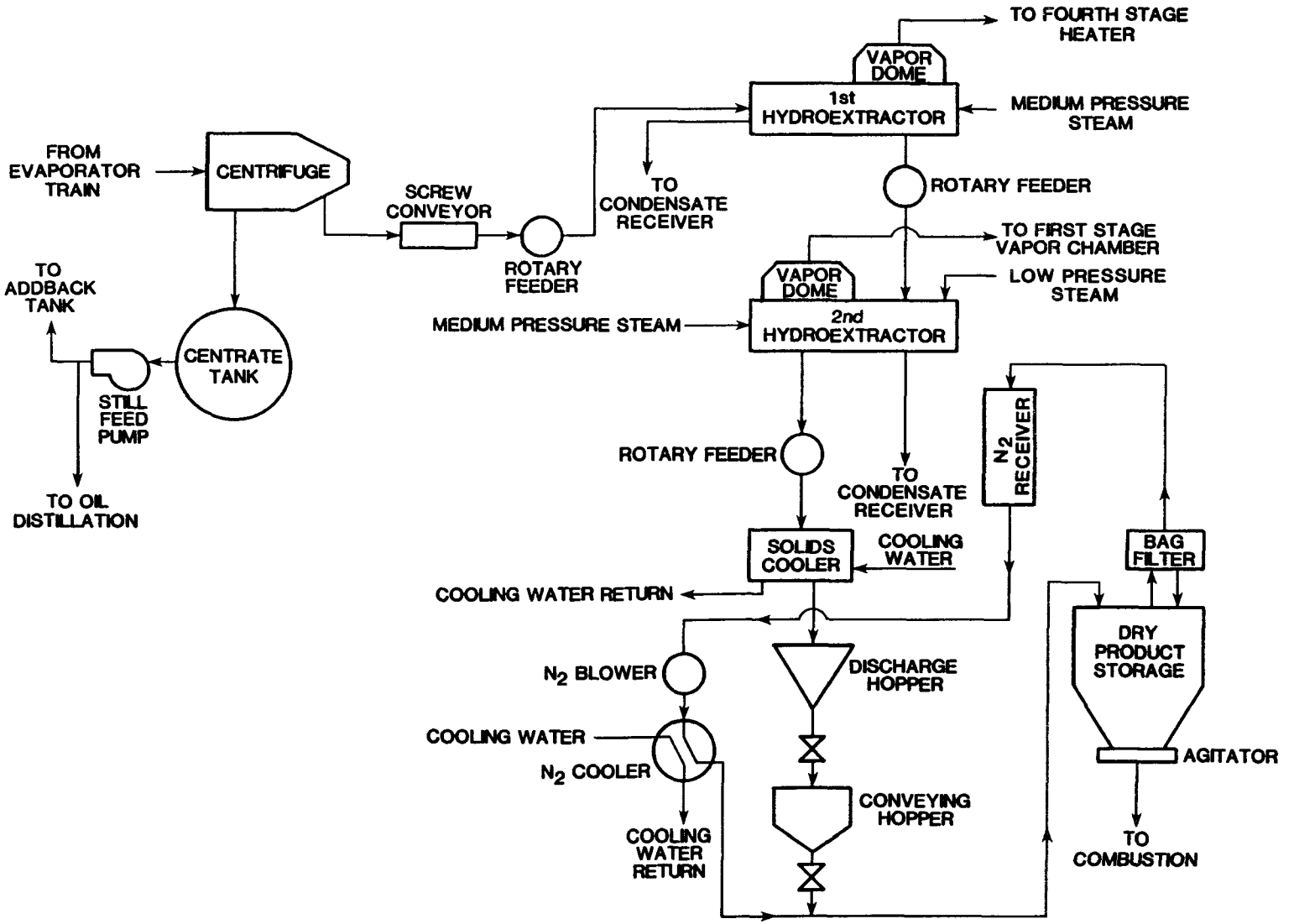


FIGURE 13 CARVER-GREENFIELD FACILITY OIL RECOVERY/DRY PRODUCT TRAIN

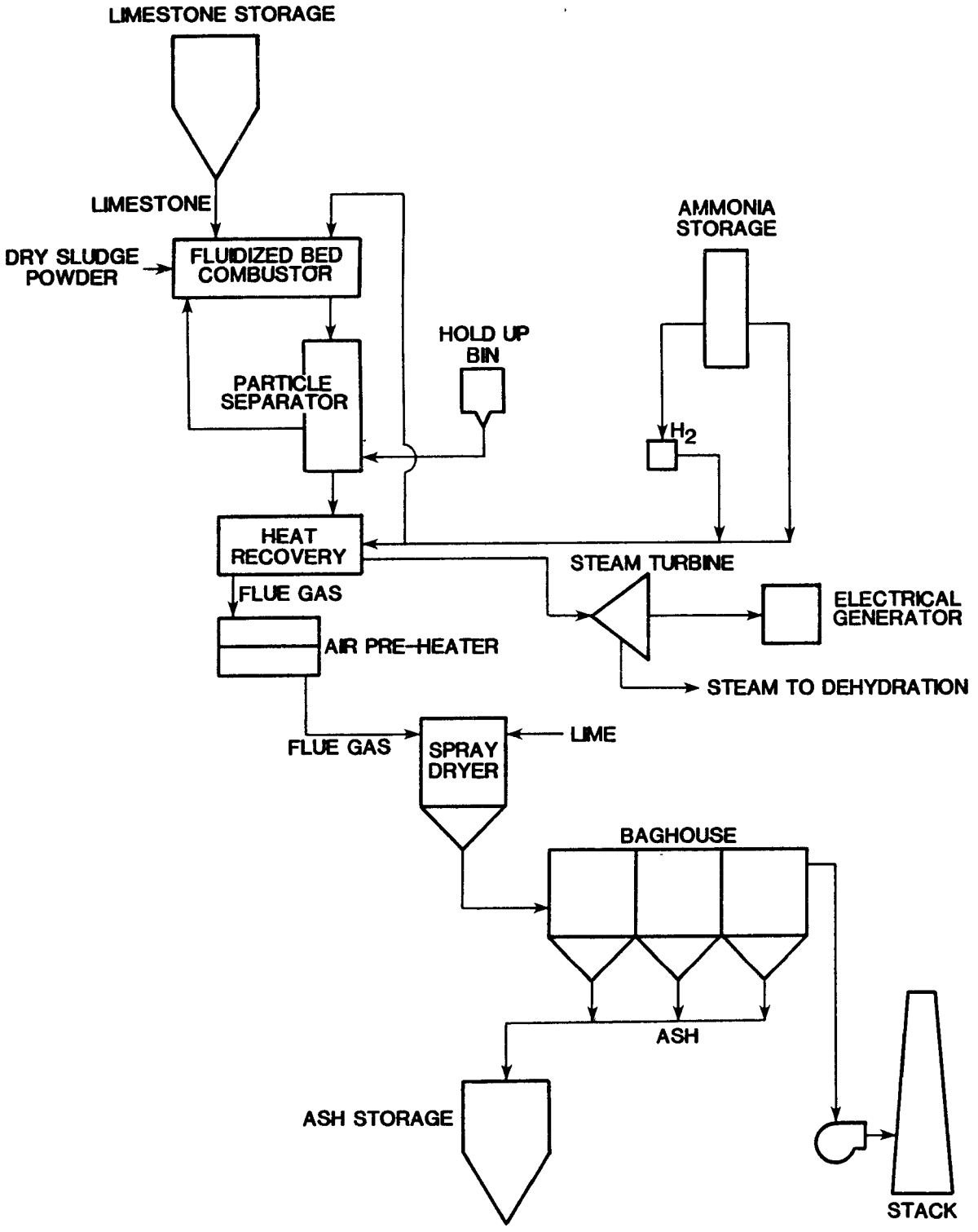


FIGURE 14 CARVER-GREENFIELD FACILITY - COMBUSTION AND AIR POLLUTION CONTROL

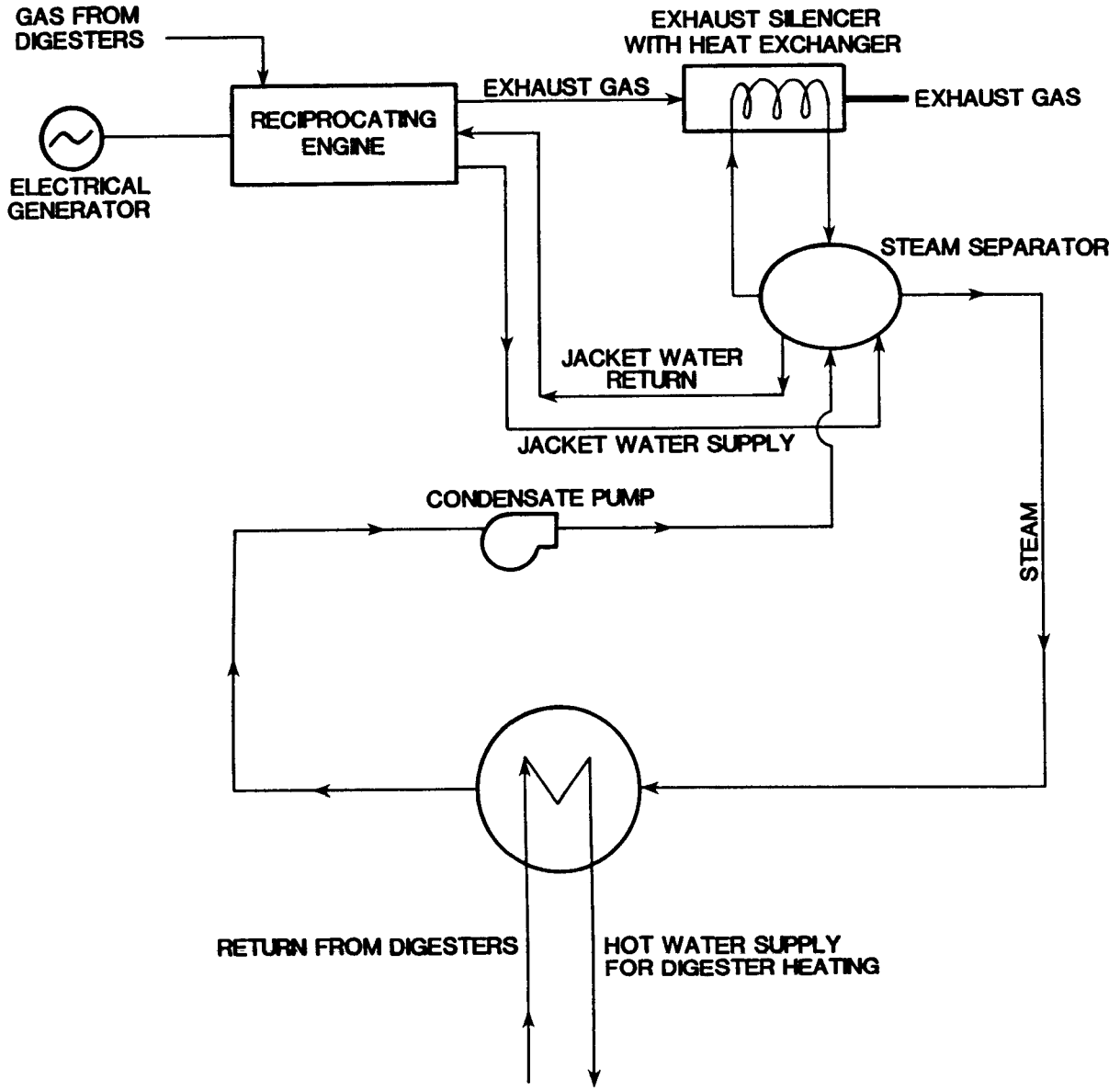


FIGURE 15 POWER GENERATION FACILITY AT VALENCIA WATER RECLAMATION PLANT

SELECTED MUNICIPAL SLUDGE TOPICS

by

Carl A. Brunner, Ph.D.
Chief, Systems & Engineering Evaluation Branch
Wastewater Research Division
Water Engineering Research Laboratory
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

Prepared for Presentation at:

Eleventh United States/Japan Conference
on Sewage Treatment Technology
Tokyo, Japan

October 12-14, 1987

SELECTED MUNICIPAL SLUDGE TOPICS

Carl A. Brunner, Ph.D.

INTRODUCTION

The amount of sludge from municipal wastewater treatment plants is slowly increasing in the United States because of a continuing increase in the number of people served by sewers and the upgrading of plants with less than secondary treatment to full secondary treatment. As the amount of sludge is increasing, the resistance to disposal is also increasing. Some municipalities and states have banned the placing of sludge in landfills. The establishment of new landfills, even when not legally banned, is becoming increasingly difficult because of resistance of residents near the sites of these proposed landfills. Ocean disposal is opposed by much of the public and is unlikely to increase in the United States. Incineration has not been popular since the large increase in fuel costs in the 1970's and is now encountering resistance because of perceived health risks from stack emissions. Application to agricultural and forest land has become an increasingly common method of disposal, but is often not a practical solution for large cities. Disposal of sludge in the United States is obviously an increasing problem.

Another factor likely to affect sludge disposal in the United States is the planned promulgation of new regulations for essentially all modes of disposal. These regulations are expected to be based upon health or environmental risk. Although the exact form of these regulations has not been established, they are very likely to place additional restrictions on sludge disposal.

The result of the pressures on treatment plant management to find more acceptable routes of sludge disposal has been at least partly responsible for the development of new methods of sludge treatment which either change the sludge to a form more acceptable for disposal or convert it to a useable material.

The following describes briefly the form of planned sludge regulations for the United States, discusses several new sludge treatment methods which have been evaluated by the United States Environmental Protection Agency (EPA) and provides some new research results on pathogenic microorganisms in sludge.

Carl A. Brunner is a Chemical Engineer with the U.S. Environmental Protection Agency, Cincinnati, Ohio.

PLANNED SLUDGE REGULATIONS

EPA adopted the very logical concept of utilizing risk assessment as the basis for a planned set of sludge disposal regulations covering land application, landfilling of sludge alone (monofill), incineration, and ocean disposal. Home use of compost and other marketed sludge products are included with land application. For each mode of disposal, all conceivable routes of exposure to both humans and other species were listed. From the list the routes judged to have the most impact were selected. Except for ocean disposal, the critical routes all involved human exposure. For each disposal method, other than ocean disposal, a list of approximately twenty toxic materials judged likely to have the most severe effect on human health was prepared. A similar list was prepared for ocean disposal, but with recognition that maintenance of water quality necessary for a healthy ocean ecosystem was the principal objective. For each disposal method other than ocean disposal a maximum exposed individual (MEI) was defined. These definitions were very conservative. For incineration this individual was defined as someone spending a lifetime of seventy years living 200 meters horizontal distance from the incinerator stack and being exposed to incinerator emissions 24 hours per day. Based on available data for health effects of the chosen toxic materials, using available models for the fate of these materials along the chosen paths, and assuming reasonable values for other pertinent parameters, concentrations of the toxic materials allowable in the sludge were calculated for levels of risk of fatality or incidence of cancer from one in 10^4 to one in 10^6 of the exposed individuals. At this time a risk level of one in 10^5 individuals is being assumed acceptable except for marketed sludge products. In the analysis of ocean disposal risk a level of one in 10^6 individuals was also assumed.

A three-tiered regulatory approach was then developed that would allow different degrees of consideration to be given to site-specific factors which would attenuate the migration of a pollutant into the environment. The three tiers are as follow:

Tier I: Sludges regulated under this tier would have to have concentrations of the modeled toxic materials less than those calculated to result in the chosen level of risk to the MEI. Site-specific factors would not need to be considered in this tier. Although a nationwide survey of the degree of compliance under Tier I has not been carried out, comparison of listed concentrations with those obtained for a number of sludges suggests that many sludges would not meet this requirement.

Tier II: A table would be provided specifying a range of concentrations of the modeled toxic materials in the sludge based on varying a limited number of site-specific conditions. Two examples would be height of stack for an incinerator and end use of the sludge for land application.

Tier III: This tier applies to sludges not meeting the requirements of Tier II. The permitting authority would have to make site-specific determination of the acceptability of a disposal method for these sludges. The management of the treatment plant would have to supply the necessary data to demonstrate

that pollutants would not violate the chosen risk level. Regulating a sludge under Tier III is a much more complicated procedure than regulating under Tiers I and II.

In parallel with the MEI risk assessment, EPA also carried out an aggregate risk assessment that produced an estimate of the national impact of sludge disposal under existing conditions, and reduction in this impact under the new regulations. This aggregate risk assessment has not been carefully reviewed, but the unreviewed results indicate a very low incidence of negative health effects under existing conditions. The final regulations will probably be somewhat modified from the preliminary version developed from the MEI results to reflect this relatively low national impact. That the MEI risk assessment appears to indicate a greater health impact than the aggregate risk assessment probably results largely from the fact that there is very little, if any, of the population that actually experiences conditions with a degree of risk equal to that of the MEI. For this reason the MEI approach usually gives conservatively safe results. If the allowable sludge concentrations from the MEI approach were much higher than those usually found in sludge, sludge producers and regulatory agencies could be assured that there was extremely small risk from sludge disposal. Where sludge concentrations calculated from the MEI are within the range actually found in sludges, further consideration must be given to whether very highly exposed individuals approaching the conditions of the MEI do actually exist and whether modifications to results can safely be made to overcome the conservative nature of the MEI approach.

Pathogenic microorganisms are presently regulated under a technology based approach. Because of the difficulty in describing mathematically the exposure routes, completion of a risk assessment methodology has not been possible. New regulations must, therefore, retain a technology basis.

LIQUID FUEL FROM WET SLUDGE

Conversion of municipal treatment plant sludge to fuel eliminates the problem of disposal and provides some income from sale of the fuel or saves operating expense through use in the treatment plant. A number of attempts have been made to produce a liquid fuel (usually called oil even though the composition is probably quite different from that of petroleum based products) from dried sludge. In all cases some amount of char is also produced. Canadian investigators have developed a process to the large pilot scale. Use of a dry sludge requires energy to remove water from the sludge and greatly reduces the net energy for fuel production. Use of a wet sludge as a raw material eliminates the energy consuming drying step. A small continuous pilot investigation of the conversion of wet sludge to oil indicated technical feasibility of producing an organic liquid with a heating value comparable to other liquid fuels, but failed to provide a reliable material balance for the process.[1] More recently a study using a one-liter stirred autoclave operated in a batchwise manner was undertaken to obtain data on the degree of conversion of energy in sludge to liquid fuel.[2] An extraction procedure using Freon TF, 1,1,2-trichloro 1,2,2-trifluoroethane, was developed to remove the oil from both treated and untreated sludges. A similar extraction procedure

using a different solvent would probably be used for oil separation in an operating plant.

In carrying out the autoclave experiments about 400 g of wet sludge was added to the 316 stainless steel vessel and the vessel was then purged with nitrogen and pressurized to 100 psig. The contents were heated to an operating temperature of 345°C. Heating time was 1.5 hours and reaction time was 1.0 hour The reactor was then cooled to room temperature over a 3.5-hour period. Although the reaction time of one hour is probably appropriate for a full-scale plant, the heating and cooling times are not. By proper heat exchange these times would be substantially reduced. Freon extraction was carried out on 50-g samples of wet sludge or reactor product with pH adjusted to 2.0 using HCl. The pH was lowered to convert fatty acid salts to free fatty acids for effective extraction. Extraction was accomplished with four additions of 50 ml of solvent. The solvent was then removed by heating to 65°C. Recovered oil consisted, therefore, of Freon-soluble substances with boiling point greater than 65°C. The water-char mixture remaining after extraction was evaporated and dried at 103°C to determine the amount of char. Oil yields were calculated in relation to volatile solids content of the sludge. Heating values were determined with an oxygen bomb colorimeter. An attempt was also made to determine oil composition using GC/MS. After carrying out a number of preliminary runs to develop proper operating procedure and to obtain approximately optimum operating conditions, three sludges were evaluated in detail.

A primary sludge from the Little Miami Wastewater Treatment Plant in Cincinnati, Ohio taken in July 1986 was run with and without additions of 5% on a wet basis of sodium carbonate and sodium hydroxide, which are thought to act as catalysts. Total solids content of the sludge was 4% and volatile solids (VS) content was 2.65%. The results of the run without catalyst were as follows:

0.218 g/g VS of light brown oil was extracted from the feed sludge
0.295 g/g VS of a black oil was extracted from the reaction product
0.355 cal in oil/cal in feed sludge was obtained from the feed sludge
0.474 cal in oil/cal in feed sludge was obtained from the reaction product
Heat of combustion of both oil samples was about 9400 cal/g
Product was solid at room temperature, liquid above 50°C.

Results indicate that there was about a 35% increase in amount of oil over that in the feed from the thermal reaction, with a recovery of 47.4% in the oil of the heating value in the original sludge. In this case, the catalysts had negligible effect on oil production. A significant amount of the oil consisted of hexadecanoic and octadecanoic acid, materials making up a large part of natural fats.

A second sample of primary sludge and a sample of secondary sludge was obtained in September 1986 from the same treatment plant. Believing that a higher solids content would increase oil yield, the sludge samples were thickened to a VS content of about 11%. Sodium carbonate was added in an amount equal to 25% of the total solids. Results of runs with these sludges were as follow:

From primary sludge -

0.181 g oil/g VS was extracted from the feed sludge
0.313 g oil/g VS was extracted from the reaction product
0.314 cal in oil/cal in feed sludge was obtained from the feed sludge
0.524 cal in oil/cal in feed sludge was obtained from the reaction product

From secondary sludge -

0.035 g oil/g VS was extracted from the feed sludge
0.221 g oil/g VS was extracted from the reaction product
0.065 cal in oil/cal in feed sludge was obtained from the feed sludge
0.387 cal in oil/cal in feed sludge was obtained from the reactor product

The recovery of oil from thermal reaction of the primary sludge was 73% more than recovered from the unreacted feed, more than double the increase obtained with the earlier, more dilute sample. Recovery of thermal energy in the oil, compared to the earlier sample, was slightly higher at 52.4% of the energy in the sludge. Thermal reaction of the secondary sludge increased oil recovery by over 500%. Much of this increase is believed to result, however, from the thermal destruction of cells with release of lipids. Since only 38.7% of the thermal energy in the secondary sludge was recovered in the oil from that sludge, the thermal energy recoverable from the combined primary and secondary sludge from this plant would be slightly less than 50% based on these two samples.

Balances were carried out on VS. About 30% of the VS was lost in the gas that is formed, most of which is carbon dioxide. Of the remaining volatile solids, the following distributions were found:

	Oil	Char	Remaining Water
Primary sludge	48.7%	16.9%	34.4%
Secondary sludge	30.4%	44.3%	25.3%

The relatively higher fraction of VS associated with oil from primary sludge undoubtedly results from the higher original free oil content. Considerably more charing of the biological solids occurred with the secondary sludge under the chosen operating conditions. Chars could be used as fuel. Because most of the metals in the original sludge will remain with the char, there is some health risk from burning this residual. Landfilling is probably the best method of disposal. The aqueous streams resulting from thermal treatment contained about 6,000 mg/l and 11,000 mg/l total organic carbon from primary and secondary sludges, respectively. It is assumed this wastewater would be returned to the sewage treatment plant. Molton, et al.[1] in the earlier study with wet sludge obtained slightly more concentrated wastewaters. Because the biochemical oxygen demand (BOD) equaled about 70% of the chemical oxygen demand (COD) of these waters, the authors assumed they would be aerobically treatable.

Although the feasibility of recovering about one-half of the thermal energy in sludge as a liquid fuel has been shown, the economics of a full-scale system still need to be determined. Moulton, et al.[1] presented costs, based on very preliminary data and without a clear understanding of how the oil would be separated from the char and water, which suggested the process could be practical for large municipalities. From the present investigation it is believed an extraction process using a low density solvent, such as propane, will be necessary. Filtration could also be necessary to obtain a solids-free fuel.

There are two lines of further investigation that need to be undertaken to clearly define the practicality of this process. The higher priority appears to be the conceptual design of a full-scale system based on existing data and carried out by designers experienced with thermal processing equipment. From the design a cost estimate could be made that would be reliable for the set of assumptions used. This design and cost study could then be followed by a continuously operating pilot plant study to affirm the assumptions made or to provide reliable revised information.

WET OXIDATION OF SLUDGE USING A VERTICAL REACTOR

A full-scale vertical reactor for wet oxidation of municipal wastewater sludge was constructed at Longmont, Colorado, in 1983 and has been operated on the sludge from that community[3]. The reactor is 1585 m long, extending vertically underground from the surface, and has an outside diameter of 25.4 cm. The reactor consists of concentric tubes which, starting from the center, include heat exchange fluid inlet, heat exchange fluid outlet, sludge inlet, and sludge outlet. Figure 1 is a diagram of the reactor. A detailed description of the installation, discussion of the possible advantages of the vertical reactor, and a progress report on operation was given at the Tenth United States/Japan Conference on Sewage Treatment[4]. The purpose of this discussion is to summarize the most important findings from operation at Longmont and to indicate possible future developments of the technology in the United States.

The performance of the vertical reactor is best characterized by COD reduction. Reduction of COD as a function of temperature at the bottom of the reactor over the period of testing is shown in Figure 2. Data shown include operation with both air and oxygen as the oxidizing medium. COD reduction increased with increasing temperature as expected. Scatter in the data results partly from variations that occurred in flow rate and in overall temperature profile. Although the average reduction was only 76.3 percent, reductions of 80 percent or more would be expected during normal operation. Early operation utilized air for oxidation which caused a number of operating problems, including the need to dilute the sludge. This condition is generally represented by the lower temperature data because maintenance of satisfactory reactor temperature of 260°C or higher was difficult with the low-energy dilute sludge. Use of oxygen increased oxidation capacity and allowed a higher rate of sludge solids processing. It appears that use of oxygen is necessary to make the vertical reactor technology practical. At a price of oxygen of $65/1000 kg, cost for this material is about $50/metric ton sludge which constitutes almost one half of the estimated operating cost. The

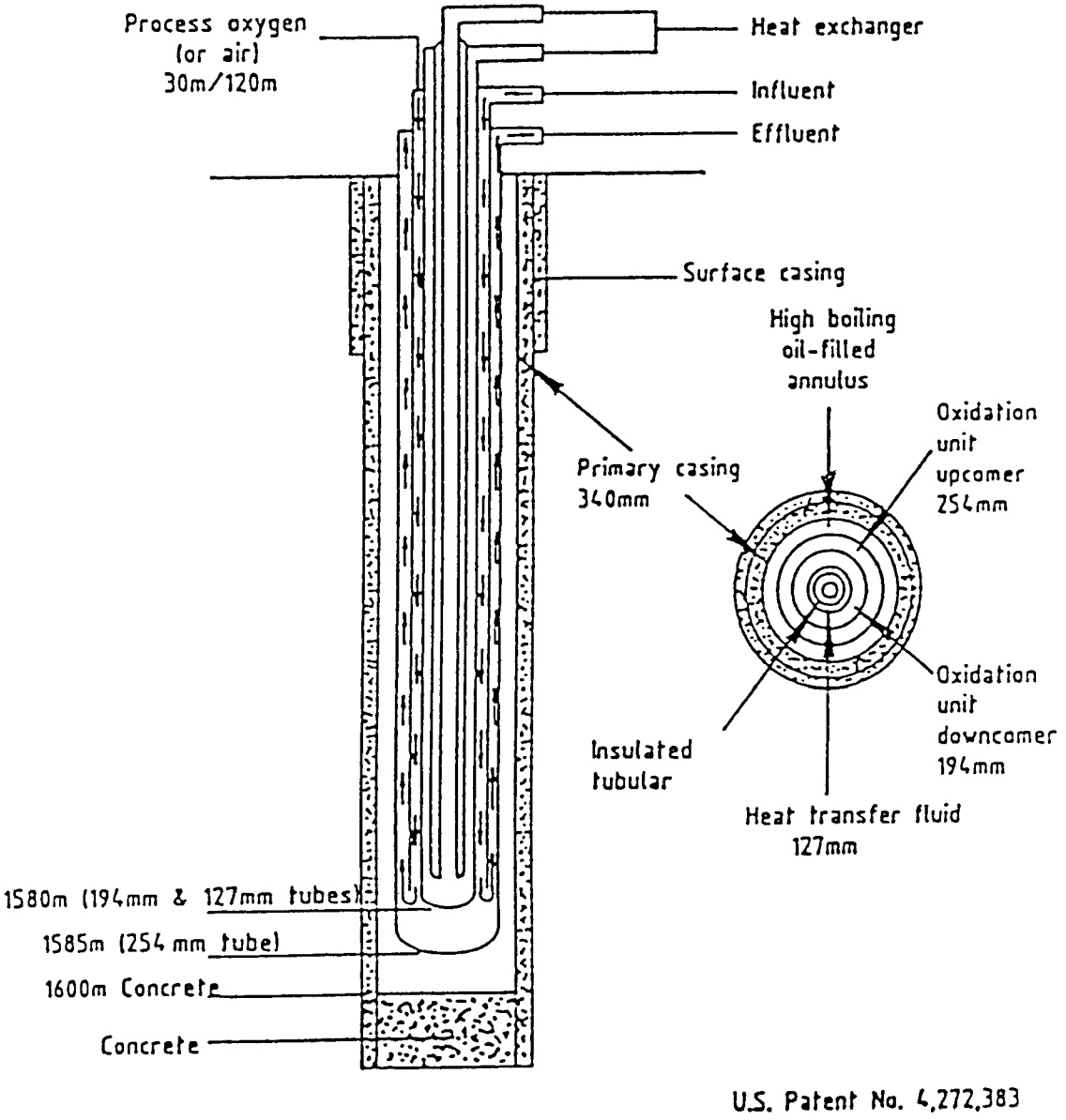


Figure 1. Subsurface cross-sectional schematic of the vertical reactor.

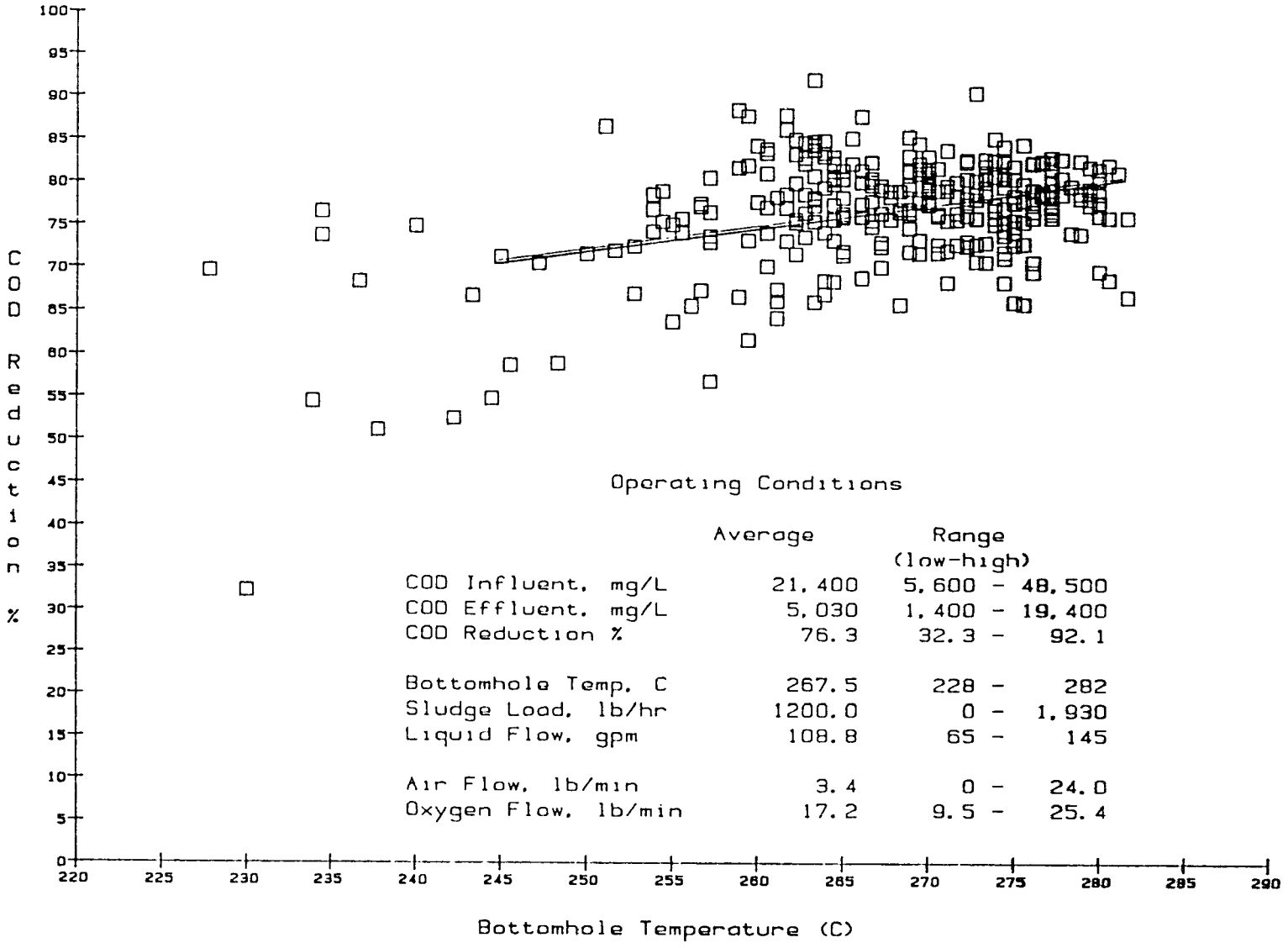


Figure 2. Reduction of COD in the vertical reactor.

overall removal of COD was shown to increase if some of the product from the reactor was recycled through the reactor. A longer residence time than the nominal 30 minutes of active oxidation time would be necessary, therefore, if a higher fraction of COD removal was desired. Another alternative would be higher operating temperatures.

Reduction of VS was slightly greater than COD reduction. Results with very low temperature data eliminated are shown on Figure 3. Reductions of 90 percent or more would be expected during normal operation.

Because the vertical reactor was large enough to treat all of the Longmont sludge, it was possible to determine, approximately, the effect of return of the settled liquor from the reactor on the biological treatment plant. An exact determination was not possible because periods with and without liquor return had to occur at different times with slightly different conditions. The biological treatment system consists of a trickling filter followed by a rotating biological contactor. The BOD of the liquor from the vertical reactor was about 4000 mg/L. Returning this stream before the trickling filter increased the influent BOD to the trickling filter by about 38 mg/L. Effect on the plant effluent BOD was an increase from 21 mg/L to 30 mg/L, with soluble BOD increasing from 13 mg/L to 17 mg/L. Suspended solids of both effluents was 27 mg/L. Although the Longmont plant was able to handle the additional load of reactor liquor, the investigators recommend evaluation of separate biological treatment, especially anaerobic, for cases where the treatment plant is close to its load limit.

Although the expected mode of residual or ash disposal would be by landfilling, the investigators carried out limited testing to determine whether the ash could be used as a brick additive. Standard brick mixes from two companies were investigated. For one mix 7.1% of wet oxidation ash was added. Test bricks exhibited 0.5 percent less dry shrinkage, 0.5 percent more fired shrinkage, and 5% reduction in fired weight. Compression strength and modules of rupture were comparable to those of other bricks. Some darkening of the brick occurred. In tests with the second brick company mix, wet oxidation ash was added in a ratio of one part to nine parts mix. This level of ash produced a slightly rougher edge during extrusion. It was concluded that 7 to 8% ash would be more satisfactory. Bricks made from the mix also exhibited less dry shrinkage, more fired shrinkage, and 3% less fired weight. Brick color was again slightly darker. It was concluded that this ash would be a satisfactory additive in brick manufacture.

This evaluation showed that wet oxidation of sludge in a vertical reactor is technically feasible, producing a biologically treatable liquid waste stream and a readily disposable residue very similar to incinerator ash. There were significant operating problems which the investigators were able to overcome. The original concept of using air for oxidation seriously limited capacity, and does not appear generally practical. Preliminary total cost of operation including amortization of the equipment is estimated at about $200/metric ton for a 25.4 cm diameter unit treating 9,100 metric tons/year. The cost is based on a relatively short operating period compared to the expected operating life of a plant and cannot reflect any longer term problems that could arise. The estimated cost is within the range experienced

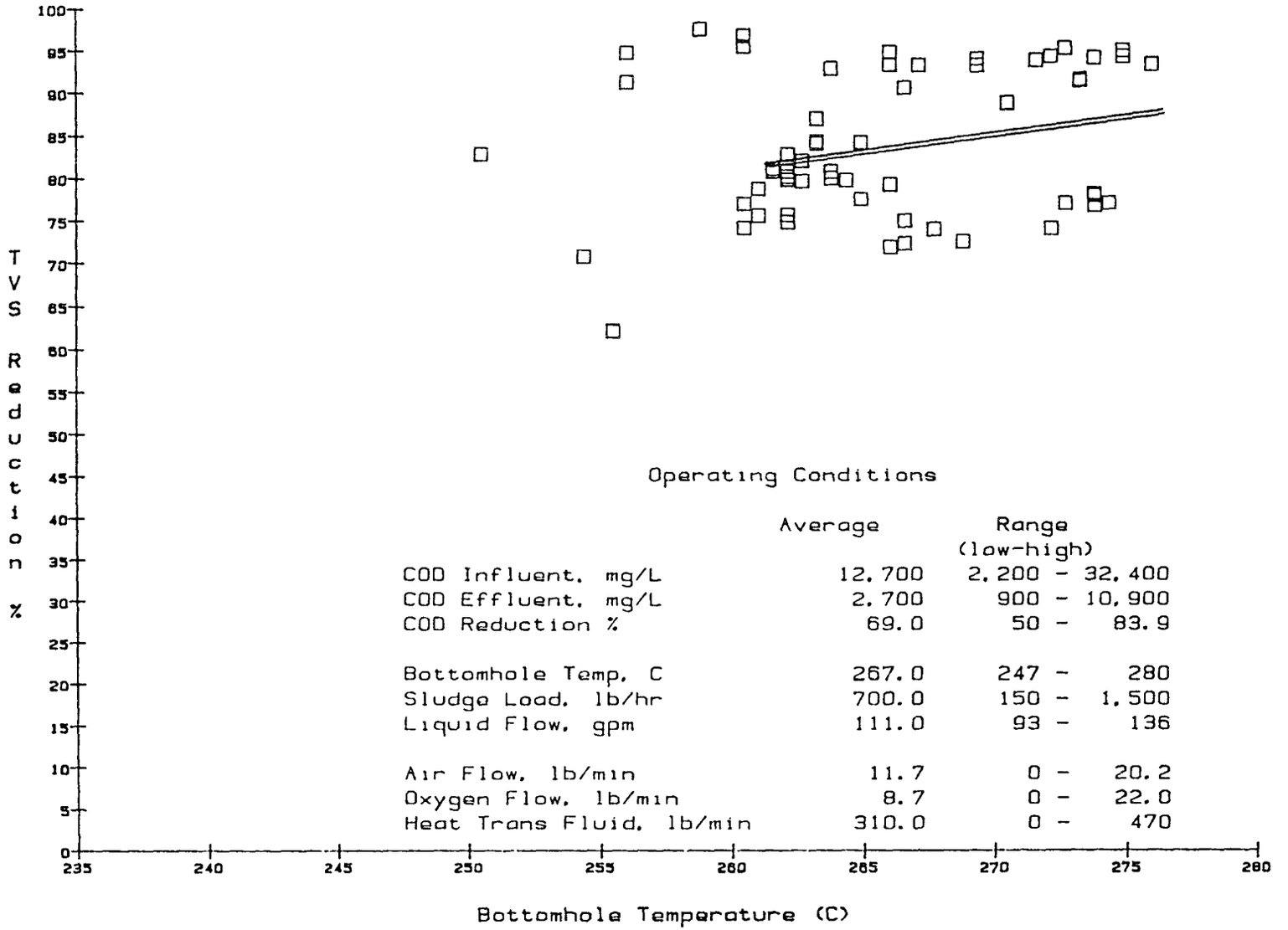


Figure 3. Reduction of total volatile solids in the vertical reactor.

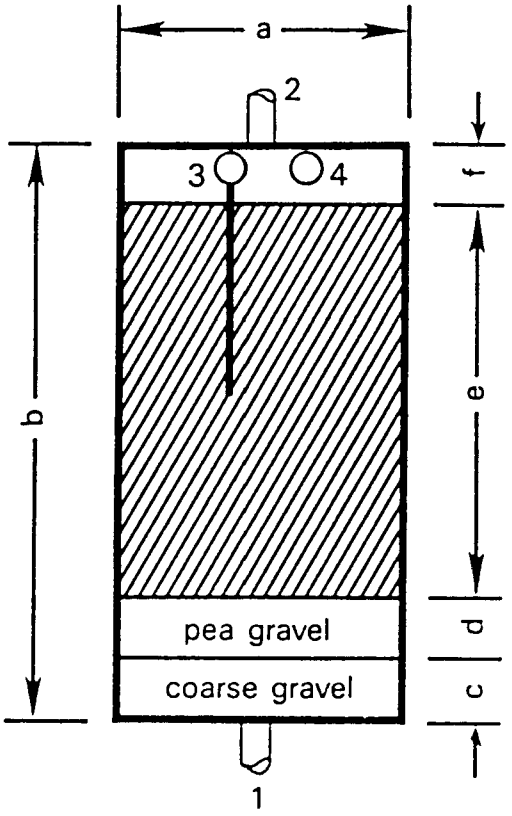
in the United States for other methods of sludge treatment and disposal.

Although a number of companies in the United States are interested in this type of technology, no plants other than that at Longmont have yet been constructed. One company is proposing use of essentially the same technology at temperatures above the critical point of water, 374°C. Under supercritical conditions, the properties of water change greatly. Organic solubility is greatly increased, and oxygen becomes miscible. The greater solubility, reduced mass transfer resistances, and inherently increased rate of oxidation that should result from higher temperature are expected to result in greatly increased capacity for a given size reactor and essentially 100% oxidation of most substances found in sludge. Batch autoclave tests with a number of chemicals, including PCB's, have shown nearly 100% destruction, usually in less than five minutes[5]. Although the application of supercritical oxidation to toxic wastes would appear to be a more promising application, the developers of this process also believe it can be cost competitive for treatment of sludge. For the supercritical approach to be competitive with the subcritical will require that the somewhat higher cost of the supercritical approach be offset by the reduced cost of handling a reactor liquid waste stream with significantly reduced organic content.

CODISPOSAL OF SLUDGE IN SOLID WASTE LANDFILLS

In the United States about 40% of the sludge on a dry weight basis is disposed to solid waste landfills. It is presumed by many in the sanitary field that adding sludge to solid waste landfills increases both the amount and strength of leachate from these landfills. As a result, some municipalities and states are prohibiting the disposal of sludge to these landfills. Because of the large amount of sludge that is disposed by this method, significant reduction in the availability of this disposal route could lead to problems for many municipalities that do not have convenient disposal alternatives. To learn the effect of sludge in solid waste landfills a study was initiated several years ago. More than four years of data has now been collected on leachate composition and biological activity as measured by production of methane[6].

Because of the difficulty of setting up different conditions within an actual landfill that would give reliable comparisons, lysimeters were used to carry out the investigation. Figure 4 shows the details of these test cells. The cells were of steel coated with epoxy sealer. A factorial experiment was designed for the solid waste-sludge (SW-SL) experiments that included two sludge types from Washington, D.C., anaerobically digested and lime stabilized, two simulated rainfall or infiltration rates, and three levels of sludge addition. Controls with no sludge were included. Solid waste was from Cincinnati, Ohio. Moisture content was 42%. Table 1 gives the design parameters. Cells 13 to 16 received a dose of small amounts of several toxic chemicals, but analytical problems prevented comparisons of toxics in leachate from these cells with leachates from cells 5 to 8. The toxics produced no significant effects on any other measured parameters so these cells duplicate cells 5 to 8. Solid waste and sludge were loaded and compacted in four 0.46-m-high lifts. Leachate was drained once per month and the volume was



1. Leachate Drain
2. Infiltration Line
3. Temperature Probe
4. Gas Port

Dimensions (m)	SW, SW-SL
a	1.8
b	2.7
c	0.3
d	0.3
e	1.8
f	0.3

Figure 4. Test cell design.

TABLE 1. EXPERIMENTAL DESIGN

Cell Contents	Test Cell	Sludge Type	Infiltration Rate (Low, High)	Sludge Loading (percent by wet weight)
SW-SL	1	AD[a]	L[c]	10
SW-SL	2	LT[b]	L	10
SW-SL	3	AD	H[c]	10
SW-SL	4	LT	H	10
SW-SL	5	AD	L	20
SW-SL	6	LT	L	20
SW-SL	7	AD	H	20
SW-SL	8	LT	H	20
SW-SL	9	AD	L	30
SW-SL	10	LT	L	30
SW-SL	11	AD	H	30
SW-SL	12	LT	H	30
SW-SL	13	AD	L	20
SW-SL	14	LT	L	20
SW-SL	15	AD	H	20
SW-SL	16	LT	H	20
SW	17	NONE	L	0
SW	18	NONE	H	0
SW	19	NONE	L	0
SW	20	NONE	H	0

[a]AD = Anaerobically digested sludge, 16% solids.
[b]LT = Lime treated sludge, 16% solids.
[c]L and H = Annual water infiltration rate (L/kg of cell waste on a dry weight basis), L = 0.5, H = 1.0.

TABLE 2. ANNUAL AVERAGE OF COD CONCENTRATIONS OF MONTHLY LEACHATE COLLECTION FOR GROUPS OF TEST CELLS

Year	1	2	3	4
Test Cell Grouping				
		COD (mg/L)		
SW	39,000	30,000	16,000	1,480
SW-SL-AD	10,600	2,190	1,090	700
SW-SL-LT	26,500	9,930	1,670	930

recorded. Infiltration water was added once per month as indicated in Table 1 after draining the leachate.

A large amount of data was obtained from this study and considerable evaluation of this information was carried out. Only the most important findings will be presented here.

Leachate was not produced immediately nor even after early additions of simulated rainwater in any of the test cells. Infiltration gradually increased until it became equal to the added water. The time for this to occur was between six months for the higher water addition rate and the highest percentage sludge to 16 months for the lower addition rate and solid waste without sludge. After the leachate rate reached the rate of water addition, it consistently maintained that rate. The only effect of the sludge was to decrease by four to six months the time for which leachate rate equalled water addition or infiltration rate.

The most significant result of this study was the effect of sludge in lowering the gross organic content (measured as COD) of the leachate. Figure 5 compares results for the 20%-sludge cells at high infiltration rate with the SW-only cells. Similar results were obtained for all cells in which sludge was added. These results are summarized in Table 2. For each of the four years, sludge addition produced lower leachate COD. For anaerobically digested sludge, the COD in the leachate from SW-SL cells had dropped sharply by the end of one year. In the SW-only cells, a similar drop did not occur until the end of the third year. Decline in leachate COD from lime-treated sludge occurred less rapidly than for anaerobically-digested sludge. One objective of this study was to show the effect of sludge on toxics in leachate from landfills. Although analytical problems prevented this objective from being attained, the very significant decline in gross organic content resulting from sludge addition provides some justification that a similar reduction would be found for at least some toxic materials.

The decline in leachate COD was accompanied by an increase in methane-rich gas. Table 3 summarizes the rate of gas production and gas composition. Clearly, the SW-only cells were slower to produce methane than the SW-SL cells. Although the type of sludge is not differentiated in the table, lime-treated sludge was slower to produce gas than the anaerobically-digested. By the third year the volume of gas from SW-SL cells had declined significanlty and was surpassed by gas production from the SW-only cells. Over the four-year period total methane production from SW-SL cells still exceeded production from the SW-only cells, but over the long term the difference would probably disappear. The pattern of methane production offers an explanation for the early marked decline in leachate COD from SW-SL cells. The decline resulted from the early initiation of anaerobic biological activity in those cells. Approximate calculation of the amount of dry solids converted to methane produced would account for the COD decline. The higher pH of the lime-treated sludge apparently suppressed somewhat the rate of anaerobic activity in those cells and resulted in the slower decline in leachate COD. The reason for leachate COD remaining low after decline in gas production and, therefore, anaerobic activity in the SW-SL cells, is probably the exhaustion of the more soluble materials in the SW-SL mixture. Addition of more

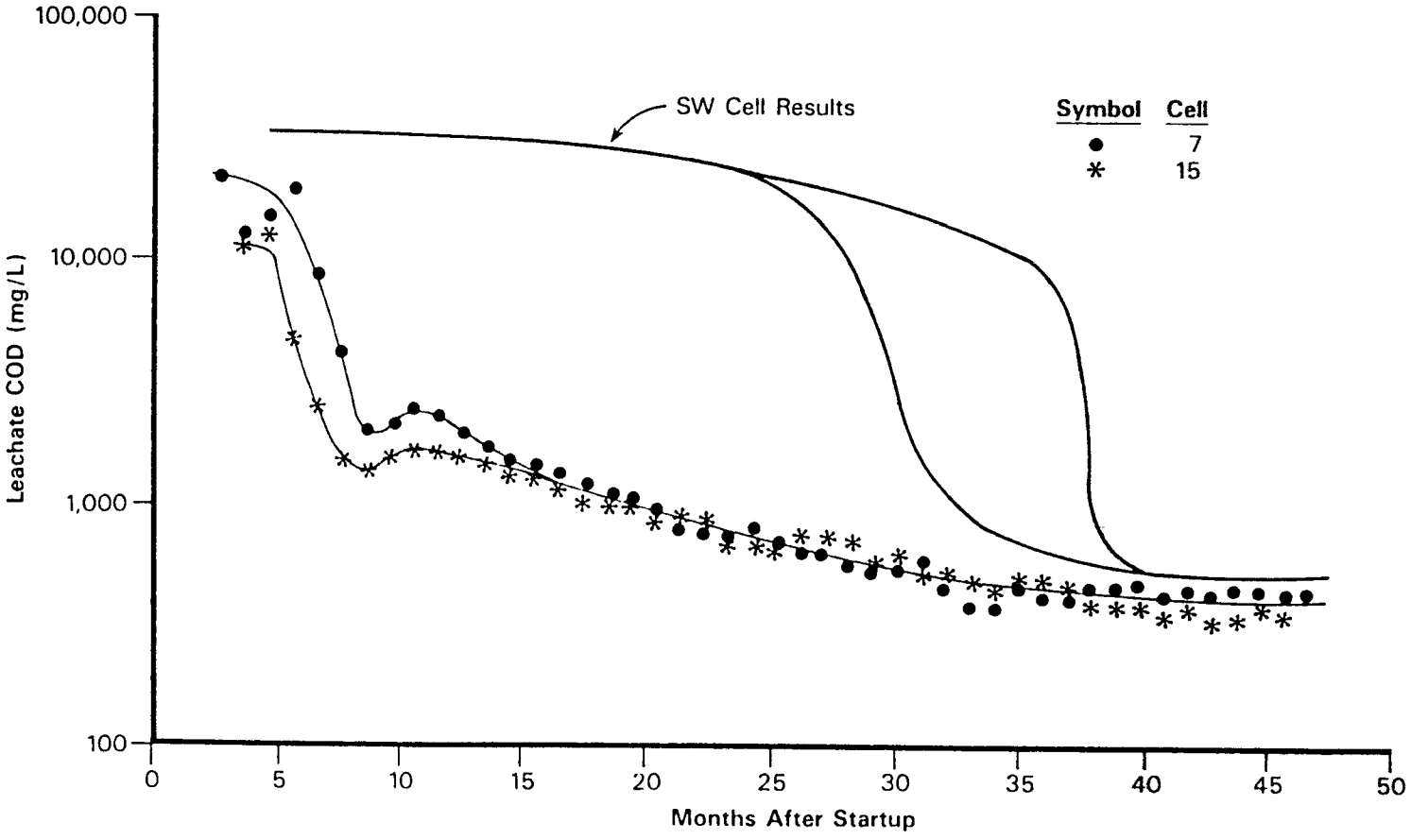


Figure 5. Comparison of leachate COD from solid waste cells and cells with 20 percent sludge.

sludge to the cells, and the resulting contribution of nutrients to the system, might be expected to stimulate more anaerobic activity and again increase methane production. Additional sludge has been added to some of the cells, but results in gas production are not yet available.

TABLE 3. ANNUAL AVERAGE GAS PRODUCTION AND METHANE CONTENT

Year	1	2	3	4
		Percent Methane		
SW	11.5	43.6	52.4	54.2
SW-SL	38.1	54.1	55.3	55.5
		Average Gas Production (L/hr)		
SW	1.2	4.2	7.0	7.5
SW-SL	10.0	12.1	4.9	3.0

Toxic metals data in the leachates were obtained over the history of the project. Summarized data for each year are shown in Table 4. All SW cells and SW-SL cells were averaged in obtaining the values shown in the table. In all cases the metals were lower in leachates from SW-SL cells even though the original metals concentrations in the sludges were all higher than in the solid waste. Except for zinc, which showed an increase in the second and third years in the SW-only cells, all metals declined with time and reached similar values in both SW-only and SW-SL cells at the end of four years. The reason for the temporary increase in leachate zinc could have been dissolving of metallic zinc under the slightly acid conditions found in the cells before methane production. Anaerobic biological activity increased the pH by about one unit. The general decrease with time is probably due to a combination of washout and precipitation from pH increase. Since the amounts of metals leached over the four-year period represent only a small percentage of the total metals in the cells, further leaching of small amounts would be expected for a long period of time.

This project has shown that the addition of municipal wastewater sludge to solid waste landfills is not environmentally detrimental, but could be beneficial. Although production of initial leachate may begin earlier, the long term volume of leachate is not measurably increased. The gross organic content of the leachate and, therefore, the annual mass emission, is greatly decreased when the landfill is new and remains lower for at least four years. If recovery of fuel gas is desired, addition of sludge hastens the process by about two years. It may be possible to continue production of gas for very long times by repeatedly injecting additional sludge every few years. With this manner of operation the landfill would serve as a long term sludge disposal site. The toxic metals content of the leachate, when the landfill is new, is also decreased by the addition of sludge. The difference appears to decrease with time and essentially disappears by the end of four years.

TABLE 4. AVERAGE ANNUAL CONCENTRATIONS OF METALS IN LEACHATE

	Year			
	1	2	3	4
		Cadmium (mg/L)		
SW	0.039	0.029	0.007	0.004
SW-SL	0.034	0.018	0.006	0.003
		Chromium (mg/L)		
SW	0.142	0.096	0.042	0.019
SW-SL	0.087	0.053	0.028	0.022
		Copper (mg/L)		
SW	0.044	0.042	0.030	0.012
SW-SL	0.039	0.034	0.027	0.013
		Iron (mg/L)		
SW	1400	1330	270	78
SW-SL	660	54	39	40
		Lead (mg/L)		
SW	0.298	0.232	0.102	0.050
SW-SL	0.229	0.129	0.063	0.043
		Nickel (mg/L)		
SW	0.64	0.60	0.35	0.21
SW-SL	0.33	0.22	0.21	0.18
		Zinc (mg/L)		
SW	2.19	12.0	3.62	0.14
SW-SL	0.60	0.30	0.12	0.12

COMPARISON OF BACTERIAL CONTENT OF SLUDGES FROM CONVENTIONAL ACTIVATED SLUDGE AND LONG AERATION PROCESSES

Present sludge regulations for land disposal in the U.S. assume that the pathogenic microorganism content of the mixture of primary sludge and activated sludge obtained from a conventional primary-activated sludge plant is

the same as from plants with long aeration and no primary treatment. As a result, the same degree of additional treatment must be given to all sludges that are disposed to the land. Arguments have been made that the long aeration time of plants such as extended aeration is comparable to separate aerobic digestion of the sludge and should reduce the requirement for further treatment of the sludge. To provide information on the relative destruction of pathogenic microorganisms in sludges from conventional activated sludge (CAS) compared to extended aeration (EA), a side-by-side continuous pilot study was carried out using the same wastewater feed[7].

The pilot study was carried out using screened wastewater from the Mill Creek Plant, Cincinnati, Ohio. The CAS system included the primary settler, aerator and final settler. Feed rate was 1.6 gal/min (7900 L/day), the aerator hydraulic residence time was about eight hours, and the approximate solids retention time (SRT) in the aerator was seven days. The EA pilot system did not include a primary clarifier. The feed rate to the EA system was 0.93 gal/min (5100 L/day), and the aerator hydraulic residence time was about 24 hours. The SRT was varied over a range from 19 days to 32 days, but reduction in microorganisms did not change within this range. Actual pathogens were not measured in the study. It was intended to include salmonella, but levels near or below the detection limit prevented this. The common indicators, total coliforms, fecal coliforms, and fecal streptococci, were used instead to characterize the effects of the two treatment systems. About 30 sets of grab samples were taken from each system for bacterial analysis.

Overall removals for each treatment system are shown in Table 5 for the three indicator organisms. Since the microorganism determinations are done on wet sludge, the results can be reported on a total volume basis or a solids basis. Regarding disposal on land, organism density on a total solids basis is usually of greatest interest. In this study, a modification was made with results reported on a volatile suspended solids (VSS) basis. Similar results would have been obtained using a total solids basis, except for very dilute samples where inorganic materials can become significant. Table 5 shows clearly that the longer-SRT process, EA, results in a greater reduction of indicator organisms. The difference expressed as log-to-base-ten ranged from 0.84 for fecal streptococci to 1.21 for total coliforms. Although not shown in Table 5, results from samples taken of the primary and secondary sludge from the CAS system show that about 85 percent of the organisms are associated with the primary sludge. Much of the improvement of the EA system results from the fact that the primary sludge undergoes what is comparable to a degree of aerobic digeston in this system.

Although actual pathogens were not measured in the pilot study, in an earlier study a bacterial pathogen, salmonella, was measured along with the three indicator organisms measured in the pilot study[8]. In this earlier study reductions across four long-SRT plants without primary were compared with reduction across one conventional primary-activated sludge plant. The long-SRT plants included two EA plants and two oxidation ditches. Reduction of the indicator organisms over the four long-SRT plants were remarkably consistent. Average reductions expressed as logs were 1.76 for total coliforms, 2.22 for fecal coliforms and 1.64 for fecal streptococci. Results are almost identical or only slightly higher than those shown in Table 5 for the pilot study.

TABLE 5. BACTERIAL DENSITIES IN SOLIDS FROM EXTENDED AERATION AND CONVENTIONAL ACTIVATED SLUDGE

Indicator	Sample	Bacterial Density[$\log_{10}$(no./g VSS)]
Total Coliforms	Influent Solids	9.28
	EA[a] Sludge	7.59
	Difference[b]	1.69
	CAS[c] Sludge	8.80
	Difference	0.48
Fecal Coliforms	Influent Solids	7.95
	EA Sludge	6.00
	Difference	1.95
	CAS Sludge	7.05
	Difference	0.90
Fecal Streptococci	Influent Solids	7.34
	EA Sludge	5.99
	Difference	1.35
	CAS Sludge	6.83
	Difference	0.51

[a]EA Extended Aeration
[b]Difference Influent Solids Bacterial Density - Sludge Bacterial Density
[c]CAS Conventional Activated Sludge

Results for salmonella were complicated by some determinations being less than the detectable limit, but estimates of the median reductions for the four longSRT plant were from 1.1 to 1.3 logs. Salmonella reduction for the CAS plant was obtained for both the waste activated sludge and the primary sludge. As was the case for indicator organisms in the pilot study, much of the salmonella was associated with the primary sludge. Reduction in the waste activated sludge was about 1.3 logs, but for the primary only 0.7 log, giving a reduction for the mixture of about 0.8 log. This reduction was, therefore, 0.3 to 0.5 log less than for the long-SRT plants.

Well operated digestion of mixed primary and activated sludge has been observed to result in about two logs indicator reduction and one log salmonella reduction. Data obtained from these studies indicate there is approximately a one log greater reduction of indicator organisms for long-SRT-no-primary systems compared to CAS and 0.3 to 0.5 log greater reduction of salmonella. Although there is a need for additional data, especially for pathogenic organisms, available data suggest that some credit regarding further sludge treatment for land disposal should be given to the long-SRT systems.

REFERENCES

1. Molton, P. M., A. G. Fassbender, and M. D. Brown. STORS: The Sludge to Oil Reactor System. EPA-600/2-86-034, U.S. Environmental Protection Agency, Cincinnati, Ohio, 1986.

2. Lee, K. M., P. Griffith, J. B. Farrell, and A. E. Eralp. Conversion of Municipal Sludge to Oil. Accepted by Water Pollution Control Federation Journal, July 1987.

3. The City of Longmont, Colorado. Aqueous-Phase Oxidaiton of Sludge Using the Vertical Reaction Vessel System. EPA/600/2-87/022, U. S. Environmental Protection Agency, Cincinnati, Ohio, 1987.

4. Morill, G. B. Municipal Sludge Oxidation with the Vertical Tube Reactor. In: Proceedings, Tenth United States/Japan Conference on Sewage Treatment Cincinnati, Ohio, 1985, pp 502-515, EPA/600/9-86/0156, 1986.

5. Modell, M. Processing Methods for the Oxidation of Organics in Supercritical Water. U.S. Patent No. 4,543,190, issued September 4, 1985.

6. Farrell, J. B., G. K. Dotson, J. W. Stamm, and J. J. Walsh. The Effects of Municipal Wastewater Sludge in Leachates and Gas Production from Sludge Refuse Landfills. Presented: U.S./U.S.S.R. Bilateral Agreement Symposium on Municipal and Industrial Wastewater Treatment, March 20-21, 1987, Cincinnati, Ohio.

7. Lee, K. M., J. B. Farrell, A. E. Eralp, and R. A. Rossi. Bacterial Density Reduction in Activated Sludge Processes. Internal Report, U.S. Environmental Protection Agency, Cincinnati, Ohio 1987.

8. Farrell, J. B., B. V. Salotto, and A. D. Venosa. Reduction in Bacterial Densities of Wastewater Solids by Three Secondary Treatment Processes. Internal Report, U.S. Environmental Protection Agency, Cincinnati, Ohio, 1987.

STATUS OF THE POTABLE WATER REUSE DEMONSTRATION PROJECT AT DENVER

by

William C. Lauer, Project Officer
Denver Reuse Plant
Denver, Colorado 80022

and

John J. Convery, Director
Wastewater Research Division
Water Engineering Research Laboratory
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

Prepared for Presentation at:

Eleventh United States/Japan Conference
on Sewage Treatment Technology
Tokyo, Japan

October 12-14, 1987

INTRODUCTION

The purpose of this paper is fourfold: (1) report on the current status of the Denver Potable Water Reuse Demonstration Project; (2) describe the Phase I process performance results; (3) evaluate these performance results for compliance with the new requirements in the recent Amendments to the Safe Drinking Water Act and (4) present a summary of the Phase I operating costs of the Denver facility.

The project is designed to determine the feasibility of renovating secondary treated wastewater to potable water including resolution of the issues of water quality safety, process dependability, economics, regulatory agency approval and social acceptability.

Process dependability includes both the concepts of reliability and utilization. Reliability is the ability of the process to produce water which satisfies specified standards such as the EPA Primary and Secondary Maximum Contaminant Levels (MCL) and Maximum Contaminant Level Goals (MCLG's) for drinking water. The MCL's are enforceable standards required to be set as near as "feasible" to the MCLG's (treatment goals), taking cost into consideration. The MCLG's are required to be set at levels that would result in no known or anticipated adverse health effects with an adequate margin of safety. The design reliability standards based upon EPA Primary and Secondary MCL's are shown in Tables 1 and Table 2, respectively. The EPA proposed MCLG's are shown in Appendix A. Utilization is measured by comparing the quantity of water produced which satisfies the reliability parameters to the maximum production capability exclusive of lost production due to power failures.

$$\text{UTILIZATION FACTOR} = \frac{(\text{Gallons produced} - \text{Gallons not meeting reliability standards}) \times 100}{\text{Maximum production in gallons (exclusive of power outages)}}$$

The 1986 Amendments to the Safe Drinking Water Act were signed by the President of the United States on June 19, 1986. Every public water supply in the Country must meet the National Primary Drinking Water Regulations which currently consist of 21 contaminants. The new Amendments require the regulation of some 83 contaminants within three years[1]. A complete list of contaminants which are currently regulated or proposed for regulation is shown in Appendix B. EPA has the option of substituting up to seven other contaminants for those on the list if the Agency finds this will provide greater health protection. The Agency is currently considering the deletion of zinc, vanadium, sodium, molybdenum, dibromomethane, aluminum and silver. Additions to the list which are being considered

include aldicarb sulfoxide, aldicarb sulfone, ethylbenzene, heptachlor, heptachlor epoxide, styrene, nitrate. In addition to the above 83, at least 25 more primary standards will be required by 1991; with 25 more standards expected every three years thereafter. By 1988, EPA must specify criteria for filtration of surface water supplies and by 1990, criteria for disinfection of surface and ground water supplies. The Act also requires that MCLG's and MCL's be proposed simultaneously and promulgated simultaneously. The Administrator of EPA must list the technology, treatment technique, and other means that he determines are feasible for meeting the MCL. This does not mean that these specific techniques must be used for meeting the MCL. To overcome the lack of information on the occurrence of contaminants of potential health significance, the Act requires cities to monitor another set of 50 contaminants shown in Appendix C.

The 1.0 MGD (.043 m^3/s) Demonstration Facility at Denver began full operation on October 1, 1985. EPA provided research grant funding of $7.05M toward the $30 million project cost which includes the capital cost of the facility, five years of operation and testing as well as health effects studies.

The Process Flow Scheme[2] consists of: single stage of lime clarification, recarbonation, pressure filtration, selective ion exchange for ammonia removal, first-stage activated carbon adsorption, ozonation, second-stage activated carbon adsorption, reverse osmosis, air stripping and chlorine dioxide disinfection. Unit processes following first-stage activated carbon treatment are operated at a reduced flow rate of 0.1 MGD (.0043 m^3/s).

Process alternatives are to be evaluated in four phases to identify the overall treatment sequence which could provide the best quality water at least cost. Special removal studies, contaminant dosing studies, unit process optimization studies and selected health effects testing will also be conducted to fully characterize the plant performance.

Following the four phase plant optimization period lasting two years, the process configuration to be used for the remainder of the project will be selected. During this phase of the study health effects testing including whole animal testing will be conducted to compare the plant product water to the Denver potable supply. A public information program introducing the concept of water recycling to future consumers and an extensive quality assurance program are also being conducted.

PHASE I TESTING PROGRAM

During Phase I, which lasted from October 1, 1985 to March 28, 1986, the ion exchange and regenerant recovery system for ammonia removal were not operated. The process schematic for Phase I is shown in Figure 1. The

operational parameters for the unit processes operated during this phase of the study are shown in Table 3.

Following first-stage activated carbon treatment the flow stream was split with 0.1 MGD (.0043 m^3/s) receiving the additional treatment. The remainder of the flow 0.9 MGD (.039 m^3/s) was disinfected with chlorine dioxide and returned to the wastewater treatment plant for use in nonpotable systems.

Phase I test results for general, radiological, microbiological, and inorganic parameters are listed in Table 4. The results for the Reuse Plant Influent, Reuse Plant Product, and the Denver Drinking Water comparison are presented for the time period October 1, 1985 - March 28, 1986. It is interesting to note that with the exception of turbidity, fluoride, asbestos, and the microbiological parameters, the measured Reuse Plant Influent Parameters satisfy all existing and proposed primary drinking water standards. The Reuse Plant Effluent satisfies all of the primary, secondary and proposed recommended MCL's except pH as shown in Tables 5, 6, and 7. The finished water pH in a full production plant could easily be adjusted to be in the appropriate range. Removals of most parameters through the Phase I treatment processes exceeded 95% except for boron, uranium, chloride, silica, TKN and ammonia-N. Figures 2 and 3 show the progressive removal of Total Organic Carbon (TOC), Membrane Heterotrophic Plate Count (M-HPC), and total coliform through the sequence of unit processes. No significant removal of the aerobic heterotrophic organisms occurred until the reverse osmosis unit. It is notable that the results for the Reuse Plant Product are either lower than or not significantly different from Denver Drinking Water for every parameter tested with the exception of three: Boron, Ammonia-N, and Total Kjeldahl Nitrogen (TKN).

There are no U.S. standards for boron in drinking water. A limit has been set at 5.0 mg/L for boron in Canada.[3] The levels found in the Reuse Plant Product and Denver Drinking Water are near the analytical detection limit (0.20 and 0.12 mg/L) and are more than a factor of twenty lower than the Canadian limit.

The higher ammonia-nitrogen concentrations in the Reuse Plant Phase I Product than in the Denver Drinking Water (3.2 mg/L vs 0.3 mg/L) were expected since the ammonia removal ion exchange process was not operated during the evaluation period. The higher TKN results can be fully explained by the increased ammonia-N values. There are no standards for the ammonia-N concentration in drinking water in the United States, Canada or set by the World Health Organization. However, there is concern regarding the conversion of ammonia-N to nitrite-N or

nitrate-N. Both of these nitrogen species are or will be regulated. The current MCL for NO_3-N is 10 mg/L. The proposed MCLG for NO_2-N is 1 mg/L. There are definite health effects associated with ingestion of these ions.[4] The residual ammonia levels experienced in this test can be reduced by various means including chlorination. Alternate methods for ammonia-N removal will be investigated in future process sequence evaluations which may omit the selective ion exchange system.

Confirmed trace organic compounds found in the Plant Influent, Plant Product, and Denver Drinking Water are listed in Table 8. The compounds are grouped by their maximum concentration over the October - March period. Compounds which have been tentatively identified by mass spectra library comparison for the Reuse Product Water and Denver Drinking Water are shown in Appendices D and E, respectively. Estimated concentrations are based upon internal standards. Compounds identified as having been confirmed are based upon known concentrations of certified standards. Three sample preparation methods were used for organic analyses: Grob closed-loop stripping, purge and trap, and liquid-liquid extraction. The results listed combined data from all of these methods. Only five compounds were found at concentrations above 1 ug/L in the Reuse Plant Product water. These represent single events and maximum concentrations. Denver drinking water contained eleven compounds with concentrations in excess of 1 ug/L. None of the compounds identified in either sample were in concentrations approaching any established or proposed standards.

Many analyses were performed which produced undetectable results or limited data with low concentrations. These included: rare earth elements, radionuclides, microscopic examination, enteric virus, arsenic, cyanide, Campylobacter, Salmonella, Shigella, parasites and asbestos. Most of these tests were intended as screening tests which were performed infrequently. If concentrations of concern were found, the frequency of sampling would have been increased. However, this was not necessary in either the Reuse Product or Denver Drinking Water samples.

UNIT PROCESS OPERATING EXPERIENCE

CHEMICAL CLARIFICATION

Annual cleaning of the influent pipe line to remove organic slime deposition was found to be necessary to maintain the raw water flow plus backwash equalization basin recycle flow at 1.0 MGD (.043 m^3/s) with the existing pump. Retrofitting the influent channel with fine bubble diffusers (1 ft^3 air/ft^3 water) to strip carbon dioxide from the secondary effluent has reduced the required chemical lime dose by ten percent resulting in a savings of $10,000 per year. Each mg of CO_2 consumes 1.7 mg of lime. The tertiary lime clarification system is operated as a single stage excess lime treatment system with ferric chloride addition as a coagulant. The pH feedback process control system has been working well since the response time was reduced to ten seconds by feeding hydrated

lime with an eductor. The pH probe requires daily cleaning with an acid rinse. Carbon dioxide is sparged beneath a radial flow turbine mixer to reduce the pH to 7.7 prior to entering the ballast pond.

Lime clarification reduced the influent turbidity from 8.6 to 1.9 NTU. Total organic carbon was reduced from 17.2 mg/L to 8.2 mg/L.

PRESSURE FILTRATION

Two of the three multi-media pressure filters are operated simultaneously at 4.42 gpm/ft^2 (0.18 m/min). Each filter has a surface area of 78.5 square feet (7.3 m^2). The media for the filters consists of 16.5 inches (0.42 m) of anthracite coal (effective size = 1.0 to 1.1 mm, sp. gr. = 1.6); 9 inches (0.23 m) of sand (effective size 0.45 to 0.55 mm, sp. gr. = 2.6); and 4.5 inches (0.11 m) of garnet (effective size = 0.3 to 0.4 mm, sp. gr. = 4.0) supported by 10 inches (0.254 m) of gravel. The filter cycles ranged from 16.0 to 19.0 hours and were terminated when the headloss reached 11 feet (3.35 m). The backwash sequence includes a surface wash step. The surface wash routine takes about 17 minutes at an average flow rate of 56.5 gpm (0.00356 m^3/s). The total backwash waste flow is equal to about 7.5% of the total filter flow. This corresponds to an average filter production efficiency of 92.5%. Filter flow was wasted for 10 minutes prior to returning the unit to production service.

The filters removed an average of 78% of the influent turbidity to a level of 0.33 NTU. Filter effluent quality was highly consistent and independent of the influent turbidity levels within the range of 0.6 to 3.3 NTU. The dissolved oxygen was also reduced across the filters from a geometric mean of 7.53 mg/L to an effluent value of 2.05 mg/L. Total coliform counts were reduced by an average of 32% across the filters. Substantial biological activity and loss of dissolved oxygen within the filters was not controlled by shock treatment of the filters for six hours with 75 mg/L of chlorine dioxide on a monthly basis.

Multiple (4) and topical additions of solid calcium hypochlorite ($Ca(OCl)_2$, HTH) with dosages as high as 390 mg/L and contact time of 24-30 hours were able to reduce the initial oxygen depletion to 15%. For Phase II, the backwash flow rate was increased to 1500 gpm (0.094 m^3/s) for 25 minutes and surface wash time extended to 25 minutes.

CARBON ADSORPTION

The activated carbon used for both the first- and second-stage adsorption is Filtrasorb 300, a crushed coal-based carbon, in the 8 by 30 mesh (0.6 to 2.4 mm) size range. The nominal bed volume is approximately

3200 cubic feet with an empty bed contact time of 33 minutes. The hydraulic loading rate is 6.04 gpm/ft^2 (0.245 m/min) and the columns are operated in a downflow mode. During a six month evaluation phase this represents a dosage of 71 mg/L. Initially the columns were backwashed and the surface fines were skimmed which resulted in an initial clean bed headloss of 8.5 feet (2.59 m). The columns were backwashed at 17.7 gpm/ft^2 (0.718 m/min) when the headloss exceeded 15 feet (4.57 m). A combination of factors including: biological activity, lack of surface wash mechanisms in the column design and 20 feet (6.09 m) excessive free-board from bed surface to backwash waste header resulted in poor backwash performance and the necessity to periodically remove 55 ft^3 (1.55 m^3) of medium. This was equivalent to removing a six inch (0.15 m) layer of carbon. During the six months of Phase I operation 5% of the carbon was physically removed by skimming.

The first-stage carbon adsorption column removed 91% of the influent TOC initially and 51% after six months of operation. Figure 2 shows the geometric mean TOC concentration for each unit process for Phase I and indicates an average removal across the first-stage carbon column of 66%.

Virgin carbon purchased to make up for inventory losses is added to the second-stage column. Second-stage carbon is used to make up first-stage inventory needs. Only first-stage carbon is thermally regenerated. The second-stage column has a nominal bed volume of 340 cubic feet (9.58 m^3). The daily average flow is 86,000 gallons (3.25 m^3) which corresponds to a hydraulic loading of 4.7 gpm/ft^2 (1.06 m/min). Over the six month period of Phase I this corresponds to a dosage of 88 mg/L. The second-stage column had an initial headloss of 7.8 feet (2.38 m) after two feet (0.61 m) of fines were removed. No further fines removal were required. TOC removals ranged from a maximum of 85.7% initially to a minimum of 38% at the end of the period. The geometric mean TOC removal average 62% from 2.6 mg/L to 1.0 mg/L.

CARBON REGENERATION

The carbon regeneration system consists of the following functions: (1) carbon transport as a slurry 1.5 inch eductors (3.8 cm), (2) drying with a manually controlled variable speed dewatering screw auger feed system to the drying section of the fluidized bed system, (3) regeneration of the carbon by gasifying adsorbed organics in an oxygen limiting environment at 1600 - 1800°F, (4) incineration of the off gases in the presence of 3 to 4% oxygen and partial recycle to the dryer, (5) quenching of the carbon and (6) off gas treatment consisting of cooling and particulate removal in a variable-throat venturi scrubber and a tray impingement scrubber.

The actual performance of the carbon regeneration furnace compared to operating goals is shown in Table 9.

OZONE

The primary purpose of ozonation is to oxidize organic substances that were not removed in the first-stage carbon columns. Oxidation was expected to either remove some of the organics directly or alter their form to facilitate their adsorption in the second-stage carbon columns. Ozone also acts as a disinfectant for bacteria and viruses.

The ozone unit is designed to treat 0.086 MGD (0.0037 m^3/s) which is 10% of the discharge from the first-stage carbon adsorption process. The contact basin is rectangular in shape with a depth of 15.8 feet (4.82 m), a length of 14 feet (4.28 m), and a width of 2.5 feet (0.76 m) which provided a mean contact time of 60 minutes. The basin is partitioned by aluminum baffles into six contacting compartments with ozone diffusers and one quiescent zone which allows the release of non-absorbed bubbles of ozone. Released gases are collected and sent to the catalytic ozone destruction units.

Ambient air is compressed to 100 psig (689 kPa), filtered and dried to a dew point of - 70°F and then refiltered prior to entering the ozone generator at 11 psig (75.8 kPa). The air flow to the generators was 4228 actual ft^3/day (0.0014 m^3/s). The two ozone generators are cylindrical stainless steel tanks with 15 glass dieletric tubes each encircled by stainless tubes. A discharge field generates a corona effect in the gap between the glass and steel tubes. As air is pumped through the gap, oxygen molecules are split and ozone is formed. The production of ozone is varied by a voltage regulator that controls the amount of electricity to create the corona discharge. A geometric mean dose of 1.89 mg/L was provided to the contact basin which had an overall transfer efficiency of 72.7%.

The ozonation unit did not remove any TOC as shown in Figure 2. Total coliform removal averaged 66.4% (412 to 139 counts/100 ml). The membrane-heterotrophic plate count was reduced 40% to 1.8 x 10^5 counts/ml. Coliphage B removals were 98.8% to 1.4 PFU/100 mL.

REVERSE OSMOSIS

The second-stage carbon column effluent is acidified with hydrochloric acid to pH 6.0, dosed with sodium hexametaphosphate to inhibit calcium sulfate precipitation (6 mg/L), filtered through 5 micron polypropylene cartridge filters and pressurized to 260 psig (1790 kPa) for processing in one of three 35 gpm (0.0022 m^3/s) reverse osmosis units. Each unit consists of four first-stage tubes, two second-stage tubes and one third-stage tube. Each tube contains six spiral wound polyamide membranes. The water recovery through each stage is approximately 50% resulting in an overall water recovery of 86% and salt rejection of >97% based upon TDS.

Due to the relative permeabilities of the various carbonate species, the pH of the recovered permeate (4.6) is lower than that of feed. It is neutralized via air stripping at a gas to liquid ratio of 100:1 to remove 98% of the dissolved carbon dioxide to a residual of 4 mg/L. The brine stream (10% of flow) is presently discharged to the sanitary sewer.

When feed pressure increases 10-15% to maintain permeate flow, the unit is first cleaned with warm (32°C) citric acid solution (21 g/L) and adjusted to pH 3.5 to remove metal hydroxide and calcium carbonate scale. Next the unit is rinsed and cleaned with a warm solution of borax, EDTA, and trisodium phosphate (10 g/L each) to remove organic and biological residues.

Ammonia nitrogen and total organic carbon removals averaged 86% to residuals of 3.2 mg/L and 0.2 mg/L respectively. The few milligrams per liter of ammonia-nitrogen can be used beneficially to provide a chloramine residual in the finished water. Persistent coliform contamination of the permeate at a mean level of 1.5 counts/100 ml was observed and attributed to permeate side colonization rather than leakage.

Total operating time ranged from a minimum of 10 days to a maximum of 131 days. Longer term operating expectations are two months for each module if all upstream units are performing satisfactorily. Membrane life is now 4 plus years which exceeds the estimate of 3 years.

CHLORINE DIOXIDE

Chlorine dioxide was chosen because it is a more effective viricide and bactericide than chlorine, is applicable over a wide pH range and with careful control, and reduces the potential for formation of chlorinated organics. Chlorine dioxide is generated on-site by reacting chlorine solution with 25% sodium chlorite ($NaClO_2$) solution with vacuum-induced delivery of reactants. Chlorine dioxide output capacity is 5.8 - 41.8 lb/day (0.044 - 0.32 kg/day). Careful monitoring of the various chlorine species produced from the generator is necessary. The primary chlorine species include chlorine dioxide, chlorite, chlorine and chlorate. Speciation analyses are performed weekly and the chlorine dioxide yield is determined twice daily with the routine absorbance (440 nm) test. The current generator design is capable of producing yields greater than 90% with minimal excess chlorine down to concentrations of 20.0 mg/L (15%). Chlorine dioxide concentrations are of the order of 245 mg/L.

Chlorine dioxide is applied to the first-stage carbon adsorption effluent and to the reverse osmosis effluent. Profiles of the mean total coliform and mean membrane heterotrophic plate count (m-HPC) through the treatment plant are shown in Figure 3. Coliforms are reduced significantly by lime at pH above 11. Ozone further reduces the viable population

to a low value. Following reverse osmosis essentially no coliforms remain. Chlorine dioxide provides protection from regrowth. The plate count organisms were not reduced significantly by the lime treatment. Ozone reduced these organisms by less than one log. The reverse osmosis units provided more than 3 log removal. After complete treatment and chlorine dioxide disinfection, mean m-HPC values are less than one count per milliliter. This excellent performance was achieved with an applied dose of only 0.25 mg/l and a contact time of 3.8 minutes.

ECONOMIC ANALYSIS

The direct operation costs for the Denver Reuse Demonstration Plant during Phase I are tabulated in Table 10 and summarized graphically in Figure 4. Fifty two percent of the unit process costs are associated with the reverse osmosis unit.

The carbon regeneration costs are based upon regeneration at 6 month intervals (i.e., treatment of 7.439 m^3 wastewater/m^3 carbon) and cost experience through the 2nd regeneration campaign. The chemical cost for activated carbon represent the cost of carbon replacement due to losses during operation (approximately 20% of total) and regeneration (approximately 80% of total). Two thirds of the utility cost is associated with normal operation and the balance with carbon regeneration.

SUBSEQUENT TESTING PROGRAM

Phase II began April 16, 1986 and was concluded in October 17, 1986. Phase II incorporated the operation of all unit processes including the ion exchange process for ammonia removal and recovery by regeneration of the exchange media. The clinoptilolite columns are operated in a downflow mode at 5 gpm/ft^2 (0.2 m/min.) to remove ammonium ion. The columns are regenerated with a concentrated (2%) sodium chloride solution in a batch-counter-current flow mode. Ammonia is then removed from the spent regenerant after pH adjustment with NaOH and clarification by air stripping and absorbed in sulphuric acid and disposed of on the plant grounds as a nitrogen fertilizer. Problems occurred with the regeneration system which resulted in periodic salt leakage into the product water and inefficient regeneration of the ion exchange media. Influent ammonia (NH_4-N) levels averaging 24 mg/L were reduced to 4.5 mg/L by the ion exchange process.

Phase III started November 10, 1986 and was terminated prematurely on February 24, 1987. The goal in Phase III was to evaluate a non-reverse osmosis process sequence to obtain information which may support a reuse treatment sequence which provides for split treatment and blending of effluents to satisfy quality objectives while minimizing costs. Phase III was terminated prematurely because nematodes occurred in the product water surviving clarification, filtration, ozonation and chlorine dioxide disinfection.

Partial nitrification of the ammonia present in the ozonated water occurred in the second stage carbon adsorption column. The resulting nitrite concentrations increased the chlorine dioxide demand of the water to an unacceptable level. Complete nitrification was demonstrated by reducing the flow rate to the ion exchange columns which reduced the ammonia level to 1 mg/L.

Chlorine dioxide was added to the filter pump wet well to provide 0.1 mg/l residual to the filter influent and eliminate the dissolved oxygen reduction which occurred across the filters. As a consequence of this operational change, filter run lengths were extended from 20 to 70 hours. Phase IV was started March 5, 1987 and is currently underway. This Phase of the evaluation was initiated without ion exchange, ozonation or carbon adsorption. The second stage of carbon adsorption and ozonation were subsequently returned to service because of excessive pressure build-up in the reverse osmosis unit.

SUMMARY

The Phase I results have shown that the Reuse Plant effluent satisifies all of the primary, secondary and proposed recommended MCL's except pH. Furthermore, the Phase I operating results indicate that this high quality water can be reliably produced at an operating cost of $3.60 per thousand gallons ($0.95/$m^3$), with approximately half of the unit process costs attributed to reverse osmosis operation.

TABLE 1. RELIABILITY STANDARDS - EPA PRIMARY MCLs

PARAMETER	MCL[1]
Physical/Aesthetic	
Turbidity	1 NTU
Major Cations, Anions	
Fluoride	1.7[2]
Nitrate-N	10
Trace Metals	
Arsenic	0.05
Barium	1
Cadmium	0.01
Chromium	0.05
Lead	0.05
Mercury	0.002
Selenium	0.01
Silver	0.05
Radiological	
Gross Alpha	5 pCi/L
Gross Beta	50 pCi/L
Microbiological	
Total Coliforms	1/100 mL
Trihalomethanes	
TTHM	0.10
Trace Organics	
Endrin	0.0002
Lindane	0.004
Methoxychlor	0.1
Toxaphene	0.005
2,4-D	0.1
2,4,5-TP (Silvex)	0.01

[1] Units in mg/L unless otherwise noted
[2] Fluoride MCL is related to the annual average maximum daily air temperature

TABLE 2. RELIABILITY STANDARDS - EPA SECONDARY MCLs

PARAMETER	MCL[1]
Physical/Aesthetic	
pH	6.5-8.5
Color	15 CU
MBAS (Foaming Agents)	0.5
Odor	3 TON
Trace Metals	
Copper	1
Iron	0.3
Manganese	0.05
Zinc	5
Major Ions	
TDS	500
Chloride	200
Sulfate	250
OTHER STANDARDS	
Microbiological	
Coliphage B	0/100mL
Coliphage C	0/100mL

[1] Units in mg/L unless otherwise noted

TABLE 3. PHASE I PROCESS OPERATING PARAMETERS

LOCATION	PARAMETERS	VALUES
Influent	Flow Rate	0.91 MGD
	Turbidity	8.6 NTU
	Backwash Recycle	0.11 MGD
Rapid Mix Basins	Flow Rate	1.02 MGD
	Detention Time	5.8 Min
	Velocity Gradient	326 1/sec
	Lime Dose	530 mg/L
	Ferric ChloriDe Dose	13 mg/L
	pH Set Point	11.2
Flocculation Basin	Detention Time	24.5 Min
	Velocity Gradient	100 L/sec
Chemical Clarifier	Flow Rate	0.98 MGD
	Detention Time	98.8 Min
	Overflow Rate	808 GPD/Ft^2
	Waste Sludge Flow Rate	0.033 MGD
	Sludge Solids Concentration	2.53%
	Sludge Wasted Daily	$6.9X10^3$ lbs/day
	Turbidity	6.3 NTU
Recarbonation Basin	Detention Time	11.67 Min
	Velocity Gradient	533 1/sec
	Carbon Dioxide Dose	230 mg/L
	pH Set Point	7.7
Ballast Pond	Detention Time	149.6 Min
	Turbidity	1.9 NTU
Filters	Flow Rate	0.32 MGD
	Hydraulic Loading Rate	4.42 GPM/Ft^2
	Avg. Filter Run Length	17.2 Hrs
	Backwash Duration	25 Min
	Backwash Flow Rate	1250 GPM
	Surface Wash Duration	17 Min
	Surface Wash Flow Rate	56.5 GPM
	Backwash Loading Rate	15.96 GPM/Ft^2
	Terminal Pressure Drop	11.0 Ft
	Turbidity	0.33 NTU

TABLE 3. (continued)

LOCATION	PARAMETERS	VALUES	
First-Stage Carbon	Flow Rate	0.98	MGD
	Hydraulic Loading Rate	6.04	GPM/Ft^2
	Throughput Rate	1.73	BV/Hr
	Empty Bed Contact Time	33.1	Min
	Backwash Duration	20	Min
	Backwash Flow Rate	2000	GPM
	Backwash Loading Rate	17.7	GPM/Ft^2
	Terminal Pressure Drop	15.0	Ft
Ozone Basin	Flow Rate	0.086	MGD
	Detention Time	59.7	Min
	Ozone Residual	0	mg/L
	Ozone Off Gas Concentration	0.06%	
	Ozone Transfer Efficiency	72.7%	
	Applied Ozone Dose	1.89	mg/L
	Ozone Absorbed Dose	1.37	mg/L
	Ozone Produced	1.46	lbs/day
	Generator Air Flow	4228	ACFD
	Generator Power Consumption	0.8	KWH
Second-Stage Carbon	Throughput Rate	1.43	BV/Hr
	Hydraulic Loading Rate	4.75	GPM/Ft^2
	Empty Bed Contact Time	42.0	Min
	Backwash Duration	20	Min
	Backwash Flow Rate	200	GPM
	Backwash Loading Rate	15.9	GPM/Ft^2
	Terminal Pressure Drop	15.0	Ft
	Turbidity	0.17	NTU
Reverse Osmosis	Flow Rate	0.042	MGD
	Feed Pressure	260	PSI
	Feed Conductivity	1013	UMHOS/CM
	Product Conductivity	50	UMHOS/CM
	Product Water Recovery	86%	
	Rejection Based on Conductivity	95%	
	Hydrochloric Acid Dose	147	mg/L
Disinfection	Chlorine Dioxide Dose	0.29	mg/L
	Chlorine Dioxide Residual	0.11	mg/L
	Detention Time	15.9	Min
	Turbidity	0.04	NTU

1 MGD = .0438 m^3/s
1 GPM = 6.309 X 10^{-5} m^3/s
1 GPM/ft^2 = 0.041 m/min.
1 ft = 0.305 m
1 lbs/day = 0.0076 kg/day
1 ACFD = 0.026 m^3/day
1 PSI = 6/89 kPa

TABLE 4. PHASE I TEST RESULTS
MEAN VALUES OCTOBER 1, 1985 - MARCH 28, 1986
(All concentrations in mg/L unless otherwise indicated)

PARAMETER	REUSE PLANT INFLUENT	REUSE PLANT PRODUCT	DENVER DRINKING WATER
General			
Total Alkalinity - $CaCO_3$	273	1	79
Hardness - $CaCO_3$	199	1	105
TSS	10	<1	<1
TDS	545	14	172
Specific Conductance-umhos/cm	1022	49	286
pH - Units	6.9	6.0	7.7
D.O.	3.3	7.9	-
Temp - °C	16	18	4
Turbidity - NTU	7.9	0.04	0.28
TKN	25	2.7	0.7
TOC	17	0.2	2.1
Color - Units	25	<1	<1
Particle Size 128µ (count/50mL)	-	*	*
Particle Size 64-128µ (count/50mL)	-	1	1
Particle Size 32-64µ (count/50mL)	-	6	68
Particle Size 16-32µ (count/50mL)	-	19	224
Particle Size 8-16µ (count/50mL)	-	65	444
Particle Size 4-8µ (count/50mL)	-	156	780
Asbestos-MFibers/L	12.2	*	*
MBAS	0.09	0.01	0.01
Radiological			
Gross Alpha - pCi/L	4.4	<1	2.0
Gross Beta - pCi/L	6.9	<1	1.9
Microbiological			
m-HPC (count/mL)	6.9×10^5	0.3	0.6
Total Coliform (count/100mL)	3.0×10^5	*	*
Fecal Strep (count/100mL)	1.1×10^4	*	*
Fecal Coliform (count/100mL)	2.2×10^4	*	*
Coliphage B - count/100mL	3.3×10^4	*	*
Coliphage C - count/100mL	6.0×10^4	*	*
Giardia - cysts/L	0.34	*	*
Endamoeba coli - cysts/L	0.283	*	*
Nematodes	+	*	*
Algae	+	*	-
Enteric Virus	-	*	-

* = below detection limit, or more than 50% of data was below detection limit - no mean calculated
- = not tested
+ = detected but not quantified
< = detection limit

TABLE 4. (continued)

PARAMETER	REUSE PLANT INFLUENT	REUSE PLANT PRODUCT	DENVER DRINKING WATER
Inorganic			
Aluminum	0.039	<1	0.139
Barium	0.026	<1	0.038
Boron	0.30	0.20	0.12
Bromide	0.27	<0.08	0.09
Cadmium	<0.0008	<0.0008	<0.0008
Calcium	53	<0.5	23
Chloride	80	11	28
Chromium	0.013	<0.001	<0.001
Copper	0.014	<0.005	0.017
Fluoride	1.9	0.11	1.1
Iron	0.208	<0.01	0.074
Potassium	10.1	0.5	1.9
Magnesium	12	<0.02	8
Manganese	0.066	<0.005	0.012
Mercury	0.00013	<0.00005	<0.00005
Molybdenum	0.008	<0.002	0.002
Ammonia-N	24	3.2	0.3
Nitrate-N	0.07	<0.05	<0.05
Nitrite-N	<0.05	<0.05	<0.05
Nickel	0.011	<0.001	<0.001
Orthophosphate	5.9	<0.08	<0.08
Total Phosphate	6.5	0.02	0.02
Silica	15	2.4	9.5
Strontium	0.50	<0.01	0.22
Sulfate	140	<0.8	34
Lead	0.002	<0.001	0.002
Uranium	0.009	0.009	0.004
Zinc	0.034	<0.004	0.013
Sodium	110	3	20
Lithium	0.021	<0.008	0.010

TABLE 5. COMPARISON OF REUSE PRODUCT WATER WITH NATIONAL PRIMARY DRINKING WATER REGULATIONS (MCLs) - mg/L UNLESS OTHERWISE NOTED

PARAMETER	MCL	REUSE PRODUCT (MEAN VALUE)
Total Coliform - count/100 mL	1.0	*
Turbidity - NTU	1.0	0.04
Fluoride[1]	1.7	0.11
Nitrate-N	10	<0.05
Arsenic	0.05	<0.001
Barium	1.0	<0.1
Cadmium	0.01	<0.0008
Chromium	0.05	<0.001
Lead	0.05	<0.001
Mercury	0.002	<0.00005
Selenium	0.01	<0.001
Silver	0.05	<0.0008
Gross Alpha - pCi/L	5	<1
Total Trihalomethanes	0.10	*
Pesticides/Herbicides	+	*

1 = Fluoride MCL is related to the annual average maximum daily air temperature
* = Below detection limit, or more than 50% of data below detection limit-no mean calculated
+ = each compound has an individual MCL. None were found
< = detection limit

TABLE 6. COMPARISON OF REUSE PRODUCT WATER WITH NATIONAL SECONDARY MCLs - mg/L UNLESS OTHERWISE NOTED

PARAMETER	MCL	REUSE PRODUCT (MEAN VALUE)
Color - Units	15	<1
pH - Units	6.5 - 8.5	6.0
MBAS	0.5	*
Chloride	250	11
Sulfate	250	<0.8
TDS	500	<14
Copper	1.0	0.005
Iron	0.3	<0.01
Manganese	0.05	<0.005
Zinc	5.0	<0.004

* = Below detection limit, or more than 50% of data below detection limit - no mean calculated
< = detection limit

TABLE 7. COMPARISON OF REUSE PRODUCT WATER MAXIMUM VALUES WITH SELECTED FINAL MCLs AND PROPOSED MCLGs - ug/L

COMPOUND	FINAL MCL	REUSE PRODUCT
Trichloroethylene	5	0.51
Carbon Tetrachloride	5	0.15
1, 2-Dichloroethane	5	<2.0
Benzene	5	0.39
1,1-Dichloroethylene	7	*
1,1,1-Trichloroethane	200	0.2
p-Dichlorobenzene	75	0.04
	PROPOSED MCLG (ug/L)	
1,2-Dichloropropane	6	<0.5
O-Dichlorobenzene	620	*
2,4-D	70	<0.01
Ethylbenzene	680	0.31
Lindane	0.2	<0.1
Methoxychlor	340	<0.1
Monochlorobenzene	60	0.08
Toluene	2000	0.88
2,4,5-TP	52	*
Toxaphene	zero	<1.0
trans-1,2-Dichloroethylene	70	*
Xylenes	440	0.41

* - Below detection limit

TABLE 8. CONFIRMED ORGANIC COMPOUNDS ug/L

COMPOUND	REUSE PLANT INFLUENT MAX	REUSE PLANT INFLUENT GEOMETRIC MEAN[1]	REUSE PLANT EFFLUENT MAX	DENVER DRINKING WATER MAX	DENVER DRINKING WATER GEOMETRIC MEAN
1,1 Dichloroethene	*		*	*	
trans-1,2 Dichloroethylene	*		*	*	
Chloroform	4.5	2.70(15)	0.92	6.85	1.81(14)[2]
1,1,1 Trichloroethane	6.9	3.74(14)	0.2	0.2	
1,2 Dichloroethane	*		*	*	
Benzene	0.28	NQ	0.39	0.83	NQ
Carbon tetrachloride	0.14		0.15	0.52	NQ
Trichloroethylene	6.4	1.79(16)	0.51	0.48	
1,2 Dichloropropane	<0.5		<0.5	<0.5	
Bromodichloromethane	0.53	NQ	0.12	1.61	0.80(11)
cis-1,3 Dichloropropene	*		0.13	*	
trans-1,3 Dichloropropene	<0.5		<0.5	<0.5	
Toluene	0.32	NQ	0.88	0.12	
1,1,2 Trichloroethane	*		*	*	
Dibromochloromethane	0.50	NQ	*	1.14	NQ
Tetrachloroethylene	60	6.27(13)	0.21	0.34	
Chlorobenzene	<0.2		0.08	<0.2	
m&p-Xylene	0.65		0.32	0.82	
o-Xylene	1.12		0.09	0.09	
Ethylbenzene	0.49		0.31	0.11	
Bromoform	0.12		<0.5	<0.5	
1,1,2,2 Tetrachloroethane	<0.5		<0.5	<0.5	
m-Dichlorobenzene	0.22		*	*	
p-Dichlorobenzene	3.04	1.93(15)	0.04	*	
o-Dichlorobenzene	5.58	1.19(15)	*	*	
trans-Decalin	0.81	NQ	<0.2	<0.2	
cis-Decalin	0.2		<0.2	<0.2	

* = below detection limit
1 = given in cases where more than 50% of the test results were above the detection limit
2 = number of samples above the detection limit are shown parenthetically
NQ = below minimum quantification limit

TABLE 9. CARBON REGENERATION FURNACE PERFORMANCE

CRITERIA	GOAL	ACTUAL
Feed Rate (lb/day)		
Carbon	2000	2167
Adsorbate	1000	403
Natural Gas (BTU/lb Carbon)		
Total	4735 - (HVT)[1]	5310
Electricity (kwh/lb carbon)		
Total	0.39	0.46
Iodine Number	852 (90% of virgin carbon)	773 (82% of virgin carbon)
Apparent Density (lb/ft^3)	28-32 (virgin carbon is 32.7± 6)	31.2

[1] Heating Value Term = (1.4) x (weight % adsorbate) x (heating value)

1 lb/day = 0.45 kg/day
1 BTU/lb = 2324 joules/kg
1 kwh/lb = 2.2 kwh/kg
1 lb/ft^3 = 1.6 kg/m^3

TABLE 10. PHASE I DIRECT OPERATION COSTS (IN $/kgal TREATED)
OCTOBER 1, 1985 - MARCH 28, 1986

	CHEMICALS	UTILITIES	TOTAL
Unit Process Costs:			
Lime Treatment	0.254	0.030	0.284
Recarbonation and Aeration	0.074	0.025	0.099
Filtration	-	0.018	0.018
Activated Carbon	0.014	0.083	0.187
Ozonation	-	0.072	0.072
R.O. plus Decarbonation	0.358	0.373	0.731
ClO_2	0.019	-	0.019
SUB-TOTALS	0.809	0.601	1.410
General Facility Operation and Maintenance:			
Utilities			0.245
Materials			0.166
Other Services			0.145
Direct Labor			1.610
SUB-TOTALS			2.176
		COMBINED TOTAL	3.586

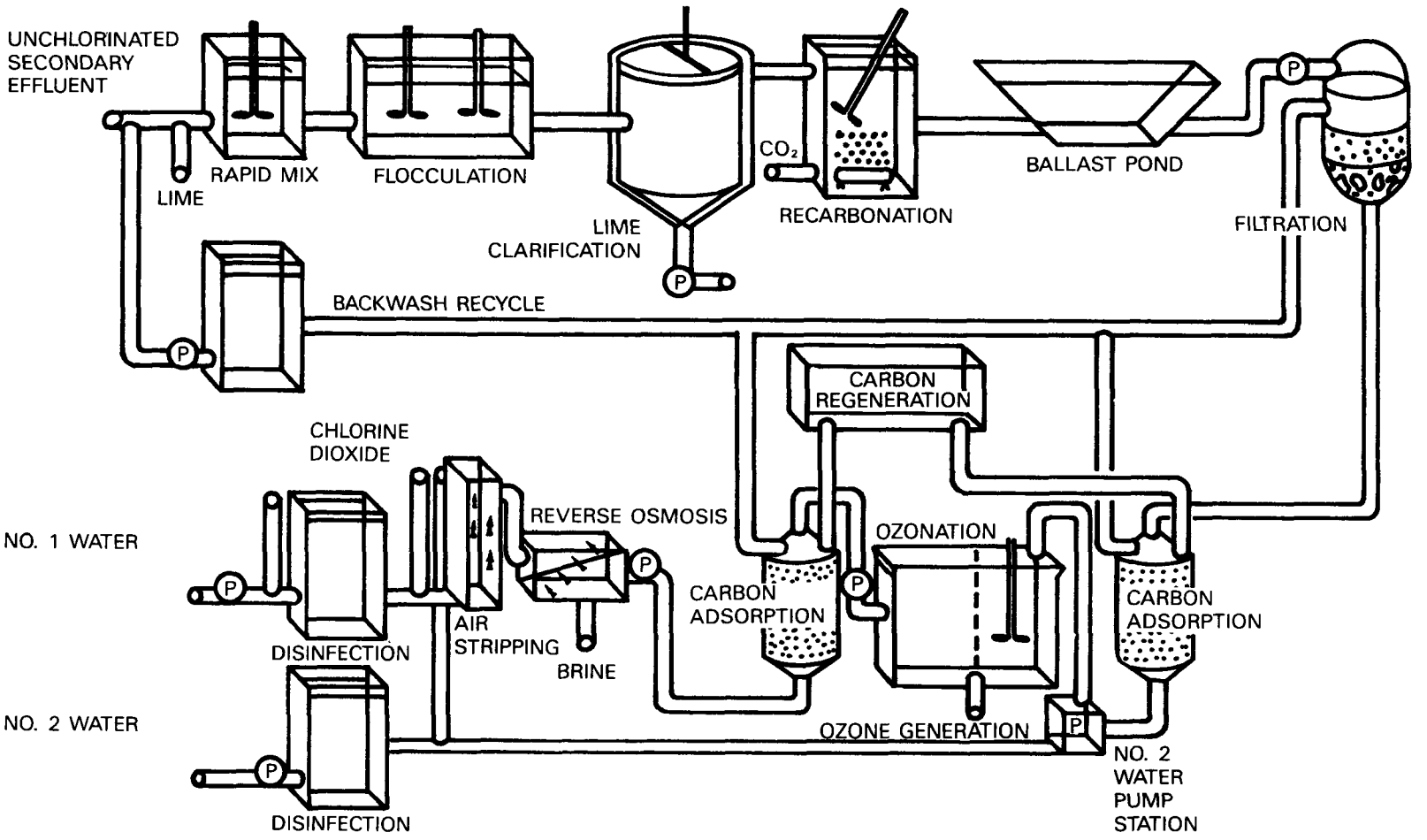


FIGURE 1. WATER REUSE TREATMENT PROCESS

SHOWING SAMPLE LOCATIONS

FIGURE 2

TOC by Phase 1 Process

Oct. 1, 1985 — Mar. 28, 1986 (mean values mg/l)

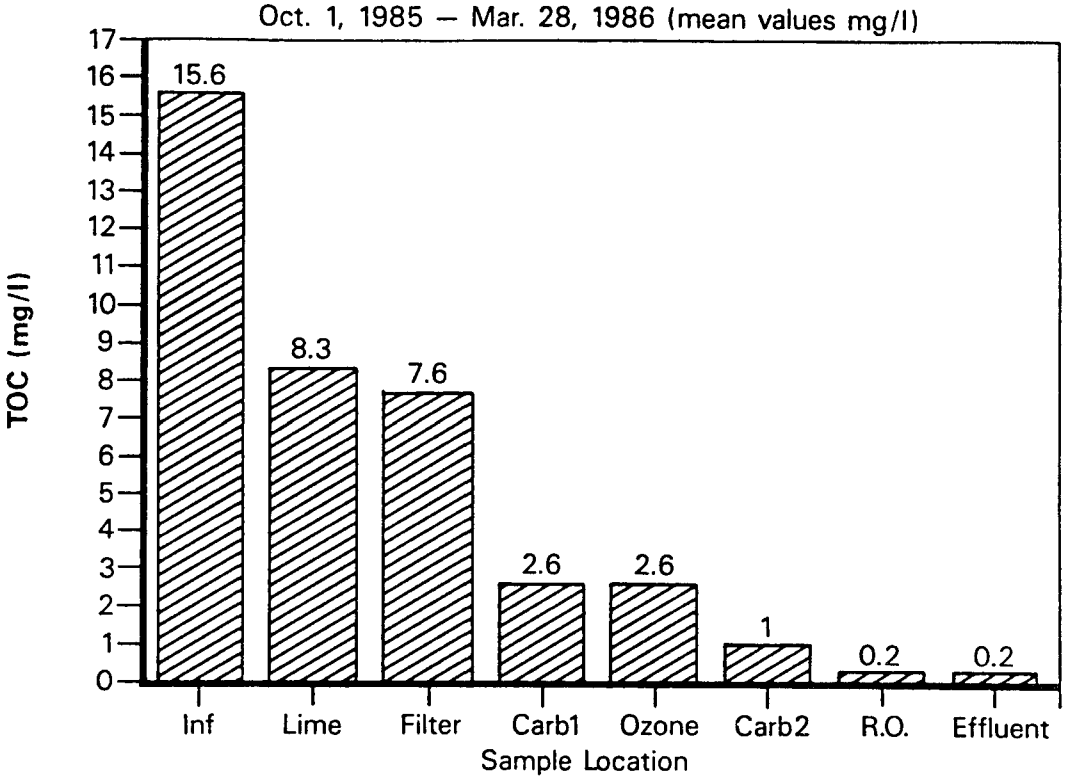


FIGURE 3

m — HPC and Total Coliform

Oct. 1, 1985 — Mar. 28, 1986 (mean values)

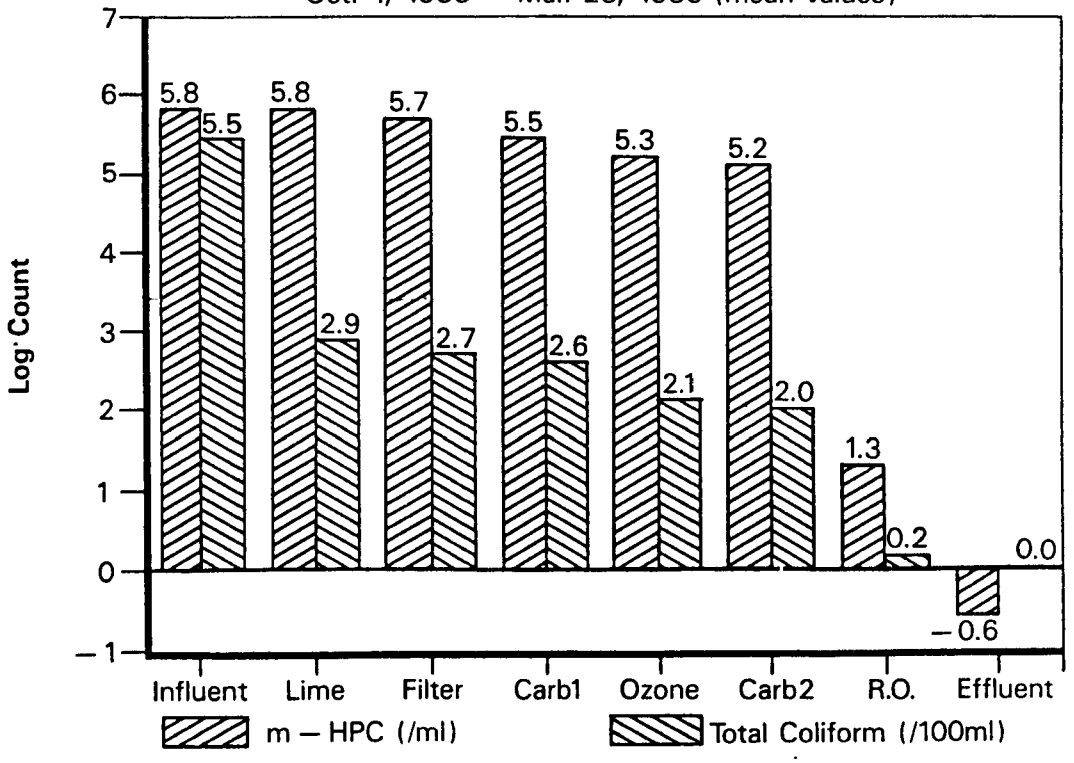
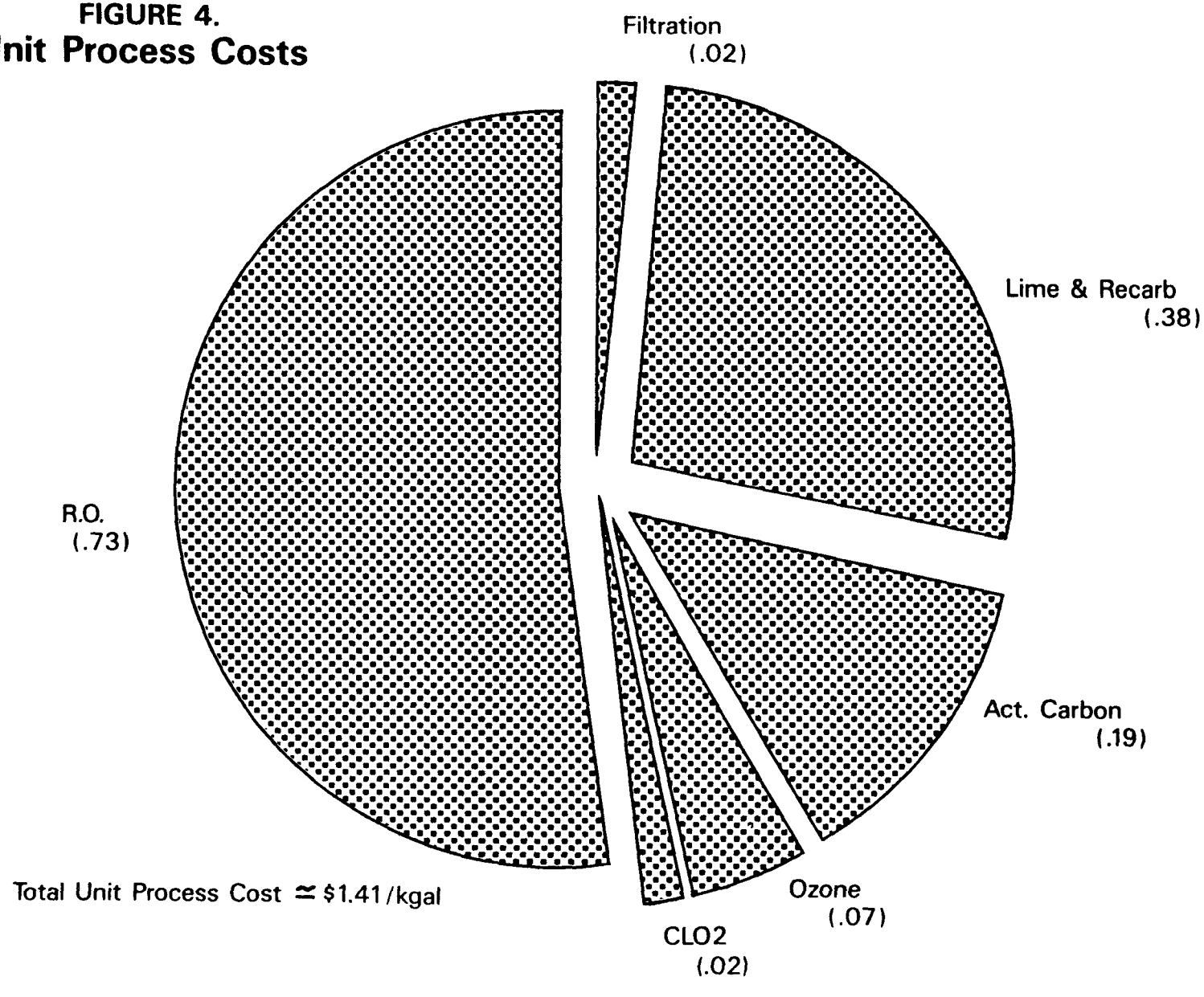


FIGURE 4.
Unit Process Costs



APPENDIX A

FINAL MCLs/PROPOSED MCLGs

VOLATILE ORGANIC CONTAMINANTS

VOC	Final MCL (µg/L)	VOC	Final MCL (µg/L)
Benzene	5	Trichloroethylene	5
Vinyl Chloride	2	1,1-Dichloroethylene	7
Carbon Tetrachloride	5	1,1,1-Trichloroethane	200
1,2-Dichloroethane	5	p-Dichlorobenzene	75

SYNTHETIC ORGANIC CONTAMINANTS

SOC	Proposed MCLGs (mg/L)	Current MCL (mg/L)
Acrylamide	0	
Alachlor	0	
Aldicarb, aldicarb sulfoxide and aldicarb sulfone	0.009	
Carbofuran	0.036	
Chlordane	0	
cis,1,2-Dichloroethylene	0.07	
DBCP	0	
1,2-Dichloropropane	0.006	
o-Dichlorobenzene	0.62	
2,4-D	0.07	0.1
EDB	0	
Epichlorohydrin	0	
Ethylbenzene	0.68	
Heptachlor	0	
Heptachlor epoxide	0	
Lindane	0.0002	0.0004
Methoxychlor	0.34	
Monochlorobenzene	0.06	
Pentachlorophenol	0.22	0.1
Styrene	0.14	
Toluene	2.0	
2,4,5-TP	0.052	0.01
Toxaphene	0	0.005
trans-1,2-Dichloroethylene	0.07	
Xylene	0.44	

MICROBIOLOGICAL PARAMETERS

Parameter	Proposed RMCL	Current MCL	Parameter	Proposed RMCL
Total coliforms	0		Giardia	0
Turbidity	0.1 NTU	1 NTU	Viruses	0

APPENDIX A (continued)

PROPOSED MCLGs

INORGANIC CONTAMINANTS

IOC	Proposed MCLGs (mg/L)	Current MCL (mg/L)
Arsenic	0.05	0.05
Asbestos(medium and long fibers)	7.1 million fibers/liter	
Barium	1.5	1.0
Cadmium	0.005	0.01
Chromium	0.12	
Copper	1.3	
Lead	0.020	0.05
Mercury	0.003	0.002
Nitrate	10.0	
Nitrite	1.0	
Selenium	0.045	0.01

APPENDIX B

VOLATILE ORGANIC CONTAMINANTS

Trichloroethylene	Vinyl Chloride	Trichlorobenzene(s)
Tetrachloroethylene	Methylene Chloride	1,1 - Dichloroethylene
Carbon Tetrachloride	Benzene	cis - 1,2 - Dichloroethylene
1,1,1-Trichloroethane	Chlorobenzene	trans - 1,2 - Dichloroethylene
1,2 - Dichloroethane	Dichlorobenzene(s)	

SYNTHETIC ORGANIC CONTAMINANTS

Endrin*	Carbofuran	Pentachlorophenol
Lindane*	1,1,2 - Trichlorethane	Picloram
Methoxychlor*	Vydate	Dinoseb
Toxaphene*	Simazine	Alachlor
2, 4, - D*	PAHs (Polynuclear Aromatic	EDB (Ethylene Dibromide)
2,4,5 - TP (Silvex)*	Hydrocarbons)	Epichlorohydrin
Total Trihalomethanes*	PCBs (Polychlorinated	Dibromomethane
Aldicarb	Biphenyls)	Toluene
Chlordane	Atrazine	Xylene
Dalapon	Phthalates	Adipates
Diquat	Acrylamide	Hexachlorocyclopentadiene
Endothall	DBCP (Dibromochloropropane)	2, 3, 7, 8-TCDD (Dioxin)
Glyphosate	1,2 - Dichloropropane	

INORGANIC CHEMICALS

Arsenic*	Silver*	Vanadium
Barium*	Fluoride*	Sodium
Cadmium*	Aluminum	Nickel
Chromium*	Antimony	Zinc
Lead*	Molybdenum	Thallium
Mercury*	Asbestos	Beryllium
Nitrate (as N)*	Sulfate	Cyanide

MICROBIOLOGICAL CONTAMINANTS

Turbidity*	Viruses	Filtration of Surface Water
Total Coliforms*	Standard Plate Count	Disinfection of All Water
Giardia Lamblia		

RADIONUCLIDE CONTAMINANTS

Radium 226 and 228*	Beta particle and Photon	Uranium
Gross Alpha Particle Activity*	Radioactivity*	Radon

* Already regulated

APPENDIX C

ORGANICS TO BE MONITORED

Chloroform*	p-Xylene	bis-2-Chloroisopropyl ether
Bromodichloromethane*	o-Xylene	2,2-Dichloropropane
Chlorodibromomethane*	m-Xylene	1,2,4-Trimethylbenzene
Bromoform*	1,1-Dichloroethane	n-Butylbenzene
trans-1,2-Dichloroethylene	1,1,2,2-Tetrachloroethane	Napthalene
Chlorobenzene	Ethylbenzene	Hexachlorobutadiene
m-Dichlorobenzene	1,3-Dichloropropane	o-Chlorotoluene
Dichloromethane	Styrene	p-Chlorotoluene
cis-1,2-Dichloroethylene	Chloromethane	1,3,5-Trimethylbenzene
o-Dichlorobenzene	Bromomethane	p-Isopropyltoluene
1,2,4-Trichlorobenzene	Bromochloromethane	1,1-Dichloropropene
Fluorotrichloromethane	1,2,3-Trichloropropane	iso-Propylbenzene
Dichlorodifluoromethane	1,2,3-Trichlorobenzene	tert-Butylbenzene
Dibromomethane	n-Propylbenzene	sec-Butylbenzene
1,2-Dibromoethane(EDB)	1,1,1,2-Tetrachloroethane	Bromobenzene
1,2-Dibromo-3-chloropropane (DBCP)	Chloroethane	
	1,1,2-Trichloroethane	
Toluene	Petachloroethane	* Already regulated

APPENDIX D

TENTATIVELY IDENTIFIED ORGANIC COMPOUNDS
PLANT EFFLUENT
ESTIMATED CONCENTRATION GROUPINGS - ug/L
(RESULTS OF 31 SAMPLES)

5 - 10 ug/L Concentration Range	Frequency
HEXADECANOICACID	1
1 - 5 ug/L Concentration Range	
DECANOICACID	1
HEXENOL, PROPANOATE	1
DIBUTYLPHTHALATE	1
FURAN, DIETHYL, TETRAHYDRO-	5
Less Than 1 ug/L Concentration Range	
OCTADIENE-DIOL	1
CYCLOHEXANE-DIOL	1
NORBORNENE, TRIMETHYL	3
CYCLOHEXADIENE	1
BENZENE, BIS (METHYLETHYL)-	1
BENZENE, BUTYL-	1
BENZENE, ETHYL-METHYL- (? ISOMERS)	11
BENZENE, METHYL-PROPYL-	1
BENZENE, METHYL-(METHYLETHYL)-	1
BENZENE, PROPYL	2
BENZENE, TRIMETHYL-(METHYLETHYL)-	1
BENZENE, TRIMETHYL- (3 ISOMERS)	5
BENZENE, (METHYLETHYL) -	5
BENZENE, (METHYLPROPYL)-	6
BENZENE, ETHENYL-	4
BENZENE, METHYL-ETHENYL-	2
BENZENE, (METHYLETHENYL)-	1
HEXENOL, PROPANOATE	2
PROPANOICACID, METHYLPROPYLESTER-	1
BUTANE, PROPOXY-	1
ETHANOL, BUTOXY-	1
HEXANONE, METHYL-	2
PENTANONE, HYDROXY-METHYL-	4
PENTANONE, METHYL-	3
BUTYL-METHYL CARBAMICACID, METHYLESTER	1

APPENDIX E

TENTATIVELY IDENTIFIED ORGANIC COMPOUNDS
DENVER DRINKING WATER
ESTIMATED CONCENTRATION GROUPINGS - ug/L
(RESULTS OF 18 SAMPLES)

5 - 50 ug/L Concentration Range	Frequency
HEXENOL, PROPANOATE	1
1 - 5 ug/L Concentration Range	
CYCLOHEXANE-DIOL	1
CYCLOPENTANE, ETHYL-METHYL-	1
CYCLOPENTENE, ETHYL-	1
DIBUTYLPHTHALATE	1
BENZENEMETHANOL	1
PROPENE, TRICHLORO-	1
PHENOL	1
Less Than 1 ug/L Concentration Range	
HEXANE, DIMETHYL-	1
4-CARENE	1
CYCLOHEXANE, METHYL-(METHYLETHENYL)-	1
CYCLOHEXADIENE	1
UNDECYNE	1
FORMAMIDE, N,N-DIMETHYL-	1
BENZENE, BUTYL-	1
BENZENE, ETHYL-METHYL- (3 ISOMERS)	4
BENZENE, METHYL-PROPYL-	1
BENZENE, METHYL-(METHYLETHYL)-	1
BENZENE, PROPYL-	1
BENZENE, TRIMETHYL-(METHYLETHYL)-	2
BENZENE, TRIMETHYL-	2
BENZENE, (METHYLETHYL)-	3
BENZENE, (METHYLPROPYL)-	2
BENZENE, ETHENYL-	2
BENZENE, (METHYLETHENYL)-	1
BENZOIC ACID	1
HEXADECANOICACID	1
NONANOICACID	1
TETRADECANOICACID	1
DIBUTYLPHTHALATE	1
HEXANONE, METHYL-	1
PENTANONE, HYDROXY-METHYL-	2

REFERENCES

1. "Summary of Revisions of the Drinking Water Regulations and Amendments to the Safe Drinking Water Act", Camp, Dresser and McKee, Inc., June, 1986.

2. Potable Water Reuse Demonstration Project - Preliminary Process Evaluations, Phase 1 Report, Denver Water Department, March, 1987.

3. Health and Welfare, Canada. "Guidelines for Canadian Drinking Water 1978", pp 42.

4. National Academy of Sciences, "Drinking Water and Health", Vol. 1, National Academy Press, Washington, D.C., 1977.

MICHIGAN'S PROCESS FOR REGULATING TOXIC SUBSTANCES IN SURFACE WATER PERMITS

by

Paul D. Zugger & James E. Grant
Surface Water Quality Division
Michigan Department of Natural Resources
Lansing, Michigan 48909

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

Prepared for Presentation at:

Eleventh United States/Japan Conference
on Sewage Treatment Technology
Tokyo, Japan

October 12-14, 1987

CONTENTS

Page

Abstract 477

Introduction 478

Rule 57 480

Rule 57(2) Guidelines 481
Aquatic Chronic Value 482
Human Life Cycle Safe Concentration 483
Terrestrial Life Cycle Safe Concentration 484
Cancer Risk Value 485
Hereditary Mutagen and Genotoxic Teratogen Values 486

Summary 487

Acknowledgements 488
References 488

Appendices 489

A. Department of Natural Resources, Water Resources Commission, General Rules, Part 4. Water Quality Standards, filed with Secretary of State November 14, 1986 490

B. State of Michigan, Department of Natural Resources, Environmental Protection Bureau, Guidelines for Rule 57(2), filed with Secretary of State January 2, 1985 507

C. Staff Report, Support Document for the Proposed Rule 57 Package, Michigan Department of Natural Resources, Environmental Protection Bureau, dated March 26, 1984 539

D. Annual Listing of Rule 57(2) Guideline Levels, dated January 27, 1987 600

ABSTRACT

A necessary aspect of a water pollution control program is a regulatory system that will not only provide nontoxic water quality conditions but also assure that toxic substances do not bioaccumulate in fish to levels unacceptable for human consumption. In 1985, Michigan promulgated revisions to Rule 323.1057 of its Water Quality Standards that establish a regulatory process that will protect public health and the environment from discharges of toxic substances from point source surface water discharges. Rule 57(2) specifically addresses the development of allowable toxicant levels in the waters of the state applicable to wastewater discharges. The universe of chemicals to which the subrule applies is defined, an upper boundary on estimated excess risk of 1 in 100,000 for non-threshold carcinogens is established, comprehensive procedural guidelines are mandated and a mechanism for issuance of scheduled abatement permits is provided. This paper reviews the development of the rule amendments and discusses key aspects of the adopted rules and guidelines.

INTRODUCTION

Michigan's unique geographic position at the heart of the Great Lakes has provided us with an enormous fresh water resource. With the privilege of having these wonderful lakes to use comes the responsibilities to protect their quality; a quality much more fragile than our predecessors earlier in this century imagined. One of the most sensitive indices of water quality is the health of the aquatic biological community. The Great Lakes's fishery is an integral part of the aquatic ecosystem and the health of this fishery reflects the quality of the Lakes.

Water pollution control programs to protect the aquatic community from acute and chronic toxicity have been in place for many years. However, in recent years it has become clear that the traditional means of regulating toxics are not sufficient. Certain substances, while not being lethal to fish, have bioaccumulated in some Great Lakes fish species to levels unacceptable for human consumption. Examples of these are the persistent chloro organic compounds such as polychlorinated biphenyls (PCBs) and 2,3,7,8 Tetrachloro-dibenzo-p-dioxin (dioxin). In recognition of this phenomena, Michigan in 1985 expanded the scope of its Water Quality Standards to assure not only nontoxic water conditions, but to assure as well that fish are fit for human consumption.

In 1972, the United States federal government passed major water pollution control legislation known as the federal Clean Water Act. Under this Act, wastewater discharges to surface waters are prohibited unless authorized by a discharge permit. Michigan was delegated the authority to administer the federal permit program in 1973, after Michigan law was amended to provide equivalent requirements at the state level.

The basic water pollution control legislation in Michigan is the Michigan Water Resources Commission Act, Act 245, Public Acts of 1929, as amended. Section 6(a) reads:

> "It shall be unlawful for any persons directly or indirectly to discharge into the waters of the state any substance which is or may become injurious to the public health, safety or welfare; or which is or may become injurious to domestic, commercial, industrial, agricultural, recreational, or other uses which are being or may be made of such waters; or which is or may become injurious to the value or utility of riparian lands; or which is or may become injurious to livestock, wild animals, birds, fish, aquatic life, or plant..."

The Act empowers the Water Resources Commission to set Water Quality Standards and to control the pollution of the waters of the state through issuance of permits which restrict the constituents of discharges to levels which assure compliance with the Standards.

One of the major charges of the surface water permit program is to protect public health and the environment from toxic substances discharged from point sources. Basic elements of the program are the development of technology-based effluent limitations determined from federal Best Available Treatment (BAT) requirements and the development of water quality-based effluent limitations to assure that Water Quality Standards are met. The process to calculate the water quality-based limitations for toxic substances is the subject of this paper. It is important to emphasize, however, that control of toxic substances inputs to the Great Lakes from nonpoint and aerial sources is also necessary before the Great Lakes will be adequately protected.

Water Quality Standards are provisions of law which define the level of protection for a water body by designating the uses to be protected and establishing water quality criteria needed to protect those uses. As used in the surface water discharge permit program, Water Quality Standards also serve as the basis for the establishment of water quality-based controls beyond the technology-based levels of treatment required by the Clean Water Act.

Part 4 of the General Rules of the Michigan Water Resources Commission contains the State Water Quality Standards (see Appendix A). Michigan's first formal Water Quality Standards were promulgated in 1967 and revised in 1973. In January, 1985, significant amendments to Rule 323.1057 were adopted. Rule 57 is Michigan's Toxic Substance Water Quality Standard.

Rule 57 was revised because the 1973 version had been promulgated at a time when the body of knowledge concerning toxic substances was much less than it is today. The 1973 version contained references to outdated literature and only addressed acute and chronic toxicity to aquatic organisms. It was apparent that the rule needed to be revised to provide protection of public health from toxic substances.

The Rule 57 revision process began in 1976, and was long and controversial. The incorporation of a risk assessment process for carcinogens into the rule was the major cause of controversy. The establishment in 1981 of a Rule 57 Advisory Committee, representing various interest groups, was the key development that ultimately led to a rule package acceptable to the regulated community and the major environmental groups in Michigan. A strong involvement by Michigan's universities on the Advisory Committee was important in gaining public confidence in the proposed package.

RULE 57

Rule 57 is considered a "narrative" Water Quality Standard as opposed to a "numerical rule" which would have absolute values specified for a list of toxic substances. In recognition of the rapid advances in the field of toxicology, the complex process required to amend a rule in Michigan, and past difficulties in attempting to promulgate a numerical rule, a narrative rule, blended with more specific guidelines was the format chosen. Guidelines, which are also promulgated pursuant to the Michigan Administrative Procedures Act, are binding only on the agency. An important aspect of guidelines is that the process for amending them is considerably less burdensome than the rule making process.

The rule is divided into two subrules. Subrule (1) is a general statement prohibiting injurious levels of toxic substances in the waters of the state and stating that the Commission determines allowable levels by using appropriate scientific data. Under the rule, determination of allowable levels for situations other than point source discharges is done on a case-by-case basis.

Subrule (2) specifically addresses the development of allowable toxicant levels in the waters of the state applicable to point source discharges. It is this subrule which is used extensively in our permitting program. Subrule (2) defines the universe of chemicals to which the subrule applies, establishes an estimated upper boundary on risk of 1 in 100,000 increased cases of cancer for carcinogens not determined to cause cancer by a threshold mechanism, specifies that the allowable toxicant levels apply after mixing with a portion of the receiving stream, mandates development of Rule 57(2) Guidelines, and provides a mechanism for establishing compliance schedules in permits to meet these requirements.

The Michigan Critical Materials Register and the United States Environmental Protection Agency's lists of priority pollutants and hazardous materials are used as the generic chemicals of concern. However, if a chemical not on these lists is of concern for a specific situation, the Commission may make a determination to include it on a case-by-case basis. Staff of the Commission routinely review the published scientific literature for emerging problem chemicals.

The risk assessment process and the upper limit on risk for chemicals assumed to be non-threshold carcinogens were major issues deliberated by the Rule 57 Advisory Committee and Michigan Department of Natural Resources (MDNR) staff. The resulting rule requires that a point source discharge not create an estimated level of increased cancer risk greater than 1 in 100,000 above background in the surface water after mixing with the allowable receiving

stream volume specified in R 323.1082 (mixing zone rule) and calculated using the model and assumptions specified in the Rule 57(2) Guidelines. Because of the conservative assumptions used, the actual risk to the individuals exposed to these levels in most surface waters of the state under these conditions, is considerably less than 1 in 100,000, and is well below common everyday risks.

The concept of blending Rule 57 with the Rule 57(2) Guidelines provides a more flexible package than placing all the details of the Guidelines in the rule. The procedures set forth in the Rule 57(2) Guidelines are practical and are being implemented. However, it is important to realize that the knowledge and understanding of toxic substances is rapidly expanding. The procedures, while valid today, will require periodic review and revision to assure current the state-of-the-art science is applied. Accordingly, Rule 57 was kept in the more general narrative form and most of the highly technical, detailed procedures were placed in the Guidelines. The Rule 57(2) Guidelines will be discussed in more detail later in this paper.

An important concern of the regulated community was the process by which discharges which cannot immediately meet the new regulation would be addressed. This concern was addressed under Rule 57(2), which states that the Commission may issue a scheduled abatement permit if immediate attainment of the allowable level of a toxic substance is not economically or technically feasible and no prudent alternative exists. Scheduled abatement permits are to be of an interim nature and include a schedule to achieve reasonable progress toward compliance with the final limits. During the developmental stages of Rule 57, considerable comments were submitted concerning the possible adverse economic impact of promulgating Rule 57. The facility specific scheduled abatement permit approach is a sound mechanism to address unacceptable economic impacts resulting from compliance with the rule.

RULE 57(2) GUIDELINES

The Rule 57(2) Guidelines (see Appendix B) are specifically mandated in Rule 57(2)(d). These Guidelines were adopted pursuant to the Administrative Procedures Act, and pursuant to that Act are only binding on the agency (Michigan Department of Natural Resources). The Guidelines set forth procedures that Michigan Department of Natural Resources staff must use in the development of permit recommendations to the Water Resources Commission on allowable levels of toxic substances in the waters of the state applicable to point source discharge permits. The Guidelines also set forth the minimum toxicity data needed for a chemical to enable staff to derive recommendations. Minimum data consists of a rat oral Lethal Dose to 50 percent of the test organisms (LD50), a 48 hour Effective Concentration to 50 percent of the test organisms (EC50) for a daphnid (*Daphnia magna*), and a 96 hour Lethal Concentration to 50 percent of the test organisms (LC50) for a fathead minnow (*Pimephales promelas*) or rainbow trout (*Salmo gairdnerii*).

The Guidelines contain detailed procedures for calculating levels necessary to protect aquatic life (Aquatic Chronic Value), wildlife (Terrestrial Life Cycle Safe Concentration), and public health from threshold effect toxic

substances (Human Life Cycle Safe Concentration); and concentrations which protect the public from cancer, hereditary mutagenic effects or genotoxic teratogenic effects. The most restrictive of the above values is used as the Rule 57(2) level which must be met in the surface water after a discharge is mixed with the appropriate receiving stream volume. Discussion on the calculation of these values follows (also see Appendix C).

1. Aquatic Chronic Value

The Aquatic Chronic Value (ACV) is the highest concentration of a chemical or combination of chemicals which theoretically will produce no adverse effects on important aquatic organisms (and their progeny) exposed continuously for a lifetime. The ACV can be calculated on a chemical specific basis or for mixtures by using biological techniques, such as bioassays, to assure that chronically toxic conditions do not exist for important aquatic life in the waters of the state. Under the chemical specific approach, a specific numerical value is derived for each chemical using the procedures in the Guidelines. The procedures also factor in the effects of various water quality characteristics (i.e., hardness, pH) on the toxicity of a chemical substance. Site specific data are preferred and used whenever possible.

The chemical specific mechanism used to calculate the ACV for a toxic substance depends upon the number of chronic data points available for that substance. When six or more appropriate chronic data points are available for a chemical, the ACV is calculated directly from fish and macroinvertebrate chronic toxicity data for that chemical. The ACVs for chemical substances calculated using this procedure are designed to be equivalent to, or less than, the chemical's chronic value for 95 percent of all fish and aquatic macroinvertebrate species resident to Michigan's waters.

Unfortunately, there exist numerous chemicals for which there are little or no chronic data available. For these chemicals, the ACV must be predicted from Final Acute Values (FAV) using appropriate application factors. An FAV corresponds to the highest concentration of a chemical in water which theoretically will kill or significantly impair 50 percent of a population of important aquatic organisms exposed continuously for a short period of time (96 hours for fish and aquatic macroinvertebrates, except 48 hours for cladocerans and chironomids). When six or more appropriate acute data points are available, the FAV is calculated. If this data base is not available, the FAV is predicted by dividing the LC50 for the most sensitive species tested (rainbow trout/daphnid; or fathead minnow/daphnid) by a species sensitivity factor of five if rainbow trout is present in the data base or ten if absent. The ACV is predicted by dividing the FAV by a chemical-specific application factor (acute LC50/chronic value ratios) for those chemical substances which have at least one acute/chronic ratio available. When chemical-specific application factors cannot be determined due to an absence of appropriate chronic data, the ACV is predicted by dividing the FAV by a general application factor of 45. This application factor corresponds to about the eightieth percentile rank of all similarly selected ratios.

The details of using biological techniques, or the whole effluent toxicity-based approach, must be established on a case-by-case basis. The advantages of using this approach are that the interaction of chemicals is inherently addressed by the test, incomplete chemical characterization of the effluent can be accounted for, chemical specific toxicity testing can be reduced in certain cases, and a more site specific determination can be made.

2. Human Life Cycle Safe Concentration

The Human Life Cycle Safe Concentration (HLSC) is the highest concentration of a chemical which causes no significant adverse effects to humans and their offspring when exposed continuously for a lifetime. The HLSCs are derived to provide an adequate margin of safety against the adverse effects of chemicals which have a toxicity threshold below which there are no adverse effects. Carcinogenic effects are handled separately.

To derive an HLSC for a chemical, the No Observable Adverse Effect Level (NOAEL) for laboratory animals or humans is determined. Although use of human data is preferred, in most cases these data are lacking, and animal data must be used instead. The NOAEL is then divided by an uncertainty factor (10-1,000) to determine the acceptable dose for a human. This factor is used to account for the uncertainties in trying to predict an acceptable exposure level for the general human population based upon experimental animal data or limited human data.

For many chemicals, appropriate toxicological data NOAELs are not available to derive an HLSC by this method. In the absence of an adequate toxicity data base, procedures have been developed to derive an HLSC from a single acute toxicity data point, i.e., an oral rat LD50. The procedure for deriving an HLSC from an oral rat LD50 involves the use of an acute to chronic application factor. The acute to chronic application factor is a numerical value by which the acute oral rat LD50 is adjusted. The value of this factor as derived by MDNR staff is 0.0001 (rationale available upon request). The oral rat LD50 is multiplied by the acute to chronic application factor (0.0001) and the value obtained from this procedure is used as a surrogate NOAEL.

The acceptable dose or milligrams of toxicant (MgT) is translated into a water concentration using the following formula:

$$HLSC = \frac{MgT\ (mg/day)}{WC + (F \times BCF)}$$

Where: HLSC = Human Life Cycle Safe Concentration
MgT = allowable milligrams of toxicant/day
WC = volume water consumed/day (liters)
F = fish consumed/day (kg/day)
BCF = bioconcentration factor of chemical (liters/kg)

For all surface waters, it is assumed that a person consumes 6.5 grams of contaminated fish per day (approximately five pounds per year) for a lifetime. This value is based upon a U.S. Environmental Protection Agency (EPA) survey of fish and shellfish consumption in the United States.

The volume of water consumed per day is assumed to be an untreated 2.0 liters for surface water protected as a drinking water source, and an untreated 0.01 liters for all other surface waters. The value of 2.0 liters was recommended by the U.S. EPA for establishing drinking water standards. The value of 0.01 liters of water per day for surface waters not protected for drinking water is to account for incidental exposure such as absorption through the skin or ingestion of small quantities of water while swimming or using the waters for other recreational purposes.

3. Terrestrial Life Cycle Safe Concentration

The purpose of establishing Terrestrial Life Cycle Safe Concentrations (TLSC) is to determine surface water concentrations which are considered acceptable for lifetime consumption by the wildlife and livestock that utilize these waters. The TLSC is defined as the highest aqueous concentration of a toxicant which causes no significant reduction in the growth, reproduction, viability, or usefulness (in the commercial and/or recreational sense) of a population of exposed organisms (utilizing the receiving waters as a drinking water source), over several generations.

To derive a TLSC, the scientific literature regarding the toxicological effects of a chemical is reviewed to determine a NOAEL for appropriate mammalian and/or avian organisms. Data on organisms native to Michigan and likely to be utilizing the particular surface water are preferred for calculating the TLSC. In most cases, however, such data are lacking, and the data from common laboratory animals (usually rodents) must be used instead. The experimental NOAEL is then divided by an uncertainty factor ranging from 10-100. This uncertainty factor is to account for: 1) species variability, since data from one species are used to predict an acceptable level for all wildlife; and 2) inadequacies in study designs or availability of data. When appropriate NOAEL data are not available, a TLSC may be calculated from an oral rat LD50 by the following equation:

$$TLSC = \frac{LD50\ (mg/kg) \times \frac{Wa}{Vw} \times M}{10}$$

Where: TLSC = Terrestrial Life Cycle Safe Concentrations
Wa = weight of test animal (kg)
Vw = volume of water consumed by test animal per day (liters)
M = acute to chronic application factor of 0.0001 derived by MDNR staff (rationale available upon request).

4. Cancer Risk Value

Due to the limits of current predictive testing, the Rule 57 Guidelines make the conservative assumption that any chemical which has been shown to be carcinogenic in one animal bioassay of good quality, is a complete carcinogen having no threshold level. However, the Guidelines do include a mechanism for evaluating a carcinogen on a case-by-case basis if the preponderance of data suggests the cancer is caused by a threshold mechanism. A committee of scientists expert in the field of carcinogenesis may be convened when MDNR staff will benefit from their advice and recommendations on this issue or other highly technical scientific issues which require additional technical expertise to resolve.

If appropriate human epidemiological data are available, an extrapolation from high doses is necessary in order to estimate the carcinogenic risk for the chemical at low concentrations. There are no standard guidelines available to estimate the risk from human epidemiology studies. However, the use of adequate human exposure data to estimate the risks associated with a carcinogenic chemical is a preferred method and when necessary, the MDNR may convene an expert committee to advise staff on an appropriate methodology in order to utilize these data. To date staff has not used human epidemiological data in setting limits.

When human epidemiological evidence is not available, the carcinogenic risk to humans is extrapolated from experimental animal data. There is no conclusive scientific evidence for the choice of one mathematical model over another; however, the linearized multistage model, GLOBAL 79 (Crump and Watson, 1979), a non-threshold extrapolation model, is used since no other extrapolation model has as much regulatory acceptance. Use of the upper 95 percent confidence limit to estimate the dose rather than extrapolation from the maximum likelihood estimate dose gives a more stable value which does not change appreciably with minor variability in the biological response at the lower doses. The use of this methodology provides a plausible upper limit estimate of cancer risk.

The Rule 57 Advisory Committee recommended that an estimated risk level of 1 in 100,000 excess cases of cancer be used as the upper boundary on risk for establishing allowable levels of carcinogens in the waters of the state applicable to point source discharges. The MDNR staff supported this position, and Rule 57 was promulgated accordingly. Greater levels of protection may be recommended at facilities where lower levels are achievable through utilization of control measures already in place.

Allowable concentrations of a carcinogen utilizing the risk-based approach are calculated using the following formula:

$$C = \frac{D \times 70\ kg}{WC + (F \times BCF)}$$

Where: C = allowable concentration of carcinogen
D = dose which theoretically would produce a risk of 1/100,000 (mg/kg/day)

WC = water consumed/day (liters)
F = fish consumed/day (kg)
BCF = bioconcentration factor for the chemical

The values are calculated on the basis of a 70 kg human and the fish and water exposure assumptions are the same as those used for the HLSC values.

5. Hereditary Mutagen and Genotoxic Teratogen Values

The levels providing an acceptable degree of protection to public health for hereditary mutagenic effects and genotoxic teratogenic effects are derived by MDNR staff on a case-by-case basis with assistance, as needed, from an expert committee of scientists.

The Rule 57 Guidelines require the Department to annually publish a listing of values calculated under the Rule. The last listing was published in February, 1987, and reflects the most restrictive level of the various criteria which meets Rule 57(2) after mixing with the receiving waters.

SUMMARY

Rule 57 has been in place for more than two years and has been utilized in the surface water permit program. In general, the application has been very successful. Michigan has approximately 200 major industrial and municipal dischargers. More than half of these permits have been reissued since the promulgation of the Rule 57 amendment. All of these dischargers have been processed consistent with Rule 57, and necessary and appropriate conditions placed in the permits. Examples of the types of pollutants limited are: heavy metals (copper, zinc, cadmium), chlorinated persistent organics (PCB, HCB), chlorinated solvents, and toxic substances such as cyanide and ammonia. The regulation of substances such as heavy metals, ammonia, and cyanide is not new. These substances were regulated in a similar manner prior to the Rule 57 amendments. The major change has been in the regulation of toxic organic chemicals, especially those that bioaccumulate in fish and the concern is with human exposure through consumption of fish. The most common types of substances in this category are the chlorinated organics. The Department annually lists the instream values for the chemicals calculated under Rule 57 and Guidelines (see Appendix D).

Recent amendments to the Federal Clean Water Act have set ambitious deadlines for the control of toxics. By 1989, control strategies for the discharge of toxics must be in place for all waters where protected uses are not currently being met because of toxic substance concerns. Within three years of that date, toxics discharges must be reduced to where all uses are restored.

Michigan is well ahead of most states since the necessary toxics regulations are in place and the majority of the major permits have been reissued under the new rule. Nevertheless, the deadlines set forth in the federal law will be extremely difficult to meet, especially for highly bioaccumulative substances such as PCBs where the acceptable Rule 57 value is extremely low. Michigan welcomes these new federal initiatives, however, which will require equivalent regulation of toxics by all states.

This paper dealt with the control of toxic substances discharged from point sources. This alone, however, will not protect the Great Lakes from toxic chemicals. Significant loadings of toxic substances are entering the Great Lakes through atmospheric transport and nonpoint source runoff. Major new state, national and international initiatives are necessary to address these issues. When Rule 57 controls are fully implemented, Michigan will have essentially eliminated point source discharges as factors in toxic chemical pollution of Great Lakes waters. Only after equivalent levels of control for atmospheric and nonpoint source loadings have been accomplished will the Great Lakes be protected. Michigan looks forward to the challenges of the next decade to accomplish this goal.

ACKNOWLEDGMENTS

The authors acknowledge the support from Michigan's regulated community and major environmental groups and the efforts of the Great Lakes and Environmental Assessment staff and the Rule 57 Advisory Committee members in the development of this process.

REFERENCES

EPA (Environmental Protection Agency). 1985. Guidelines for Deriving Numerical National Water Quality Criteria for the Protection of Aquatic Organisms and Their Uses. U.S. Environmental Protection Agency, NTIS Number PB 85-227049, Environmental Research Laboratory, Duluth, Minnesota.

Crump, Kenny S. and Warren W. Watson. 1979. GLOBAL 79. A FORTRAN program to extrapolate dichotomous animal carcinogenicity data to low doses. National Institute of Environmental Health Sciences Contract NOI-ES-2123.

APPENDICES

A. Department of Natural Resources, Water Resources Commission, General Rules, Part 4. Water Quality Standards, field with Secretary of State November 14, 1986.

B. State of Michigan, Department of Natural Resources, Environmental Protection Bureau, Guidelines for Rule 57(2), filed with Secretary of State January 2, 1985.

C. Staff Report, Support Document for the Proposed Rule 57 Package, Michigan Department of Natural Resources, Environmental Protection Bureau, dated March 26, 1984.

D. Annual Listing of Rule 57(2) Guideline Levels, dated January 27, 1987.

APPENDIX A

DEPARTMENT OF NATURAL RESOURCES

WATER RESOURCES COMMISSION

GENERAL RULES

Filed with the Secretary of State on November 14, 1986
These rules take effect 15 days after filing with the Secretary of State

(By authority conferred on the water resources commission by sections 2 and 5 of Act No. 245 of the Public Acts of 1929, as amended, being §§323.2 and 323.5 of the Michigan Compiled Laws)

R 323.1041 to R 323.1050, R 323.1053, R 323.1055, R 323.1058 to R 323.1065, R 323.1070, R 323.1075, R 323.1082, R 323.1092 to R 323.1098, R 323.1100, and R 323.1116 of the Michigan Administrative Code, appearing on pages 1630 and 1632 to 1639 of the 1979 Administrative Code and pages 162 to 164, 166, and 167 of the 1984 Annual Supplement to the Code, are amended, and R 323.1099 is added, to read as hereinafter set forth.

R 323.1074, R 323.1080, R 323.1091, R 323.1110, and R 323.1115 of the Michigan Administrative Code, appearing on pages 1636 to 1644 of the 1979 Michigan Administrative Code, are rescinded.

PART 4. WATER QUALITY STANDARDS

R 323.1041 Purpose.

Rule 41. The purpose of the water quality standards as prescribed by these rules is to establish water quality requirements applicable to the Great Lakes, the connecting waters, and all other surface waters of the state, to protect the public health and welfare, to enhance and maintain the quality of water, to protect the state's natural resources, and serve the purposes of Public Law 92-500, as amended, 33 U.S.C. §466 et seq., Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws, and the Great Lakes water quality agreement enacted November 22, 1978. These standards may not reflect current water quality in all cases, but are minimum water quality requirements for which the waters of the state are to be managed.

R 323.1043 Definitions; A to N.

Rule 43. As used in this part:

(a) "Agricultural use" means a use of water for agricultural purposes, including livestock watering, irrigation, and crop spraying.

(b) "Anadromous salmonids" means those trout and salmon which ascend streams to spawn.

(c) "Carcinogen" means a substance which causes an increased incidence of benign or malignant neoplasms or a substantial decrease in the latency period between exposure and onset of neoplasms through oral or dermal exposure or through inhalation exposure when the cancer occurs at nonrespiratory sites, in at least 1 mammalian species, or man through epidemiological or clinical studies, unless the commission, on the basis of credible scientific evidence, determines that there is significant

uncertainty regarding the credibility, validity, or significance of such study or studies, in which case it shall refer the question of carcinogenicity to experts on carcinogenesis and shall consider the recommendations of those experts in making its final determination.

(d) "Coldwater fish" means those fish species whose populations thrive in relatively cold water, including trout, salmon, whitefish, and cisco.

(e) "Commission" means the Michigan water resources commission established pursuant to Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws.

(f) "Connecting waters" means any of the following:

(i) The St. Marys river.

(ii) The Keweenaw waterway.

(iii) The Detroit river.

(iv) The St. Clair river.

(v) Lake St. Clair.

(g) "Designated use" means a use of the waters of the state as established by these rules, including use for any of the following:

(i) Industrial, agricultural, and public water supply.

(ii) Recreation.

(iii) Fish, other aquatic life, and wildlife.

(iv) Navigation.

(h) "Dissolved oxygen" means the amount of oxygen dissolved in water and is commonly expressed as a concentration in terms of milligrams per liter.

(i) "Dissolved solids" means the amount of materials dissolved in water and is commonly expressed as a concentration in terms of milligrams per liter.

(j) "Effluent" means a wastewater discharged from a point source to the waters of the state.

(k) "Fecal coliform" means a type of coliform bacteria found in the intestinal tract of humans and other warm-blooded animals.

(l) "Final acute value" means the level of a chemical or mixture of chemicals that does not allow the mortality of important fish or fish food organisms to exceed 50% when exposed for 96 hours, except where a shorter time period is appropriate for certain species.

(m) "Fish, other aquatic life, and wildlife use" means the use of the waters of the state by fish, other aquatic life, and wildlife for any life history stage or activity.

(n) "Industrial water supply" means a water source intended for use in commercial or industrial applications or for noncontact food processing.

(o) "Inland lake" means an inland body of standing water of the state situated in a topographic depression other than an artificial agricultural pond less than one acre, unless it is otherwise determined by the commission. The commission may designate a dammed river channel or an impoundment as an inland lake based on aquatic resources to be protected.

(p) "Keweenaw waterway" means the entire Keweenaw waterway, including Portage lake, Houghton county.

(q) "MATC" means the maximum acceptable toxicant concentration obtained by calculating the geometric mean of the lower and upper chronic limits from a chronic test. A lower chronic limit is the highest tested concentration which did not cause the occurrence of a specified adverse effect. An upper chronic limit is the lowest tested concentration which

did cause the occurrence of a specified adverse effect and above which all tested concentrations caused such an occurrence.

(r) "Mixing zone" means that portion of a water body wherein a point source discharge is mixed with the receiving water.

(s) "Natural water temperature" means the temperature of a body of water without an influence from an artificial source or a temperature as otherwise determined by the commission.

(t) "NOAEL" means the highest level of toxicant which results in no observable adverse effects to exposed test organisms.

(u) "Non-point source" means a source of material other than a source defined as a point source.

R 323.1044 Definitions; P to W.

Rule 44. As used in this part:

(a) "Palatable" means the state of being agreeable or acceptable to the sense of sight, taste, or smell.

(b) "Plant nutrients" means those chemicals, including nitrogen and phosphorus, necessary for the growth and reproduction of aquatic rooted, attached, and floating plants, fungi, or bacteria.

(c) "Point source" means a discernible, confined, and discrete conveyance from which wastewater is or may be discharged to the waters of the state, including the following:

(i) A pipe.
(ii) A ditch.
(iii) A channel.
(iv) A tunnel.
(v) A conduit.
(vi) A well.
(vii) A discrete fissure.
(viii) A container.
(ix) A concentrated animal feeding operation.
(x) A boat or other watercraft.

(d) "Public water supply sources" means a surface raw water source which, after conventional treatment, provides a source of safe water for various uses, including human consumption, food processing, cooking, and as a liquid ingredient in foods and beverages.

(e) "Raw water" means the waters of the state before any treatment.

(f) "Receiving waters" means the waters of the state into which an effluent is or may be discharged.

(g) "Sanitary sewage" means treated or untreated wastewaters which contain human metabolic and domestic wastes.

(h) "Standard" means a definite numerical value or narrative statement promulgated by the commission to maintain or restore water quality to provide for, and fully protect, a designated use of the waters of the state.

(i) "Suspended solids" means the amount of materials suspended in water and is commonly expressed as a concentration in terms of milligrams per liter.

(j) "Total body contact recreation" means any activity where the human body may come into direct contact with water to the point of complete submergence, including swimming, waterskiing, and skin diving.

(k) "Toxic substance" means a substance, except heat, when present in sufficient concentrations or quantities which are or may become harmful to plant life, animal life, or designated uses.

(l) "Warmwater fish" means those fish species whose populations thrive in relatively warm water, including any of the following:

(i) Bass.

(ii) Pike.

(iii) Walleye.

(iv) Panfish.

(m) "Wastewater" means storm water runoff which could result in injury to a use designated in R 323.1100; liquid waste resulting from commercial, institutional, domestic, industrial, and agricultural activities, including cooling and condensing waters; sanitary sewage; and industrial waste.

(n) "Waters of the state" means all of the following, but does not include drainage ways and ponds used solely for wastewater conveyance, treatment, or control:

(i) The Great Lakes and their connecting waters.

(ii) All inland lakes.

(iii) Rivers.

(iv) Streams.

(v) Impoundments.

(vi) Open drains.

(vii) Other surface waterbodies within the confines of the state.

R 323.1050 Physical characteristics.

Rule 50. The waters of the state shall not have any of the following unnatural physical properties in quantities which are or may become injurious to any designated use:

(a) Turbidity.

(b) Color.

(c) Oil films.

(d) Floating solids.

(e) Foams.

(f) Settleable solids.

(g) Suspended solids.

(h) Deposits.

R 323.1051 Dissolved solids.

Rule 51. (1) The addition of any dissolved solids shall not exceed concentrations which are or may become injurious to any designated use. Point sources containing dissolved solids shall be considered by the commission on a case-by-case basis and increases of dissolved solids in the waters of the state shall be limited through the application of best practicable control technology currently available as prescribed by the administrator of the United States environmental protection agency pursuant to section 304(b) of Public Law 92-500, as amended, 33 U.S.C. §466 et seq., except that in no instance shall total dissolved solids in the waters of the state exceed a concentration of 500 milligrams per liter as a monthly average nor more than 750 milligrams per liter at any time, as a result of controllable point sources.

(2) The waters of the state designated as a public water supply source shall not exceed 125 milligrams per liter of chlorides as a monthly average, except for the Great Lakes and connecting waters, where chlorides shall not exceed 50 milligrams per liter as a monthly average.

R 323.1053 Hydrogen ion concentration.

Rule 53. The hydrogen ion concentration expressed as pH shall be maintained within the range of 6.5 to 9.0 in all waters of the state. Any artificially induced variation in the natural pH shall remain within this range and shall not exceed 0.5 units of pH.

R 323.1055 Taste- or odor-producing substances.

Rule 55. The waters of the state shall contain no taste-producing or odor-producing substances in concentrations which impair or may impair their use for a public, industrial, or agricultural water supply source which impair the palatability of fish as measured by test procedures approved by the commission.

R 323.1057. Toxic substances.

Rule 57. (1) Toxic substances shall not be present in the waters of the state at levels which are or may become injurious to the public health, safety, or welfare; plant and animal life; or the designated uses of those waters. Allowable levels of toxic substances shall be determined by the commission using appropriate scientific data.

(2) All of the following provisions apply for purposes of developing allowable levels of toxic substances in the surface waters of the state applicable to point source discharge permits issued pursuant to Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws:

(a) Water quality-based effluent limits developed pursuant to this subrule shall be used only when they are more restrictive than technology-based limitations required pursuant to R 323.2137 and R 323.2140.

(b) The toxic substances to which this subrule shall apply are those on the 1984 Michigan critical materials register established pursuant to Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws; the priority pollutants and hazardous chemicals in 40 C.F.R. §122.21, appendix D (1983); and any other toxic substances as the commission may determine are of concern at a specific site.

(c) Allowable levels of toxic substances in the surface water after a discharge is mixed with the receiving stream volume specified in R 323.1082 shall be determined by applying an adequate margin of safety to the MATC, NOAEL, or other appropriate effect end points, based on knowledge of the behavior of the toxic substance, characteristics of the receiving water, and the organisms to be protected.

(d) In addition to restrictions pursuant to subdivision (c) of this subrule, a discharge of carcinogens, not determined to cause cancer by a threshold mechanism, shall not create a level of risk to the public health greater than 1 in 100,000 in the surface water after mixing with the allowable receiving stream volume specified in R 323.1082. The commission may require a greater degree of protection pursuant to R 323.1098 where achievable through utilization of control measures already in place or where otherwise determined necessary.

(e) Guidelines shall be adopted pursuant to Act No. 306 of the Public Acts of 1969, as amended, being §24.201 et seq. of the Michigan Compiled Laws, setting forth procedures to be used by staff in the development of recommendations to the commission on allowable levels of toxic substances and the minimum data necessary to derive such recommendations. The commission may require the applicant to provide the minimum data when

otherwise not available for derivation of allowable levels of toxic substances.

(f) For existing discharges, the commission may issue a scheduled abatement permit pursuant to R 323.2145 upon a determination by the commission that the applicant has demonstrated that each of the following conditions is met:

(i) Immediate attainment of the allowable level of a toxic substance is not economically or technically feasible.

(ii) No prudent alternative exists.

(iii) During the period of scheduled abatement, the permitted discharge will be consistent with the protection of the public health, safety, and welfare.

(iv) Reasonable progress will be made toward compliance with this rule over the term of the permit, as provided for in a schedule in the permit.

R 323.1058 Radioactive substances.

Rule 58. The control and regulation of radioactive substances discharged to the waters of the state shall be pursuant to the criteria, standards, or requirements prescribed by the United States nuclear regulatory commission in 10 C.F.R. §20.1 et seq. and by the United States environmental protection agency.

R 323.1060 Plant nutrients.

Rule 60. (1) Consistent with Great Lakes protection, phosphorus which is or may readily become available as a plant nutrient shall be controlled from point source discharges to achieve 1 milligram per liter of total phosphorus as a maximum monthly average effluent concentration unless other limits, either higher or lower, are deemed necessary and appropriate by the commission.

(2) In addition to the protection provided under subrule (1) of this rule, nutrients shall be limited to the extent necessary to prevent stimulation of growths of aquatic rooted, attached, suspended, and floating plants, fungi or bacteria which are or may become injurious to the designated uses of the waters of the state.

R 323.1062 Microorganisms.

Rule 62. (1) All waters of the state shall contain not more than 200 fecal coliform per 100 milliliters. This concentration may be exceeded if such concentration is due to uncontrollable non-point sources. The commission may suspend this rule from November 1 through April 30 upon determining that designated uses will be protected.

(2) Compliance with the fecal coliform standards prescribed by subrule (1) of this rule shall be determined on the basis of the geometric average of any series of 5 or more consecutive samples taken over not more than a 30-day period.

(3) Protection of the waters of the state designated for total body contact recreation and public water supply source by the water quality standards prescribed by this rule may be subject to temporary interruption during or following flood conditions, accidents, or emergencies which affect a sewer or wastewater treatment system. In the event of such occurrences, notice shall be served to those affected in accordance with procedures established by the commission. Prompt corrective action shall be taken by the discharger to restore the designated use.

R 323.1064 Dissolved oxygen in Great Lakes, connecting waters, and inland streams.

Rule 64. (1) A minimum of 7 milligrams per liter of dissolved oxygen in all Great Lakes and connecting waterways shall be maintained, and, except for inland lakes as prescribed in R 323.1065, a minimum of 7 milligrams per liter of dissolved oxygen shall be maintained at all times in all inland waters designated by these rules to be protected for coldwater fish. In all other waters, except for inland lakes as prescribed by R 323.1065, a minimum of 5 milligrams per liter of dissolved oxygen shall be maintained. These standards do not apply for a limited warmwater fishery use subcategory or limited coldwater fishery use subcategory established pursuant to R 323.1100(10) or during those periods when the standards specified in subrule (2) of this rule apply.

(2) Waters of the state which do not meet the standards set forth in subrule (1) of this rule shall be upgraded to meet those standards. For existing point source discharges to these waters, the commission may issue permits pursuant to R 323.2145 which establish schedules to achieve the standards set forth in subrule (1) of this rule. If existing point source dischargers demonstrate to the commission that the dissolved oxygen standards specified in subrule (1) of this rule are not attainable through further feasible and prudent reductions in their discharges or that the diurnal variation between the daily average and daily minimum dissolved oxygen concentrations in those waters exceeds 1 milligram per liter, further reductions in oxygen-consuming substances from such discharges will not be required, except as necessary to meet the interim standards specified in this subrule, until comprehensive plans to upgrade these waters to the standards specified in subrule (1) of this rule have been approved by the commission and orders, permits, or other actions necessary to implement the approved plans have been issued by the commission. In the interim, all of the following standards apply:

(a) For waters of the state designated for use for coldwater fish, except for inland lakes as prescribed in R 323.1065, the dissolved oxygen shall not be lowered below a minimum of 6 milligrams per liter at the design flow during the warm weather season in accordance with R 323.1090(3) and (4). At the design flows during other seasonal periods, as provided in R 323.1090(4), a minimum of 7 milligrams per liter shall be maintained. At flows greater than the design flows, dissolved oxygen shall be higher than the respective minimum values specified in this subdivision.

(b) For waters of the state designated for use for warmwater fish and other aquatic life, except for inland lakes as prescribed in R 323.1065, the dissolved oxygen shall not be lowered below a minimum of 4 milligrams per liter, or below 5 milligrams per liter as a daily average, at the design flow during the warm weather season in accordance with R 323.1090(3) and (4). At the design flows during other seasonal periods as provided in R 323.1090(4), a minimum of 5 milligrams per liter shall be maintained. At flows greater than the design flows, dissolved oxygen shall be higher than the respective minimum values specified in this subdivision.

(c) For waters of the state designated for use for warmwater fish and other aquatic life, but also designated as principal migratory routes for anadromous salmonids, except for inland lakes as prescribed in R 323.1065, the dissolved oxygen shall not be lowered below 5 milligrams per liter as a minimum during periods of migration.

(3) The commission may cause a comprehensive plan to be prepared to upgrade waters to the standards specified in subrule (1) of this rule taking into consideration all factors affecting dissolved oxygen in these waters and the cost effectiveness of control measures to upgrade these waters and, after notice and hearing, approve the plan. After notice and hearing, the commission may amend a comprehensive plan for cause. In undertaking the comprehensive planning effort the commission shall provide for and encourage participation by interested and impacted persons in the affected area. Persons directly or indirectly discharging substances which contribute towards these waters not meeting the standards specified in subrule (1) of this rule may be required after notice and order to provide necessary information to assist in the development or amendment of the comprehensive plan. Upon notice and order, permit, or other action of the commission, persons directly or indirectly discharging substances which contribute toward these waters not meeting the standards specified in subrule (1) of this rule shall take the necessary actions consistent with the approved comprehensive plan to control these discharges to upgrade these waters to the standards specified in subrule (1) of this rule.

R 323.1065 Dissolved oxygen; inland lakes.

Rule 65. (1) The following standards for dissolved oxygen shall apply to lakes designated as trout lakes by the natural resources commission or lakes listed in the publication entitled "Coldwater Lakes of Michigan":

(a) In stratified coldwater lakes which have dissolved oxygen concentrations less than 7 milligrams per liter in the upper half of the hypolimnion, a minimum of 7 milligrams per liter dissolved oxygen shall be maintained throughout the epilimnion and upper 1/3 of the thermocline during stratification. Lakes capable of sustaining oxygen throughout the hypolimnion shall maintain oxygen throughout the hypolimnion. At all other times, dissolved oxygen concentrations greater than 7 milligrams per liter shall be maintained.

(b) Except for lakes described in subdivision (c) of this subrule, in stratified coldwater lakes which have dissolved oxygen concentrations greater than 7 milligrams per liter in the upper half of the hypolimnion, a minimum of 7 milligrams per liter of dissolved oxygen shall be maintained in the epilimnion, thermocline, and upper half of the hypolimnion. Lakes capable of sustaining oxygen throughout the hypolimnion shall maintain oxygen throughout the hypolimnion. At all other times, dissolved oxygen concentrations greater than 7 milligrams per liter shall be maintained.

(c) In stratified coldwater lakes which have dissolved oxygen concentrations greater than 7 milligrams per liter throughout the hypolimnion, a minimum of 7 milligrams per liter shall be maintained throughout the lake.

(d) In unstratified coldwater lakes, a minimum of 7 milligrams per liter of dissolved oxygen shall be maintained throughout the lake.

(2) For all other inland lakes not specified in subrule (1) of this rule, during stratification, a minimum dissolved oxygen concentration of 5 milligrams per liter shall be maintained throughout the epilimnion. At all other times, dissolved oxygen concentrations greater than 5 milligrams per liter shall be maintained.

R 323.1069. Temperature; general considerations.

Rule 69. (1) In all waters of the state, the points of temperature measurement normally shall be in the surface 1 meter; however, where turbulence, sinking plumes, discharge inertia or other phenomena upset the natural thermal distribution patterns of receiving waters, temperature measurements shall be required to identify the spatial characteristics of the thermal profile.

(2) Monthly maximum temperatures, based on the ninetieth percentile occurrence of natural water temperatures plus the increase allowed at the edge of the mixing zone and in part on long-term physiological needs of fish, may be exceeded for short periods when natural water temperatures exceed the ninetieth percentile occurrence. Temperature increases during these periods may be permitted by the commission, but in all cases shall not be greater than the natural water temperature plus the increase allowed at the edge of the mixing zone.

(3) Natural daily and seasonal temperature fluctuations of the receiving waters shall be preserved.

R 323.1070 Temperature of Great Lakes and connecting waters.

Rule 70. (1) The Great Lakes and connecting waters shall not receive a heat load which would warm the receiving water at the edge of the mixing zone more than 3 degrees Fahrenheit above the existing natural water temperature.

(2) The Great Lakes and connecting waters shall not receive a heat load which would warm the receiving water at the edge of the mixing zone to temperatures in degrees Fahrenheit higher than the following monthly maximum temperature:

(a) Lake Michigan north of a line due west from the city of Pentwater.

J	F	M	A	M	J	J	A	S	O	N	D
40	40	40	50	55	70	75	75	75	65	60	45

(b) Lake Michigan south of a line due west from the city of Pentwater.

J	F	M	A	M	J	J	A	S	O	N	D
45	45	45	55	60	70	80	80	80	65	60	50

(c) Lake Superior and the St. Marys river:

J	F	M	A	M	J	J	A	S	O	N	D
38	36	39	46	53	61	71	74	71	61	49	42

(d) Lake Huron north of a line due east from Tawas point:

J	F	M	A	M	J	J	A	S	O	N	D
40	40	40	50	60	70	75	80	75	65	55	45

(e) Lake Huron south of a line due east from Tawas point, except Saginaw bay.

J	F	M	A	M	J	J	A	S	O	N	D
40	40	40	55	60	75	80	80	80	65	55	45

(f) Lake Huron, Saginaw bay:

J	F	M	A	M	J	J	A	S	O	N	D
45	45	45	60	70	75	80	85	78	65	55	45

(g) St. Clair river:

J	F	M	A	M	J	J	A	S	O	N	D
40	40	40	50	60	70	75	80	75	65	55	50

(h) Lake St. Clair:

J	F	M	A	M	J	J	A	S	O	N	D
40	40	45	55	70	75	80	83	80	70	55	45

(i) Detroit river:

J	F	M	A	M	J	J	A	S	O	N	D
40	40	45	60	70	75	80	83	80	70	55	45

(j) Lake Erie:

J	F	M	A	M	J	J	A	S	O	N	D
45	45	45	60	70	75	80	85	80	70	60	50

R 323.1075 Temperature of rivers, streams, and impoundments.

Rule 75. (1) Rivers, streams, and impoundments naturally capable of supporting coldwater fish shall not receive a heat load which would do either of the following:

(a) Increase the temperature of the receiving waters at the edge of the mixing zone more than 2 degrees Fahrenheit above the existing natural water temperature.

(b) Increase the temperature of the receiving waters at the edge of the mixing zone to temperatures greater than the following monthly maximum temperatures:

J	F	M	A	M	J	J	A	S	O	N	D
38	38	43	54	65	68	68	68	63	56	48	40

(2) Rivers, streams, and impoundments naturally capable of supporting warmwater fish shall not receive a heat load which would warm the receiving water at the edge of the mixing zone more than 5 degrees Fahrenheit above the existing natural water temperature.

(3) Rivers, streams, and impoundments naturally capable of supporting warmwater fish shall not receive a heat load which would warm the receiving water at the edge of the mixing zone to temperatures greater than the following monthly maximum temperatures:

(a) For rivers, streams, and impoundments north of a line between Bay City, Midland, Alma and North Muskegon:

J	F	M	A	M	J	J	A	S	O	N	D
38	38	41	56	70	80	83	81	74	64	49	39

(b) For rivers, streams, and impoundments south of a line between Bay City, Midland, Alma, and North Muskegon, except the St. Joseph river:

J	F	M	A	M	J	J	A	S	O	N	D
41	40	50	63	76	84	85	85	79	68	55	43

(c) St. Joseph river:

J	F	M	A	M	J	J	A	S	O	N	D
50	50	55	65	75	85	85	85	85	70	60	50

(4) Non-trout rivers and streams that serve as principal migratory routes for anadromous salmonids shall not receive a heat load during periods of migration at such locations and in a manner which may adversely affect salmonid migration or raise the receiving water temperature at the edge of the mixing zone more than 5 degrees Fahrenheit above the existing natural water temperature.

R 323.1082 Mixing zones.

Rule 82. (1) A mixing zone to achieve a mixture of a point source discharge with the receiving waters shall be considered a region in which the response of organisms to water quality characteristics is time dependent. Exposure in mixing zones shall not cause an irreversible response which results in deleterious effects to populations of aquatic life or wildlife. As a minimum restriction, the final acute value for aquatic life shall not be exceeded in the mixing zone at any point inhabitable by these organisms, unless it can be demonstrated to the commission that a higher level is acceptable. The mixing zone shall not prevent the passage of fish or fish food organisms in a manner which would result in adverse impacts on their immediate or future populations. Watercourses or portions thereof which, without 1 or more point source discharge, would have no flow except during periods of surface runoff may be considered as a mixing zone for a point source discharge. The area of mixing zones should be minimized. To this end, devices for rapid mixing, dilution, and dispersion are encouraged where practicable.

(2) For toxic substances, not more than 25% of the receiving water design flow, as stated in R 323.1090, shall be utilized when determining effluent limitations for surface water discharges, unless it can be demonstrated to the commission that the use of a larger volume is acceptable. The commission shall not base a decision to grant more than 25% of the receiving water design flow for purposes of developing effluent limitations for discharges of toxic substances solely on the use of rapid mixing, dilution, or dispersion devices. However, where such a device is or may be employed, the commission may authorize the use of a design flow greater than 25% if the effluent limitations which correspond to such a design flow are shown, based upon a site-specific demonstration, to be consistent with Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws, and other applicable law.

(3) For substances not included in subrule (2) of this rule, the design flow, as stated in R 323.1090, shall be utilized when determining effluent limitations for surface water discharges if the provisions in subrule (1) of this rule are met, unless the commission determines that a more restrictive volume is necessary.

(4) For all substances, defined mixing zone boundaries may be established and shall be determined on a case-by-case basis.

(5) Mixing zones in the Great Lakes, their connecting waters, and inland lakes shall be determined on a case-by-case basis.

R 323.1090. Applicability of water quality standards.

Rule 90. (1) The water quality standards prescribed by these rules shall not apply within mixing zones, except for those standards prescribed in R 323.1082(1) and R 323.1050.

(2) Water quality standards prescribed by these rules are minimally acceptable water quality conditions. Water quality shall be equal to or better than such minimal water quality conditions not less than 95% of the time.

(3) Water quality standards shall apply at all flows equal to or exceeding the design flow. The design flow is equal to the most restrictive of the 12 monthly 95% exceedance flows, except where the commission determines that a more restrictive design flow is necessary or where the commission determines that seasonal design flows may be granted pursuant to R 323.1090(4). The 95% exceedance flow is the flow equal to or exceeded 95% of the time for the specified month.

(4) A maximum of 4 seasonal design flows may be granted when determining effluent limitations for a surface water discharge if it is determined by the commission that the use of such design flows will protect water quality and be consistent with the protection of the public health, safety, and welfare. The seasonal design flows shall be the most restrictive of the monthly 95% exceedance flow for the months in each season. Seasonal design flows shall not be granted for toxic substances which, on the basis of credible scientific evidence, may bioaccumulate in biota inhabiting or using the waters of the state unless, taking into account the receiving water characteristics the persistence and environmental fate characteristics of the substance or substances and the presence of other discharges of bioaccumulative toxic substances into the same receiving waters, the commission determines that the increased mass loading of the substance or substances resulting from granting seasonal design flows is consistent with Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws, and other applicable law.

R 323.1092 Applicability of water quality standards to dredging or construction activities.

Rule 92. Unless the commission determines, after consideration of dilution and dispersion, that such activities result in unacceptable adverse impacts on designated uses, the water quality standards prescribed by these rules shall not apply to dredging or construction activities within the waters of the state where such activities occur or during the periods of time when the aftereffects of dredging or construction activities degrade water quality within such waters of the state, if the dredging operations or construction activities have been authorized by the United States army corps of engineers or the department of natural resources. The

water quality standards shall apply, however, in nonconfined waters of the state utilized for the disposal of spoil from dredging operations, except within spoil disposal sites specifically defined by the United States army corps of engineers or the department of natural resources.

R 323.1096 Determinations of compliance with water quality standards.

Rule 96. Analysis of the waters of the state to determine compliance with the water quality standards prescribed by these rules shall be made pursuant to procedures outlined in 40 C.F.R. §136, as amended by F.R. pp. 43234 to 43442 October 26, 1984, and F.R. pp. 690 to 697 January 4, 1985, or pursuant to other methods prescribed or approved by the commission and the United States environmental protection agency.

R 323.1097 Materials applications not subject to standards.

Rule 97. The application of materials for water resource management projects pursuant to state statutory provisions is not subject to the standards prescribed by these rules, but all projects shall be reviewed and approved by the commission before application.

R 323.1098 Antidegradation.

Rule 98. (1) This rule applies to waters of the state in which the existing water quality is better than the water quality standards prescribed by these rules or than needed to protect existing uses.

(2) These waters shall not be lowered in quality by action of the commission unless it is determined by the commission that such lowering will not do any of the following:

(a) Become injurious to the public health, safety, or welfare.

(b) Become injurious to domestic, commercial, industrial, agricultural, recreational, or other uses which are or may be made of such waters.

(c) Become injurious to the value or utility of riparian lands.

(d) Become injurious to livestock, wild animals, including birds, fish, and other aquatic animals, or plants, or their growth or propagation.

(e) Destroy or impair the value of game, fish, and wildlife.

(f) Be unreasonable and against the public interest in view of the existing conditions.

(3) All of the following waters are designated as protected waters:

(a) All Michigan waters of the Great Lakes, except as these waters may be affected by discharges to the connecting waters and tributaries.

(b) Trout streams south of a line between Bay City, Midland, Alma, and North Muskegon.

(c) Inland lakes.

(d) Reaches of country-scenic and wild-scenic rivers designated under Act No. 231 of the Public Acts of 1970, being §281.761 et seq. of the Michigan Compiled Laws.

(e) Scenic and recreational rivers designated under the wild and scenic rivers act of 1968, 16 U.S.C. §1721 et seq.

(4) In addition to the requirements of subrule (2) of this rule, the waters specified in subrule (3) of this rule shall not be lowered in quality unless, after opportunity for public hearing, it has been demonstrated by the applicant to the commission that a lowering in quality will not be unreasonable, is in the public interest in view of existing conditions, is necessary to accommodate important social or economic

development, and that there are no prudent and feasible alternatives to lowering water quality.

(5) Reaches of the following rivers have been designated pursuant to Act No. 231 of the Public Acts of 1970, being §281.761 et seq. of the Michigan Compiled Laws:

(a) Jordan river - October, 1972, natural river plan.
(b) Betsie river - July, 1973, natural river plan.
(c) Rogue river - July, 1973, natural river plan.
(d) White river - May, 1975, natural river plan.
(e) Boardman river - December, 1975, natural river plan.
(f) Huron river - May, 1977, natural river plan.
(g) Pere Marquette river - July, 1978, natural river plan
(h) Flat river - October, 1979, natural river plan.
(i) Rifle river - May, 1980, natural river plan.
(j) Kalamazoo river - June, 1981, natural river plan.
(k) Pigeon river - June, 1982, natural river plan.

Designated reaches of these rivers are provided in the department of natural resources natural river plan for each respective river.

(6) Reaches of the AuSable river - November, 1984, have been designated pursuant to the wild and scenic rivers act of 1968, 16 U.S.C. §1721 et seq.

(7) Michigan's waters of the Great Lakes are of special significance and are designated as outstanding state resource waters. In addition to the protection specified under subrules (2), (3) and (4) of this rule, mixing zones shall not be used for new or increased discharges to the Great Lakes of toxic substances, as defined by R 323.1057(2)(b), which would result in a lowering of water quality. However, the commission may grant a mixing zone for certain toxic substances on a case-by-case basis, taking into account credible scientific evidence, including persistence and environmental fate characteristics of the substances. Mixing zones for existing discharges of these toxic substances to the Great Lakes and for all discharges of these toxic substances to the connecting waters shall be minimized.

(8) Before authorizing a new or increased discharge of wastewater directly to the Great Lakes or connecting waters, the commission shall provide, in addition to the public notice required by commission rules, supplemental notice of its intent to authorize such discharge, of its proposed determination with respect to the applicable factors set forth in subrule (4) of this rule, and the proposed national pollutant discharge elimination system permit terms and conditions, to the administrator of the United States environmental protection agency, the director of the state or provincial water pollution control agency for all states or provinces which border the lake or connecting waters which receive the new or increased discharge, the United States fish and wildlife service, and the international joint commission. The commission shall allow not less than 30 days for comments from the recipients of the supplemental notice and shall carefully consider all comments received in making its determination.

(9) Wild rivers designated under the wild and scenic rivers act of 1968, 16 U.S.C. §1721 et seq., rivers flowing into, through, or out of national parks or national lakeshores, and wilderness rivers designated under Act No. 231 of the Public Acts of 1970, being §281.761 et seq. of the Michigan Compiled Laws, shall not be lowered in quality. Reaches of

the Two Hearted river - December, 1973, natural river plan - are designated under Act No. 231 of the Public Acts of 1970 as a wilderness river.

R 323.1099 Waters which do not meet standards.

Rule 99. Waters of the state which do not meet the water quality standards prescribed by these rules shall be improved to meet those standards. Where the water quality of certain waters of the state does not meet the water quality standards as a result of natural causes or conditions, further reduction of water quality is prohibited.

R 323.1100 Designated uses.

Rule 100. (1) As a minimum, all waters of the state are designated for, and shall be protected for, all of the following uses:

(a) Agriculture.

(b) Navigation.

(c) Industrial water supply.

(d) Public water supply at the point of water intake.

(e) Warmwater fish.

(f) Other indigenous aquatic life and wildlife.

(g) Partial body contact recreation.

(2) All waters of the state are designated for, and shall be protected for, total body contact recreation from May 1 to October 31 in accordance with R 323.1062. The commission will annually publish a list of those waters of the state located immediately downstream of municipal sewage system discharges where total or partial body contact recreation is contrary to prudent public health practices.

(3) All inland lakes identified in the publication entitled "Coldwater Lakes of Michigan," as published in 1976 by the department of natural resources, are designated for, and shall be protected for, coldwater fish.

(4) All Great Lakes and their connecting waters, except the entire Keweenaw waterway, including Portage lake, Houghton county, and Lake St. Clair, are designated for, and shall be protected for, coldwater fish.

(5) All lakes designated as trout lakes by the natural resources commission under the authority of Act No. 165 of the Public Acts of 1929, as amended, being §301.1 et seq. of the Michigan Compiled Laws, are designated for, and shall be protected for, coldwater fish.

(6) All waters of the state designated as trout streams by the director of the department pursuant to section 8 of Act No. 165 of the Public Acts of 1929, as amended, being §301.8 et seq. of the Michigan Compiled Laws, shall be protected for coldwater fish.

(7) All waters of the state which are designated by the Michigan department of public health as existing or proposed for use as public water supply sources are protected for such use at the point of water intake and in such contiguous areas as the commission may determine necessary for assured protection.

(8) Water quality of all waters of the state serving as migratory routes for anadromous salmonids shall be protected as necessary to assure that migration is not adversely affected.

(9) Discharges to wetlands, as defined by Act No. 203 of the Public Acts of 1979, being §281.701 of the Michigan Compiled Laws, that result in quality less than that prescribed by these rules may be permitted after a use attainability analysis shows that designated uses are not and cannot be attained and shows that attainable uses will be protected.

(10) After completion of a comprehensive plan developed pursuant to R 323.1064(3), upon petition by a municipality or other person, and in conformance with the requirements of 40 C.F.R. §131.10 (1983), the commission may determine that attainment of the dissolved oxygen standards of R 323.1064(1) is not feasible and designate, by amendment to this rule, a limited warmwater fishery use subcategory of the warmwater fishery use or a limited cold water fishery use subcategory of the cold water fishery use. For waters so designated, the dissolved oxygen standards specified in R 323.1064(2) and all other applicable standards of these rules apply. For waters so designated, the dissolved oxygen standards specified in R 323.1064(1) do not apply. Not less than sixty days before filing a petition under this subrule by a municipality or other person, a petitioner shall provide written notice to the executive secretary of the water resources commission and the clerk of the municipalities in which the affected waters are located of its intent to file such petition.

R 323.1105. Multiple designated uses.

Rule 105. When a particular portion of the waters of the state is designated for more than 1 use, the most restrictive water quality standards for one or more of those designated uses shall apply to that portion.

R 323.1116 Availability of documents.

Rule 116. Documents referenced in R 323.1057, R 323.1058, R 323.1065, R 323.1096, and R 323.1100 may be obtained at current costs as follows:

(a) "EPA Priority Pollutants and Hazardous Substances," 40 C.F.R. §122.21, appendix D (1983); copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost, or from the Office of Water Enforcement, United States Environmental Protection Agency, Washington, D.C. 20460, at no cost.

(b) "1984 Michigan Critical Materials Register;" copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(c) "Guidelines Establishing Test Procedures for Analysis of Pollutants," 40 C.F.R. §136 as amended by F.R. pp 43234 to 43442, October 26, 1984, and F.R. pp. 690 to 697, January 4, 1985; copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(d) "Designated Trout Lakes," a publication of the department of natural resources; copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(e) "Coldwater Lakes of Michigan," August, 1976, a publication of the department of natural resources, fisheries division, copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(f) "Designated Trout Streams for the State of Michigan," April, 1975, a publication of the department of natural resources; copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(g) "Standards for Protection Against Radiation," 10 C.F.R. §20, January 1, 1985. Copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(h) "Designation of uses," 40 C.F.R. §131.10, as published in November 8, 1983 F.R. pp. 51406 and 51407; copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

APPENDIX B

State of Michigan

Department of Natural Resources

Environmental Protection Bureau

Guidelines for Rule 57(2)

Rule 323.1057 of the Part 4 Water Quality Standards filed with the Secretary of State, January 2, 1985 , establishes standards for toxic substances. These guidelines set forth procedures, pursuant to Rule 57(2), that Environmental Protection Bureau staff will use in the development of recommendations to the Water Resources Commission on allowable levels of toxic substances in the waters of the state applicable to point source discharge permits. The most recent of such toxic substance levels developed pursuant to these guidelines shall be compiled on an annual basis and be available for distribution by February 1 of each year.

(1) As used in these guidelines:

(a) "Acute toxicity" means the ability of a chemical to cause a debilitating or injurious change in an organism which results from a single or short-term exposure to the chemical.

(b) "Bioconcentration" is the increase in concentration of the chemical of concern and its metabolities in or on the target organisms (or specified tissues thereof) relative to the concentration of the chemical of concern in the ambient water.

(c) "Carcinogen" means a chemical which causes an increased incidence of benign or malignant neoplasms, or a substantial decrease in the latency period between exposure and onset of neoplasms through oral or dermal exposure, or through inhalation exposure when the cancer occurs at non-respiratory sites in at least one mammalian species, or man through epidemiological and/or clinical studies.

(d) "Chronic toxicity" means the ability of a chemical to cause an injurious or debilitating effect in an organism which results from repeated exposure to a chemical for a time period representing a substantial portion of the natural life expectancy of that organism.

(e) "EC50" is the median effective concentration which is the concentration of a test material in a suitable diluent at which 50 percent of the exposed organisms exhibit a specified response during a specified time period.

(f) "Genotoxic teratogen" is a chemical which fits all of the following descriptions:

(i) It is positive in tests of gene mutation (with or without metabolic activation).

(ii) It or its genotoxic metabolites are placentally transferred in any mammalian species through oral, dermal or inhalation exposure.

(iii) It elicits a teratogenic response when administered orally, dermally or by inhalation in at least one mammalian species.

(g) "Hereditary mutagen" is a chemical which has the ability to cause a heritable change in the genome of the germinal cells through oral, dermal, or inhalation exposure in at least one mammalian species.

(h) "LC50" is the median lethal concentration which is the concentration of a test material in a suitable diluent at which 50 percent of the exposed organisms die during a specified time period.

(i) "Life Cycle Safe Concentration" is the highest concentration of a chemical to which an organism is exposed continuously for a life time, and which results in no observable adverse effects to this organism and its progency.

(j) "log Kow" means the log (base 10) of the n-octanol/water partition coefficient.

(k) "MATC" is the maximum acceptable toxicant concentration obtained by calculating the geometric mean of the lower and upper chronic limits from a chronic test. A lower chronic limit is the highest tested concentration which did not cause the occurrence of a specified adverse effect. An upper chronic limit is the lowest tested concentration which did cause the occurrence of a specified adverse effect and above which all tested concentrations caused such an occurrence.

(l) "NOAEL" means the highest level of toxicant which results in no observable adverse effects to exposed test organisms.

(m) "n-octanol/water partition coefficient" (Kow) means the ratio of the octanol to water equilibrium concentrations of a compound.

(n) "Risk" means the probability that a chemical, when released to the environment, will cause an adverse effect in exposed humans, other living organisms, or abiotic environmental compartments.

(o) "Risk assessment" means the analytical process used to determine the level of risk.

(2) The allowable level in the surface water after a discharge is mixed with the receiving stream volume specified in R 323.1082 shall not exceed any of the following:

(a) The Aquatic Chronic Value as derived in (3).

(b) The Terrestrial Life Cycle Safe Concentration as derived in (4).

(c) The Human Life Cycle Safe Concentration as derived in (5).

(d) The concentration providing an acceptable degree of protection to public health for cancer, as derived in (6), when sufficient data are available in the scientific literature to establish that a chemical is a carcinogen.

(e) The concentration providing an acceptable degree of protection to public health for hereditary mutagenic or genotoxic teratogenic effects, as derived in (7), when sufficient data are available in the scientific literature to establish that a chemical is a hereditary mutagen or genotoxic teratogen.

(3) The aquatic chronic value (ACV) shall be derived by one of the following methods. The minimum toxicity data requirements to calculate an ACV in (a) and (b) shall consist of a 96 hour LC50 or EC50 for a rainbow trout or fathead minnow and a 48 hour EC50 or LC50 for a daphnid.

(a) The ACV is calculated in the following manner (modified EPA, 1983) if chronic maximum acceptable toxicant concentration (MATC) data for a chemical are available for at least six North American freshwater species and:

- one species is a salmonid
- one species is a cyprinid or centrarchid
- one species is a daphnid
- one species is a benthic macroinvertebrate
- one species is in the order osteichythyes and not represented in a family above
- one species is any other freshwater species not represented above.

Resident species data are preferred for the above required data set. If one or more of the required species

are not site resident, the requirement may be waived and appropriate substitution will be made.

(i) If the chronic toxicity of the chemical has not been adequately shown to be related to a water quality characteristic, an ACV is calculated using the following procedures. If the toxicity of the chemical is related to a water quality characteristic, go to (a)(ii).

(A) For each species for which at least one chronic MATC is available, the Species Mean Chronic Value (SMCV) is calculated as the geometric mean of the results of all flow-through tests in which the concentrations of test material were measured. For a species for which no such result is available, the geometric mean of all available MATC's is calculated, i.e., results of flow-through tests in which the concentrations were not measured and the results of static and renewal tests based on initial total concentrations of test material.

(B) The SMCV's are ordered from high to low.

(C) Ranks (R) are assigned to the SMCV's from "1" for the lowest to "N" for the highest. If two or more SMCV's are identical, successive ranks are arbitrarily assigned.

(D) The cummulative proportion (P) for each SMCV is calculated as R/(N+1).

(E) The SMCV's (T) (T=3 for 6-7 SMCVs; T=4 for 8 or more SMCVs) are selected which have cummulative proportions closest to 0.05. If there are less than 59 SMCVs, these will always be the three (6-7 SMCVs) or four (8 or more SMCVs) lowest SMCVs.

(F) Using the selected SMCVs and Ps, the ACV is calculated as:

$$S^2 = \frac{\Sigma((\ln SMCV)^2) - ((\Sigma(\ln SMCV))^2/T)}{\Sigma(P) - ((\Sigma(\sqrt{P}))^2/T)}$$

$$L = (\Sigma(\ln SMCV) - S(\Sigma(\sqrt{P})))/T$$

$$A = S(\sqrt{0.05}) + L$$

$$ACV = e^A$$

(ii) If data for a chemical are available to show that chronic toxicity to two or more species is similarly related to a water quality characteristic, an Aquatic Final Chronic Equation must be calculated using the following procedures:

(A) For each species for which comparable chronic toxicity values are available at three or more different values of the water quality characteristic, perform a least squares regression of the chronic toxicity values on the values of the water quality characteristic. Because the best documented relationship is that between hardness and toxicity of metals in fresh water and a log-log relationship best fits the available data, natural logarithms of both toxicity and water quality are used here. For relationships based on other water quality characteristics, such as pH or temperature, no transformation or a different transformation may fit the data better, and appropriate changes will be necessary throughout this section.

(B) Each chronic slope is evaluated as to whether or not it is meaningful, taking into account the range and number of the tested values of the water quality characteristic. For example, a slope based on four data points may be of limited value if it is based

only on data for a narrow range of values of the water quality characteristic. On the other hand, a slope based on only three data points may be meaningful if it is consistent with other information and if the three points cover a broad enough range of the water quality characteristic. If meaningful slopes are not available for at least one fish and one invertebrate or if the available slopes are not similar or if too few data are available to adequately define the shape of the curve, return to (a)(i).

(C) The mean chronic slope (V) is calculated as the arithmetic average of all the meaningful chronic slopes for individual species.

(D) For each species the geometric mean (W) of the chronic toxicity values and the geometric mean (X) of the related values of the water quality characteristic are calculated.

(E) For each species the logarithmic intercept (Y) is calculated using the equation: $Y = \ln W - V(\ln X)$.

(F) For each species the species mean chronic intercept is calculated as the antilog of Y.

(G) The Aquatic Chronic Intercept is obtained by using the procedure described in (a)(i)(A-G), except "Intercept" is inserted for "Value".

(H) The Aquatic Chronic Equation is written as $e^{(V \ \ln(\text{water quality characteristic}) \ + \ln Z)}$, where V = mean chronic slope and Z = Aquatic Chronic Intercept.

(b) If chronic MATC data for a chemical is not available for at least six freshwater species meeting the requirements in (a) an ACV is calculated as follows:

(i) The Final Acute Value (FAV) is divided by an acute/chronic ratio (or geometric mean if more than one is available) for at least one North American freshwater species. The FAV is derived in the following manner (listed in order of preference);

(A) The FAV is derived in the same manner as the ACV described in (a) by substituting FAV for ACV, SMAV (Species Mean Acute Value) for SMCV, and acute for chronic, and LC50 or EC50 for MATC.

(B) If the required data to derive a FAV in (A) is not present in the acute toxicity data base, a FAV is calculated by dividing the most sensitive 96 hr. LC50 or EC50 for rainbow trout or fathead minnow and 48 hr. EC50 or LC50 for a daphnid by the appropriate application factor from the following table:

Species Combination	Application Factor
Rainbow trout/Fathead minnow/ Daphnid	5
Rainbow trout/Daphnid	5
Fathead minnow/Daphnid	10

If appropriate, the FAV will be made a function of a water quality characteristic in a manner similar to that described in (a)(ii).

(ii) If no acute/chronic ratio is available in the aquatic chronic toxicity data base, the ACV is calculated by dividing the FAV by 45.

(c) As an alternative to the chemical specific approaches to calculating an ACV described in (a) and (b), biological techniques may be used to assure that chronically toxic conditions do not exist for aquatic life in the waters of the state. This approach will be used on a case-by-case basis.

(d) On the basis of all available pertinent laboratory and field information, the FAV and ACV are evaluated as to whether they are consistent with sound scientific evidence and are protective of ecologically, commercially, or recreationally important resident species. If not, staff will evaluate appropriate modifications of the procedures, including site specific techniques for deriving acute and chronic values for chemicals.

Reference

U.S. Environmental Protection Agency. 1983. Draft Guidelines for Deriving National Water Quality Criteria for the Protection of Aquatic Life and Its Uses (July 5, 1983). U.S. Environmental Protection Agency Development Document, Environmental Research Laboratory, Duluth, Minnesota.

(4) The concentration to protect wildlife is the Terrestrial Life Cycle Safe Concentration (TLSC). The TLSC shall be derived by one of the following methods, depending on the type and quality of the toxicity data base. The minimum toxicity data requirement for derivation of a TLSC shall consist of an acute oral LD50 for rats. When mammalian and avian toxicity data are available, a TLSC shall be calculated for both groups. The final TLSC is the lowest of the two values.

(a) If a chronic or subacute NOAEL from mammalian or avian species exposed to toxicant contaminated water is available,

$$TLSC = \frac{NOAEL\ (mg/l)}{U}$$

Where: U = uncertainty factor (U = 10-100 depending on quality of study)

(b) If a chronic or subacute NOAEL from mammalian or avian species exposed to toxicant contaminated feed is available,

$$TLSC = \frac{NOAEL\ (ppm) \times C \times \frac{W_a}{V_w}}{U}$$

Where: C = weight of feed consumed daily expressed as a fraction of test animals body weight

W_a = weight of test animal (kg)

V_w = volume of water consumed daily by the test animal (l)

(c) If a chronic or subacute NOAEL from mammalian or avian species exposed to toxicant by gavage is available,

$$TLSC = \frac{\text{NOAEL (mg/kg)} \times \frac{W_a}{V_w} \times Fw}{U}$$

Where: Fw = fraction of days dosed per week

(d) If an oral rat LD50 is available,

$$TLSC = \frac{\text{LD50 (mg/kg)} \times \frac{W_a}{V_w} \times M}{10}$$

Where: M = acute to chronic application factor (M = 0.0001)

(e) TLSCs are best derived from data involving oral exposure. However, if available oral data are insufficient, it may be useful to use data from other exposure routes. Use of such data will depend on the specific pharmacokinetic and toxicological properties of each chemical.

(f) If an acceptable NOAEL is lacking, the lowest observable adverse effect level (LOAEL) may be substituted in some cases, with an additional uncertainty factor of 1 to 10.

(g) On the basis of available information, the TLSC is evaluated as to whether it is consistent with sound scientific judgement. If not, staff will evaluate appropriate modifications of these procedures.

(5) The concentration to protect public health from threshold effect toxicants is the Human Life Cycle Safe Concentration (HLSC). The HLSC shall be derived in the following manner. The minimum toxicity data requirement for derivation of an HLSC shall consist of an acute oral LD50 for rats.

(a) The HLSC shall be derived from appropriate toxicological data using the following formula:

$$HLSC = \frac{MgT\ (mg/day)}{WC + (F \times BCF)}$$

Where: MgT = maximum milligrams of toxicant per day causing no adverse effects to humans when ingested daily for lifetime.

WC = volume of water consumed daily in liters (2 liters (l) for surface water protected for drinking water supply; 0.01 liters (l) for surface water protected for total and partial body contact)

F = daily consumption of fish by humans (F = 0.0065 kg)

BCF = bioconcentration factor as determined in (8).

(i) The MgT shall be derived by one of the following methods depending on the type and quality of the toxicity data base.

(A) If a scientifically valid Maximum Contaminant Level (MCL) from the National Interim Primary Drinking Water Regulations is available,

$$MgT = MCL\ (mg/l) \times V_h$$

Where: V_h = volume of water consumed daily by humans (V_h = 2 l)

(B) If a chronic or subacute no observable adverse effect level (NOAEL) for humans exposed to toxicant contaminated drinking water is available,

$$MgT = \frac{NOAEL\ (mg/l) \times V_h}{U}$$

Where: U = uncertainty factor (U = 10-100)

(C) If a scientifically valid Acceptable Daily Intake (ADI) is available from the Food and Drug Administration Regulations, MgT = ADI.

(D) If a chronic or subacute NOAEL from mammalian test species exposed to toxicant contaminated drinking water is available,

$$MgT = \frac{NOAEL\ (mg/l) \times \frac{V_w}{W_a} \times W_h}{B}$$

Where: V_w = volume of water consumed daily by test animal (l)

W_a = weight of test animal (kg)

W_h = weight of human (W_h = 70 kg)

B = uncertainty factor (B = 100-1,000 depending on quality of study)

(E) If a chronic or subacute NOAEL from mammalian test species exposed to toxicant contaminated food is available,

$$MgT = \frac{NOAEL\ (ppm) \times C \times W_h}{B}$$

Where: C = daily food consumption expressed as a fraction of the animal's body weight

(F) If a chronic or subacute NOAEL from mammalian test species exposed to toxicant by gavage is available,

$$MgT = \frac{NOAEL\ (mg/kg) \times F_w \times W_h}{B}$$

Where: F_w = fraction of days dosed per week

(G) If an oral rat LD50 is available,

$$MgT = \frac{LD50\ (mg/kg) \times M \times W_h}{100}$$

Where: M = acute to chronic application factor

(M = 0.0001)

(H) If an acceptable NOAEL is lacking, the lowest observable adverse effect level (LOAEL) may be substituted in some cases, with an additional uncertainty factor of 1 to 10.

(I) HLSCs are best derived from data involving oral exposure. However, if available oral data are insufficient, it may be useful to use data from other exposure routes. Use of such data will depend on the specific pharmacokinetic and toxicological properties of each chemical.

(J) On the basis of available information, the HLSC is evaluated as to whether it is consistent with sound scientific judgement. If not, staff will evaluate appropriate modifications of these procedures.

(6) The concentration providing an acceptable degree of protection to public health for cancer shall be derived in (a), except for carcinogens that are assumed to cause cancer by a non-threshold mechanism. For these chemicals a greater degree of protection than that derived in (a) may be developed pursuant to R 323.1098 where achievable through utilization of control measures already in place.

(a) A water concentration of the carcinogen shall be derived from human epidemiological data or from appropriate animal research data using the following formula.

$$C = \frac{D \times W_h}{WC + (F \times BCF)}$$

Where: C = concentration of the carcinogen (mg/l)

D = dose derived in (i) (ii) or (iii) (mg/kg/day)

W_h = weight of an average human (W_h = 70 kg)

WC = daily water consumption 0.01 l for surface water protected for total or partial body contact; 2.0 l for surface waters protected for drinking water supply)

F = daily fish consumption (F = 0.0065 kg)

BCF = bioconcentration factor as determined in (8)

(i) The dose (D) may be derived from appropriate human epidemiological data on a case-by-case basis with staff seeking the advice of an expert committee established in (b) as needed.

(ii) Whenever appropriate human epidemiological data are not available, a non-threshold mechanism shall be assumed for carcinogens which have not been adequately demonstrated to cause cancer by a threshold mechanism. The dose (D) shall be the concentration estimated to cause one additional cancer over the background rate in 100,000 individuals exposed to that concentration calculated using the following method:

(A) All carcinogenesis bioassay data are reviewed and data of appropriate quality are used for the quantitative risk estimations. The data are fitted into the multistage model using the computer model GLOBAL 79 developed by Crump and Watson (1979). The upper 95% confidence limit on risk at the 1 in 100 risk level is divided by the maximum likelihood dose at the same level of risk which determines the slope, q_1^*. This is taken as an upper bound of the potency of the chemical in inducing cancer at low doses. Whenever the multistage model does not fit the data, as determined by the Chi-square goodness of fit statistical test, the model is refitted to the data omitting the highest dose. This is continued until an acceptable fit is determined as described in, U.S. Environmental Protection Agency, 1980. If a single study in which a chemical induces more

than one type of tumor is available, then the response for the tumor type predicting the highest estimate of q_1^* is generally used for the risk assessment. If two or more studies of equal quality are available, but vary in any of the following: species, strain, sex or tumor type, then the data set giving the highest estimate of q_1^* is generally used for the risk assessment. If two or more studies exist which are identical regarding species, strain, sex, tumor type, and are of equal quality, then the geometric mean of the q_1^* values from these data sets is used.

(B) The dose corresponding to an estimated one additional cancer in 100,000 exposed test organisms is determined by dividing 10^{-5} by the value for q_1^*.

(C) A species sensitivity factor is used to account for differences between test species and man. It is assumed that mg/surface area/day is an equivalent dose between species. The value may be calculated by dividing the average weight of a human (70 kg) by the weight of the test species and taking the cube root of this value; the slope q_1^* is multiplied by this factor. However, if adequate pharmacokinetic and metabolism studies are available, this data may be factored into the adjustment for species

differences on a case-by-case basis. Staff may seek the advice of an expert committee established in (b) as needed.

(D) All doses are adjusted to give a lifetime average daily dose. If dosing was only for a fraction of a lifetime, then the total dose is averaged over the entire lifespan.

(E) If the duration of experiment (L_e) is less than the natural lifespan of the test animal (L), the slope, q_1^*, is multiplied by the factor $(\frac{L}{L_e})^3$.

(iii) Whenever appropriate human epidemiological data are not available, and the preponderance of data suggests that the chemical causes cancer by a threshold mechanism and does not interact with DNA, the dose (D) for chemicals shall be calculated from animal research data by applying a safety factor to an appropriate toxicity end point. Staff may seek the advice of an expert committee established in (b), as needed.

(A) The appropriate toxicity end point shall be determined by staff on a case-by-case basis.

(B) The safety factor shall be determined by staff based on an evaluation of appropriate toxicological and pharmacological considerations including, mechanism of carcinogenesis, number and type of tumors induced, the spontaneous incidence of tumors, the number of animal species tested and affected, metabolic considerations, epidemiologic observation on exposed humans, extent of the data supporting a nongenetic mechanism, and other pertinent information.

(C) A species sensitivity factor may be used to account for differences between test species and man.

(b) A committee of scientists expert in the field of carcinogenesis shall be established by the director, as needed.

(i) A committee may be convened when the director determines that staff will benefit from advice and recommendations on a highly technical scientific issue which staff requires additional technical expertise to resolve. Such issues include, but are not limited to: specific mechanisms of carcinogenicity (i.e. epigenetic/genetic or promotor/initiator), species sensitivity, and determination of carcinogenicity for chemicals with data not fitting the criteria to determine carcinogenicity established in

these guidelines. Social, political and economic issues shall not be within the charge of this committee. The director shall provide a specific charge to the committee on the issue(s) which they are to address and a time frame for completing the task.

(ii) The committee shall consist of five (5) members.

(A) Three members shall be selected by the director. Two additional members shall be recommended on a unanimous basis to the director by the initial three members.

(B) Each member shall have a PhD degree (or equivalent) in toxicology or a related field, and extensive experience in technical areas pertinent to the issue to be addressed.

(C) Nominees can be recommended by any interested party.

(D) Committee members shall be reimbursed for actual travel expenses within Michigan.

(iii) The committee shall select a chairperson and adopt operating procedures. The procedures shall provide for meeting announcements, agendas and minutes. All meetings shall be open to the public. Provisions for written and oral public comment shall be provided in the procedures.

(iv) The department shall provide staff to the committee.

(v) The committee shall provide a written report stating their recommendations and supporting documentation and rationale.

(c) On the basis of available information, the concentration providing an acceptable degree of protection to public health for cancer is evaluated as to whether it is consistent with sound scientific judgement. If not, staff will evaluate appropriate modifications of these procedures.

References

Crump, Kenny S. and Warren W. Watson. 1979. GLOBAL 79. A FORTRAN program to extrapolate dichotomous animal carcinogenicity data to low doses. National Institute of Environmental Health Science Contract NOI-ES 2123.

U.S. Environmental Protection Agency. 1980. Water Quality Criteria Availability. Appendix C - Guidelines and Methodology Used in the Preparation of Health Effect Assessment Chapters of the Consent Decree Water Criteria Documents. 45 Federal Register 79347-79357.

(7) The level providing an acceptable degree of protection to public health for hereditary mutagenic effects and genotoxic teratogenic effects shall be derived in the following manner.

(a) A committee of scientists expert in the field of hereditary mutagens and genotoxic teratogens may be established, as in (6)(b), to advise staff on these chemicals.

(b) As needed, staff may seek the advice of the expert committee established in (a) above regarding the mutagenic potential of a chemical or the potential of a chemical to act as a genotoxic teratogen and methods for assessing risks.

(c) Staff shall derive acceptable concentrations for these chemicals on a case-by-case basis.

(8) The final bioconcentration factor (BCF_f) standardized to reflect the value for fresh fish tissue having a lipid content of 9.6% shall be determined as follows in order of preference:

(a) Measured, steady-state bioconcentration factors from standardized laboratory tests shall be recorded as BCF_m. If more than a single value is available the BCF_m shall be equal to the geometric mean of the reported values:

(b) If bioconcentration factors are available from other laboratory tests, the BCF_m will be the highest of the following values:

(i) The projected steady-state BCF as calculated from the test data.

(ii) The highest individual BCF reported during the study.

(iii) The apparent steady-state BCF if steady-state was reached but the test duration was not of sufficient length.

(iv) The BCF obtained by dividing the highest tissue concentration of the chemical by the nominal water concentration.

(c) If bioconcentration factors are not available from laboratory studies, the measured bioconcentration factor from a field study may be used as the BCF_m if the following conditions were met:

(i) Data are available to show that the concentration of the chemical in the water to which the organism is exposed remained reasonably constant throughout the study.

(ii) Competing mechanisms for chemical removal from solution did not markedly affect the bioavailability of the chemical.

(iii) The concentration of the chemical to which the organism was exposed is known to be less than the concentration causing any adverse effects on the test species.

(iv) The field measured values agree reasonably well with values reported for similar compounds or with values calculated from regression equations.

(d) If measured bioconcentration factors (BCF_m) are not available from field or laboratory studies, a calculated bioconcentration factor (BCF_c) will be determined by the following equation:

$$\log BCF_c = 0.847 \log Kow - 0.628$$

(e) If a measured Kow is not available for the chemical of interest the Kow may be calculated according to standard references and used in the regression equation in (d).

(f) If a Kow cannot be calculated, BCF_c may be estimated on a case-by-case basis using other regression equations or correlations as appropriate.

(g) The final bioconcentration factor (BCF_f) will be obtained by normalization to 9.6% lipids as follows:

(i) For measured bioconcentration factors:

$$BCF_f = BCF_m \left(\frac{9.6}{L}\right)$$

where BCF_m = measured bioconcentration factor

L = percent lipid content of fish used in the test

(ii) For calculated bioconcentration factors:

$$BCF_f = BCF_c \left(\frac{9.6}{4.8}\right)$$

where BCF_c = calculated bioconcentration factor from log Kow or other regression equations.

4.8 = average percent lipid for test fish used to develop the regression equation in (d)

APPENDIX C

STAFF REPORT

Support Document

for the

Proposed Rule 57 Package

Michigan Department of Natural Resources
Environmental Protection Bureau

March 26, 1984

TABLE OF CONTENTS

	Page
Introduction	541
Proposed Rule 57	542
Proposed Rule 82	544
Proposed Rule 90	545
Proposed Rule 57(2) Guidelines	547
1. Aquatic Chronic Values	547
2. Human Health Values	556
a. Human Life Cycle Safe Concentrations	558
b. Cancer Risk Values	563
3. Terrestrial Life Cycle Safe Concentrations	567
4. Bioconcentration Factors	568
General Considerations	576
Summary	577
Example Calculations	578
References	585
General Questions and Responses	591

INTRODUCTION

This document contains supporting information for the February 28, 1984 draft of the Rule 57 package which includes proposed Rule 57, Rule 82, Rule 90, and Rule 57(2) Guidelines. Justification for amendments proposed, assumptions made and safety factors used will be outlined. Example calculations of Rule 57(2) guideline values will be presented and a list of commonly asked questions and our responses are included. A qualitative assessment of the conservatism incorporated into the approach to calculate allowable levels of toxic substances in the waters of the state applicable to point source discharges will be presented to demonstrate that public health and the environment will be protected with an adequate margin of safety. Uncertainties of the approach are also discussed.

The Rule 57 package is the result of years of effort. Past public comment and recommendations by both the Water Quality Standards Task Force and the Rule 57 Advisory Committee have been considered. Many meetings with interested parties have been held. The Department has been attempting to amend the 1973 water quality standards since 1976. A more definitive approach to control the discharge of toxic substances for the protection of public health is needed. We feel that if the proposed rule amendments and guidelines are adopted, Michigan will have taken a giant step forward in the regulation of toxic substances. The package will also give the regulated community the regulatory certainty that they have been requesting. The approach of blending rule and guidelines for this highly technical and controversial area will enable the Department to factor in recent advances in this field much easier than if the detailed procedures of the guidelines were in rule form.

Rule 82 (mixing zones) and Rule 90 (design flows) were included in the Rule 57 package because they are directly involved in the calculation of allowable levels of toxicants applicable to point source discharges.

The underlying philosophy of the proposed rule package is to promulgate water quality standards with specific authority to protect the public health and environment with an adequate margin of safety. It is important to keep in mind that in dealing with point source discharge permits under both Federal and State laws, treatment based numbers and water quality-based values are addressed separately. The more restrictive of the water quality-based value and the treatment based number becomes the basis for the limit. The Federal government is to develop treatment based standards. Where they do not exist, DNR staff will use best professional judgment to evaluate the need for treatment based numbers more restrictive than water quality-based values.

The determination of allowable levels of toxic substances is a very technical area and the guidelines are detailed. We have attempted to write this document as non-technical as possible; however, some understanding of the basic principles of toxicology is required to fully understand the issues.

PROPOSED RULE 57

The 1973 Rule 57 needs to be amended because it contains outdated literature citations and places its primary emphasis on protecting aquatic life.

The proposed rule is considered a narrative water quality standard as opposed to a numerical rule which would have absolute values specified for a list of toxic substances. Because of the rapid advances in the field of toxicology, difficulties we have had in the past when attempting to promulgate a numerical rule, and the complexities of amending rules in this state, we feel the narrative approach blended with guidelines is the best way to proceed at this time.

The rule is divided into two subrules. The first being a general statement prohibiting injurious levels of toxic substances in the waters of the state and indicating that the Commission will determine allowable levels by using appropriate scientific data. Determination of allowable levels for situations other than point source discharges will need to be done on a case-by-case basis.

Subrule (2) specifically addresses the development of allowable toxicant levels in the waters of the state applicable to point source discharges. More detail is provided by defining the universe of chemicals to which the subrule applies, placing an upper boundary on risk of 1 in 100,000 for carcinogens not determined to cause cancer by a threshold mechanism, indicating that the allowable toxicant levels apply at the edge of mixing zones, referencing the Rule 57(2) guidelines and establishing a mechanism and conditions for issuing non-conforming use permits.

The Department has received comments from both the regulated community and some environmental groups recommending that we restrict the number of chemicals to which Rule 57 would be applied. Staff would like to have no limits on the universe of chemicals but we feel that a workable solution is presented in the rule. The Michigan Critical Materials List and EPA's lists of priority pollutants and hazardous materials will be used as the generic lists of concern. However, if a chemical not on the lists is of concern for a specific permit, the Commission can make a determination to include it on a case-by-case basis.

The risk assessment process and upper limit on risk for chemicals assumed to be non-threshold carcinogens was agreed upon by the Rule 57 Advisory Committee and staff. The determination of an acceptable level of risk is a complex socio-economic issue in which many factors need to be considered. Rule 57 requires that a point source discharge not create an estimated level of risk to public health greater than 1 in 100,000 above background in the surface water after mixing with the allowable receiving stream volume specified in Rule 82 and calculated using the model and assumptions specified in the guidelines. DNR staff feels that the actual risk to the public health associated with exposure to these chemicals in most surface waters of the state under these conditions, will be considerably less than 1 in 100,000 and will be well below that of common everyday

risks since the background rate of a person contacting cancer is 1 in 3. More discussion on carcinogens will be presented later in this document.

The concept of blending Rule 57 with the Rule 57(2) guidelines provides a more flexible package than if all the details of the guidelines were in rule form. We believe that the proposed procedures are practical and can be implemented at this time. However, it is important to realize that the knowledge and understanding of toxic substances is rapidly expanding. The proposed procedures, while valid today, will require periodic review and revision to keep up with the state-of-the-art of the science involved. With this in mind and the fact that guidelines can be changed easier than rules, we kept Rule 57 in the more general narrative form and placed most of the highly technical detailed procedures in the guidelines. The Rule 57(2) guidelines will be discussed in more detail later in this report.

Rule 57(2) states that the Commission may issue a non-conforming use permit if immediate attainment of the allowable level of a toxic substance is not economically or technically feasible and no prudent alternative exists. In addition, the permitted discharge during the period of non-conformance cannot be of a quality which causes long-term adverse impacts to the public health, safety and welfare. These permits are meant to be of an interim nature and must include a schedule to achieve reasonable progress toward compliance with the final limits. During the developmental stages of Rule 57, considerable comments were submitted concerning the possible adverse economic impact of promulgating Rule 57. Staff feel that the facility specific non-conforming use permit is the most appropriate mechanism to address the economic impact of Rule 57.

PROPOSED RULE 82

Rule 82 establishes mixing zones for point source discharges. Changes to this rule are primarily proposed to clarify the intent of the rule. It is specifically stated that for toxic substances, not more than 25 percent of the design flow will be utilized for determining point source discharge limits, unless it can be demonstrated that the use of a larger volume is acceptable. Limiting our initial mixing zone determination for toxics to 25 percent is done when there are insufficient data on the site-specific mixing characteristics of the discharge with the receiving stream to evaluate the impacts on the biological communities within the mixing zone and fish passage.

In general, the entire design flow of the receiving stream is utilized as a mixing zone for conventional substances. Less concern for adverse environmental impacts exists for these substances than for toxics and use of the entire design flow in the general case will not adversely impact the receiving stream.

Because an acceptable technique for determining mixing zones in all inland lakes and Great Lakes situations does not exist, they will be established on a case-by-case basis.

PROPOSED RULE 90

Rule 90 addresses the applicability of the water quality standards. The major proposed change is the mechanism for allowing a maximum of four seasonal design flows when deriving effluent limitations when it is determined by the Commission to be acceptable. This option would allow the use of additional assimilative capacity of the stream which is available during periods of high stream flows and can result in substantial treatment cost saving. However, water quality standards would still be met 95 percent of the time because the design flow would be the most restrictive of the monthly 95 percent exceedance flows for the months in each season. The 95 percent exceedance flow concept was chosen as the most appropriate method to estimate a monthly low flow value.

The most restrictive of the 12 monthly 95 percent exceedance flows was also substituted for the 7-day Q_{10} flow. This was done to be consistent with the seasonal design flow concept. We feel that this approach is a close approximation of the 7-day Q_{10} as shown by flows from some randomly selected streams in Table 1.

Table 1. The 7 day Q10 flows and the most restrictive monthly 95% exceedance flows at selected sites in Michigan

	7Q10 (cfs)	95% Exceedance (cfs)
Black River at Fargo	5.9	5.9
Bell River at Memphis	3.6	4.4
N. Br. Bell River at Imley City	0.25	0.3
Mill Creek NR Abbottsford	4.1	4.0
Mill Creek NR Avoca	1.4	1.8
Pigeon River NR Owendale	0.64	1.0
Deer Creek NR Dansville	0.15	0.2
Red Cedar River at E. Lansing	9.4	11
Grand River at Jackson	22	22
Grand River at Lansing	78	85
Lookingglass River NR Eagle	16.1	18
Grand River at Grand Rapids	707	750
Flat River at Smyrna	127	140
Quaker Brook NR Nashville	1.1	1.3
Thornapple River NR Hastings	49.8	52
Rouge River NR Rockford	71	72
Muskegon River at Evart	310	320
Bear Creek NR Muskegon	2.4	2.4
White River NR Whitehall	187	190
Big Sable River NR Freesoil	85.2	87
Manistee River NR Sherman	704	720
North Br. Kawkawlin	0	0
S. Br. Cass River NR Cass City	1.1	1.2
E. Br. Coon Creek at Armada	0	0
St. Joe River NR Burlington	16.6	15
Trap Rock River NR Linden	8	8.7
Ford River NR Hyde	26.5	32
Peshekee River NR Champion	3.0	3.2

PROPOSED RULE 57(2) GUIDELINES

The Rule 57(2) guidelines are specifically referenced in Rule 57(2)(d). These guidelines will be adopted pursuant to the Administrative Procedures Act and are only binding on the Department. They set forth procedures that Environmental Protection Bureau staff will use in the development of recommendations to the Water Resources Commission on allowable levels of toxic substances in the waters of the state applicable to point source discharge permits. They also set forth the minimum toxicity data needed for a chemical to enable staff to derive their recommendations. Minimum data consists of a rat oral LD50, a 48 hr. EC50 for a daphnid, and a 96 hr. LC50 for a fathead minnow or rainbow trout. Because the allowable toxic substance levels can change based on new toxicity data, a list of the most recent of these toxicant levels will be compiled on an annual basis and will be available for distribution by February 1 of each year.

The guidelines contain detailed procedures for calculating levels necessary to protect aquatic life (Aquatic Chronic Value), wildlife (Terrestrial Life Cycle Safe Concentration), and public health from threshold effect toxic substances (Human Life Cycle Safe Concentration); and concentrations providing an acceptable degree of protection to public health for cancer. The concentration providing an acceptable degree of protection to public health for hereditary mutagenic effects or genotoxic teratogenic effects will be calculated on a case-by-case basis because no recognized methods are presently available. The most restrictive of the above values is used as the allowable level in the surface water after a discharge is mixed with the receiving stream volume specified in Rule 82. Discussion on the calculation of these values and methods for calculating bioconcentration factors follow. The reader is referred to the proposed Rule 57(2) guidelines for the specific procedures.

1. Aquatic Chronic Values

   The aquatic chronic value (ACV) is the highest concentration of a chemical or combination of chemicals which theoretically will produce no adverse effects on important aquatic organisms (and their progeny) exposed continuously for a lifetime. The ACV can be calculated on a chemical specific basis or by using biological techniques, such as bioassays, to assure that chronically toxic conditions do not exist for important aquatic life in the waters of the state. With the chemical specific approach, a specific numerical value is derived for each chemical using the procedures in the guidelines. The details of using biological techniques will be established on a case-by-case basis for each facility.

   The procedures are based on the belief that effects observed in appropriate laboratory tests will generally occur to the same species in comparable field situations. The procedures also account for the effects of various water quality characteristics (i.e., hardness, pH, etc.) on the toxicities of chemical substances. Site specific data are preferred and used whenever possible.

The mechanism used to calculate the ACV for a toxic substance depends upon the number of chronic data points available for that substance. When six or more appropriate chronic data points are available for a chemical, the ACV is calculated directly from fish and macroinvertebrate chronic toxicity data for that chemical using procedures similar to those described in U.S. EPA, 1983. ACVs for chemical substances calculated using this procedure theoretically are designed to be equivalent to, or less than, the chemical's chronic value for 95 percent of all fish and aquatic macroinvertebrate species resident to Michigan's waters.

Unfortunately, there exists a large number of chemical substances for which there are little or no chronic data available. For these chemical substances, the ACV must be predicted from Final Acute Values (FAV) using appropriate application factors. A FAV corresponds to the highest concentration of a chemical in water which theoretically will kill or significantly impair 50 percent of a population of important aquatic organisms exposed continuously for a short period of time (96 hours for fish and aquatic macroinvertebrates, except 48 hours for cladocerans and chironomids). The FAV is calculated using a modified U.S. EPA, 1983 approach when six or more appropriate acute data points are available. If this data base is not available, the FAV is predicted by dividing the most sensitive species tested (rainbow trout/daphnid; or fathead minnow/daphnid) by a species sensitivity factor of 5 if rainbow trout is present in the data base or 10 if rainbow trout is absent. These species sensitivity factors were derived by first assembling those chemicals (Table 2) that have acute toxicity data meeting the minimum data requirements to calculate a FAV using the modified EPA methodology described above and in (3)(b)(i)(A) of the guidelines. Least squares regressions of the $\log_e$ of the FAV versus the $\log_e$ of the most sensitive species LC50 value from the rainbow trout/daphnid and fathead minnow/daphnid combinations were used to determine the expected (average) $\log_e$ of the FAV for a specific species LC50 value (Tables 3, 4 and 5). The equations in Table 6 were used to determine for a specific species LC50 value the 80 percent confidence interval for predicted values of the $\log_e$ of the FAV over the range of LC50 values in the combined data bases. The formula for the 80 percent confidence line is:

$$\bar{Y}_{x_f} - t_{n,.8} \times S^2_{y.x} \left[1 + \frac{1}{N} + \frac{(X_f - \bar{X})^2}{S_x^2 (N-1)} \right]$$

Where:

$\bar{Y}_{x_f}$ is the log FAV calculated for X_f by the regression equations in Table 1;

$t_{n,.8} = 0.851$;

TABLE 2

DATA USED TO CALCULATE AQUATIC ACUTE SENSITIVITY FACTORS

Chemical	Rainbow Trout/*Daphnia* sp. FAV ug/l	Rainbow Trout/*Daphnia* sp. Species Value ug/l	Rainbow Trout/*Daphnia* sp. Sensitive Species	Fathead Minnow/*Daphnia* sp. FAV ug/l	Fathead Minnow/*Daphnia* sp. Species Value ug/l	Fathead Minnow/*Daphnia* sp. Sensitive Species
Lead *	115	2400	Daphnia	115	2400	Daphnia
Lindane	2.1	32	Rainbow	2.1	67	Fathead
Heptachlor	0.90	11.7	Rainbow	0.90	42	Daphnia
Cadmium *	27	93	Daphnia	27	93	Daphnia
Phosmet	0.70	7.8	Daphnia	0.70	7.8	Daphnia
Chlordane	2.6	20	Daphnia	2.6	20	Daphnia
DDD	0.50	3.2	Daphnia	0.50	3.2	Daphnia
Parathion ethyl	0.07	0.42	Daphnia	0.07	0.42	Daphnia
Toxaphene	1.7	9.2	Rainbow	1.7	10	Daphnia
Phosphamidon	2	9.4	Daphnia	2	9.4	Daphnia
Dieldrin	0.48	2.1	Rainbow	0.48	17.7	Fathead
Endrin	0.18	0.71	Rainbow	0.18	0.41	Fathead
Fenithrothion	2.9	11	Daphnia	2.9	11	Daphnia
Benzene Hexa.	5	18	Rainbow	5	125	Fathead
Arsenic	408	1348	Daphnia	408	1348	Daphnia
Selenium	258	710	Daphnia	258	710	Daphnia
Glyphosate	1330	3000	Daphnia	1330	2300	Fathead
Naled	0.16	0.35	Daphnia	0.16	0.35	Daphnia
Carbaryl	4.4	6.4	Daphnia	4.4	6.4	Daphnia
Endosulfan	0.23	0.34	Rainbow	0.23	0.83	Fathead
Chromium +6	4167	6400	Daphnia	4167	6400	Daphnia
Mercury	3.4	5	Daphnia	3.4	5	Daphnia
Trichlorfon	0.12	0.18	Daphnia	0.12	0.18	Daphnia
Ethion	0.04	0.056	Daphnia	0.04	0.056	Daphnia
Mexacarbate	7	10	Daphnia	7	10	Daphnia
Pentachlorophenol	55	71	Rainbow	55	212	Fathead
Silver *	2.2	2.8	Daphnia	2.2	2.8	Daphnia
Malathion	0.79	1	Daphnia	0.79	1	Daphnia
Methoxychlor	0.60	0.78	Daphnia	0.60	0.78	Daphnia
Trifluralin	33	41	Rainbow	33	105	Fathead
Fenthion	0.52	0.62	Daphnia	0.52	0.62	Daphnia
Parathion methyl	0.12	0.14	Daphnia	0.12	0.14	Daphnia
Dichlobenil	3400	3700	Daphnia	3400	3700	Daphnia
Phenol	10150	10200	Rainbow	10150	14000	Daphnia
Cyanide	45	46	Rainbow	45	83	Daphnia
Zinc *	800	749	Daphnia	800	749	Daphnia
Aldrin	4.3	4	Rainbow	4.3	21	Fathead
Chlorine	36	28	Daphnia	36	28	Daphnia
Copper *	43	30	Daphnia	43	30	Daphnia
DDT	0.90	0.36	Daphnia	0.90	0.36	Daphnia

* Hardness dependent
Hardness = 200 mg/l $CaCO_3$

Table 3

Analysis	Regression Line Formula	Significance	Correlation	R^2 Value
Rainbow Trout/Daphnia	$\log_e$ FAV = -0.748 + 0.972 $\log_e$ SPNUM	<0.0001	0.959	0.920
FATHEAD /Daphnia	$\log_e$ FAV = -0.919 + 0.940 $\log_e$ SPNUM	<0.0001	0.938	0.879

Table 4

Analysis	Regression Line
Rainbow Trout/Daphnia	$FAV = 0.473 \times SPNUM^{0.972}$
FATHEAD /Daphnia	$FAV = 0.399 \times SPNUM^{0.940}$

TABLE 5

ANALYSIS	STATISTICAL VALUES			
	$S^2_{y.x}$	N	$\bar{X}$	Sx^2
Rainbow Trout/Daphnia	0.8688	40	2.6156	10.2650
FATHEAD /Daphnia	1.3074	40	2.8868	10.4918

TABLE 6

ANALYSIS	80% CONFIDENCE LIMIT FOR LOG FAV
Rainbow Trout/Daphnia	80% C.L. $\log_e FAV(X_f) = \log_e FAV(X_f)$ .7578 - .001847$(X_f - 2.6156)^2$
FATHEAD /Daphnia	80% C.L. $\log_e FAV(X_f) = \log_e FAV(X_f)$ - 1.1404 - .002234$(X_f - 2.8868)^2$

N is the number of observations used in the regression analysis;

$S^2_{y.x}$ is the mean square error from the simple linear regression analysis;

S_x^2 is the variance of the logged species numbers;

$\overline{X}$ is the mean logged species numbers; and

x_f is the log of the specific species number for which the 80 percent confidence point for the FAV is being calculated.

Dividing the species LC50 value by the 80 percent confidence interval of the predicted FAV, yielded the ratios shown in Tables 7 and 8. Based on these data, species sensitivity factors of 5 and 10 were chosen for the rainbow trout/daphnid and fathead minnow/daphnid combinations, respectively, to simplify the calculation process. FAVs calculated using this procedure will produce an adequate approximation of the FAV derived by using the modified U.S. EPA, 1983 approach.

The ACV is predicted by dividing the FAV by a chemical-specific application factor (acute LC50/chronic value ratios) for those chemical substances which have at least one acute/chronic ratio available. The FAV of a chemical substance when divided by the chemical-specific application factor yields a predicted ACV which will provide an adequate margin of safety to protect fish and aquatic macroinvertebrate species for chronic adverse effects elicited by the chemical.

When chemical-specific application factors cannot be determined for a chemical substance due to an absence of appropriate chronic data, the ACV is predicted by dividing the FAV by a general application factor of 45. This application factor was derived by assembling all appropriate fish and macroinvertebrate acute LC50/chronic value ratios available for chemical substances (Table 9). The ratios were serially arranged from smallest to largest and the ratios versus percentile rank were plotted (Figure 1). The application factor of 45 corresponds to about the 80th percentile rank of all similarily selected and plotted ratios. To provide protection greater than the 80th percentile would require a much larger application factor and was not considered to be appropriate or necessary. An ACV produced by dividing the FAV of a chemical substance by 45 will provide an adequate margin of safety to protect fish and aquatic macroinvertebrates for chronic adverse effects elicited by that chemical substance.

Aquatic acute and chronic values calculated for chemical substances according to the procedure described in the guidelines will be routinely evaluated by staff to ensure their consistency with sound scientific evidence. If not, staff will evaluate appropriate modifications of these procedures.

TABLE 7

ESTIMATED FAV AND SPECIES NUMBER - FAV RATIO
FOR SELECTED SPECIES NUMBERS
RAINBOW TROUT - DAPHNIA ANALYSIS

SPECIES NUMBER	80% CONFIDENCE INTERVAL OF FAV	SPECIES NUMBER - 80% C.I. FAV RATIO
.056	.01	4.4
.42	.09	4.5
1.49	.32	4.6
4.02	.86	4.7
9.32	1.94	4.8
19.50	3.98	4.9
37.77	7.55	5.0
69.00	13.53	5.1
120.20	23.12	5.2
201.20	37.96	5.3
325.60	60.30	5.4
512.00	93.09	5.5
785.00	140.18	5.6
1177.00	206.49	5.7
1730.00	298.27	5.8
2497.00	423.21	5.9
3546.00	591.00	6.0
4964.00	813.76	6.1
6856.00	1105.79	6.2
9350.00	1484.13	6.3
10200.00	1611.70	6.3

TABLE 8
ESTIMATED FAV AND SPECIES NUMBER - FAV RATIO
FOR SELECTED SPECIES NUMBERS
FATHEAD - DAPHNIA ANALYSIS

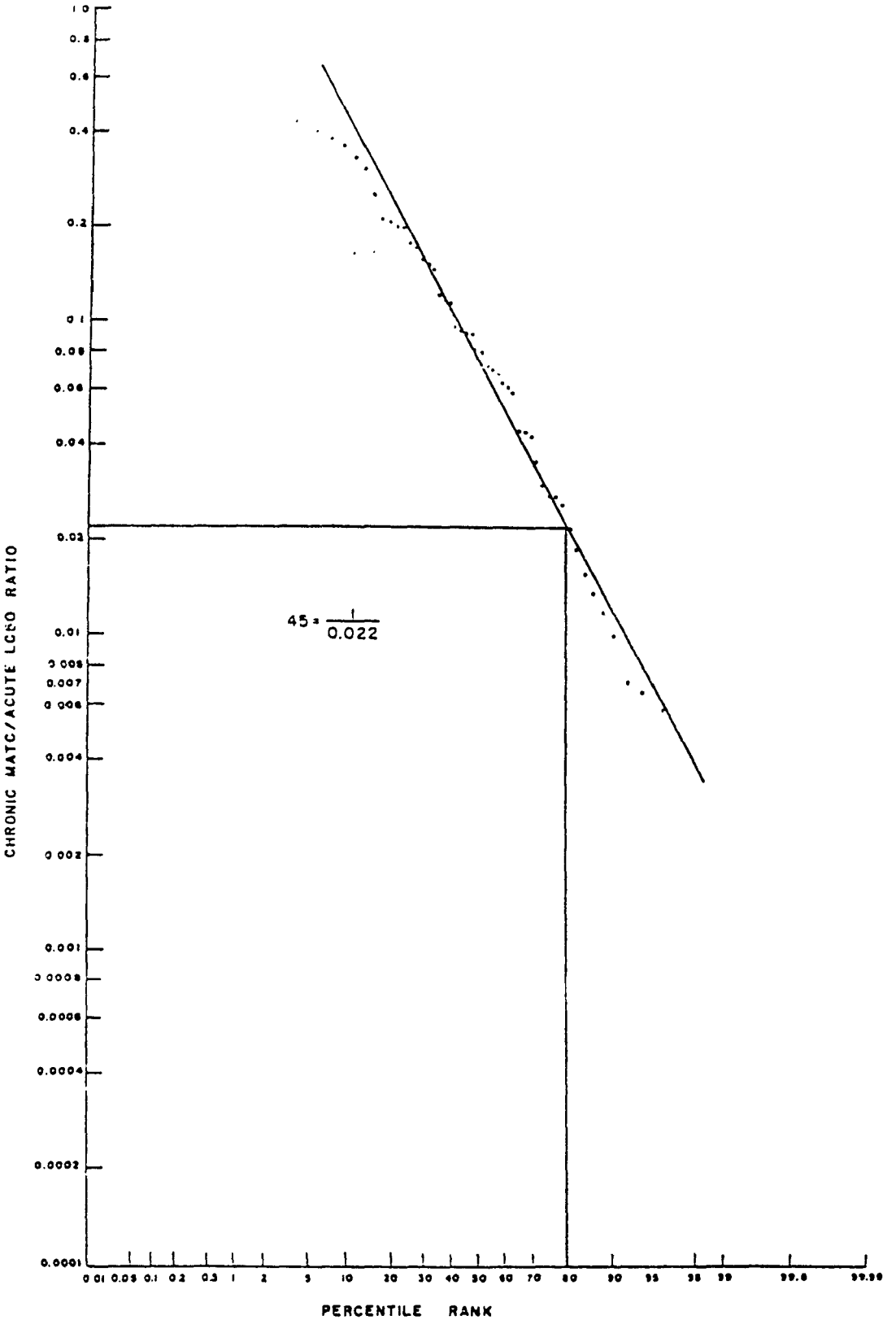
SPECIES NUMBER	80% CONFIDENCE INTERVAL OF FAV	SPECIES NUMBER - 80% C.I. FAV RATIO
.056	.01	7.22
.27	.03	7.60
.94	.12	8.00
2.67	.32	8.40
6.53	.74	8.80
14.33	1.56	9.20
28.85	3.01	9.60
39.94	4.08	9.80
54.50	5.45	10.00
97.40	9.36	10.40
166.00	15.37	10.80
272.00	24.35	11.20
432.00	37.24	11.60
666.00	55.49	12.00
1000.00	80.63	12.40
1468.00	114.66	12.80
2105.00	159.46	13.20
2968.00	218.22	13.60
3505.00	253.95	13.80
4114.00	293.85	14.00
5620.00	390.26	14.40
7590.00	512.68	14.80
10060.00	661.82	15.20
14000.00	892.46	15.69

TABLE 9

DATA USED TO CALCULATE AQUATIC LIFE ACUTE/CHRONIC APPLICATION FACTOR

Chemical	Chronic Value	Acute Value	C/A Ratio	Rank
Hexachlorocyclopentadiene	5.2	7	.7429	1.96
Acrolein	24	57	.4211	3.92
Captan	25.2	64	.3938	5.88
Endrin	.22	.60	.3667	7.84
Hexachloroethane	540	1530	.3529	9.80
Pentachlorophenol	116	364	.3214	11.76
1,2,3,4-Tetrachlorobenzene	.318	1070	.2972	13.72
1,2,4-Trichlorobenzene	705	2870	.2456	15.68
Zinc	138	671	.2057	17.64
LAS	870	4350	.2000	19.60
1,3-Dichlorobenzene	1510	7790	.1938	21.56
1,4-Dichlorobenzene	763	4000	.1908	23.52
Arsenic	912	5278	.1728	25.48
1,2-Dichloroethane	20000	118000	.1695	27.44
Pentachloroethane	1100	7300	.1507	29.40
2,4-Dimethylphenol	2475	16750	.1478	31.36
Antimony	2939	20291	.1448	33.32
1,1,2,2-Tetrachloroethane	2400	20300	.1182	35.28
1,1,2-Trichloroethane	9400	81700	.1151	37.24
Copper	14.5	131	.1107	39.20
Naphthalene	620	6600	.0939	41.16
Hexachlorobutadiene	9.3	102	.0912	43.12
Butyl benzyl phthalate	311	3494	.0890	45.08
Dieldrin	.22	2.5	.0880	47.04
2,4,6-Trichlorophenol	720	9040	.0796	49.00
Lindane	8.1	104	.0779	50.96
Phenol	2560	36000	.0711	52.92
Cyanide	16	233	.0687	54.88
Thallium	86	1280	.0672	56.84
Tetrachloroethylene	840	13460	.0624	58.80
Selenium	159	2500	.0612	60.76
1,2-Dichloropropane	8100	139300	.0581	62.72
2,4-Dichlorophenol	365	8230	.0443	64.68
1,3-Dichloropropene	5700	131100	.0435	66.64
Carbaryl	.38	9	.0422	68.60
Malathion	25	738	.0352	70.56
Nickel	130	4355	.0299	72.52
Lead	39	1393	.0279	74.48
Atrazine	187	6900	.0271	76.44
Endosulfan	4.3	166	.0259	78.40
Trifluralin	4.2	193	.0218	80.36
Silver	.12	6.4	.0188	82.32
DDT	.74	48	.0154	84.28
Chlordane	.8	59	.0136	86.24
Chromium $^{+3}$	260	21700	.0120	88.20
Chromium $^{+6}$	519	52970	.0098	90.16
Mercury	.52	74	.0070	92.12
Toxaphene	.14	22	.0064	94.08
Cadmium	2.4	414	.0058	96.04
Beryllium	5.3	2500	.0021	98.00

Figure 1 – Relationship between aquatic life chronic MATC and acute LC50



One of the uncertainties associated with the proposed approach to determine an ACV is that the actual percent level of species protection is difficult to predict when insufficient chronic data are available to use the modified U.S. EPA, 1983 procedures. To account for this uncertainty and add an additional margin of safety, we selected the 80 percent confidence interval rather than best fit when doing the regression analysis to predict FAVs. We also feel that our generic acute to chronic ratio of 45 is higher than most ratios for common industrial chemicals we deal with.

We also feel that the concept of untested species protection is a conservative element of our process. The vast majority of ACVs we derive will be below the available data base for a chemical. No ecologically, commercially, or recreationally important resident species will knowingly go unprotected.

2. Human Health Values

The purpose of establishing human health values is to estimate surface water concentrations which are considered acceptable or do not present a significant risk to the public. Different procedures have been developed to estimate these levels based on whether the effect produced by a chemical is considered a threshold or non-threshold phenomenon. The threshold response assumes that an organism has a physiological reserve which must be depleted before an effect is manifested. For such chemicals, there exists a dose below which no adverse response will likely occur in exposed animals. The non-threshold concept assumes that exposure to any dose of the chemical, no matter how small, will produce a response. For the purposes of these regulations, the toxicity endpoints that are regarded as non-threshold are carcinogenicity and mutagenicity. Since the mechanism of action for causing teratogenesis for some chemicals may be through gene mutations, some teratogenic chemicals (referred to as genotoxic teratogens) may also be regarded as non-threshold toxicants. For carcinogenic chemicals the concentration corresponding to an acceptable level of risk is determined. Similar procedures may be used for mutagens and genotoxic teratogens, however, since methodologies for addressing these types of chemicals have not been developed they will be handled on a case-by-case basis. All other toxic effects will be considered to act by a threshold mechanism. For these chemicals, the acceptable concentration in water is called the human life cycle safe concentration (HLSC). HLSCs and acceptable levels of risk will be further discussed later in this document.

Regardless of whether a chemical produces a threshold or non-threshold effect, in calculating the HLSC or acceptable risk concentration similar assumptions are made regarding exposure potential and the standard reference human. The basis for the exposure assumptions used in the Rule 57 guidelines, is discussed below.

For all surface waters, it is assumed that a person consumes 6.5

grams of contaminated fish per day (approximately 5 lb/yr.). This value is based upon a U.S. EPA survey (U.S. EPA 1980a) of fish and shellfish consumption in the U.S. Based on this data it was estimated that the average per capita consumption of freshwater and estuarine, fish and shellfish in the U.S. was 6.5 g/day. An estimate of consumption of inland fish by Michigan anglers was calculated at about 4 grams/day. The larger EPA value was used because it was calculated on better data and was slightly more conservative.

The volume of water consumed per day is assumed to be an untreated 2.0 liters for surface waters protected as a drinking water source, and an untreated 0.01 liters for all other surface waters. The value of 2.0 liters was recommended by the U.S. EPA for establishing drinking water standards (U.S. EPA 1976). This value was considered to be representative of the fluid consumption of a normal adult male, based on EPA's review of the literature regarding water consumption. The EPA recognized that wide variation in individual consumption would exist, but felt that 2.0 liters was defensible as a reference standard, since this value was 30-100 percent greater than the average value cited by several authors. In addition it was noted that women and children drink less than the average man, so that it is likely that a large percent of the population consumes less than 2.0 liters.

The value of 0.01 liters of water per day for surface waters not protected for drinking water is to account for incidental exposure such as absorption through the skin or ingestion of small quantities of water while swimming or using the waters for other recreational purposes. These exposures are likely to be highly variable and difficult to quantify. Therefore, to account for these exposures, MDNR staff felt that the daily consumption of a small amount (0.01 liters) of water could be used to represent such exposures. This value is believed to provide a conservative estimate of the actual exposure occuring from such incidents.

The duration of exposure is assumed to be daily for a lifetime. Exposure for less than lifetime is likely to result in a lower risk or larger margin of safety.

Daily ingestion of more than 6.5 grams of fish or 0.01 or 2.0 liters of water will increase an individuals risk. Conversely, ingestion of lesser amounts should decrease the risk.

It is also assumed that the sole source of exposure for a chemical occurs from the consumption of contaminated fish and water. Other sources of exposure, e.g., occupational, dietary (other than fish), and ambient air are not considered. To factor contributions from other sources into the calculations would require extensive amounts of monitoring data, which in many cases are not available. Additionally, exposures from other sources will vary widely. It is recognized, however, that significant exposure may occur from other sources,

and that the actual risk for a chemical will be dependent upon the combined exposure from all sources.

A 70 kg (approximately 150 lb.) person is used as the representative human model for calculating HLSCs and cancer risk concentrations. This value is assumed to be the average weight of a human adult. It is the same value used by the U.S. EPA (U.S. EPA, 1980) in developing their ambient water quality criteria for priority pollutants, and by the Safe Drinking Water Committee of the National Research Council (NAS, 1977) in calculating no adverse effect levels in drinking water.

a. Human Life Cycle Safe Concentrations

The human life cycle safe concentration (HLSC) is the highest concentration of a chemical which causes no significant adverse effects to humans and their offspring when exposed continuously for a lifetime.

HLSCs are derived to provide an adequate margin of safety against the adverse effects of chemicals which elicit threshold responses, excluding carcinogenic effects. The threshold response assumes that an organism has a physiological reserve which must be depleted before an effect is manifested. For such chemicals there exists a dose below which no significant adverse response will occur in exposed animals. Experimentally such a dose is called the no observable adverse effect level (NOAEL). Exposure to doses less than the NOAEL will not elicit an observable response.

To derive an HLSC for a chemical, the NOAEL for laboratory animals or man is determined. Although use of human data is preferable, in most cases these data are lacking, and animal data must be used instead. The NOAEL is then divided by the appropriate uncertainty factor to determine the acceptable dose for humans. Uncertainty factors are used to account for the uncertainties in trying to predict an acceptable exposure level for the general human population based upon experimental animal data or limited human data.

Some of the factors that contribute to this uncertainty include:

Interspecies variation in response to a toxicant. Both qualitative and quantitative differences may exist between species in their response to a toxicant. Therefore, the uncertainty factor is meant to account for the possibility that humans may be more sensitive than the animal model used to predict an acceptable level.

Individual variation in the susceptibility to the effects of a toxicant. Such variation present in a genetically diverse human population would not be seen in the highly

inbred strains of laboratory animals used in most toxicological testing.

Inadequacies in study designs or availability of data, e.g., less than lifetime exposures.

Generally the uncertainty factor applied to an experimental NOAEL will range from 10-1,000. The following guidelines for application of uncertainty factors were recommended by the National Academy of Science (NAS, 1977) and are used in the Rule 57(2) guidelines for derivation of the HLSC.

Uncertainty factor (U = 10) "Valid experimental results from studies on prolonged ingestion by man, with no indication of carcinogenicity."

Uncertainty factor (U = 100) "Experimental results of studies of human ingestion not available or scanty (e.g., acute exposure only). Valid results of long-term feeding studies on experimental animals or in the absence of human studies, valid animal studies on one or more species. No indication of carcinogenicity."

Uncertainty factor (U = 1,000) "No long-term or acute human data. Scanty results on experimental animals. No indication of carcinogenicity."

In cases where the data do not meet all the conditions for one of these categories, and appear to fall between requirements for two categories, an intermediate uncertainty factor is used. Such an intermediate uncertainty factor may be developed based on a logarithmic scale (U.S. EPA, 1980a).

In some cases where a NOAEL is not available, a lowest observed adverse effect level (LOAEL) may be used for lack of better data. In such cases an additional uncertainty factor is applied. The magnitude of this uncertainty factor is judgmental, but should lie in the range of 1 to 10. (U.S. EPA, 1980a).

For many chemicals appropriate toxicological data (NOAELs, LOAELs) are not available to derive an HLSC by these methods. To develop this data could take many years of costly testing. In the absence of an adequate toxicity data base, procedures have been developed to derive an HLSC from a single acute toxicity data point, i.e., an oral rat LD50.

The procedure for deriving an HLSC from an oral rat LD50 involves the use of an acute to chronic application factor. The acute to chronic application factor is a numerical value by which the acute oral rat LD50 is adjusted. The value of this factor as derived by MDNR staff is 0.0001. The oral rat LD50 is multiplied by the acute to chronic application factor (0.0001)

and the value obtained from this procedure is used as a surrogate NOAEL.

To derive the acute to chronic application factor, the scientific literature was reviewed, and all chemicals having both an oral rat LD50 and a NOAEL from a lifetime (two year) study in rats were collected (Table 10). Chronic data for the individual chemicals were evaluated for various parameters such as body weight/organ weight changes, sensitive histopathological changes, alterations in blood composition, abnormal serum enzyme levels, inhibition of important enzymes, reproductive impairment or behavioral changes. Chronic studies of a chemical which did not report evaluating a majority of these parameters were not used in the data base.

After the data were collected, the acute to chronic ratio (NOAEL/LD50) was determined for each chemical. This set of acute to chronic ratios was then serially arranged from largest to smallest, the percentile rank of each ratio was determined, and the ratio value vs. percentile rank were then plotted on log-probability paper. A straight line was then fitted through these data points and from this line the ratio corresponding to the 95th percentile rank was determined. The value for this ratio was 0.0001 (Figure 2). Such an application factor when multiplied by any randomly selected or newly generated oral rat LD50 will theoretically give a surrogate NOAEL equivalent to, or less than, the experimentally derived chronic NOAEL, 95 percent of the time.

The appropriateness of the 95th percentile rank for derivation of the application factor is like an "acceptable" level of risk, and therefore, a value judgment. MDNR staff and the Rule 57 Advisory Committee agreed that the 95th percentile rank was appropriate and provided an adequate margin of safety.

It is recognized that there are a number of uncertainties in trying to estimate an HLSC from acute toxicity data. Variation in the mechanism of action for acute and chronic toxicity, pharmacokinetic differences from different exposures (acute vs. chronic), and different measurements of toxicity (NOAEL vs. lethality) are some of the uncertainties involved. Furthermore, estimates of HLSCs from acute data do not necessarily represent the value that would be expected from an actual chronic NOAEL.

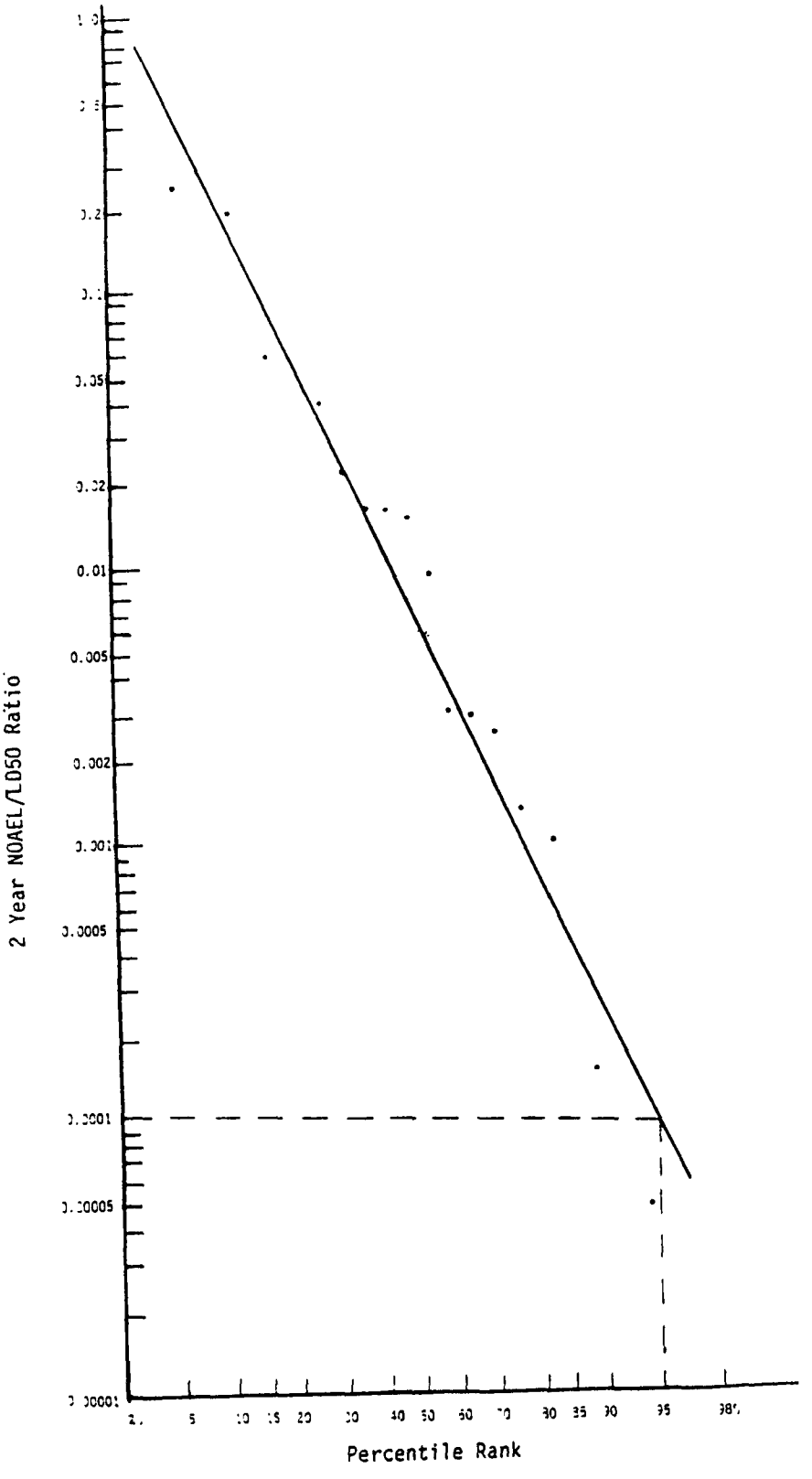
The application factor approach used in deriving HLSCs from acute toxicity data relies on two assumptions, the first being that the data base used to calculate the application factor is a representative sampling of data from all types of toxicants. Since the ratio values varied widely in the collected acute/chronic data, it was assumed that this data base exemplified the variation necessary to be representative for such a process. The second assumption made was that the logs of the ratio values were normally distributed.

Table 10

Data Used to Calculate Acute/Chronic Ratio for HLSC and TLSC

Chemical	2 Yr. NOAEL (mg/kg)	LD50 (mg/kg)	Ratio of NOAEL/LD50	Percentile Rank	References
Di tertiary butyl methyl phenol	400	1,700	0.235	5.6	9
Azinphos methyl	2.5	13	0.192	11.1	31, 10
Methyl salicylate	50	887	0.0564	16.7	13, 29
Sodium (2-ethylhexyl) alcohol sulfate	290	5,760	0.0504	22.2	25
O-phenyl phenol	100	2,700	0.0370	27.8	11
Pentachlorophenol	3	142	0.0211	33.3	23
Carbaryl	7	510	0.0155	38.9	4
Biphenyl	50	3,280	0.0152	44.4	1, 8
3',4'-dichloropropionalide	20	1,384	0.0145	50.0	2
Pimaricin	25	2,730	0.00916	55.6	17
Ronnel	5	1,740	0.00287	61.1	18
1,4-dioxane	14	5,170	0.00271	66.7	14, 16
Bromacil	12.5	5,200	0.0024	72.2	24
2,2-dichloroproprionic acid, Na salt	9.75	7,744	0.00126	77.8	20
Methyl methacrylate	8.4	8,410	0.000999	83.3	3, 7
2,4,5,4'-tetrachlorodiphenyl sulfide	0.5	3,550	0.000141	88.9	28
2,3,7,8-TCDD	0.000001	0.022	0.000045	94.4	15, 22

Figure 2. Relationship Between Oral Pat LD50 and 2 Year NOAEL



b. Cancer Risk Values

The regulation of carcinogenic chemicals is a complex issue since the mechanisms involved in the carcinogenic process are poorly understood. In recent years advances have been made in understanding this process; however, the fundamental mechanisms and causes of cancer still remain uncertain.

Currently, a dominant theory regarding the process by which a chemical causes cancer involves two stages: a) initiation which is believed to be a gene mutation through the direct or indirect interaction of a chemical with the genetic material (DNA) of a cell, and b) promotion, a process which allows the expression of the altered genome.

Chemicals which act on the initiation stage of the carcinogenic process are referred to as initiators. These chemicals are capable of directly altering in an irreversible manner the native structure of the DNA of the cell. These alterations may result from the covalent binding to DNA of the initiator of one of its metabolites, or distortion of the DNA structure without covalent binding from chemicals such as intercalating agents. Initiators may also cause complete scissions of the DNA chain, elimination of one of its component parts (e.g., bases or sugars) or errors in DNA repair (Pitot and Sirica, 1980). If initiation involves a mutational event, then the carcinogenic process of these chemicals may be characterized by a linear non-threshold dose response curve since the biological dose response curve associated with mutagenesis, especially at low doses, is believed to be a linear non-threshold one.

Chemicals which act on the promotional stage of the carcinogenic process are referred to as promotors. Promoting agents do not directly react with the genetic material, but instead affect its expression by a variety of mechanisms. Although the process of promotion is not clearly understood, it may involve various non-threshold mechanisms such as chronic tissue injury, hormonal imbalance, interference with cell-cell communication, or immunologic mechanisms.

Because different mechanisms of action appear to be involved in initiation and promotion, different procedures for estimating acceptable exposure levels for a chemical might be used depending upon which stage in the carcinogenic process the chemical affected. However, it must be kept in mind that the initiation-promotion model of carcinogenesis is still a hypothesis even though it is well founded and currently the most accepted theory.

Due to the limits of current predictive testing, the Rule 57 guidelines make the conservative assumption that any chemical which has been shown to be carcinogenic in one animal bioassay

of good quality, is a complete carcinogen having no threshold. However, the guidelines do include a mechanism for evaluating a carcinogen on a case-by-case basis if the preponderance of data suggests the cancer is caused by a threshold mechanism and does not interact with DNA. A committee of scientists expert in the field of carcinogenesis, may be convened when staff will benefit from advice and recommendations on this issue or other highly technical scientific issues which staff requires additional technical expertise to resolve. At the present time, there are no acceptable methods to establish a threshold mechanism. There is such diversity in the human population with wide genetic variation among those at risk, it will be difficult to establish a threshold.

In addition, even though the word cancer, by strict pathological definition, means malignant tumors only, chemicals which cause benign tumors will also be regulated as carcinogens. The reasons for this approach are that there is not sufficient evidence to show that chemicals are capable of inducing only permanently benign tumors without ever inducing malignant ones, and benign tumors may progress to malignant tumors (IRLG, 1979).

In order to establish acceptable exposure levels for carcinogens, it is necessary to determine the shape of the dose response curve for that chemical and the effect observed. If a linear non-threshold dose-response curve is assumed, then exposure to any dose, no matter how small, may produce a response. Thus, for maximum protection of human health, the concentration of a carcinogen in water is zero. In general, however, a zero exposure policy is not considered technically or economically feasible.

If in theory exposure to any dose above zero may produce a response, the assessment of risk and determination of a response level low enough to be considered insignificant or acceptable can be used in establishing allowable levels of carcinogens in water for point source discharges. The adoption of this policy involves the consideration of two separate issues: 1) selection of an extrapolation procedure to estimate low levels of risk, and 2) judgment as to what constitutes an acceptable level of risk.

If appropriate human epidemiological data are available, an extrapolation from high doses is necessary in order to estimate the carcinogenic risk for the chemical at low concentrations. There are no standard guidelines available to estimate the risk based on all human epidemiology studies since the conditions and variables from these studies are not standardized. However, the use of adequate human exposure data to estimate the risks associated with a carcinogenic chemical is a preferred method and when necessary, the Department may convene an expert committee

to advise staff on an appropriate methodology in order to utilize these data.

When human epidemiological evidence is not available, the carcinogenic risk to humans will be extrapolated from experimental animal data. There is no conclusive scientific evidence for the choice of one mathematical model over another; however, the linearized multistage model, GLOBAL 79 (Crump and Watson, 1979), a non-threshold extrapolation model, is used since no other extrapolation model has as much regulatory acceptance. It is used by various agencies within the state such as the MDNR Air Quality Division and the Department of Public Health and by the U.S. EPA Carcinogen Assessment Group, the State of New York and the State of California in their risk assessment procedures. Use of the upper 95 percent confidence limit to estimate the dose rather than extrapolation from the maximum likelihood estimate dose gives a more stable value which does not change appreciably with minor variability in the biological response at the lower doses. The use of this methodology provides a plausible upper limit estimate of risk. Because of the uncertainties involved, the true risk may range from this upper limit to some lower level, possibly approaching zero, although this cannot be proven. This extrapolation procedure generally follows those outlined in the health effects guidelines for the Ambient Water Quality Criteria Documents which are utilized by the Carcinogen Assessment Group (U.S. EPA, 1980).

Selection of an acceptable level of risk is difficult and may be controversial because risk perception and the degree of public acceptance are not easily analyzed from the available statistics, knowledge about benefits versus costs of the reduced risk, and the degree of voluntary compared to involuntary risks. A list of commonly accepted estimated risks given in Table 11 may help make a more meaningful interpretation of a given level of risk.

The Rule 57 Advisory Committee felt that the risk associated with exposure to these chemicals in ambient water should generally be below that of common everyday risks and recommended that an estimated risk level of 10^{-5} (1 in 100,000) be used as the upper boundry on risk for establishing allowable levels of carcinogens in the waters of the state applicable to point source discharges. Staff agree with this recommendation. Greater levels of protection will also be evaluated at facilities where achievable through utilization of control measures already in place.

Conservative assumptions in this methodology include the use of the linearized multi-stage model with the selection of the 95 percent confidence bound on the estimated carcinogenicity potency and the linear-non-threshold assumptions made when

TABLE 11 Risks of Other Activities

Everyday Risks

	Time to Accumulate a One in 100,000 Risk of Death	Average Annual Risk per Capita	Extrapolated to* Risk/Lifetime
Living in the United States			
Motor vehicle accident	15 days	2×10^{-4}	1.4×10^{-2}
Falls	60 days	6×10^{-5}	4.2×10^{-3}
Drowning	100 days	4×10^{-5}	2.8×10^{-3}
Fires	130 days	3×10^{-5}	2×10^{-3}
Firearms	360 days	1×10^{-5}	7×10^{-4}
Electrocution	20 months	5×10^{-6}	3.5×10^{-4}
Tornados	200 months	6×10^{-7}	4×10^{-5}
Floods	200 months	6×10^{-7}	4×10^{-5}
Lightening	20 years	5×10^{-7}	3.5×10^{-5}
Animal bite or sting	40 years	2×10^{-7}	1.4×10^{-5}
Occupational Risks			
General			
manufacturing	45 days	8×10^{-5}	5.6×10^{-3}
trade	70 days	5×10^{-5}	3.5×10^{-3}
service and government	35 days	1×10^{-4}	7×10^{-3}
transport and public utilities	10 days	4×10^{-4}	3×10^{-2}
agriculture	150 hours	6×10^{-4}	4×10^{-2}
construction	140 hours	6×10^{-4}	4×10^{-2}
mining and quarrying	90 hours	1×10^{-3}	7×10^{-2}
Specific			
coal mining (accidents)	140 hours	6×10^{-4}	3×10^{-2}
police duty	15 days	2×10^{-4}	1.4×10^{-2}
railroad employment	15 days	2×10^{-4}	$1.4 - 10^{-2}$
fire fighting	110 hours	8×10^{-4}	5.4×10^{-2}

Some One in a Million Cancer Risks

Source of Risk	
Cosmic Rays	one transcontinental round trip by air; living 1.5 months in Colorado compared to New York; camping at 15,000 feet for 6 days compared to sea level
Other radiation	20 days of sea level natural background radiation 2.5 months in masonry rather than wood building 1/7 of a chest x ray using modern equipment
Eating and drinking	40 diet sodas (saccharin) 6 pounds of peanut butter (aflatoxin) 180 pints of milk (aflatoxin) 200 gallons of drinking water from Miami or New Orleans 90 pounds of broiled steak (cancer risk only)
Smoking	2 cigarettes

Adapted from Crouch and Wilson (1982)

*Risk/Lifetime = $1 - (1-p)^{70}$

extrapolating from experimentally administered doses. There are other factors such as the use of total animals with malignant and benign organ-specific tumors as input data to the model as well as the selection of the highest potency value (largest estimate of q_1^*) which add some additional conservativism. We also assume that unless there can be an adequate demonstration to the contrary, the potency is adjusted by a animal to man sensitivity factor. On a dose per unit of body surface basis, the effects seen in man are in the same range as those seen in experimental animals. Thus on a body weight basis, man is assumed to be more sensitive than the experimental animals by factors of approximately 5 and 13 for rats and mice, respectively.

Assuming that all animal carcinogens are human carcinogens may also be considered a conservative approach. However, since every known human carcinogen, with the exception of arsenic, has also been found to be carcinogenic in animals, prudent policy is to accept the use of such data, rather than wait for the proof of human carcinogenicity.

Uncertainties and limitations are recognized in the quantitative risk assessment process. The choice of a mathematical extrapolation model may change the risk considerably. Furthermore, the validity of any one model cannot be established, given the limits of current predictive testing methods.

Further uncertainty in the risk estimates comes from the diverse environmental conditions to which the human population is exposed. Various factors such as diet, stress, sex, age, and exposure to the chemical by other routes can alter the response to a carcinogen. Additionally, genetic variability may result in differences in susceptibility between various human subgroups. Because of these limitations, the extent to which the estimated risk reflects the true human risk will always be uncertain. These guidelines also are not able to utilize predictive screening tests to estimate risks without some parallel positive animal data.

3. Terrestrial Life Cycle Safe Concentrations

The purpose of establishing terrestrial life water quality values is to determine surface water concentrations which are considered acceptable for the wildlife and livestock that utilize these waters. For the purpose of these regulations, this concentration is called the terrestrial life cycle safe concentration (TLSC).

The TLSC is defined as the highest aqueous concentration of a toxicant which causes no significant reduction in the growth, reproduction, viability, or usefulness (in the commercial and/or recreational sense) of a population of exposed organisms (utilizing the receiving waters as a drinking water source), over several generations.

To derive a TLSC, the scientific literature regarding the toxicological effects of a chemical is reviewed to determine a no observable adverse effect level (NOAEL) for appropriate mammalian and/or avian organisms. (See discussion in HLSC section 2a regarding rationale for use of NOAELs.) Data on organisms native to Michigan and likely to be utilizing the particular surface water are preferable for calculating the TLSC. In most cases, however, such data are lacking, and data from common laboratory animals (usually rodents) must be used instead. The experimental NOAEL is then divided by an uncertainty factor ranging from 10-100. This uncertainty factor is to account for 1) species variability, since data from one species are used to predict an acceptable level for all wildlife, and 2) inadequacies in study designs or availability of data. When appropriate NOAEL data are not available, a TLSC may be calculated from a lowest observable adverse effect level (LOAEL), or an oral rat LD50. (See discussion in HLSC section 2a regarding use of LOAELs and oral rat LD50s.)

In calculating a TLSC it is assumed that 100 percent of the exposure for a chemical occurs through drinking water alone. Therefore, for bioaccumulative substances, there may be an additional risk to wildlife whose diet consists largely of fish. Additionally, dermal absorption of a chemical may also increase the risk for that toxicant.

4. Bioconcentration Factors

One critical property of chemicals that has a major influence on the calculation of allowable toxicant levels is the ability of persistent, apolar organic compounds to accumulate in aquatic biota to concentrations which are orders of magnitude higher than the concentration of the chemical in the water. For toxic chemicals, this bioaccumulation in aquatic organisms can increase the exposure of consumers of these organisms to the chemicals in question. This increased exposure of consumers and subsequent increase in risk of adverse health effects must be considered by the regulatory agency in the development of allowable toxicant levels.

Historical releases of toxic substances in the Great Lakes basin have resulted in the bioaccumulation of hundreds of different chemical compounds by fish and other organisms. In a recent report, Hesselberg and Seelye (1982) list four hundred seventy-six compounds which they have identified in samples of lake trout and walleye (Stizostedion v. vitreum) collected from selected areas of the Great Lakes. Restrictions of the commercial fishery and advisories against consumption of some fish from a number of waters of the State currently limit the utilization of certain fishery resources due to the elevated concentrations of pesticides, PCBs, and other chemicals present in these fish (MDNR, 1984).

The bioconcentration factor (BCF) value is a key element in the derivation of allowable levels of toxic substances as proposed

by the proposed Rule 57(2) guidelines. For those chemicals which are moderately to highly bioaccumulative in aquatic organisms, the toxicant concentration in fish tissue can represent the most significant exposure for humans consuming these organisms. The dose received via this route of exposure may be the limiting factor in establishing an allowable toxicant level. Conversely, for materials which do not bioconcentrate to a high degree in fish tissue, the bioconcentration factor will be only one of several pieces of data determining a final level. In either case, bioconcentration factors must be estimated as precisely as possible to ensure that the appropriate factor is used.

To calculate equilibrium residue concentrations of toxic materials in aquatic organisms, it is necessary to determine the rates of uptake and depuration of specific chemicals by the species of interest. Alternatively, measurements of the chemical residue concentration must be made over a sufficient period of time to ensure that equilibrium, steady-state conditions have been reached. For purposes of these guidelines, the final, steady-state bioconcentration factor (BCF_f) is the best available determination of the degree to which a chemical accumulates in fish tissue over the water concentration of that chemical.

The term bioconcentration factor must be distinguished from other terms such as biomagnification, bioaccumulation, and ecological magnification. These other terms take into account the observed effects of an organism's trophic level in a given ecosystem and the subsequent increases in chemical residues due to the chemical burden from food chains. Throughout this paper the terms "bioconcentration", "biomagnification" and "bioaccumulation" will be used as defined by Macek et al. (1979). Bioconcentration refers to the process by which chemicals become concentrated in the tissues of fish and aquatic invertebrates via direct partitioning across the gills or epithelial tissue. Biomagnification refers to the added residues accumulated from the food chain which can be a significant source of toxicant accumulation in longer-lived predatory fish in certain ecosystems. Bioaccumulation is a broader term referring to the total toxicant residue accumulated via bioconcentration and any additional uptake from dietary sources.

For these guidelines, the term bioconcentration is used with the assumption that uptake across external membranous surfaces from water is the main source of the material that is accumulated in the organism. The Department recognizes that this assumption does not hold for all chemicals under all ecological conditions. Unfortunately, it is not yet possible to identify in a consistent manner those chemicals for which this assumption is in error; nor is it possible to quantify the magnitude of the underestimation of steady-state body burden for a particular chemical on the basis of its measured or estimated physicochemical properties. Where food-chain biomagnification is known to have a major effect on equilibrium tissue concentrations of toxic materials, the BCF will be appropriately adjusted to account for this effect.

Many variables are known to affect the process of bioaccumulation of chemicals by aquatic organisms. The combination of these variables has produced measured bioconcentration factors for a single chemical that vary by as much as thirteen fold for different species and life stages of aquatic organisms tested using a single chemical under uniform test conditions (Kenaga and Goring, 1980). In assessing measured bioconcentration factors for a single compound, the proposed Rule 57(2) guidelines specify that the geometric mean of reported BCFs will be used when more than one measured BCF is available. It is felt that this value provides a reasonably accurate determination of the bioconcentration factor given the potential variability of this parameter.

When measured, steady-state bioconcentration factors are not available and kinetic data on uptake and depuration rates are not available, a variety of methods to estimate a bioconcentration factor have been described in the literature which relate the measured bioconcentration factor to some physical property of the chemical of interest such as water solubility or n-octanol:water partition coefficient. Equations describing these relationships are presented in Table 12.

Although correlation coefficients for most of these equations are very high, several authors have discussed the limits of these estimator techniques when used to calculate a bioconcentration factor. Veith and Kosian (1983), after examining data on 122 bioaccumulation tests conducted on thirteen species of fish, developed a refined log BCF/log P regression equation. However, the authors cautioned that the estimation of BCFs by this method provides roughly an order of magnitude level of accuracy. Furthermore, although the regession equation holds for Kow values over several orders of magnitude, Veith and Kosian (1983) caution that its application is limited for chemicals with a molecular weight greater than 600 or a log Kow in excess of 6.0. Shaw and Connell (1984) discuss the limiting effect of what they term a "steric effect coefficient" on bioconcentration. Both molecular size and structural orientation of the molecule were shown to influence the relative accumulation of PCB isomers by aquatic invertebrates and fish.

It is recognized by the Department that bioconcentration factors which are calculated from Kow regression equations may vary by as much as an order of magnitude in either direction from the calculated value. In practice, measured values for BCFs that are known appear to fall within a factor of two from the value which would be calculated from the regression equation used in the proposed Rule 57(2) guidelines.

Table 13 lists log bioconcentration factors calculated from five different, published regression equations and compares the calculated value with the log of the measured value obtained in laboratory testing. Fish species and test conditions were constant for all measured values reported. The mean estimated log BCF from these five regression equations is also listed in Table 13. Regression of the log BCF against the mean log Kow results in the following

TABLE 12

Regression Equations for Estimating BCF and Related Parameters

Equation	Reference
$\log BCF = 0.79 \log P^{a} - 0.40$	(Veith, 1981)
$\log BCF = 0.76 \log P^{a} - 0.23$	(Veith, et al., 1980)
$\log BCF = 0.85 \log P^{a} - 0.70$	(Veith, et al., 1979)
$\log BCF = 0.66 \log Kow - 0.44$ (for H-bonding compounds)	(Briggs, 1981)
$\log BCF = 0.87 \log Kow - 0.62$ (for hydrocarbons and chlorohydrocarbons)	(Briggs, 1981)
$\log BCF = 0.935 \log Kow - 1.495$	(Kenaga and Goring, 1980)
$\log BCF = 0.635 \log Kow + 0.7285$	(Brown, 1978)
$\log BCF = 0.79 \log P - 0.40$	(Veith and Kosian, 1983)
$\log BCF = 0.542 \log P^{a} + 0.124$	(Neely et al., 1974)
$\log BCF = 2.791 - 0.564 \log WS^{c}$	(Kenaga and Goring, 1980)
$\log BCF = 1.119 \log K_{oc}^{g} - 1.579$	(Kenaga and Goring, 1980)
$K_{B}^{b} = 0.048 Kow$	(Mackay, 1982)
$K_{B}^{b} = 86/C_{L}^{d}$	(Mackay, 1982)
$\log BF^{e} = 3.41 - 0.508 \log S^{f}$	(Chiou et al., 1977)
$\log EM^{h} = -0.7504 + 1.1587 \log Kow$	(Metcalf et al., 1975)
$\log EM^{b} = 4.4806 - 0.4732 \log WS^{c}$	(Metcalf et al., 1975)
$\log K_{os}^{i} = 1.08 \log Kow - 1.3$	(Steen and Karickhoff, 1981)

a. $\log P = \log Kow$

b. $K_{B} = BCF$

c. WS = water solubility

d. C_{L} = liquid solute solubility or the "corrected" solid solute solubility

e. BF = BCF

f. S = aqueous solubility

g. Koc = soil sorption coefficient (% carbon normalized)

h. EM = ecological magnification

i. K_{os} = biosorption partition coefficient

Table 13

Compound	log Kow	Estimated Log BCF: Veith et al. 1979	Veith et al. 1980	Veith 1981	Briggs 1981	Mackay 1982	Mean Log Estimated BCF (σ)	Measured Log BCF
heptachlor	5.44	3.92	3.904	3.90	4.11	4.12	3.99 (0.11)	3.98
heptachlor epoxide	5.40	3.89	3.87	3.87	4.08	4.08	3.95 (0.12)	4.16
p,p'DDE	5.69	4.14	4.09	4.09	4.33	4.37	4.08 (0.18)	4.71
pentachlorophenol	5.01	3.56	3.58	3.56	3.74	3.69	3.63 (0.08)	2.89
hexabromobiphenyl	6.39	4.73	4.63	4.65	4.94	5.07	4.80 (0.19)	4.26
methoxychlor	4.3	2.96	3.04	3.0	3.12	2.98	3.02 (0.06)	3.92
mirex	6.89	5.16	5.01	5.04	4.94	5.57	5.14 (0.25)	4.26
aroclor 1016	5.88	4.3	4.24	4.24	4.49	4.56	4.37 (0.15)	4.63
aroclor 1248	6.11	4.49	4.41	4.43	4.69	4.79	4.56 (0.17)	4.85
aroclor 1254	6.47	4.8	4.69	4.71	5.01	5.15	4.87 (0.20)	5.00
aroclor 1260	6.91	5.17	5.02	5.06	5.39	5.59	5.25 (0.24)	5.29
chlordane	6.0	4.4	4.33	4.34	4.6	4.68	4.47 (0.16)	4.58
octachlorostyrene	6.29	4.65	4.55	4.57	4.85	4.97	4.72 (0.18)	4.52
p,p'DDT	5.75	4.19	4.14	4.14	4.38	4.43	4.26 (0.14)	4.47
o,p'DDT	5.75	4.19	4.14	4.14	4.38	4.43	4.26 (0.14)	4.47
hexachlorobenzene	5.23	3.74	3.74	3.73	3.93	3.91	3.81 (0.10)	4.27
1,2,4-trichloro-benzene	4.23	2.9	2.98	2.94	3.06	2.91	2.96 (0.06)	3.32
lindane	3.89	2.61	2.73	2.67	2.76	2.57	2.67 (0.08)	2.26
5-bromoindole	2.97	1.82	2.03	1.95	1.96	1.65	1.85 (0.15)	1.15
tricresyl phosphate	3.42	2.21	2.34	2.30	2.36	2.10	2.26 (0.11)	2.22
toluene diamine	3.16	1.99	2.17	2.1	2.13	1.84	2.05 (0.13)	1.96

equation which is used in the Rule 57(2) guidelines to calculate a BCF if measured BCFs are not available from field or laboratory studies:

$$\log BCF = 0.847 \log Kow - 0.628$$

Figure 3 illustrates this relationship and includes the 95 percent confidence interval on the line. A plot of the estimated BCF using this equation against the ideal line (Figure 4) shows very little variance from the ideal situation. In general, this equation gives a slight underestimation of the BCF for chemicals with a log Kow < 2.0 and overestimates the BCF for chemicals with a log Kow > 3.0. In practice this adds an additional level of conservativism to the allowable levels calculated for materials with higher log Kow values. Substances with log Kow < 2.0 are not expected to bioaccumulate to any significant degree in fish due to the ability of aquatic organisms to excrete or metabolize these more water soluble materials. Veith et al. (1980) have recommended that substances with log Kow < 3.0 be excluded from further bioconcentration testing due to the lack of accumulation of these chemicals in fish tissue from monitoring data. BCFs calculated from this Kow/BCF regression equation will generally provide a value biased on the high side and will allow an additional level of conservatism in the calculation.

Additional safety factors are built into the process by using a BCF value for whole fish when only lean muscle tissue is generally consumed by humans. A final adjustment is also made to account for the higher fat content of the average Michigan fish sample (OTMC, 1982) which in most cases doubles the BCF used in the calculations. This percent lipid value is the mean of over 2,000 lipid measurements of Michigan fish samples listed in the "STORET" data base of USEPA.

These adjustment are felt to have a secondary effect of providing an extra margin of safety to account for the probabilities of underestimation discussed above. In nearly all cases where test BCF calculations have been performed according to the proposed Rule 57 procedures, the lipid-standardized bioconcentration factor is higher than values reported from field studies.

Figure 3.

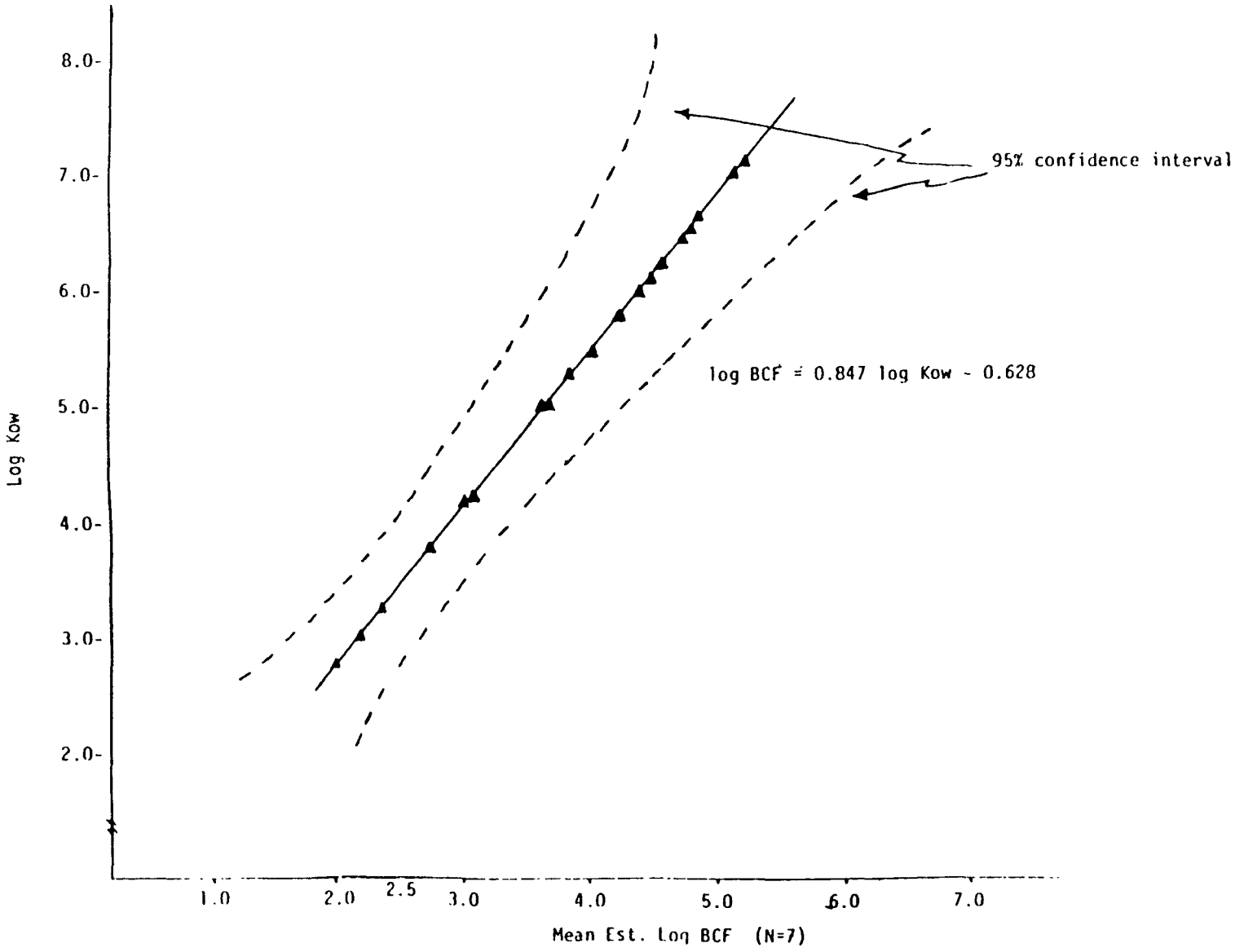
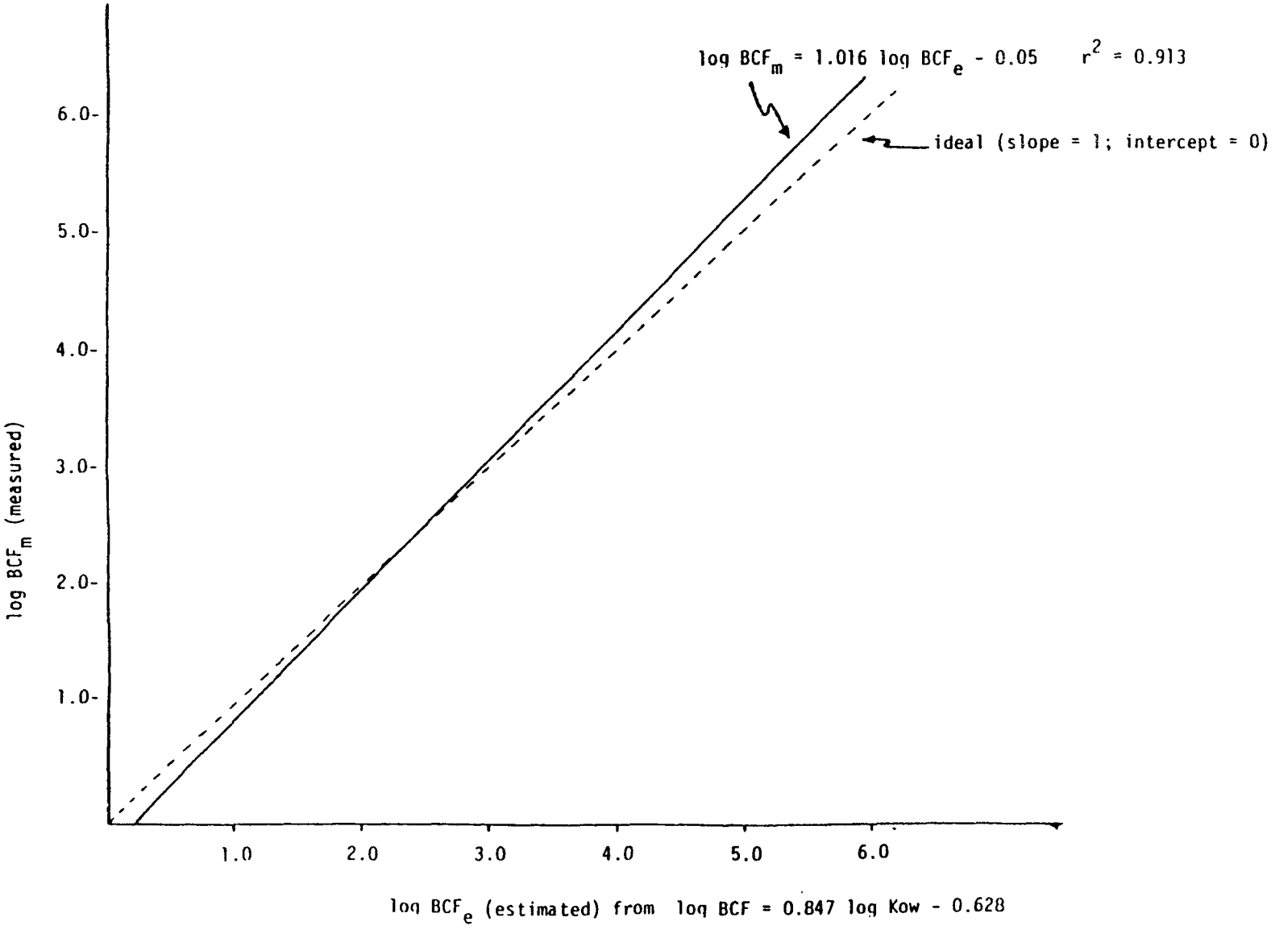

8.0
7.0
6.0
5.0
4.0
3.0
2.0
Log Kow
95% confidence interval
log BCF = 0.847 log Kow - 0.628
1.0
2.0
2.5
3.0
4.0
5.0
6.0
7.0
Mean Est. Log BCF (N=7)

Figure 4.


log BCF$_m$ = 1.016 log BCF$_e$ - 0.05 r^2 = 0.913
ideal (slope = 1; intercept = 0)
log BCF$_m$ (measured)
6.0
5.0
4.0
3.0
2.0
1.0
1.0
2.0
3.0
4.0
5.0
6.0
log BCF$_e$ (estimated) from log BCF = 0.847 log Kow - 0.628

GENERAL CONSIDERATIONS

The guidelines do not contain chemical specific procedures to address chemical interaction (additivity, synergism, antagonism). However, the proposed guidelines do allow for use of whole effluent testing or other biological techniques which would address chemical interactions from an aquatic toxicity perspective. It is recognized that in complex effluents containing more than one chemical, such interactions may occur. For example, if two chemicals acted additively or synergistically, the risk would be at least twice that assuming independence. However, if the chemicals acted antagonistically, the risk would be less than that predicted for independence. In most cases, no data are available to determine the effect of chemical interaction. Although no generic assumptions are made regarding chemical interactions, if in specific cases information is available indicating such interaction, it will be considered when developing allowable levels.

The protection of aquatic and terrestrial plant life or microorganisms has not been adequately addressed by these procedures since testing protocols and methodologies for deriving such allowable levels are not currently available. However, it is felt that in most situations the values calculated to protect aquatic life, wildlife or public health will be sufficient to protect plants and microorganisms.

In addition to the conservatism pointed out in the discussion on the individual guideline values, other more general conservative elements are applied in the effluent limit calculations to add an additional margin of safety to the resultant allowable toxicant levels. One hundred percent conservation of the chemical substance is assumed. In reality, most chemicals undergo some form of degradation due to environmental fate processes (i.e., volatilization, photolysis, microbial degradation, etc.) and may become bound to particulates and organic matter present in the receiving stream.

Rules 82 and 90 allow the use of only 25 percent of the most restrictive monthly 95 percent exceedance flow for establishing effluent limitations, unless a demonstration is made for a larger mixing volume. Even if the entire design flow was used, the concentration of the chemical at the edge of the mixing zone would be lower than the allowable level 95 percent of the time.

SUMMARY

Currently, there are no rules, guidelines or policies for surface waters which provide specific written direction and authority for protecting the public health and environment from the complete spectrum of possible adverse toxic effects. Aquatic toxicity and human carcinogenicity are currently being addressed in surface water permits on a case-by-case basis using general wording of Act 245 and existing Rule 57 as authority. However, this approach is currently being challenged on the basis of "ad hoc rule making". If a challenge is successful, there will be no authority to regulate toxics in surface water permits (with the possible exception of aquatic toxicity). The proposed Rule 57 package would provide both the specific authority and guidelines for staff on development of water quality standards. Clearly, the amended rule package would represent considerable improvement over the 1973 rules in terms of public health and water quality protection and administrative efficiency.

The assumptions and procedures in the proposed Rule 57 package must be evaluated as a whole to obtain the overall feeling for the degree of conservatism in the final calculated value. Staff feel that public health and the environment will be protected with an adequate margin of safety by developing toxic substance discharge levels for surface water permits based on the proposed procedures.

EXAMPLE CALCULATIONS

The step-by-step procedures to develop allowable edge of mixing zone levels for toxic substances according to the Rule 57(2) guidelines are presented in Examples 1-5. A hypothetical chemical (Chemical X) was used for illustrative purposes. The allowable level for Chemical X was derived for a surface water receiving stream protected for warmwater fish and partial body contact and was not considered to be a drinking water source.

The allowable level in the surface water after a discharge is mixed with the receiving stream volume specified in Rule 82 is the most restrictive of the following derivations:

- Aquatic Chronic Value (ACV) - Example 1
- Terrestrial Life Cycle Safe Concentration (TLSC) - Example 2
- Human Life Cycle Safe Concentration (HLSC) - Example 3
- 1 x 10^{-5} Cancer Risk Value - Example 4

For Chemical X, the allowable level is 33 parts per billion (33 ug/1) based on the 1 x 10^{-5} Cancer Risk Value.

EXAMPLE 1 -- CALCULATION OF AQUATIC CHRONIC VALUE (ACV)

Chemical X

Acute Toxicity Data

Species	LC_{50} or EC_{50}	SMAV	Test Duration
Rainbow trout	590 ug/1*	586	96 hr.
Rainbow trout	510 ug/1		96 hr.
Rainbow trout	670 ug/1		96 hr.
Fathead minnow	820 ug/1	820	96 hr.
Bluegill	685 ug/1	685	96 hr.
Largemouth bass	705 ug/1	705	96 hr.
Yellow perch	935 ug/1	935	96 hr.
Daphnia magna	640 ug/1**	617	48 hr.
Daphnia magna	595 ug/1		48 hr.
Scud	1,210 ug/1	1,210	96 hr.
Crayfish	2,140 ug/1	2,140	96 hr.

Chronic Toxicity Data

Species	MATC	Test Type
Rainbow trout	39 ug/1*	Embryo-Larval
Daphnia magna	70 ug/1**	Life Cycle

*From same study on Rainbow trout.
**From same study on *D. magna*.

Calculation of Final Acute Value (FAV)

N = Total Number of SMAVs in Data Set = 8

Rank	SMAV	lnSMAV	$(\ln SMAV)^2$	P = R/(N + 1)	$\sqrt{P}$
4	705	6.5582	43.0100	0.44444	0.66667
3	685	6.5294	42.6333	0.33333	0.57735
2	617	6.4249	41.2789	0.22222	0.47140
1	586	6.3733	40.6192	0.11111	0.33333
Sum		25.8858	167.5414	1.11110	2.04875

$$S^2 = \frac{167.5414 - (25.8858)^2/4}{1.11110 - (2.04875)^2/4} = 0.3682$$

$S = 0.6068$

$L = \left[25.8858 - (0.6068)(2.04875)\right] \div 4 = 6.1607$

$A = (0.6068)(\sqrt{0.05}) + 6.1607 = 6.2964$

$FAV = e^{6.2964} = 543$ ug/l

Calculation of Chemical Specific Acute/Chronic Ratio

Rainbow trout	590 ÷ 39 = 15.1	
D. magna	640 ÷ 70 = 9.1	$\bar{X}g = 11.7$

Calculation of the Aquatic Chronic Value

ACV = 543 ÷ 11.7 = 46 ug/l

EXAMPLE 2 -- CALCULATION OF TERRESTRIAL LIFE CYCLE SAFE CONCENTRATION (TLSC)

The TLSC for Chemical X was derived from a study in which groups of 25 male and female rats were gavaged with Chemical X at 15, 150, 450, and 1,250 mg/kg/day, five days per week, for six months. No effects were observed at 15 mg/kg/day, while adverse effects were observed at higher doses.

I. Data

NOAEL = 15 mg/kg/day
Fraction of days dosed per week (Fw) = 5/7
Weight of rat (Wa) = 0.350 kg
Volume of water consumed per day for rat (Vw) = 0.049 l
Uncertainty factor (U) = 100 (Less than lifetime study; species variability)

II. Calculations

$$TLSC = \frac{NOAEL \times Fw \times Wa/Vw}{U}$$

$$TLSC = \frac{15\ mg/kg/day \times 5/7 \times 0.350kg/0.049\ 1}{100} = 0.765\ mg/l$$

EXAMPLE 3 -- CALCULATION OF HUMAN LIFE CYCLE SAFE CONCENTRATION (HLSC)

The HLSC for Chemical X was derived from a study in which groups of 25 male and female rats were gavaged with Chemical X at 15, 150, 450, and 1,250 mg/kg/day, five days per week for six months. No effects were observed at 15 mg/kg/day, while adverse effects were observed at higher doses.

I. Data

NOAEL = 15 mg/kg/day
Fraction of days dosed per week (Fw) = 5/7
Weight of human (W_H) = 70 kg
Uncertainty factor (U) = 1,000 (No human data; less than lifetime animal study)
Bioconcentration factor (BCF) = 123 (See Example 5)

II. Calculation

$$MgT = \frac{NOAEL \times Fw \times W_H}{U}$$

$$MgT = \frac{15\ mg/kg/day \times 5/7 \times 70\ kg}{1{,}000} = 0.75\ mg$$

$$HLSC = \frac{MgT}{0.01\ 1 + (0.0065\ kg \times BCF)}$$

$$HLSC = \frac{0.75\ mg}{0.01\ 1 + (0.0065\ kg \times 123)} = 0.926\ mg/l$$

EXAMPLE 4 -- CALCULATION OF CANCER RISK VALUE (1×10^{-5})

Groups of 50 male and 50 female Fisher 344 rats were dosed with Chemical X in corn oil vehicle by gavage five days a week for 104 weeks. The Time Weighted Average (TWA) doses for both sexes were 169 and 339 mg/kg/day. However, since the doses were administered five days per week, the daily average doses were calculated as: 169 x 5/7 = 121 and 339 x 5/7 = 242 mg/kg/day. Groups of 20 animals of both sexes received corn oil as the vehicle controls.

Under the conditions of this bioassay, Chemical X was carcinogenic to both male and female F-344 rats, inducing statistically significant increases in hepatocellular carcinomas.

In males, the incidences of animals with hepatocellular carcinomas/animals at risk were 0/20, 16/48, and 40/45 in the vehicle control, low dose and high dose groups, respectively. In females, the incidences of animals with hepatocellular carcinomas/animals at risk were 2/20, 18/49, and 37/49 in the vehicle control, low dose and high dose groups, respectively.

The estimated average mature weight for female rats was 0.35 kg, that for male rats was 0.40 kg.

For male rats, the carcinogenicity potency q_1^* is calculated by dividing the 95 percent confidence limit on risk at the 1 in 100 risk level by the maximum likelihood estimate (MLE) dose as determined from the multi-stage model printout (Figure 5).

$$q_1^* = \frac{0.499458 \times 10^{-1}}{0.173728 \times 10^{-2}} = 2.875 \times 10^{-3} \text{ (mg/kg/day)}^{-1}$$

This is multiplied by an animal to man species adjustment factor $\sqrt[3]{70/0.40} = 5.59$ and q_1^* becomes 1.61×10^{-2} (mg/kg/day)^{-1}

For female rats, q_1^* as determined from the multi-stage model printout (Figure 6):

$$q_1^* = \frac{0.621775 \times 10^{-1}}{0.137924 \times 10^{-2}} = 4.51 \times 10^{-3} \text{ (mg/kg/day)}^{-1}$$

This is multiplied by a species adjustment factor $\sqrt[3]{70/0.35} = 5.85$ and q_1^* becomes 2.64×10^{-2} (mg/kg/day)^{-1}

Using the female rat data, since it gives the highest estimate of q_1^*, the dose at the 10^{-5} level of risk:

$$D = \frac{1 \times 10^{-5}}{2.64 \times 10^{-2} \text{ (mg/kg/day)}^{-1}} = 3.79 \times 10^{-4} \text{ mg/kg/day}$$

Using a calculated BCF of 123 (See Example 5), the water concentration at the 10^{-5} level of risk, surface water not protected as a drinking water source:

$$C = \frac{3.79 \times 10^{-4} \text{ mg/kg/day} \times 70 \text{ kg}}{0.01 \text{ 1/day} + (0.0065 \times 123)} = 3.28 \times 10^{-2} \text{ mg/1} = 33 \text{ ug/1}$$

Figure 5

```
chemical x  hepatocellular carcinomas  male rats
        TITLE :  CHEMICAL X  HEPATOCELLULAR CARCINOMAS  MALE RATS


?
3,1
?
20,48,45
?
0,16,40
?
0,121,242
CLASS( 1) HAS    20 MEMBERS WITH     0 RESPONSES TO DOSE OF          0.00000
CLASS( 2) HAS    48 MEMBERS WITH    16 RESPONSES TO DOSE OF        121.00000
CLASS( 3) HAS    45 MEMBERS WITH    40 RESPONSES TO DOSE OF        242.00000
? 00049
1

KOP = 1 GRADIENT MOVES =  7

THE COEFFICIENTS OF THE POLYNOMIAL OF DEGREE  2 THAT MAXIMIZES THE LIKELIHOOD OF THE
DATA ARE :          Q( 0) = 0.0000000000D+00
                    Q( 1) = 0.0000000000D+00
                    Q( 2) = 0.3329968677D-04

     TEST OF HYPOTHESIS: Q(1) = 0

     PR(>TEST STATISTIC) =  0.5000000000D+00

     LIKELIHOOD RATIO  0.0000000000D+00

=============================================================================

CONFIDENCE LIMITS BASED ON THE MULTI-STAGE MODEL  ( KOP = 1 )

       UPPER CONFIDENCE LIMITS FOR Q(1):
            90%              95%              97.5%            99%
        0.159222D-02     0.237071D-02     0.310030D-02     0.399995D-02

CONFIDENCE LIMITS FOR A RISK OF  0.1000000D+00  M.L.E. DOSE =  0.5624952631D+02

       UPPER CONFIDENCE LIMITS ON EXTRA RISK:
            90%              95%              97.5%            99%
        0.177101D+00     0.212358D+00     0.244028D+00     0.281332D+00

       LOWER CONFIDENCE LIMITS ON SAFE DOSE:
            90%              95%              97.5%            99%
        0.372119D+02     0.309701D+02     0.264625D+02     0.222274D+02




CONFIDENCE LIMITS FOR A RISK OF  0.1000000D-01  M.L.E. DOSE =  0.1737281573D+02

       UPPER CONFIDENCE LIMITS ON EXTRA RISK:
            90%              95%              97.5%            99%
        0.370095D-01     0.499458D-01     0.619118D-01     0.764596D-01

       LOWER CONFIDENCE LIMITS ON SAFE DOSE:
            90%              95%              97.5%            99%
        0.564557D+01     0.401316D+01     0.313609D+01     0.246215D+01
```

Figure 6

```
chemical x  hepatocellular carcinomas  female rats
        TITLE :  CHEMICAL X  HEPATOCELLULAR CARCINOMAS  FEMALE RATS


?
3,1
?
20,49,49
?
2,18,37
?
0,121,242
CLASS( 1) HAS   20 MEMBERS WITH    2 RESPONSES TO DOSE OF          0.00000
CLASS( 2) HAS   49 MEMBERS WITH   18 RESPONSES TO DOSE OF        121.00000
CLASS( 3) HAS   49 MEMBERS WITH   37 RESPONSES TO DOSE OF        242.00000
? 00049
1

KOP = 1 GRADIENT MOVES = 12

THE COEFFICIENTS OF THE POLYNOMIAL OF DEGREE  2 THAT MAXIMIZES THE LIKELIHOOD OF THE
DATA ARE :          Q( 0) = 0.1053605157D+00
                    Q( 1) = 0.4476742943D-03
                    Q( 2) = 0.2037456378D-04

     TEST OF HYPOTHESIS: Q(1) = 0
     PR(>TEST STATISTIC) =  0.4216851400D+00
     LIKELIHOOD RATIO  0.3903957552D-01

================================================================================

CONFIDENCE LIMITS BASED ON THE MULTI-STAGE MODEL  ( KOP = 1 )
       UPPER CONFIDENCE LIMITS FOR Q(1):
            90%             95%             97.5%           99%
         0.3464790-02    0.4373340-02    0.5118310-02    0.5648000-02

CONFIDENCE LIMITS FOR A RISK OF  0.1000000+00  M.L.E. DOSE =  0.6175915714D+02
       UPPER CONFIDENCE LIMITS ON EXTRA RISK:
            90%             95%             97.5%           99%
         0.2530040+00    0.2937640+00    0.3255210+00    0.3471480+00
       LOWER CONFIDENCE LIMITS ON SAFE DOSE:
            90%             95%             97.5%           99%
         0.2633170+02    0.2186440+02    0.1912850+02    0.1754970+02

CONFIDENCE LIMITS FOR A RISK OF  0.1000000-01  M.L.E. DOSE =  0.1379237384D+02
       UPPER CONFIDENCE LIMITS ON EXTRA RISK:
            90%             95%             97.5%           99%
         0.5035170-01    0.6217750-01    0.7174420-01    0.7849550-01
       LOWER CONFIDENCE LIMITS ON SAFE DOSE:
            90%             95%             97.5%           99%
         0.2852850+01    0.2274000+01    0.1948490+01    0.1763790+01
```

EXAMPLE 5 -- CALCULATION OF THE BIOCONCENTRATION FACTOR

A search of the published scientific literature revealed no measured, steady-state bioconcentration studies reported for Chemical X. One published report of a measured partition coefficient for this substance was found. A standard n-octanol:water partitioning experiment was found and a Kow value of 713.6 was reported. A lipid-adjusted bioconcentration factor for Chemical X was calculated using the log BCF_c/log Kow regression equation listed in the proposed Rule 57(2) guidelines. Calculations are shown below:

$$\text{Kow} = 713.6$$
$$\log \text{Kow} = 2.85$$

$$\log BCF_c = 0.847 \log \text{Kow} - 0.628$$
$$\log BCF_c = 0.847\ (2.85) - 0.628$$
$$\log BCF_c = 1.79$$
$$BCF_c = 61.6$$

lipid adjustment for Michigan fish:

$$BCF_f = BCF_c \left(\frac{9.6}{4.8}\right)$$

$$BCF_f = (61.6) \left(\frac{9.6}{4.8}\right)$$

$$BCF_f = 123$$

REFERENCES

Aquatic Chronic Value Reference

U.S. Environmental Protection Agency. 1983. Draft Guidelines for Deriving National Water Quality Criteria for the Protection of Aquatic Life and Its Uses (July 5, 1983). U.S. Environmental Protection Agency Development Document, Environmental Research Laboratory, Duluth, Minnesota.

Due to the large number of references used in the development of the species sensitivity factors and the acute/chronic application factor, they have not been included. They are available for inspection in the Toxic Chemical Evaluation Section.

Human Health References

1. Ambrose, A. M. 1959. A toxicological study of biphenyl, a citrus fungistat. Food Research. 25:328-336.

2. Ambrose, A. M., et al. 1972. Toxicologic studies on 3',4'-dichloropropionanilide. Toxicol. and Applied Pharmacol. 23:650-659.

3. Borzella, J. F. et al. 1964. Studies on the chronic oral toxicity of monomeric ethyl acrylate and methyl methacrylate. Toxicol. and Applied Pharmacol. 6:29-36.

4. Carpenter, C. P. et al. 1961. Mammalian toxicity of 1-naphthyl-N-methylcarbamate (Sevin insecticide). Agricultural and Food Chem. 9:30-39.

5. Crouch, E. A. C. et al. 1983. The risks of drinking water. Water Resources Res. 19:1359-1375.

6. Crump, Kenny S. and Warren W. Watson. 1979. GLOBAL 79. A FORTRAN program to extrapolate dichotomous animal carcinogenicity data to low doses. National Institute of Environmental Health Sciences Contract NOI-ES-2123.

7. Deichmann, W. B. 1941. Toxicity of methyl, ethyl, and N-butylmethacrylate, J. Ind. Hyg. Toxicol. 23:343-351.

8. Deichmann, W. B. et al. 1947. Observations on the effects of diphenyl, o- and p-aminodiphenyl, o- and p-nitrodiphenyl and dihydroxyoctachlorodiphenyl upon experimental animals. J. Ind. Hyg. and Toxicol. 29:1-13.

9. Deichmann, W. B. et al. 1955. Toxicity of ditertiarybutylmethyl phenol. AMA Archives Ind. Health. 11:93-101.

10. Gaines, T. B. 1960. The acute toxicity of pesticides to rats. Toxicol. and Applied Pharmacol. 2:88-99.

11. Hodge, H. C. et al. 1951. Toxicological studies of ortho-phenylphenol. J. Pharmacol. Experiment Ther. 104:202-210.

12. Interagency Regulatory Liason Group (IRLG) 1979. Scientific Basis for Identifying Carcinogens and Estimating their Risks. A report of the Interagency Regulatory Liaison Group, Work Group on Risk Assessment. Washington, D.C.

13. Jenner, P. M. et al. 1964. Food flavorings and compounds of related structure. I. Acute oral toxicity. Fd. Cosmet. Tox. 2:327-343.

14. Kociba, R. J. et al. 1971. 1,4-Dioxane. I. Results of a two year ingestion study in rats. Toxicol. and Applied Pharmacol. 30:275-286.

15. Kociba, R. J. et al. 1978. Results of a two year chronic toxicity and oncogenicity study of 2,3,7,8-tetrachlorodibenzo-p-dioxin (TCDD) in rats. Toxicol. and Applied Pharmacol. 46:279-303.

16. Laug, E. P. et al. 1939. The toxicity of some glycols and derivatives. J. Ind. Hyg. Tox. 21:173-199.

17. Levinskas, G. J. et al. 1966. Acute and chronic toxicity of pimaricin. Toxicol. and Applied Pharmacol. 8:97-109.

18. McCollister, D. D. et al. 1959. Toxicological studies of o,o-dimethyl-o-(2,4,5-trichlorophenyl) phosphorothioate (Ronnel) in laboratory animals. Ag. and Fd. Chem. 7:689-693.

19. National Academy of Sciences. 1977. Drinking Water and Health. National Academy of Sciences, Washington, D.C.

20. Paynter, O. E. et al. 1960. Toxicology of dalpon sodium (2,2-dichloropropionic acid, sodium salt). Agric. Food Chem. 8:47-51.

21. Pitot, Henry C. and Alphonse E. Sirica. 1980. The stages of initiation and promotion in hepatocarcinogenesis. Biochemica et Biophysica Acta. 605:191-215.

22. Schwetz, B. A. et al. 1973. Toxicology of chlorinated dibenzo-p-dioxins. Env. Health Perspectives. 5:87-99.

23. Schwetz, B. A. et al. 1978. Results of two year toxicity and reproduction studies on pentachlorophenol in rats. In Pentachlorophenol. Edited by K. Ranga. Rao. Plenum Publishing Corp., NY, NY.

24. Sherman, H. and A. M. Kaplan. 1975. Toxicity studies with 5-bromo-3-sec-butyl-6-methyluracil. Toxicol. and Applied Pharmacol. 34:189-196.

25. Smyth, H. F. et al. 1970. Experimental toxicity of sodium 2-ethyl hexyl sulfate. Toxicol. and Applied Pharmicol. 17:53-59.

26. U.S. Environmental Protection Agency. 1976. National Interim Primary Drinking Water Regulations. EPA-570/9-76-003.

27. U.S. Environmental Protection Agency. 1980. Water Quality Criteria Availability. Appendix C - Guidelines and Methodology Used in the Preparation of Health Effect Assessment Chapters of the Consent Decree Water Criteria Documents. 45 Federal Register 79347-79357.

28. Verschuen, H. G. et al. 1973. Toxicity studies on tetrasul. 1. Acute, long-term, and reproduction studies. Toxicology. 1:63-78.

29. Webb, W. K. and W. H. Hansen. 1963. Chronic and subacute toxicology and pathology of methyl salicylate in dogs, rats, and rabbits. Toxicol. and Applied Pharmacol. 5:576-587.

30. Williams, C. S. F. 1976. Practical Guide to Laboratory Animals. C. V. Mosby Company.

31. Worden, A. N. et al. 1973. Toxicity of gusathion for the rat and dog. Toxicol. and Applied Pharmacol. 24:405-412.

Bioconcentration Factor References

ASTM. 1984. Proposed new standard practice for conducting bioconcentration tests with fishes and saltwater bivalve molluscs. Draft #8. January, 1984. Committee E-47.01. American Society for Testing and Materials, Philadelphia, PA. 117 pp.

Branson, D. R., G. E. Blau, H. C. Alexander, and W. B. Neely. 1975. Bioconcentration of 2,2',4,4' tetrachlorobiphenyl in rainbow trout as measured by an accelerated test. TRANS. AMER. FISH SOC. 104:785-792.

Briggs, G. G. 1981. The theoretical and experimental relationships between soil adsorption, octanol water partition coefficients, water solubilities, bioconcentration factor and the parachor. J. AGRIC. FOOD CHEM. 29(5):1050-1059.

Brown, D. 1978. U.S. EPA, Environmental Research Laboratory, Athens, GA - unpublished data.

Brungs, W. A. and D. I. Mount. 1978. Introduction to a discussion of the use of Aquatic Toxicity Tests for evaluation of the effects of toxic substances. pp. 15-26 in ESTIMATING THE HAZARD OF CHEMICAL SUBSTANCES TO AQUATIC LIFE - ASTM STP 657. J. Cairns, K. L. Dickson, and A. W. Maki (eds). American Society for Testing and Materials, Philadelphia, PA. 278 pp.

Chiou, C. T., V. H. Freed, D. W. Schmedding, and R. L. Kohnert. 1977. Partition coefficient and bioaccumulation of selected organic chemicals. ENVIRON. SCI. TECHNOL. 11(5):475-478.

Hamelink, J. L., R. C. Waybrant, and R. C. Ball. 1971. A proposal: exchange equilibria control the degree chlorinated hydrocarbons are biologically magnified in lentic environments. TRANS. AMER. FISH SOC. 100:207-214.

Hamelink, J. L., R. C. Waybrant, and P. R. Yant. 1976. Mechanisms of bioaccumulation of mercury and chlorinated hydrocarbon pesticides by fish in lentic ecosystems. In Fate of Pollutants in the Air and Water Environments. ADV. ENVIRON. SCI. TECHNOL. #9.

Hamelink, J. L. and A. Spacie. 1977. Fish and chemicals: the process of accumulation. ANN. REV. PHARMACOL. TOXICOL. 17:167-177.

Hesselberg, R. J. and J. G. Seelye. 1982. Identification of organic compounds in Great Lakes fishes by gas chromatography/mass spectrometry: 1977. ADMINISTRATIVE REPORT #82-1. Great Lakes Fishery Laboratory. U.S. Fish and Wildlife Service, Ann Arbor, Michigan 48 pp.

Johnson, D. W. 1968. Pesticides and fishes -- a review of selected literature. TRANS. AMER. FISH SOC. 97:398-424.

Johnson, D. W. 1973. Pestidice residues in fish. In Environmental Pollution by Pesticides, ed. C. A. Edwards. pp. 181-212. Plenum Press, New York. 542 pp.

Kenaga, E. E. and C. A. I. Goring. 1980. Relationship between water solubility, soil sorption, octanol-water partitioning, and concentration of chemicals in biota. pp. 78-115 in AQUATIC TOXICOLOGY. ASTM-STP 707. J. Eaton, P. R. Parrish and A. C. Hendricks (eds).

MDNR. 1984. Michigan Fishing Guide. Fisheries Division, Michigan Department of Natural Resources, Lansing, Michigan. 16 pp.

Macek, K. J. and S. Korn. 1970. Significance of the food chain in DDT accumulation by fish. J. FISH. RES. BOARD CAN. 27:1496-1498.

Macek, K. J., S. R. Petrocelli, and B. H. Sleight, III. 1979. Considerations in assessing the potential for, and significance of, biomagnification of chemical residues in aquatic food chains. pp. 251-268 in AQUATIC TOXICOLOGY, ASTM-STP 667, L. L. Marking and R. A. Kimerle (eds). American Society for Testing and Materials, Philadelphia, PA.

Mackay, D. 1982. Correlation of bioconcentration factors. ENVIRON. SCI. TECHNOL. 16(5):274-278.

Metcalf, R. L., J. R. Sanborn, P. Y. Lu, and D. Nye. 1975. Laboratory model ecosystem studies of the degradation and fate of radiolabelled tri-, tetra-, and pentachlorobiphenyl comparted with DDE. ARCH. ENVIRON. CONTAM. TOXICOL. 3(2):151-165.

Neely, W. B., D. R. Branson, and G. E. Blau. 1974. The use of the partition coefficient to measure the bioconcentration potential of organic chemicals in fish. ENVIRON. SCI. TECHNOL. 8:1113-1115.

Niimi, A. J. and B. G. Oliver. 1983. Biological half-lives of PCB congeners in whole fish and muscle of rainbow troug (Salmo gairdneri). CAN. J. FISH. AQUATIC. SCI. 40:1388-1394.

OTMC. 1982. Rule 57 Advisory Committee Report on Proposed Surface Water Quality Standard Deviation Procedures for Chemical Substances. December 14, 1982. Office of Toxic Materials Control, Michigan Department of Natural Resources. Unpublished. 83 pp.

Reinert, R. E., L. J. Stone, H. L. Bergman. 1974a. Dieldrin and DDT: Accumulation from water and food by lake trout (Salvelinus namaycush) in the laboratory. PROC. 17TH CONF. GREAT LAKES RES. pp. 52-58.

Reinert, R. E., L. J. Stone, W. A. Willford. 1974b. Effect of temperature on accumulation of methylmercuric chloride and p,p' DDT by rainbow trout (Salmo gairdneri). J. FISH. RES. BOARD CAN. 31:1649-1652.

Steen, W. C. and S. W. Karickhoff. 1981. Biosorption of hydrophilic organic pollutants by mixed microbial populations. CHEMOSPHERE (10):27-32.

Shaw, G. R. and D. W. Connell. 1984. Physicochemical properties controlling polychlorinated biphenyl (PCB) concentrations in aquatic organisms. ENVIRON. SCI. TECHNOL. 18(1):18-23.

Tulp, M. T. M. and O. Hutzinger. 1978. Some thoughts on aqueous solubilities and partition coefficients of PCB and the mathematical correlation between bioaccumulation and physico-chemical properties. CHEMOSPHERE. 10:849-860.

U. S. EPA. 1980. Water Quality Criteria Documents: Appendix B -- Guidelines for deriving water quality criteria for the protection of aquatic life and its uses. FED. REG. 45(231):79341-79347.

U. S. EPA. 1982. Draft Guidelines for Deriving National Water Quality Criteria for the Protection of Aquatic Life and Its Uses (April 29, 1982). U.S. Environmental Protection Agency Development Document. Environmental Research Laboratory, Duluth, MN.

Veith, G. D. 1981. State-of-the-art report on structure-activity methods development (II). Structure-activity research at the Environmental research Laboratory - Duluth. U.S. EPA/ERL., Duluth, MN. 61 pp.

Veith, G. D., D. L. Defoe, and B. V. Bergstedt. 1979. Measuring and estimating the bioconcentration factor of chemicals in fish. J. FISH. RES. BOARD CANADA. 36:1040-1048.

Veith, G. D., K. J. Macek, S. R. Petrocelli, and J. Carroll. 1980. An evaluation of using partition coefficients and water solubility to estimate bioconcentration factors for organic chemicals in fish. pp. 116-129 in AQUATIC TOXICOLOGY, ASTM-STP 707. J. G. Eaton, P. R. Parrish, and A. C. Hendricks (eds). American Society for Testing and Materials, Philadelphia, PA.

Veith, G. D. and P. Kosian. 1983. Estimating bioconcentration potential from octanol/water partition coefficients. Chapter 15 in PCBs In the Great Lakes. D. Mackay, R. Patterson, S. Eisenreich, and M. Simmons (eds). ANN ARBOR SCIENCE.

Willford, W. A., R. A. Bergstedt, W. H. Berlin, N. R. Foster, R. A. Hesselberg, M. J. Mac, D. R. M. Passino, R. E. Reinert, and D. V. Rottiers. 1981. Executive summary. pp. 1-7 in Chlorinated Hydrocarbons as a Factor in the Reproduction and Survival of Lake Trout (Salvelinus namaycush) in Lake Michigan. U.S. FISH AND WILDLIFE SERVICE TECHNICAL PAPER NO. 105.

GENERAL QUESTIONS AND RESPONSES

1. What are the Water Quality Standards?

   The Water Quality Standards establish water quality requirements applicable to all surface waters of the state which protect the public health and welfare; enhance and maintain the quality of water; and protect the quality of water for recreation, public water supplies, agriculture, navigation, and use by fish, other aquatic life, and wildlife. The Water Quality Standards are promulgated as rules (Part 4 of the General Rules of the Water Resources Commission) under Act 245, P.A. 1929 (Michigan's basic water quality protection act).

2. How are the Water Quality Standards used?

   The Water Quality Standards are generally used to protect the quality of the state's surface waters. One of the primary specific uses is to provide authority and direction for protection of the surface waters from discharges of industrial or municipal wastewaters which may be harmful.

3. What is the purpose of Rule 57?

   Rule 57 is the rule of the Water Quality Standards which pertains specifically to toxic substances. The purpose of this rule is to provide specific authority to manage the surface waters of Michigan in a manner which protects the public health and environment with an adequate margin of safety.

4. Why does Rule 57 need to be revised?

   The existing Rule 57 was promulgated in 1973. It needs to be revised to bring it up to date technically and to include specific authority to regulate potential human and mammalian toxicants. The U.S. EPA also requires that states revise and update their Water Quality Standards, as needed, every three years.

5. What are the alternatives to the proposed Rule 57 package?

   One alternative is not to revise the rule. Staff would then have to continue to address aquatic toxicity and human carcinogenicity in surface water permits on a case-by-case basis. This approach is currently being challenged on the basis of "ad hoc rule making". If a challenge is successful, there will be no authority to regulate toxic substances in surface water permits (with the possible exception of aquatic toxicity). Discharge of toxic substances would then take place without state mandated controls.

   A second alternative is to rely on EPA technology based standards.

However, EPA has not promulgated many standards to date. Additionally, toxic substances may cause adverse impacts at concentrations below the technology based standards.

A final alternative is to attempt to promulgate either a very general rule alone or a rule with very specific requirements (possibly to the extent of specific numerical standards for individual chemicals). These approaches have been tried in the past and have been unsuccessful. People objected that the general rule did not state how staff would implement it. People objected to the specific rule because it lacked flexibility.

6. Why are rules and guidelines proposed in the Rule 57 package?

The Rule 57 package includes Rule 57, other rules pertaining to implementation of Rule 57, and guidelines providing specifics on how staff will implement Rule 57 as it applies to surface water discharges. The other rules which are included in the package are Rules 43 and 44 which contain definitions of terms used in the rules, Rule 51 which previously inappropriately addressed drinking water standards for toxic substances other than chlorides, and Rules 82 and 90 which deal with mixing zone determinations. Rule 57, as proposed, contains a section with a general narrative statement on Water Quality Standards for all waters of the state and a section specific to deriving allowable levels of toxic substances in the waters of the state for development of point source discharge permits. The rule gives the general authority and direction to develop water quality standards for use in development of limitations in permits. The guidelines set forth specific methods which staff will use to develop recommendations to the Water Resources Commission on allowable levels for the development of permit limitations. The package was developed in this manner largely to address prior public comments that very specific Water Quality Standards do not allow for flexibility and that the very general Water Quality Standards do not describe the methods staff will use to develop specific allowable levels. DNR staff feels that this approach is a reasonable compromise which addresses both concerns.

7. Why does the proposed Rule 57 contain a section which deals specifically with deriving allowable levels of toxic substances for development of point source discharge permits?

Proposed Rule 57(1) provides a narrative statement on Water Quality Standards for all surface waters of the state. Rule 57(2) deals specifically with derivation of allowable levels of toxic substances for development of point source discharge permits. The rule was designed this way so that the concentrations of toxic substances corresponding to the design risk would only occur at the edge of the mixing zone during low flows. At all other times, the risk would be less than the design risk due to additional dilution, losses due to degradation or volatility, sorption and removal from bioavailability, and other factors. This assures that the majority

of our inland waters and the Great Lakes will be protected to an even greater degree and will not be degraded to the allowable levels utilized in development of limitations for permits.

8. Is an Environmental Impact Statement (EIS) necessary for the Rule 57 package?

An EIS would add significant delay to enactment of regulations to control toxic substances in surface waters without any obvious benefits. An EIS is not normally prepared for rules because the procedure for promulgating rules under the Michigan Administrative Procedures Act assures ample opportunity for public comment. The Water Resources Commission and the Joint Legislative Rules Committee must act affirmatively on the Rule 57 package after public comment is received. The Attorney General also reviews the rules for legality and constitutionality. Additionally, the data on discharges and environmental and public health impacts necessary to do a complete EIS are not available. Finally, the proposed Rule 57 package would provide a greater degree of protection of the environment and public health than the present rule.

9. How is the proposed Rule 57 package an improvement over existing Rule 57?

Currently there are no rules, guidelines or policies for surface waters which provide specific written direction and authority for protecting the public health and environment from the complete spectrum of possible adverse effects from toxic substances. Aquatic toxicity and human carcinogenicity are presently being addressed in surface water permits on a case-by-case basis using the general wording of Act 245 and existing Rule 57 as authority. However, this approach is currently being challenged on the basis of "ad hoc rule making". If a challenge is successful, there will be no authority to regulate toxics in surface water permits (with the possible exception of aquatic toxicity). The proposed Rule 57 package would provide both the specific authority and guidelines for staff on development of specific allowable levels. Additional aspects of potential adverse impacts on humans and terrestrial animals (wildlife) will also be evaluated and controlled.

10. Where do requirements for treatment of wastewater fit into Rule 57?

Generally, requirements for treatment of wastewater are not included as a part of the Water Quality Standards or Rule 57. Under both Federal and State law, treatment based numbers and water quality-based considerations are addressed separately. Rule 57 and the other Water Quality Standards are water quality-based, i.e., deal with development of criteria (numbers) to protect the public health and environment. The U.S. EPA is developing treatment based standards (BAT). Whenever the standards are developed, they will be considered in the development of discharge permits. Where BAT standards do not exist, DNR staff will use professional judgment to evaluate

the need for treatment based numbers. A limitation in a surface water discharge permit generally is the more restrictive of the water quality-based or treatment based numbers.

11. What is "risk assessment"?

"Risk assessment" is a process for estimating the likelihood that a toxic response could take place if people or animals were exposed to certain concentrations of a substance with hazardous properties over a given period of time.

12. Does the Michigan Constitution and state environmental law allow risk assessments?

The Michigan Constitution requires a balance between protection of our natural resources and maintenance of a viable economy. The Rule 57 package is consistent with the constitution.

The language of Act 245 requires a risk assessment in order to determine what is injurious. Risk assessments for aquatic toxicity evaluation have been carried out for several years under the existing Rule 57.

13. What are the alternatives to a risk assessment approach?

The only real alternative is zero discharge. This approach is considered unacceptable because zero cannot be measured analytically and in general it is technologically infeasible or unreasonable to achieve.

14. Why isn't the use and discharge of all toxic substances simply prohibited?

This approach is essentially the same as zero discharge. At first glance, this simple approach seems to have great merit. However, closer examination reveals serious problems which make it unworkable. It is impossible in many cases to avoid the use of a chemical in a particular manufacturing process or to remove all of the chemical from a discharge. The only alternative then is to do without the process, the chemical, and products derived from that chemical. Furthermore, the costs of treating wastes to remove chemicals increase dramatically as treatment is installed to remove lower and lower concentrations.

We have become a society dependent on chemicals. Chemicals are involved in virtually every aspect of our daily lives. Agricultural chemicals, including feed additives, growth regulators, pesticides, fertilizers, and pharmaceuticals, have played a large part in the dramatic increases in agricultural productivity achieved over the past few decades. Plastics constitute a major portion of the components used to produce consumer goods such as automobiles, household appliances, and packaging materials which have greatly

changed our lives. Pharmaceuticals (drugs and medicines) have contributed to increased longevity and the improved health of our citizens. Cosmetics, soaps, deodorants and other personal hygiene products are largely a direct result of the chemical industry. Even many of our clothes contain artificial fibers (nylon, polyester, rayon, etc.) derived from chemicals. Chemicals play perhaps an even more important, if less obvious, role in many production and manufacturing processes as raw materials, intermediates, catalysts, and solvents.

If we wish to continue to enjoy our current lifestyle, we must accept chemicals as a part of our daily lives, accept some level of risk associated with these chemicals, and expect some additional cost of living associated with improved treatment of wastes to remove chemicals. Most chemicals, when manufactured or used under the appropriate conditions, can be controlled so that they represent little risk of adverse impacts on human health or the environment. The goal of the proposed regulation is to assure that discharge of toxic substances is properly regulated and controlled.

15. How was the Rule 57 risk assessment process developed?

The proposed process was developed by DNR staff with the assistance of the Rule 57 Advisory Committee. The committee consisted of experts in the area of toxicology, chemistry, and biology from universities, industry, municipalities, environmental groups and state government. The proposed process utilizes and builds upon the work of many scientists over the past few years. The process is also similar to that proposed by the U.S. EPA for development of numbers to protect surface waters. The "Rule 57 Advisory Committee Report" is available from the DNR Toxic Chemical Evaluation Section.

16. Does the proposed Rule 57 package protect the environment and public health with an adequate margin of safety?

Staff are confident that the proposed package protects the public health and environment with an adequate margin of safety. The basis for staff's confidence is scientific judgment and a thorough review and understanding of the subject areas and specific risk assessment processes and incorporated assumptions. It is important that the risk assessment processes and assumptions be evaluated as a whole to obtain an overall feeling for the extent of the margin of safety.

17. Is the Rule 57 risk assessment process pure science?

No, the risk assessment process is a combination of science, value judgments and policy. It is impossible to separate these elements into black and white areas.

18. How does the proposed Rule 57 risk assessment process compare to programs in the U.S. EPA and other states?

The proposed package represents one of the most comprehensive

attempts by any state to address the issue of toxic substances in surface waters. Relatively few other states are even attempting to comprehensively address this issue. The comprehensive risk assessment process is an innovative approach to control the discharge of toxic substances to surface waters and utilizes the most current knowledge and information available. Many other states have expressed an interest in this approach. The approach is also similar to that proposed by the U.S. EPA to develop numbers to protect surface waters.

19. What toxic substances are covered by the proposed Rule 57 process?

An allowable level for any toxic substance could be derived under the general narrative statement in Rule 57(1). Rule 57(2) pertaining to allowable levels for development of permit limitations deals primarily with toxic substances on the 1983 Critical Materials Register and the EPA Priority Pollutants. These lists contain over 300 distinct chemicals which are of general concern to human health or the environment. However, other toxic substances could be addressed if the Water Resources Commission determines that they are of concern at a specific site.

20. What does the result of the cancer risk assessment process mean?

The cancer risk assessment process utilizes a mathematical model to estimate the upper boundary (95 percent) on risk of increased incidence of cancer over background cancer rates for a population exposed to certain concentrations of a chemical over a lifetime under an assumed set of conditions. This result is usually expressed in terms of additional cases of cancer in a given number of individuals (i.e., 1 in 100,000). The true risk to humans is unknown, but is expected to be lower than 1 in 100,000. The process is not intended to be used to actually determine the precise numbers of cancers which may develop in selected individuals under specific exposure scenarios and does not estimate deaths from cancers.

21. Why was 1 in 100,000 selected as the design risk level for the cancer risk assessment process?

The 1 in 100,000 design level of risk for carcinogens was recommended by the Rule 57 Advisory Committee and concurred with by DNR staff. The appropriateness of the 1 in 100,000 design risk level can be judged only after a thorough evaluation of the model and assumptions used in the process. The 1 in 100,000 design risk level is generally below risks incurred in everyday life (i.e., driving a car, flying in an airplane, boating). It is also extremely small in relation to the background cancer rate in the U.S. (1 in 3).

22. Is the 1 in 100,000 design risk level for cancer appropriate in all cases?

The 1 in 100,000 design risk level may not be appropriate in all

cases. The appropriate design level of risk depends, in part, on the circumstances of the situation being addressed and the risk assessment process utilized. A different level of risk could be judged appropriate in groundwater or air. Even in surface waters, special circumstances could arise which would require reassessing the appropriateness of the 1 in 100,000 level.

23. Why would it be appropriate to use different design risk levels in other programs?

The 1 in 100,000 design risk level incorporated in Rule 57 was developed specifically for surface waters based on the incorporated factors and assumptions, current knowledge of the aquatic environment, and the statutory and policy bases of the surface water regulatory program. Design risk levels for other programs must be similarly developed. For example, it would be easy to argue that the design risk level for groundwater discharges should be less than the 1 in 100,000 level proposed for surface waters. Differences between surface water and groundwater could lead to a lower level of confidence that the risk assessment process for surface water would not provide an adequate margin of safety to protect the public health if applied to groundwater. Some of the differences which could lead to this conclusion include: 1) groundwater movement is more difficult to predict and determine than surface water movement; 2) groundwater monitoring is more difficult than surface water monitoring; 3) groundwater is the sole source of drinking water for a major portion of Michigan's people; 4) people commonly consume groundwater with no treatment; 5) dilution, degradation and removal of toxic substances in groundwater is often minimal; and 6) cleanup of groundwater is often more difficult, lengthy and costly.

24. Why is exposure for cancer risk assessment based on a population, average consumption of fish, and "average adult" basis?

The decision to use lifetime exposure of an average adult human weighing 70 kilograms in the cancer risk assessment process was based on the recommendation of the Rule 57 Advisory Committee. This approach has also been widely accepted in the scientific community for dealing with diversified populations of people and is used by the U.S. Environmental Protection Agency in the development of their water quality criteria documents under the Clean Water Act. Additionally, many of the animal studies used as a basis for the risk assessment are based on "lifetime" exposure. The 0.0065 kilograms of fish per day is also an EPA number based on average national fish consumption, including marine and shellfish. An estimate of consumption of inland fish by Michigan anglers was calculated at about 0.0040 kilograms per day. Staff decided to use the larger EPA number because it was calculated on better data and was slightly more conservative.

25. What significant impacts are expected on aquatic and terrestrial life when minimum data are used in the proposed process to calculate allowable levels to protect these organisms?

Staff would expect a very small percentage of aquatic or terrestrial organisms, if any, to experience significant adverse chronic effects when exposed to a concentration of toxicant in water corresponding to an aquatic or terrestrial value derived from minimum data. For the aquatic value, this is based on staff's belief that the acute/chronic ratio of 45 is a conservative application factor for the vast majority of industrial chemicals in common use that are not adequately characterized to allow use of the modified EPA method of calculating an aquatic chronic value. For the terrestrial value, this is based on inclusion of two separate uncertainty factors; a ten-fold uncertainty factor to account for the interspecies variability in response to a toxicant and a 10,000-fold factor to account for the uncertainty associated with using acute data to calculate criteria to protect chronic effects. Hypothetically, a worst-case scenario could be envisioned where these numbers might not provide adequate protection. However, the only concentration which would guarantee protection for 100 percent of all species, 100 percent of time is zero. In addition, any limit developed from the calculated values would be expected to occur infrequently after mixing with the receiving stream because the allowable level is used to back-calculate the discharge limit which is an "end-of-the-pipe number". Generally, this back-calculation assumes a dilution factor of 25 percent of the 95 percent exceedance flows, which is a conservative assumption in and of itself.

26. Does the proposed Rule 57 package protect all components of the environment?

No, the proposed Rule 57 process would directly protect only certain components of the total ecosystem such as people, terrestrial mammals and birds, fish and aquatic macroinvertebrates. Other components, such as bacteria and plants, would not be directly protected. Staff is unable to develop procedures at this time to protect these components because of a lack of previously developed procedures, standardized testing methods, and usable data. However, these components would receive an unknown degree of protection from the values calculated to protect the components covered in the Rule 57 process. Staff will evaluate new methods and procedures as they become available and suggest revisions to the Rule 57 package as appropriate.

27. What is a mixing zone?

A mixing zone is an area of a body of water which a discharger is allowed to use for mixing his wastes prior to meeting Water Quality Standards. Water Quality Standards, in general, do not have to be met within a mixing zone. Generally, for toxic substances, dischargers are allowed 25 percent of the design drought flow in a river for mixing with their wastewater. The mixing zone is protected against acute toxicity to aquatic organisms.

28. Why does the proposed Rule 57 package use the 95 percent exceedance flow as the design flow?

The Q10-7 was previously used as the design flow. The Q10-7 is the low flow which occurs for seven days in every 10 years. The 95 percent exceedance flow is the flow which is exceeded in the river 95 percent of the time.

The primary reasons for recommending use of the 95 percent exceedance flows are administrative ease, simplicity and consistency. The option to use seasonal flows (particularly for conventional parameters) requires the use of an approach other than the Q10-7. Water Management Division recommended the use of the 95 percent exceedance flows because they closely approximate the Q10-7 and could be calculated on a monthly or seasonal basis. Staff feels that for administrative ease, simplicity and consistency, the 95 percent exceedance flow should be used across the board. Staff does not feel that this approach significantly reduces the margin of safety.

29. Why does the proposed Rule 57 package allow issuance of "non-conforming use permits"?

The "nonconforming use permit" section of Rule 57 was designed to provide an alternative to putting a facility out of business if they could not immediately meet the Water Quality Standards. The Water Resources Commission would have to determine, based on a demonstration by the applicant, that immediate attainment of the Water Quality Standard is not economically or technically feasible; no prudent alternative exists; the permitted discharge would be consistent with the protection of public health, safety and welfare; and that reasonable progress would be made toward achieving compliance during the life of the permit.

30. Does the proposed Rule 57 package account for interactions between chemicals?

Interactions between toxic chemicals would be considered only where data is available suggesting an interaction. Since there is very little data on this subject, these interactions would be considered only infrequently. Generally, toxicologists do not have a clear idea on how important interactions might be or how to regulate for interactions in the absence of specific data. Staff is confident that the margin of safety in the process is adequate to protect for possible interactions. For aquatic toxicity, the proposed package does allow for use of whole effluent testing or other biological technique, which would address chemical interactions.

NATURAL RESOURCES COMMISSION
THOMAS J ANDERSON
MARLENE J FLUHARTY
KERRY KAMMER
O STEWART MYERS
DAVID O OLSON
RAYMOND POUPORE

STATE OF MICHIGAN

APPENDIX D

JAMES J. BLANCHARD, Governor
DEPARTMENT OF NATURAL RESOURCES
STEVENS T. MASON BUILDING
P.O. BOX 30028
LANSING, MI 48909

GORDON E. GUYER, Director

January 27, 1987

TO: All Interested Parties

FROM: Paul D. Zugger, Chief
Surface Water Quality Division

SUBJECT: Rule 57(2) Guideline Levels

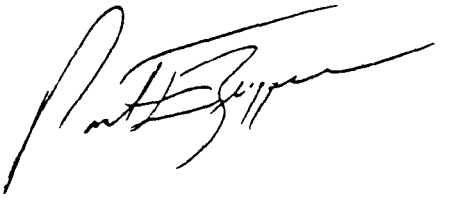
The Rule 57(2) Guidelines state that the most recent calculations of water quality-based levels of toxic substances developed pursuant to the Guidelines shall be compiled on an annual basis and be available for distribution by February 1 of each year. The following list is in fulfillment of that requirement, and is complete as of January 27, 1987. The values are subject to change as new data or information becomes available.

Rule 57(2) Guideline Levels are utilized in making water quality-based permit recommendations to the Water Resources Commission concerning toxic substances in the surface water after a point source discharge is mixed with the receiving stream volume specified in R323.1082. These levels do not represent acceptable ambient levels in all waters of the state, nor do they represent or reflect necessary treatment-based considerations.

This list is informational only and is not a mechanism to establish water quality-based permit limits. It is advisory in nature and not meant to be binding on anyone.

Water quality-based permit limitations for toxic chemicals are developed pursuant to existing procedures by staff in the Great Lakes and Environmental Assessment Section using the R323.1057(2) Guidelines and appropriate scientific data.

Questions concerning this list should be directed to Linn Duling, of the Great Lakes and Environmental Assessment Section at 517/335-4188.



R1026
1/86

CHEMICAL NAME	CAS NUMBER	Rule 57(2) Level Non-Drinking Water Value (ug/l)	Basis
Arsenic	Class 011	150	ACV
Cadmium	Class 013	@exp(0.83(@ln(H))-4.84)	ACV
Chromium	Class 015	@exp(0.83(@ln(H))+0.131)	ACV
Copper	Class 017	@exp(0.94(@ln(H))-1.3)	ACV
Cyanide	Class 018	5	ACV
Lead	Class 019	@exp(1.53(@ln(H))-5.92)	ACV
Nickel	Class 022	@exp(0.92(@ln(H))+0.12)	ACV
Selenium	Class 023	13	ACV
Silver	Class 024	0.15	ACV
Zinc	Class 027	@exp(0.85(@ln(H))+0.87)	ACV
PCB #	Class 079	0.000012	CRV
DDT #	50293	0.00013	CRV
Carbon tetrachloride #	56235	27	CRV
Phenol, 4-chloro-3-methyl	59507	4.4	ACV
Aniline #	62533	0.4	ACV
Acetone	67641	500	TLSC
Chloroform #	67663	43	CRV
Hexachloroethane #	67721	13	CRV
Benzene #	71432	51	TLSC
Ethane, 1,1,1-trichloro	71556	120	ACV
Methylene chloride #	75092	430	ACV
Ethylene oxide #	75218	56	CRV
Ethylene, 1,1-dichloro #	75354	3	CRV
Hexachlorocyclopentadiene	77474	0.5	ACV
Propane, 1,2-dichloro	78875	160	TLSC
Trichloroethylene #	79016	94	ACV
Pentachlorophenol	87865	@exp(1.0051*pH-3.8617)/4.8	ACV
2,4,6-Trichlorophenol #	88062	1.5	CRV
Dinoseb	88857	@exp(1.5837*pH-8.8767)/55.5	ACV
Naphthalene	91203	29	ACV
Benzidine, 3,3-dichloro #	91941	0.04	CRV*
Benzidine #	92875	0.0051	CRV*
Silvex	93721	3	HLSC
Benzene, 1,2-dichloro	95501	7	ACV
Phenol, 2-chloro	95578	10	ACV
Ethylbenzene	100414	82	ACV
Styrene #	100425	19	CRV
Benzene, 1,4-dichloro	106467	43	ACV
Phenol, 4-chloro	106489	9.3	ACV
Ethane, 1,2-dibromo #	106934	1.2	CRV*
Acrolein	107028	3	ACV
Ethane, 1,2-dichloro #	107062	560	CRV
Acrylonitrile #	107131	2.2	CRV*
Toluene	108883	100	ACV
Chlorobenzene	108907	71	ACV
Phenol	108952	230	HLSC
Bis(2-chloroethoxy) methane	111911	4.6	TLSC

27-Jan-87

CHEMICAL NAME	CAS NUMBER	Rule 57(2) Level Non-Drinking Water Value (ug/l)	Basis
Hexachlorobenzene #	118741	0.0019	CRV*
Benzene, 1,2,4-trichloro	120821	22	HLSC
Phenol, 2,4-dichloro	120832	@exp(0.3589*pH+3.395)/13.95	ACV
1,4-dioxane #	123911	360	ACV
Tetrachloroethylene #	127184	20	CRV
Ethylene, t-1,2-dichloro	156605	90	TLSC*
Benzene, 1,3-dichloro	541731	20	HLSC
Xylene	1330207	40	ACV
Di-N-propyl formamide	6282004	63	TLSC
Mercury, methyl	7439976	0.0006	HLSC
Ammonia (Coldwater)	7664417	20	ACV
Ammonia (Warmwater)	7664417	50	ACV
Chlorine	7782505	6	ACV
Chromium, hexavalent	18540299	6	ACV

NOTES:

- This chemical is regulated as a carcinogen. The Rule 57(2) Level is not necessarily based on its 1 in 100,000 cancer risk value.
* - Professional judgement was used - minimum data not available.

ACV- Aquatic Chronic Value
TLSC- Terrestrial Life-cycle Safe Concentration
HLSC- Human Life-cycle Safe Concentration
CRV- Cancer Risk Value
CAS = Chemical Abstract Service Number

Exponential equations: e.g., @exp(0.83(@ln(H))-4.84) = $e^{0.83(\ln H)-4.84}$

where H = Hardness (mg/l)

$$@exp(1.0051*pH-3.6617)/4.6 = \frac{e^{1.0051(pH)-3.6617}}{4.6}$$

where pH is in Standard Units

SUMMARY OF CHEMICAL AND BIOLOGICAL PHOSPHORUS REMOVAL EXPERIENCE IN NORTH AMERICA

by

Richard C. Brenner
Environmental Engineer
Wastewater Research Division
Water Engineering Research Laboratory
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

and

Denis J. Lussier
Chief, Environmental Control Systems Staff
Center for Environmental Research Information
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

Prepared for Presentation at:

Eleventh United States/Japan Conference
on Sewage Treatment Technology
Tokyo, Japan

October 12-14, 1987

SUMMARY OF CHEMICAL AND BIOLOGICAL PHOSPHORUS REMOVAL EXPERIENCE IN NORTH AMERICA

Richard C. Brenner and Denis J. Lussier

INTRODUCTION

Discharge of nutrients to receiving waters has been a major water pollution concern in North America since the late 1960's and early 1970's. Eutrophication and oxygen depletion of critical water bodies, particularly the Great Lakes and numerous inland lakes and estuarine waters in Florida, stimulated intensive private and public R&D efforts directed at nutrient control throughout most of the past decade. These efforts emphasized phosphorus removal, nitrification, and, where necessary, nitrogen removal.

During the early years of this period, implementation of phosphorus removal technology focused on mainstream chemical precipitation of soluble phosphorus in primary and biological treatment systems, primarily with metallic salts of aluminum or iron. Mineral (metallic salt) addition has proven to be a reliable phosphorus removal process, but it possesses the inherent disincentives of significant chemical costs and substantially increased sludge production. Consequently, most recent R&D programs have directed their attention to activated sludge processes that accomplish enhanced phosphorus removal biologically with minimal increases in waste sludge quantities. These processes may require supplemental mineral addition for effluent polishing to reach very low effluent phosphorus concentrations, but only at a fraction of the chemical cost of a conventional chemical-biological approach. One activated sludge phosphorus removal process utilizes sidestream addition of lime to an elutriant stream from a portion of the return sludge flow to precipitate high concentrations of phosphorus removed biologically from the mainstream flow, again at a fraction of the chemical cost of conventional mineral addition. Enhanced or "luxury" uptake biological phosphorus removal processes will be referred to hereinafter simply as biological phosphorus removal processes. Phosphorus removal that is accomplished via mainstream addition of mineral salts to treatment trains employing biological processes will be referred to hereinafter as chemical phosphorus removal.

Richard C. Brenner is an Environmental Engineer with the Wastewater Research Division, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency, Cincinnati, Ohio.

Denis J. Lussier is Chief, Environmental Control Systems Staff, Center for Environmental Research Information, U.S. Environmental Protection Agency, Cincinnati, Ohio.

Nitrogen removal has been achieved to date principally through the process of denitrification. Initial development centered on staged systems using externally added carbon sources, such as methanol, to force the denitrification reaction. This approach provides effective nitrogen removal down to very low effluent nitrogen concentrations, but at a considerable chemical cost and expanded facility size. Recently, interest in North America has shifted to managed single-stage biological processes that utilize organic carbon contained in raw wastewater or primary effluent to trigger denitrification. These processes require less overall treatment volume and save chemical costs, but generally cannot achieve as efficient nitrogen removal as the staged approach.

The need to control phosphorus and nitrogen concurrently is being encountered more frequently today in North America than in the past. These requirements may take the form of combined phosphorus removal-nitrification or combined phosphorus removal-nitrogen removal. Proven process options available for accomplishing dual nutrient control range from combining conventional mineral addition and conventional staged biological systems, wherein the control of phosphorus and nitrogen is essentially independent of one another, to combining biological phosphorus removal with nitrification and substrate-induced denitrification, if needed, in managed single-stage systems, wherein phosphorus and nitrogen control are inextricably interrelated.

Although the level of North American research activity in phosphorus removal in recent years is significantly less than at its zenith in the 1970's, important pilot- and field-scale studies are still being carried out both in the United States and Canada. For the most part, these studies involve biological phosphorus removal. In addition, an increasingly large data base is becoming available on chemical phosphorus removal as a result of the extensive implementation of that technology over the last 15 years.

The objectives of this paper are to summarize the substantial existing data base on chemical phosphorus removal available in North America and to describe emerging results on biological phosphorus removal. Comparisons of the costs and relative advantages and disadvantages of the two approaches will be offered insofar as generally accepted at this point in time. Nitrogen control will be addressed only as it affects phosphorus removal in dual nutrient control schemes.

DATA SOURCES

Two editions of a Process Design Manual for Phosphorus Removal were published by U.S. EPA Technology Transfer in the 1970's (1,2). A third edition entitled Design Manual - Phosphorus Removal (3) has recently been published (September 1987) by the U.S. EPA Center for Environmental Research Information (CERI) and Water Engineering Research Laboratory (WERL). Initial distribution of this third manual was made at the Water Pollution Control Federation (WPCF) Conference in Philadelphia, Pennsylvania, in October 1987.

A companion document to the third edition of the above Design Manual entitled Handbook - Retrofitting POTW's for Phosphorus Removal in the Chesa-

peake Bay Drainage Basin (4) was also copublished by WERL and CERI in September 1987. It, too, was first distributed at the Philadelphia WPCF Conference in October 1987. The intent of this Handbook is to utilize the extensive experience developed in other parts of the country as a guide for Chesapeake Bay Drainage Basin (CBDB) plant designers, operators, owners, and regulators to implement an efficient and cost-effective program of phosphorus removal. The contributing states discharging to the CBDB are parts of Maryland, Pennsylvania, and Virginia, as well as the District of Columbia.

Environment Canada published a comprehensive report in October 1986 that evaluated the technical and economic feasibility of modifying (retrofitting) existing municipal wastewater treatment plants in Canada for biological phosphorus removal (5). This report examined retrofitting of three types of existing treatment plants found in abundance in the province of Ontario: primary plants, conventional activated sludge plants, and extended aeration plants. Costs of retrofitting these three process configurations with biological phosphorus removal were compared with the costs of retrofitting the same configurations with chemical phosphorus removal.

The last three references cited above (3,4,5) comprise the most recent and thorough compilation of phosphorus removal experience in North American. Information from these documents was used extensively in preparing this summary paper. Figures used herein were taken directly from the three documents. In some instances, tables of data were also reproduced.

MATRIX OF ADVANCED AND SUPPLEMENTARY SYSTEMS USED IN U.S. MUNICIPAL TREATMENT PLANTS

Advanced and/or supplementary treatment has been or is being incorporated in many municipal treatment facilities in the United States. As indicated in Table 1, over 4000 advanced or supplementary treatment systems were in operation or in the design/construction phases in 1982 (6). Some plants incorporated two or more of these advanced or supplementary systems, so the number of actual treatment plants represented is not known.

The types of advanced and supplementary processes and/or equipment selected are being utilized for improved soluble organic carbon removal, improved suspended solids removal, phosphorus removal, nitrogen control, disinfection, and land treatment, among other uses. As shown, the number of phosphorus removal systems in use or being designed/constructed in 1982 totalled 897, or 21.9 percent of the advanced/supplementary system total. The corresponding figures for nitrogen control/removal were 1550 systems and 41.3 percent of the total.

Over the past 5 years, the number of facilities employing or planning to implement nutrient control in the United States has risen significantly beyond the figures presented in Table 1. An exact breakdown of this increase was not available to the author at the time of this writing, but it is known to include over 400 treatment plants in the CBDB that will be required to remove phosphorus in the near future (4). Currently, approximately 100 plants are already removing phosphorus in the CBDB (4).

TABLE 1. MATRIX OF ADVANCED AND SUPPLEMENTARY MUNICIPAL TREATMENT SYSTEMS IN UNITED STATES IN 1982 (6)

System Description	No. of Operating Systems	No. of Systems in Design or Construction Phases	Total No. of Systems	Percent of Total
Soluble Organic Carbon Removal	19	2	21	0.5
Activated carbon-granular	5	0	5	0.1
Activated carbon-powdered	24	2	26	0.6
Subtotal				
Suspended Solids Removal				
Microstrainers-primary	13	6	19	0.5
Microstrainers-secondary	78	13	91	2.2
Sand filters	954	283	1237	30.1
Mixed media filters	242	73	315	7.7
Misc. filters	26	6	32	0.8
Subtotal	1313	381	1694	41.3
Phosphorus Removal				
Aluminum addition-primary	45	15	60	1.5
Aluminum addition-secondary	310	86	396	9.6
Aluminum addition-tertiary	69	12	81	2.0
Iron addition-primary	30	1	31	0.7
Iron addition-secondary	98	19	117	2.9
Iron addition-tertiary	24	2	26	0.6
*Single-stage lime-raw	16	0	16	0.4
*Single-stage lime-tertiary	55	13	68	1.7
*Two-stage lime-raw	9	3	12	0.3
*Two-stage lime-tertiary	19	3	22	0.5
Misc. chemical addition	58	10	68	1.7
Subtotal	733	164	897	21.9
Nitrogen Control/Removal				
Nitrification-separate	247	101	348	8.5
Nitrificaton-combined	559	230	789	19.2
Denitrification	51	6	57	1.4
Ammonia stripping	8	0	8	0.2
Subtotal	865	337	1202	29.3
Disinfection				
Breakpoint chlorination	11	2	13	0.3
Land Treatment				
Primary	1	2	3	<0.1
Secondary	84	13	97	2.4
Intermediate	5	2	7	0.2
Subtotal	90	17	107	2.6
Miscellaneous				
Recarbonization	38	7	45	1.1
Recalcination	9	2	11	0.3
Ion exchange	1	0	1	<<0.1
Neutralization	12	8	20	0.5
Others	61	24	85	2.1
Subtotal	121	41	162	4.0
Grand Total	3157	944	4101	100.0

*These systems may have been installed to achieve improved suspended solids as well as phosphorus removal.

CHEMICAL PHOSPHORUS REMOVAL

Chemistry

Because of the large sludge quantities produced and the considerable operation and maintenance (O&M) problems encountered in its handling, lime addition is no longer recommended by North American engineers for mainstream chemical phosphorus removal, except under special circumstances such as exceedingly low effluent total phosphorus (TP) limitations ($\leq$0.1 mg/L). The addition of metallic salts of aluminum and iron has become the backbone of North American phosphorus removal technology.

Insolubilization of phosphorus with metallic ions is most effectively accomplished when phosphorus is in the orthophosphate form. A large fraction of the phosphorus present in raw wastewater is in the form of polyphosphates. Hydrolysis of polyphosphates to orthophosphates is efficiently accomplished in wastewater treatment plants only after the flow reaches the biological section of the treatment process train. Consequently, addition of metallic salts before or during primary treatment is less efficient than if the salts are added to the secondary process. Once conversion to orthophosphorus is complete, insolubilization of phosphorus with metallic cations (M^{+3}) can be expressed as follows:

$$M^{3+} + PO_4^{3-} \longrightarrow MPO_4 \downarrow$$

Retrofit Considerations

Injection sites for metallic salts in municipal treatment plants can include raw wastewater, the primary clarifier, primary effluent, the biological reactor, biological reactor effluent, the secondary clarifier, secondary effluent, and a tertiary reactor, if provided. As discussed above, metal ion addition to the primary clarifier is less efficient than addition to the secondary system. However, if metals are to be added in the primary portion of the plant, it is recommended that injection to the primary clarifier influent be utilized rather than the primary clarifier itself. Prior addition allows for better mixing before entering the clarifier. This will maximize the capture of organics and suspended solids in the primary clarifier, thereby increasing the quantity of primary sludge available for methane production if anaerobic sludge digestion is practiced and reducing the organic load on the secondary process to the maximum degree possible.

Because of the varying effluent TP limits imposed in some parts of North America, generally ranging from a high of 2.0 mg/L to a low of 0.2 mg/L in some watersheds, a consensus chemical dosing scheme has not emerged. Many plants today are being provided with multiple injection point capability. This flexibility becomes increasingly important the lower the effluent TP limit. Some engineers are choosing to provide multiple dosing site capability even where the current effluent TP limitation is 2.0 mg/L if they suspect that lower limits may be imposed during the design life of the dosing equipment.

Many plants in North America have found that the addition of polymer in conjunction with mineral salts improves overall suspended solids removals and enhances the probability of meeting effluent TP requirements. Efficient suspended solids control becomes more critical the lower the effluent TP limit. For example, a typical suspended solids phosphorus content is 4.5 percent where chemical phosphorus removal is employed. If the effluent soluble phosphorus (SP) concentration is 0.2 mg/L, effluent total suspended solids (TSS) could not exceed 18 mg/L to meet a 1.0-mg/L TP effluent limit. To meet a 0.5-mg/L TP effluent limit, effluent TSS could not exceed 7 mg/L under the same circumstances.

Anionic polymers are generally more effective than cationic or nonionic polymers in combination with metallic salts. Specific types and concentrations of polymers for best results should be determined on site with the aid of jar testing.

Polymers are normally added at each site where metal salts are injected. For maximum effectiveness, the insolubilization reaction between metallic salt and orthophosphate must be completed before polymer is added. A time lag of 1 to 5 minutes is usually sufficient. Many plant operators find it convenient to ensure the appropriate time lag by adding the mineral salt to clarifier influent flows and the polymer to the center feed wells of circular clarifiers.

Rapid mixing is necessary at both the mineral salt and polymer injection points to provide intimate contact between reacting molecules, prevent shortcircuiting of unreacted chemical, and overcome the stratification tendencies of the more dense and viscous chemical solutions. Supplemental in-line mixers may have to be added if existing turbulence levels are not adequate to ensure satisfactory rapid mix conditions. Following rapid mixing, a period of gentle mixing is required to promote floc agglomeration and growth. Clarifier center wells are often used for this purpose. Liquid/solids separation zones in clarifiers, of course, provide the quiescent conditions essential for floc settling and removal. An idealized sequence of the chemical and physical steps involved in efficient chemical precipitation of phosphorus is shown in Figure 1.

The quantity of additional sludge produced from chemical addition will vary depending on the chemical additive employed, the amount of chemical required to reach a desired effluent TP concentration, and the point or points of chemical dosing. Reference 4 cites an Ontario, Canada survey of 185 plants that documented an average increase in sludge mass of 40 percent to reach an effluent TP of 1.0 mg/L with metal salt addition at primary treatment facilities and an increase of 26 percent to reach the same effluent TP concentration with metal salt addition at activated sludge plants. These data, summarized in Table 2, depict an increase in the quantity of dry solids produced of 49 kg/ 1000 m^3 (410 lb/Mgal) for the primary plants vs. 44 kg/1000 m^3 (367 lb/Mgal) for the activated sludge plants. The percent increase in the volume of sludge produced was approximately the same for both types of plants, although mineral addition resulted in a more pronounced percentage reduction in dry solids content of the sludges produced at the primary plants than at the activated sludge plants.

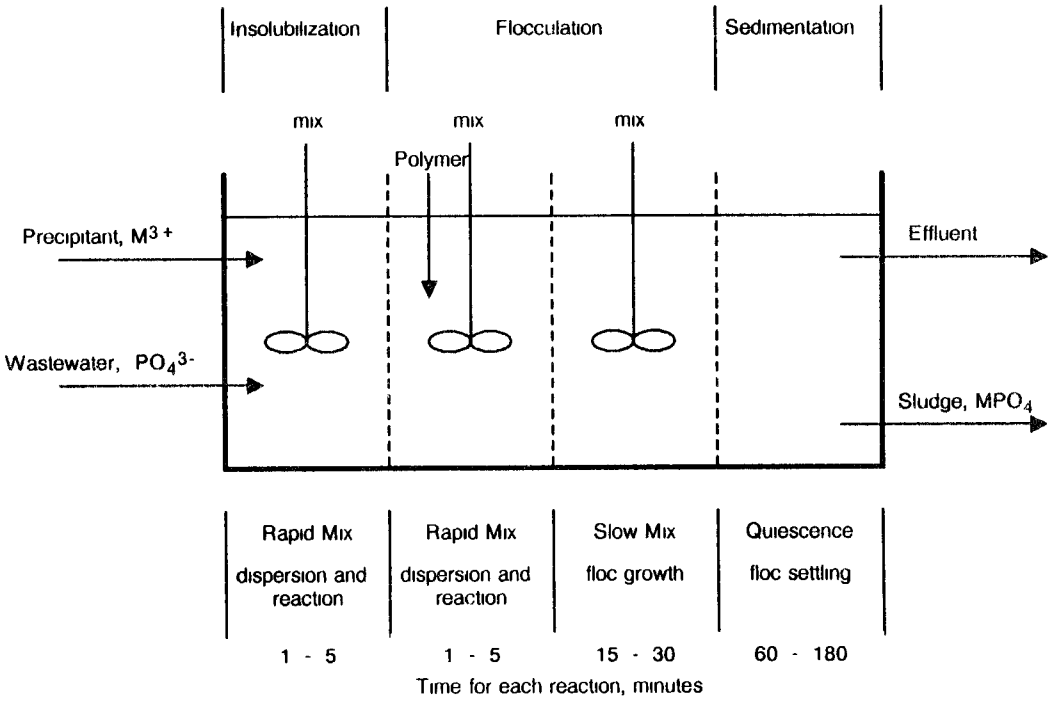


Figure 1. Idealized Sequence of Reactions Involved in Chemical Phosphorus Removal (4).

TABLE 2. SLUDGE PRODUCTION WITH AND WITHOUT MINERAL ADDITION AT TREATMENT PLANTS IN ONTARIO, CANADA (4)

	Type of Treatment			
	Primary		Activated Sludge[1]	
	Conventional	Metal Salt Addition[2]	Conventional	Metal Salt Addition[2,3]
Dry Solids Produced, kg/1,000 m^3	120	169	173	217
Dry Solids Produced, lb/Mgal	1000	1410	1443	1810
Dry Solids Content, percent	6.0	5.3	4.5	4.2
Sludge Volume, percent of influent flow	0.20	0.32	0.38	0.51

[1]Primary plus waste activated sludges.
[2]Plant influent TP = 7 mg/L; final effluent TP = 1 mg/L.
[3]Metal salt added to aerator.

A survey of 174 plants conducted by U.S. EPA (3) found similar increases in sludge production resulting from mineral addition to those observed in the Canadian survey. As shown in Figure 2, the increase in total plant sludge mass at secondary plants where the mineral was added to the primary clarifier was 100 kg/1000 m^3 (834 lb/Mgal) for iron salts and 50 kg/1000 m^3 (417 lb/Mgal) for aluminum salts. For secondary plants where the mineral was added to the secondary system, the increase was 40 kg/1000 m^3 (334 lb/Mgal) for both types of salts. A wide range of influent and effluent characteristics and treatment plant process schemes are represented in these data.

Figure 2 also illustrates the substantially greater quantities of sludge production that can be expected from lime addition contrasted to mineral salt addition. The average increases in sludge volume before thickening resulting from chemical addition for the plants in the above U.S. survey (3) averaged 25 percent for iron, 58 percent for aluminum, and several hundred percent for lime.

The sludge production data presented above for both the Canadian and U.S. surveys are average values for large numbers of plants. Contributing to those average values were plants exhibiting wide ranges of sludge production. Perhaps the single most significant factor impacting metal salt dose and the resulting sludge generation rate is the required effluent TP limitation. As illustrated in Figure 3, the sludge generation rate (mass total plant sludge produced/unit mass of TSS in raw wastewater) to achieve an effluent TP concentration of 0.2 mg/L is two to three times that required to reach a 0.7- to 1.0-mg/L effluent TP (4). The considerable effect of the metal ion dose-to-influent TP (or TP removed) weight ratio, which is dictated primarily by the effluent TP requirement, on the sludge generation rate is shown in Figure 4 (4). It should be noted, however, that the high sludge generation rates for Little Hunting Creek, Piscataway, and Dale City were affected not only by the low effluent TP limit of 0.2 mg/L for all three plants but also the addition of lime for non-phosphorus removal purposes. At Little Hunting Creek and Piscataway, lime was used for sludge stabilization, while lime was utilized for alkalinity adjustment at Dale City.

Process Descriptions

A variety of municipal treatment systems have been retrofitted for chemical phosphorus removal in North America, including plug flow, complete mix, step aeration, and contact stabilization air activated sludge; extended aeration and oxidation ditch air activated sludge; single-stage nitrification air activated sludge; pure oxygen activated sludge; standard- and high-rate trickling filters; rotating biological contactors (RBC's); lagoons; and different types of two-stage alternatives. Recommended flow diagrams for these process options, as tailored to existing plants in the CBDB, are reproduced directly from Reference 5 in Figures 5 through 12.

The above figures depict recommended dosing points for metal salt and polymer. Suggested ranges of chemical doses as a function of final effluent TP are also given. Four effluent TP limits (0.2, 0.5, 1.0, and 2.0 mg/L) are considered. Multiple chemical injection point capability is recommended for all flow schemes except extended aeration plants and oxidation ditches. The

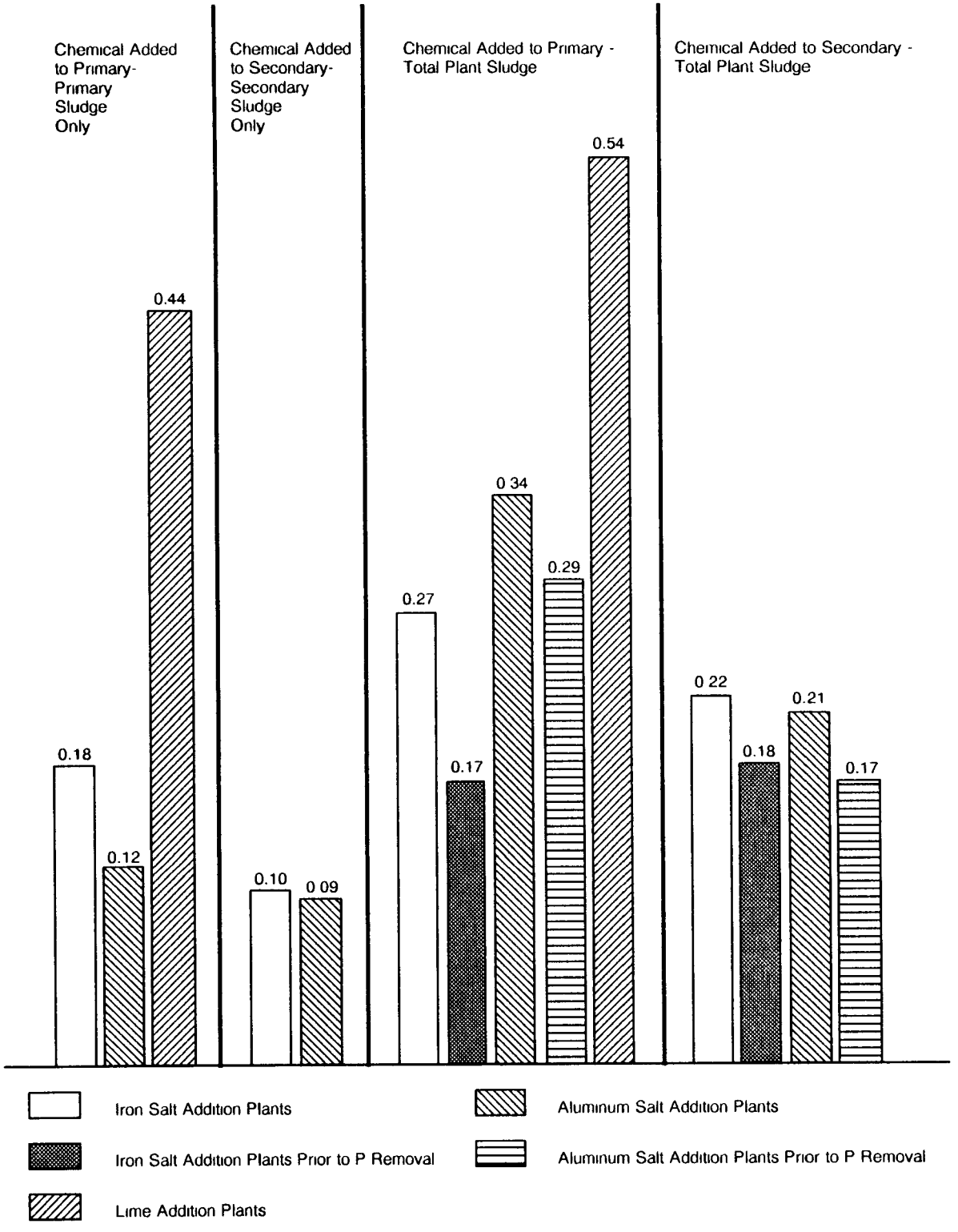


Figure 2. Mass Sludge Production Resulting from Chemical Addition Based on U.S.EPA Survey - [kg sludge produced/m^3 plant influent] (3).

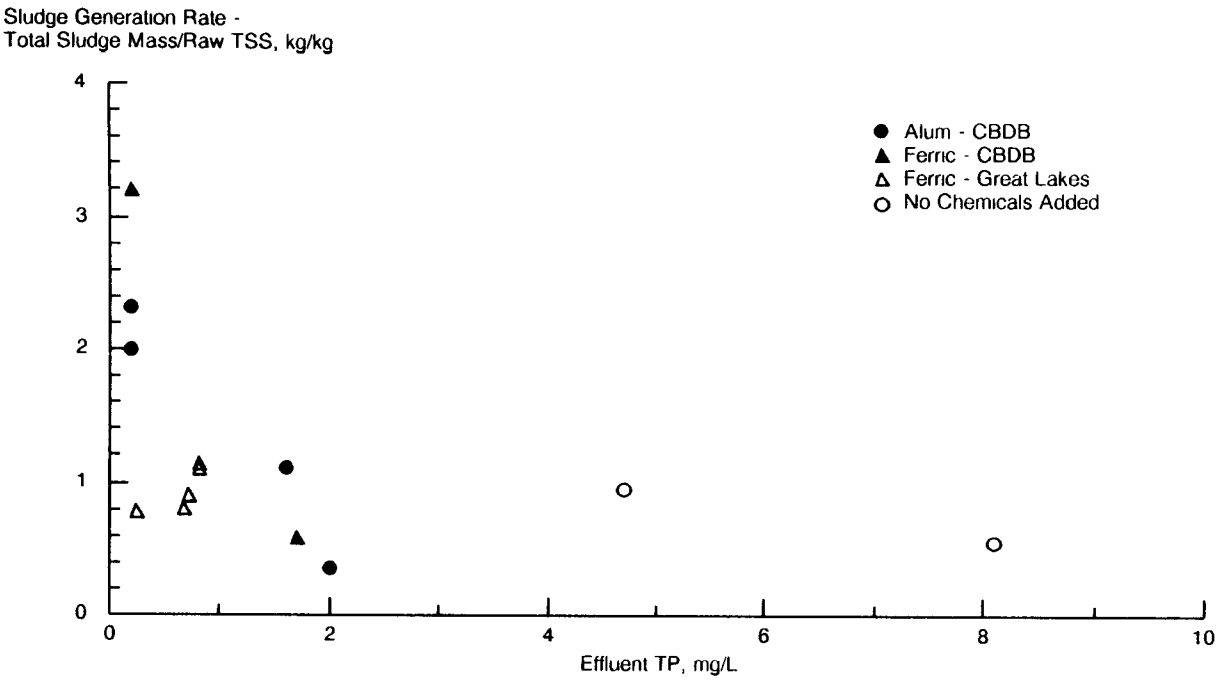


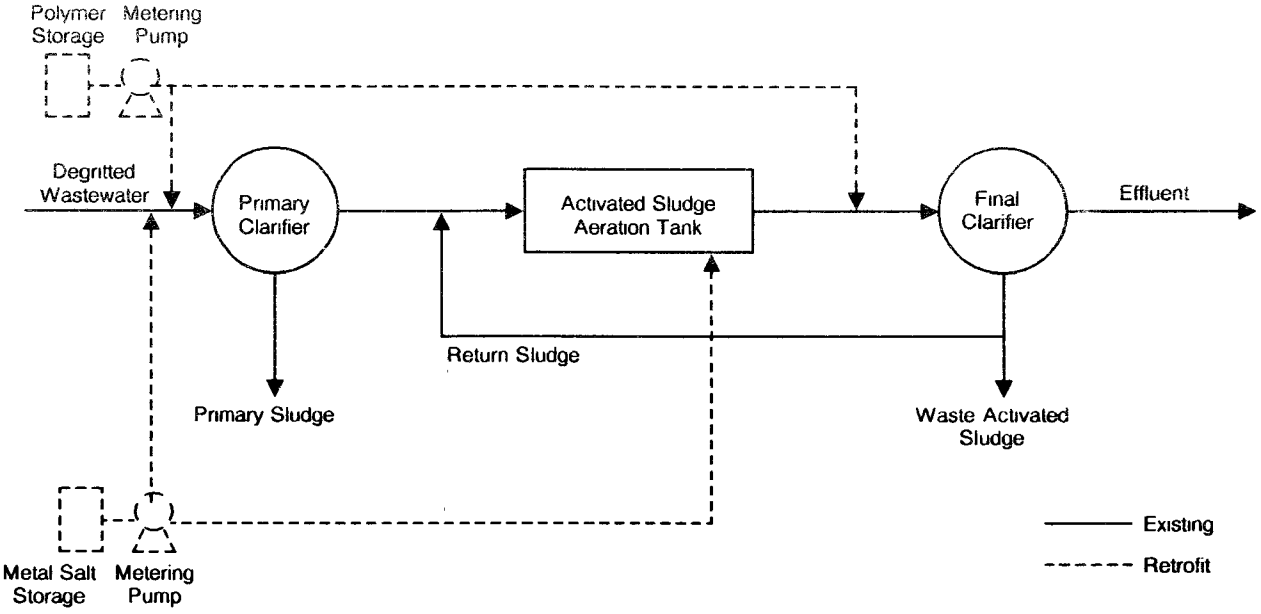
Figure 3. Sludge Generation Rate for Chemical Phosphorus Removal as a Function of Effluent TP (4).

need to utilize multiple chemical addition sites increases, as stated before, the more stringent the effluent TP limit. Also shown on these figures are suggested final clarifier surface overflow rates (SOR's) for the various effluent TP limits as well as recommendations on whether final effluent filtration would be required to reach the respective limits.

Well operated plug flow and complete mix activated sludge systems should offer no problems in retrofitting to chemical phosphorus removal other than locating the optimum injection points for metal salt and polymer addition. The location of the primary effluent feed points in step aeration systems may have to be adjusted to allow sufficient time for enzymatic conversion of polyphosphates to orthophosphates to occur prior to metal salt addition. The long aeration detention times and sludge ages in extended aeration systems combined with metal salt addition may result in a mixed liquor with undesirably low volatile solids concentrations. If this occurs, sludge wasting rates must be increased and/or part of the aeration volume must be taken out of service, either one of which will decrease the sludge age and increase the mixed liquor volatile fraction.

Similar strategies are recommended for retrofitting standard-rate trickling filters and RBC's to chemical phosphorus removal since both of these fixed film processes will produce efficient secondary treatment under proper loading and operating conditions. High-rate trickling filters, however, which typically were not designed to meet secondary treatment requirements in the

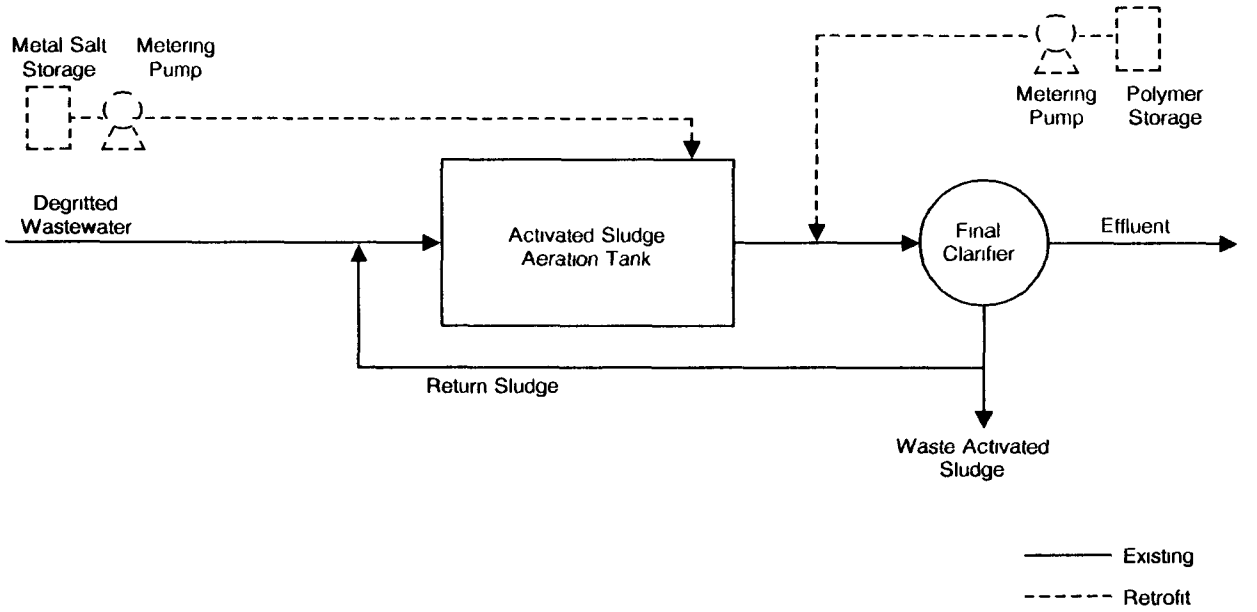
Figure 4. Sludge Generation Rate for Chemical Phosphorus Removal as a Function of Metal Ion-to-Influent TP Weight Dosage Ratio (4).



Final Effluent TP	Polymer Dose	M^{3+}/Inf. TP Ratio	Final Clarifier SOR *	Final Effluent Filtration Required
mg/L	mg/L	mole	$m^3/m^2/d$	
2	0.1 - 0.2	1.0 - 1.2	33	No
1	0.1 - 0.2	1.2 - 1.5	24	No
0.5	0.1 - 0.2	1.5 - 2.0	20	Maybe
0.2	0.5 - 1.0	3.5 - 6.0	20	Yes

* at peak sustained flow

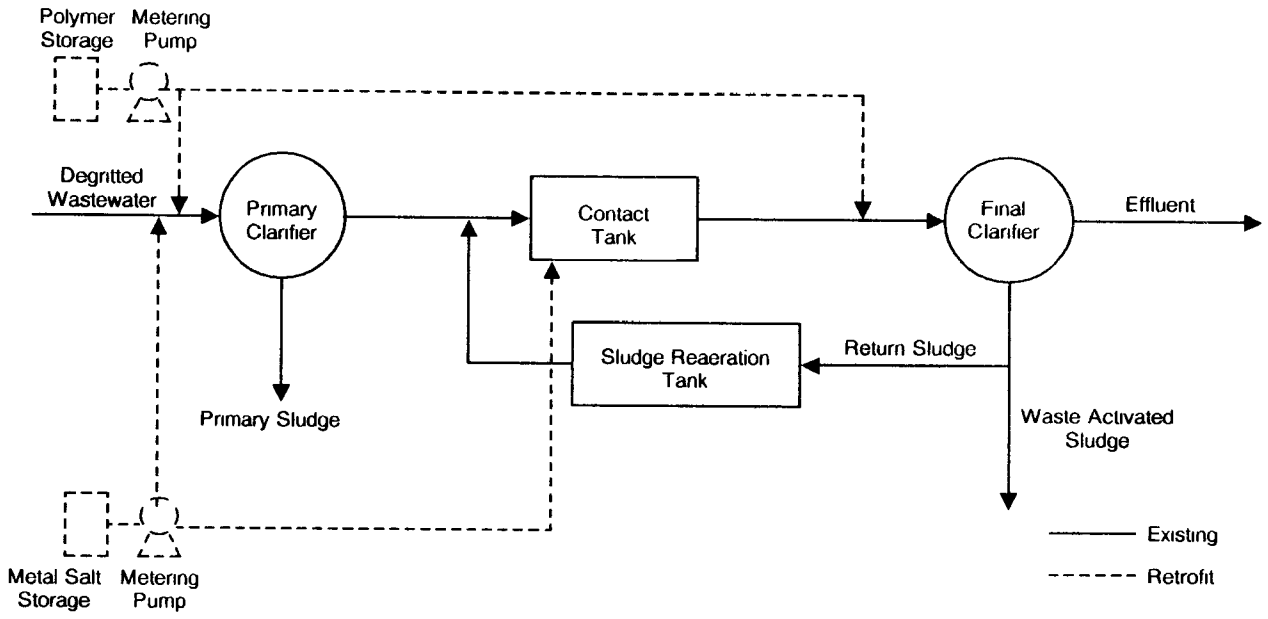
Figure 5. Flow Diagram and Design Recommendations for Retrofitting Plug Flow, Step Aeration, Complete Mix, Pure Oxygen, and Single-Stage Nitrification Activated Sludge Systems to Chemical Phosphorus Removal in the CBDB (4).



Final Effluent TP	Polymer Dose	M^{3+}/Inf TP Ratio	Final Clarifier SOR*	Final Effluent Filtration Required
mg/L	mg/L	mole	$m^3/m^2/d$	
2	0.1 - 0.2	1.0 - 1.2	33	No
1	0.1 - 0.2	1.2 - 1.5	24	No
0.5	0.1 - 0.2	1.5 - 2.0	20	Maybe
0.2	0.5 - 1.0	3.5 - 6.0	20	Yes

*at peak sustained flow

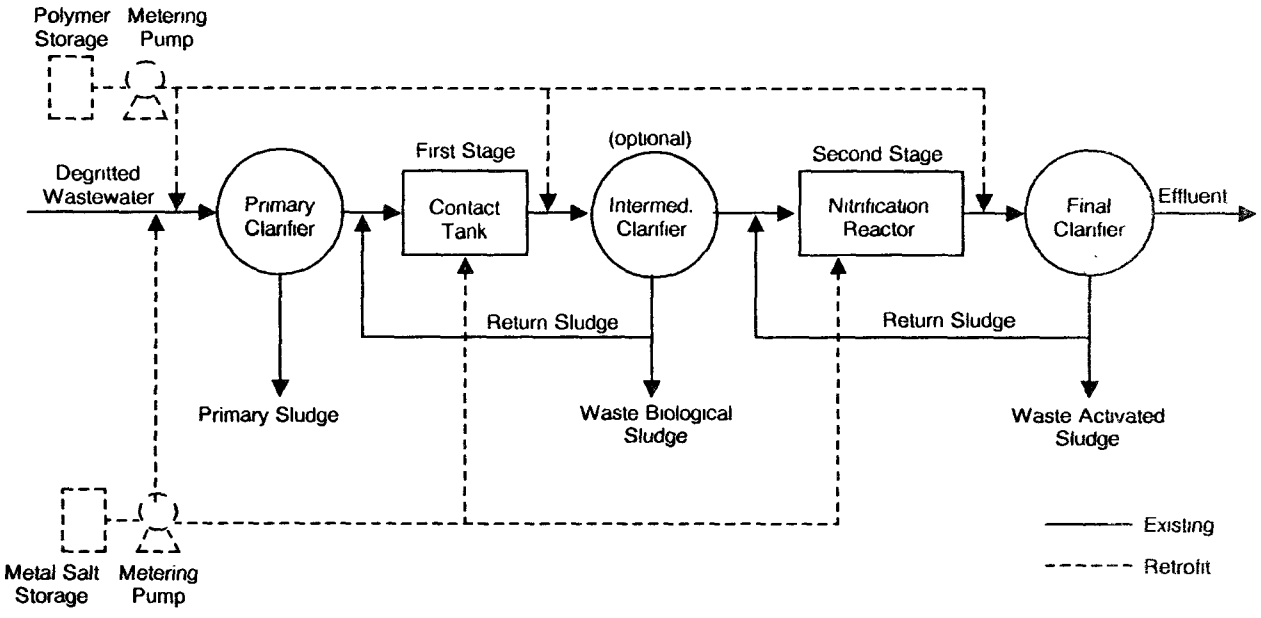
Figure 6. Flow Diagram and Design Recommendations for Retrofitting Extended Aeration and Oxidation Ditch Activated Sludge Systems for Chemical Phosphorus Removal in the CBDB (4).



Final Effluent TP mg/L	Polymer Dose mg/L	M^{3+}/Inf. TP Ratio mole	Final Clarifier SOR * $m^3/m^2/d$	Final Effluent Filtration Required
2	0.1 - 0.2	1.0 - 1.2	33	No
1	0.1 - 0.2	1.2 - 1.5	24	No
0.5	0.1 - 0.2	1.5 - 2.0	20	Maybe
0.2	0.5 - 1.0	3.5 - 6.0	20	Yes

*at peak sustained flow

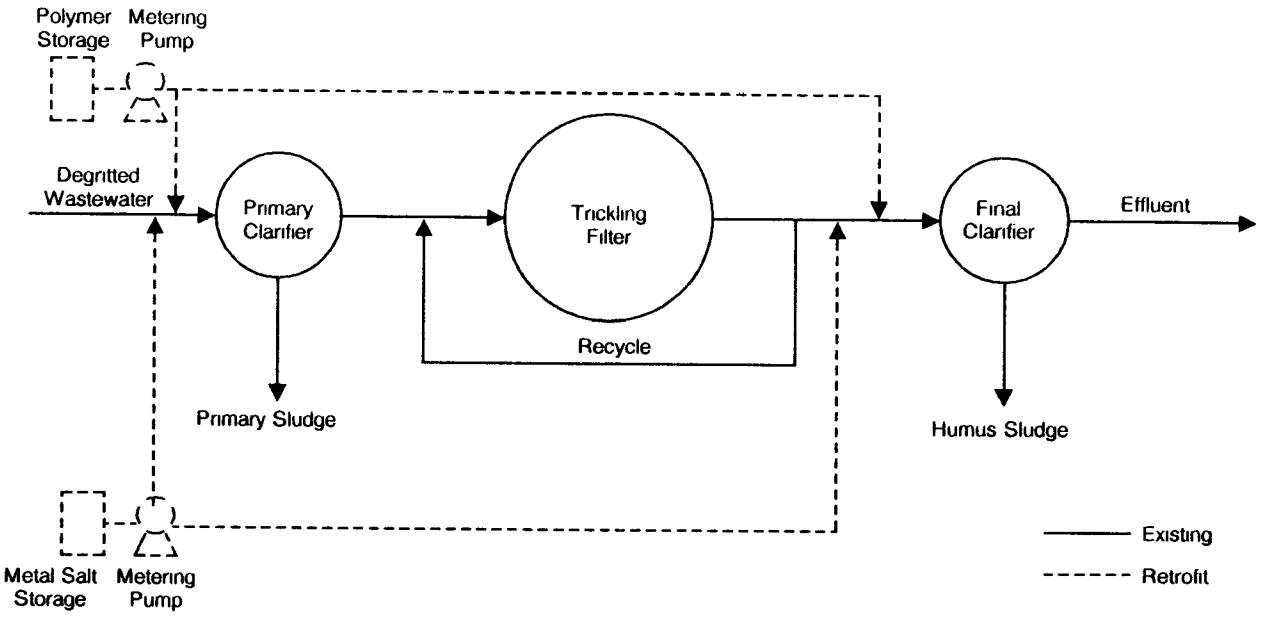
Figure 7. Flow Diagram and Design Recommendations for Retrofitting Contact Stabilization Activated Sludge Systems to Chemical Phosphorus Removal in the CBDB (4).



Final Effluent TP	Polymer Dose	M^{3+}/Inf. TP Ratio	Final Clarifier SOR *	Final Effluent Filtration Required
mg/L	mg/L	mole	$m^3/m^2/d$	
2	0.1 - 0.2	1.0 - 1.2	33	No
1	0.1 - 0.2	1.2 - 1.5	24	No
0.5	0.1 - 0.2	1.5 - 2.0	20	Maybe
0.2	0.5 - 1.0	3.5 - 6.0	20	Yes

* at peak sustained flow

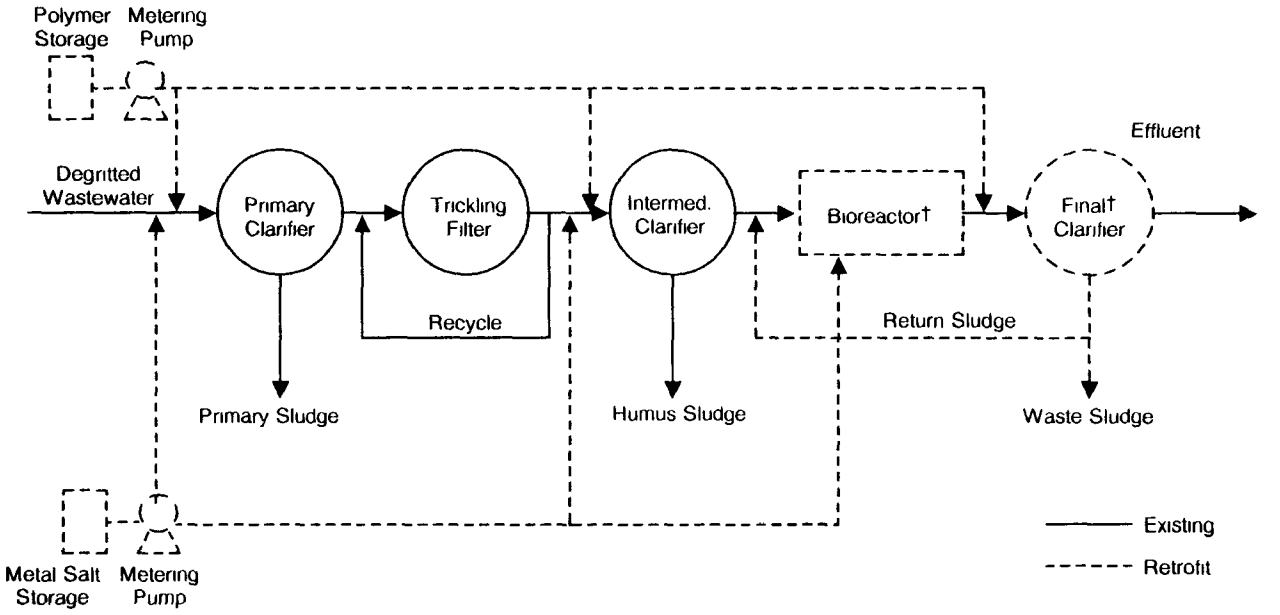
Figure 8. Flow Diagram and Design Recommendations for Retrofitting Two-Stage Biological Nitrification Systems to Chemical Phosphorus Removal in the CBDB (4).



Final Effluent TP mg/L	Polymer Dose mg/L	M^{3+}/Inf. TP Ratio mole	Final Clarifier SOR * $m^3/m^2/d$	Final Effluent Filtration Required
2	0.1 - 0.2	1.0 - 1.2	33	No
1	0.1 - 0.2	1.2 - 1.5	24	No
0.5	0.1 - 0.2	1.5 - 2.0	20	Maybe
0.2	0.5 - 1.0	3.5 - 6.0	20	Yes

*at peak sustained flow

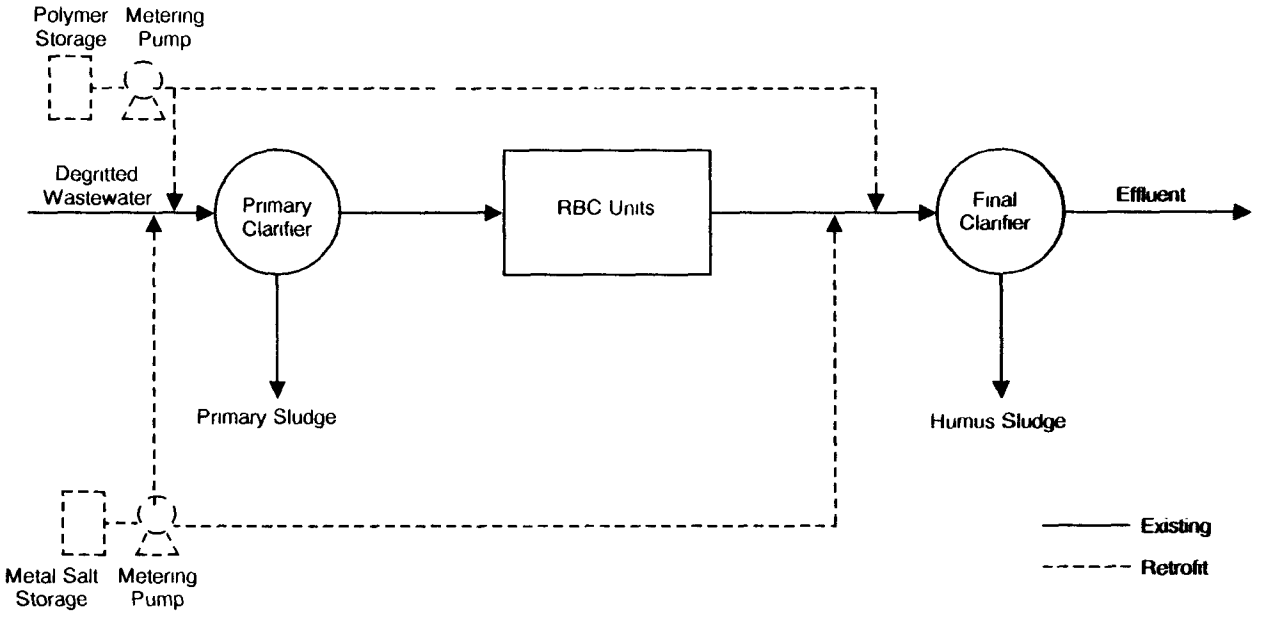
Figure 9. Flow Diagram and Design Recommendations for Retrofitting Standard-Rate Trickling Filters to Chemical Phosphorus Removal in the CBDB (4).



Final Effluent TP mg/L	Polymer Dose mg/L	M^{3+}/Inf. TP Ratio mole	Final Clarifier SOR * $m^3/m^2/d$	Final Effluent Filtration Required
2	0.1 - 0.2	1.0 - 1.2	33	No
1	0.1 - 0.2	1.2 - 1.5	24	No
0.5 **	0.1 - 0.2	1.5 - 2.0	20	Maybe
0.2 **	0.5 - 1.0	3.5 - 6.0	20	Yes

* at peak sustained flow
** major expansion may be necessary
† may be necessary for 0.5 and 0.2 mg/L effluent TP

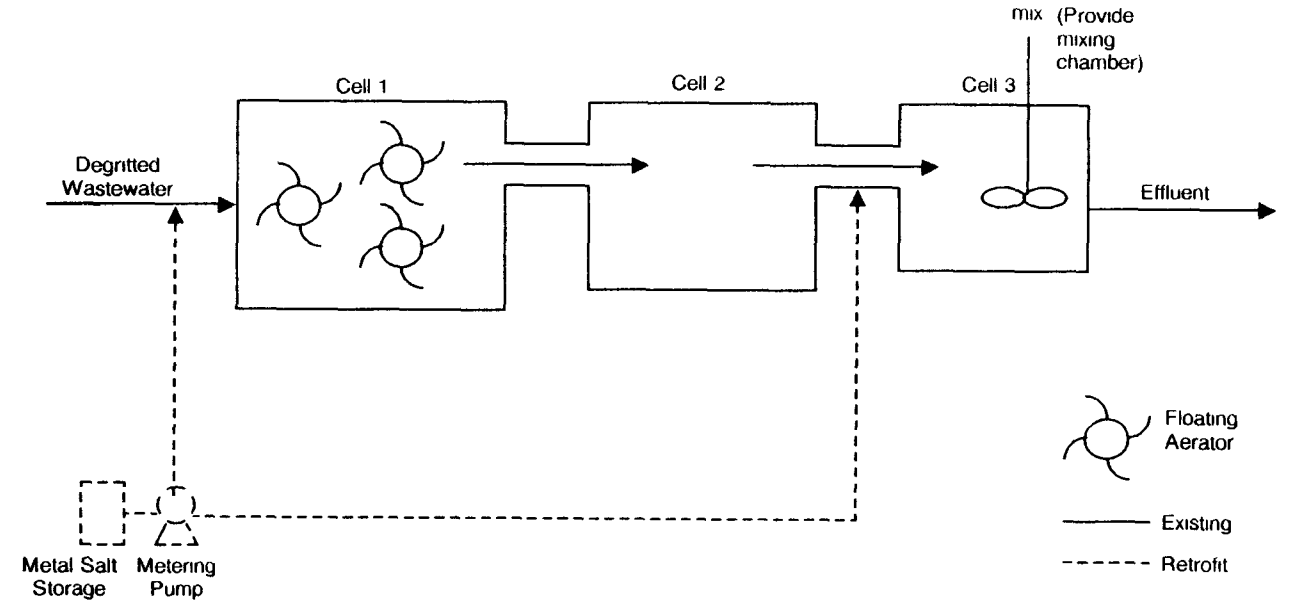
Figure 10. Flow Diagram and Design Recommendations for Retrofitting High-Rate Trickling Filters to Chemical Phosphorus Removal in the CBDB (4).



Final Effluent TP mg/L	Polymer Dose mg/L	M^{3+}/Inf. TP Ratio mole	Final Clarifier SOR * $m^3/m^2/d$	Final Effluent Filtration Required
2	0.1 - 0.2	1 0 - 1 2	33	No
1	0.1 - 0.2	1.2 - 1.5	24	No
0.5	0.1 - 0.2	1 5 - 2.0	20	Maybe
0.2	0.5 - 1.0	3 5 - 6 0	20	Yes

* at peak sustained flow

Figure 11. Flow Diagram and Design Recommendations for Retrofitting RBC's to Chemical Phosphorus Removal in the CBDB (4).



Final Effluent TP mg/L	M^{3+}/Inf. TP Ratio mole
2	1.2 - 1.5
1	1.5 - 2.0

Figure 12. Flow Diagram and Design Recommendations for Retrofitting Wastewater Lagoons to Chemical Phosphorus Removal in the CBDB (4).

CBDB, may require addition of a second-stage bioreactor and clarifier to improve overall treatment levels before the desired level of phosphorus removal can be achieved, particularly if effluent TP limits of less than 1.0 mg/L are imposed. The authors of the CBDB Handbook do not believe lagoon wastewater treatment systems can be reliably retrofitted to achieve effluent TP concentrations less than 1.0 mg/L without construction of a tertiary chemical treatment system (4).

Many configuration options are possible for developing a multi-stage system to remove both phosphorus and nitrogen. One such configuration, taken from Reference 4, is presented in Figure 13. This system utilizes a first-stage activated sludge system to remove the bulk of the carbonaceous matter contained in the flow-equalized influent wastewater stream, a second-stage RBC unit to oxidize ammonium nitrogen to nitrate nitrogen, and a third-stage suspended growth denitrification system with methanol addition to remove nitrogen. Separate clarification is not required after the second-stage RBC unit. Metal salt is added to the first and third stages, and the final effluent is subjected to dual media filtration. The plant was designed and is operated to comply with final effluent limitations of 5 mg/L total BOD_5 (TBOD), 5 mg/L TSS, 1 mg/L TP, and 3 mg/L total nitrogen.

Phosphorus Removal Performance

Performance data are given in Table 3 for 58 selected secondary treatment plants in the United States and one in Canada that use mineral salt addition for phosphorus removal (3). The list of plant types encompasses all the secondary treatment plant flow schemes illustrated in Figures 5 through 12 except wastewater lagoons. All of the facilities except six are attaining effluent TP concentrations of 1 mg/L or less. Polymer addition is practiced at 30 of the 59 facilities. Polymer dosage ranged from 0.04 to 3.8 mg/L, although most plants required a dose of less than 1 mg/L. Metal ion-to-influent TP weight dosages were significantly higher, on the average, for the plants with fixed film secondary treatment systems than for those with suspended growth secondary systems. For most activated sludge plants, the weight dosage ratio was between 0.5 and 3.5, while the corresponding ratio for the fixed film plants generally varied between 1.2 and 4.0. The average metal ion-to-influent TP dose ratio for the 41 activated sludge plants listed was 1.90. If the Algoma, Wisconsin contact stabilization plant average dosage value (Fe^{3+}/Inf. TP = 10.0) is excluded, the average weight dosage ratio for the remaining 40 activated sludge plants drops to 1.70. In contrast, the metal ion-to-influent TP weight dosage ratio for the 18 fixed film plants listed averaged 2.58.

The potential improvement in phosphorus removal as well as TBOD and TSS removals with mineral addition is summarized in Table 4 as predicted by Reference 3. Estimates of improvement are given for primary, activated sludge, and trickling filter treatment.

The relationship of metal ion-to-influent TP weight dosage to effluent TP concentration for selected municipal plants in the Chesapeake Bay and Great Lakes Drainage Basins is shown in Figures 14 and 15 for alum and ferric iron, respectively (4). The substantially higher metal salt dosage required

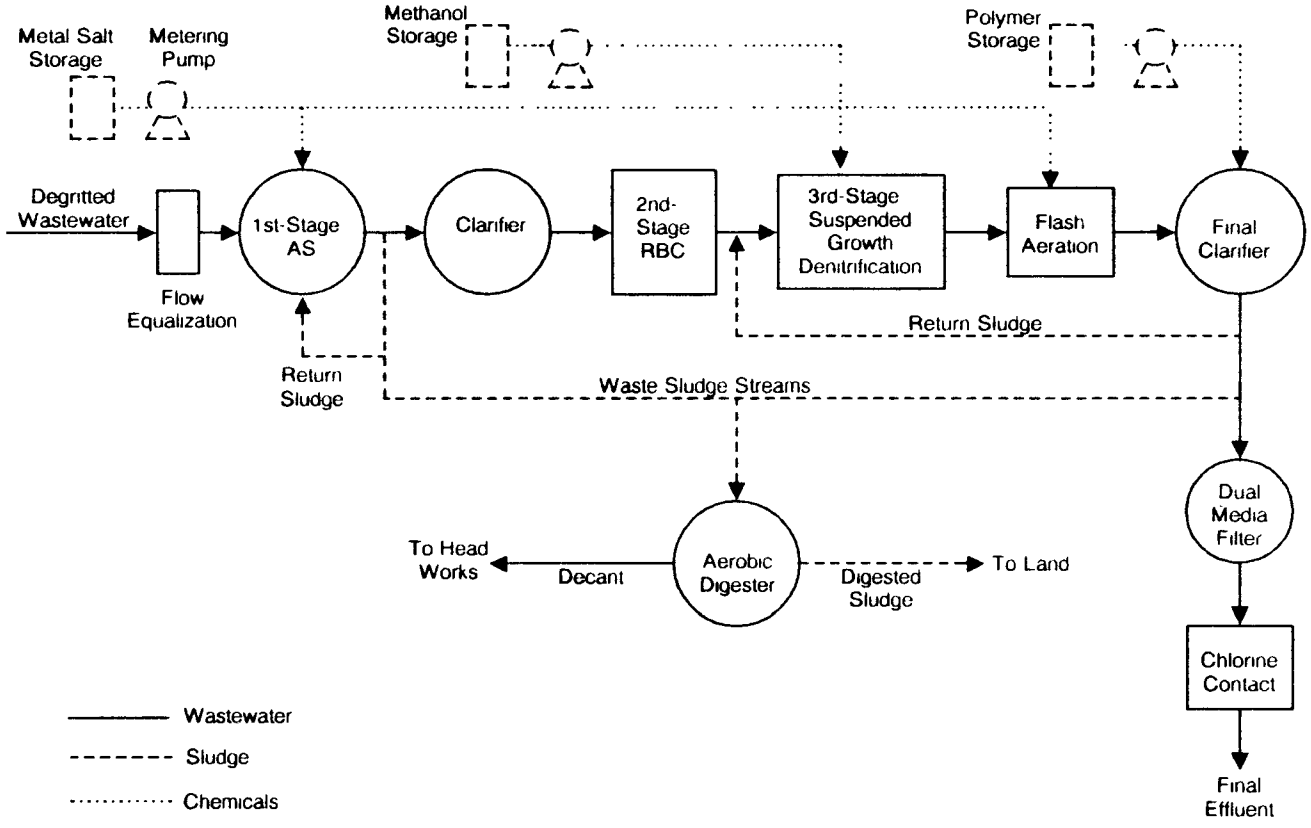


Figure 13. Flow Diagram of Multi-Stage Biological-Chemical Process for Removing Phosphorus and Nitrogen (4).

TABLE 3. OPERATING AND PERFORMANCE DATA SUMMARY FOR SELECTED NORTH AMERICAN PLANTS THAT UTILIZE CHEMICAL PHOSPHORUS REMOVAL (3)

Plant Type and Location	Design Flow	Average Flow	Chemicals	Chemical Feed Point	Chemical Dosage*	Metal Ion: Inf. TP	Inf. TP	Eff. TP
	m^3/d	m^3/d				weight	mg/L	mg/L
Plug Flow AS								
Waupaca, WI	4,760	2,200	Alum	Sec. Clarifier	24 6	3.25	7 56	0.86
East Chicago, IN	75,700	59,800	Alum	Sec. Clarifier	7 7	3 99	1 93	0.38
			Polymer	Sec. Clarifier	1 0			
Mason, MI	5,700	5,000	Ferric Chloride	Prim. Clarifier	9.1	1.4	6.5	0.88
			Polymer	Prim. Clarifier	0.05			
Flushing, MI	4,400	6,000	Ferric Chloride	Sec. Biol. Process	5.3	1.56	3.4	0.48
			Polymer	Sec. Biol. Process	0.15			
Appleton, WI	62,500	52,200	Ferrous Chloride	Plant Influent	16.8	1.6	10.5	0.8
Grand Ledge, MI	5,700	3,000	Ferrous Chloride	Sec. Biol. Process	5.6	1.24	4 5	0.7
Bowling Green, OH	30,300	20,100	Ferrous Chloride	Sec. Clarifier	5.2	0.62	8.4	0.75
			Polymer	Sec. Clarifier				
Kenosha, WI	106,000	90,500	Ferrous Sulfate	Prim. Clarifier	5.35	1.43	3.74	0.36
Toledo, OH	386,100	310,400	Ferrous Sulfate	Prim. Clarifier	3.6	1.3	2.76	0.35
			Polymer	Prim. Clarifier				
Clintonville, WI	3,800	2,700	Ferrous Sulfate	Sec. Clarifier	5.3	1.47	3.6	0.75
Complete Mix AS								
Thiensville, WI	900	3,300	Alum	Sec. Biol. Process	9.3	2.46	3.78	0.29
			Polymer	Sec. Biol. Process	0.82			
Two Harbors, MN	4,500	3,400	Alum	Sec. Clarifier	9.6	1.6	6.0	0.25
Escanaba, MI	8,300	7,600	Ferric Chloride	Prim. Clarifier	4 7	1.04	4.5	0.82
			Polymer	Prim. Clarifier	0.35			
Sheboygan, WI	69,600	46,600	Ferric Chloride	Sec. Clarifier	10.2	1.6	6.38	0.9
Lima, OH	70,000	15,100	Ferrous Chloride	Prim. Clarifier	13.2	3.38	3.9	0.5
			Polymer	Prim. Clarifier	0.07			
Niles, MI	22,000	12,100	Ferrous Chloride	Sec. Biol. Process	10.9	2.66	4.1	0.7
Crown Point, IN	13,600	8,700	Ferrous Chloride	Sec. Clarifier	11.0	2.0	5.5	0.7
			Polymer	Sec. Clarifier	0.94			
Cedarburg, WI	11,400	7,600	Ferrous Sulfate	Sec. Clarifier	9.9	2.99	3.31	0.67
			Polymer	Sec. Clarifier				
Contact Stabilization AS								
Neenah, WI	5,700	4,000	Alum	Prim. Clarifier	7.7	2.2	3.5	0.7
Neenah, WI	14,800	16,700	Alum	Sec. Biol. Process	4 1	1.0	4.1	0.8
Algoma, WI	2,800	3,000	Ferric Chloride	Prim. Clarifier	33.0	10.0	3.3	0.23
			Polymer	Prim. Clarifier	0.07			
Grafton, WI	8,100	3,600	Ferrous Chloride	Prim. Clarifier	16.2	2.31	7.0	0.69
Port Washington, WI	4,700	5,800	Ferrous Chloride	Prim. Clarifier	8.5	1.44	5.9	1.0
Port Clinton, OH	5,700	6,400	Ferrous Chloride	Sec. Biol. Process	10.2	1.96	5.2	0.5
Oberlin, OH	5,700	5,700	Ferrous Chloride	Prim. Clarifier	6.4	1.08	5.9	1.0
North Olmstead, OH	34,000	21,200	Sodium Aluminate	Sec. Biol. Process	8.3	2.86	2.9	0.7
Pure Oxygen AS								
Fon du Lac, WI	41,600	26,900	Alum	Sec. Clarifier	8.5	1.18	7.2	0.73
			Polymer	Sec. Clarifier	0.75			
Extended Aeration AS								
Aurora, MN	1,900	1,700	Alum	Prim. Clarifier	16.9	5.83	2.9	0.76
Upper Allen, PA	1,800	1,200	Alum	Sec. Biol. Process	8.2	0.92	8.9	2.0
			Polymer	Sec. Biol. Process	0.37			
Corunna, Ontario	3,800	2,000	Alum	Sec. Clarifier	5.0	0.65	7 74	0.36
Saukville, WI	7,600	2,400	Ferrous Chloride	Prim. Clarifier	10.3	1.61	6.4	0.59
Plymouth, WI	6,200	5,800	Ferrous Chloride	Sec. Biol. Process	7.7	1.15	6.7	0.77
Trenton, OH	13,200	9,600	Ferrous Chloride	Sec. Biol. Process	2.56	0.42	6.1	0.65
Seneca, MD	18,900	15,100	Sodium Aluminate	Plant Influent	4.3	0.61	7.1	1.6
			Polymer	Sec. Clarifier	2.4			

(continued)

* Precipitant, mg/L as metal ion; polymer, mg/L of chemical.

TABLE 3 (CONTINUED). OPERATING AND PERFORMANCE DATA SUMMARY FOR SELECTED NORTH AMERICAN PLANTS THAT UTILIZE CHEMICAL PHOSPHORUS REMOVAL (3)

Plant Type and Location	Design Flow	Average Flow	Chemicals	Chemical Feed Point	Chemical Dosage*	Metal Ion. Inf TP	Inf. TP	Eff. TP
	m^3/d	m^3/d				weight	mg/L	mg/L
Step Aeration AS								
Fort Wayne, IN	227,100	170,100	Ferrous Chloride	Sec. Biol. Process	4.3	0.54	7 9	0.67
East Lansing, MI	71,200	42,800	Ferrous Chloride Polymer	Sec. Clarifier Sec. Clarifier	5.9 0 05	1 11	5 3	0 9
Oak Creek, WI	454,200	340,650	Ferrous Sulfate	Sec. Biol. Process	4 4	0.96	4.6	0.54
Elkhart, IN	75,700	60,200	Ferrous Sulfate	Sec. Clarifier	1 6	0.63	2.56	0.83
2-Stage Nitrification AS								
Piscataway, MD	113,600	54,900	Alum Polymer	Sec. Clarifier Sec. Clarifier	8.8 3.8	1.44	6 13	0.2
High Rate TF								
Geneva, OH	7,600	3,900	Alum	Sec. Clarifier	12.1	4.03	3.0	0.4
Coldwater, MI	8,700	7,400	Ferric Chloride Polymer	Sec. Clarifier Sec. Clarifier	8.3 0.1	2.02	4.1	0.88
Oconto Falls, WI	1,900	1,400	Ferric Chloride	Sec. Biol. Process	8.81	2.4	3.67	0.45
Kendalville, IN	10,100	7,600	Ferric Chloride Polymer	Sec. Biol. Process Sec. Biol. Process	14.7 0.25	4.05	3.63	0.35
Standard Rate TF								
Willard, OH	5,100	4,800	Alum Polymer	Prim. Clarifier Prim. Clarifier	6.3 0.14	1.21	5.2	0 82
Elizabethtown, PA	11,400	6,500	Alum Polymer	Sec. Clarifier Sec. Clarifier	12.8 0.4	2 51	5 1	1 7
Durand, MI	3,000	2,700	Ferric Chloride	Prim. Clarifier	11.2	2.2	5.1	0.83
Saginaw, MI	16,700	6,400	Ferric Chloride Polymer	Prim. Clarifier Prim. Clarifier	9.6 0.1	0.99	9.7	1.3
Little Hunting Creek, VA	17,000	14,400	Ferric Chloride Polymer	Prim. Clarifier Prim. Clarifier	42.5 2.8	4.57	9.3	0.3
Bay City, MI	75,700	33,300	Ferric Chloride Polymer	Sec. Clarifier Sec. Clarifier	9.5 0.29	2.07	4.6	0.5
Coloma, MI	8,300	5,300	Ferrous Chloride	Prim. Clarifier	4.1	1 71	2.4	0 65
RBC								
Romeo, MI	6,100	3,300	Alum Polymer	Prim. Clarifier Prim. Clarifier	7 1 0.77	2.4	2.98	0.46
Chesaning, MI	2,200	2,000	Ferric Chloride Polymer	Prim. Clarifier Prim. Clarifier	9.0 0.4	3.46	2.6	0.6
Negaunee, MI	6,100	3,300	Ferric Chloride Polymer	Prim. Clarifier Prim. Clarifier	7.5 1.0	3.75	2.0	0.93
Dexter, MI	2,200	800	Ferric Chloride Polymer	Sec. Clarifier Sec. Clarifier	10.2 0.5	2.0	5.11	0.46
Hartford, MI	1,300	800	Ferrous Chloride Polymer	Prim. Clarifier Prim. Clarifier	13.0 0.6	3.25	4.0	0.75
St. Johns, MI	7,200	6,300	Ferrous Chloride Polymer	Prim. Clarifier Prim. Clarifier	5.01 0.04	1.38	3.7	0.5
Charlotte, MI	4,500	2,700	Ferrous Chloride Polymer	Prim. Clarifier Sec. Clarifier	13.7 0.18	2.45	5.6	0.68
Oxidation Ditch								
Lapeer, MI	7,000	7,200	Ferric Chloride	Sec. Clarifier	4 65	0.88	5.3	1.2
Portage, IN	13,200	8,400	Ferrous Chloride	Sec. Clarifier	9.9	1.65	6.0	1.5

* Precipitant, mg/L as metal ion; polymer, mg/L of chemical.

TABLE 4. POTENTIAL EFFECTIVENESS OF PRIMARY AND SECONDARY TREATMENT WITH AND WITHOUT MINERAL ADDITION FOR PHOSPHORUS REMOVAL (3)

	TP Removal (%)		TSS Removal (%)		TBOD Removal (%)	
	Without	With	Without	With	Without	With
Primary Treatment	5-10	70-90	40-70	60-75	25-40	40-65
Secondary Treatment						
Trickling Filter	10-20	80-95	80-90	85-95	75-90	80-95
Activated Sludge	10-20	80-95	80-95	85-95	85-95	85-95

to achieve effluent TP concentrations of less than 0.5 mg/L are evident.

BIOLOGICAL PHOSPHORUS REMOVAL

Chemistry and Biology

The ability of some activated sludge organisms such as Acinetobacter and Pseudomonas to first release stored phosphorus under anaerobic conditions and then remove phosphorus from solution in enhanced quantities under subsequent aerobic conditions forms the basis for biological phosphorus removal technology. During the anaerobic first step, return sludge and influent wastewater are mixed in the absence of both dissolved oxygen (DO) and nitrate nitrogen. The above organisms, among others, apparently gain a competitive advantage by taking up soluble substrate, such as acetate, under anaerobic conditions. This uptake results in the accumulation of carbonaceous storage products called polyhydroxybutyrates (PHB's) that are metabolized during follow-on aerobic treatment.

Soluble substrate removal in the anaerobic step is accompanied by a release of orthophosphorus into solution by the above organisms. During subsequent aerobic treatment, these same organisms restore their intracellular polyphosphate supply by taking up the orthophosphorus released in the anaerobic zone plus additional orthophosphorus contained in the influent wastewater. During this latter period, a portion of the carbonaceous substrate removed and synthesized into carbonaceous storage products (PHB's) in the anaerobic phase is oxidized to provide energy for phosphorus uptake. A simplified schematic of the phosphorus release/uptake cycle is presented in Figure 16.

If either DO or nitrate nitrogen (NO_3-N) is present at the point where return sludge and influent wastewater are mixed, the polyphosphate-storing organisms do not compete successfully with organisms that use either NO_3^- or DO as terminal electron acceptors. Under these conditions, the biological phosphorus removal mechanism is interrupted and can be significantly diminished or even curtailed altogether if DO persists and/or NO_3^- remains present in moderate to high concentrations throughout the entire anaerobic zone.

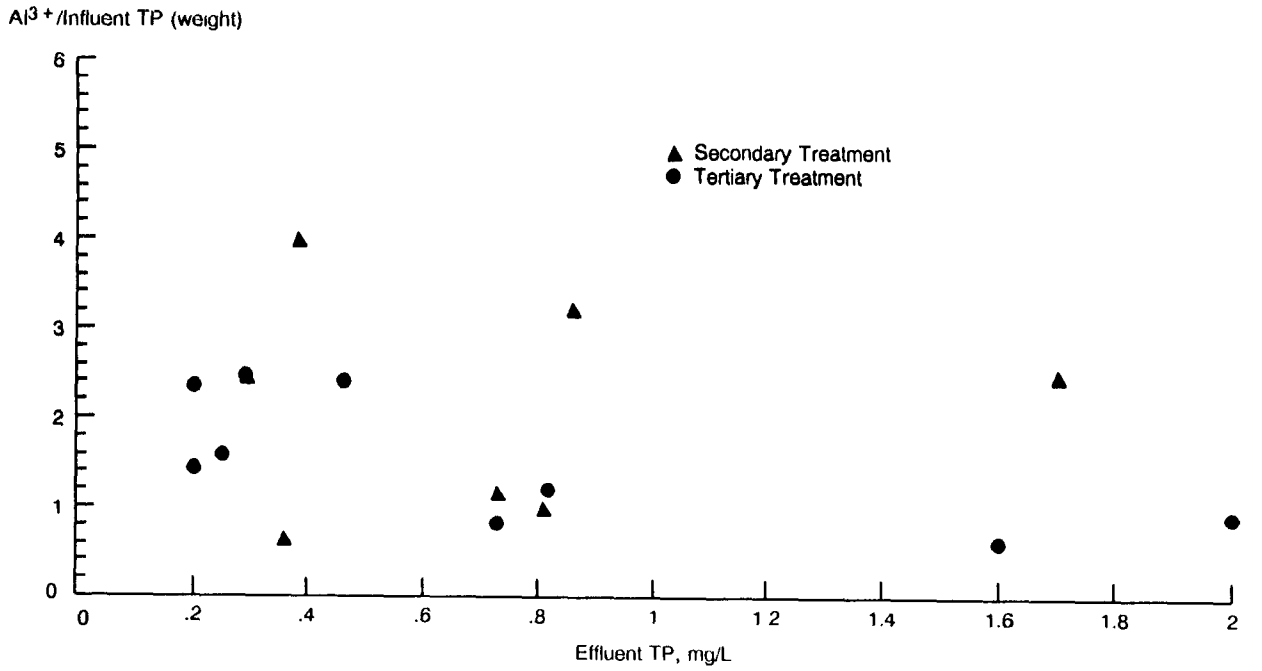


Figure 14. Relationship of Aluminum Ion-to-Influent TP Weight Dosage Ratio to Effluent TP for Chemical Phosphorus Removal in the Chesapeake Bay and Great Lakes Drainage Basins (4).

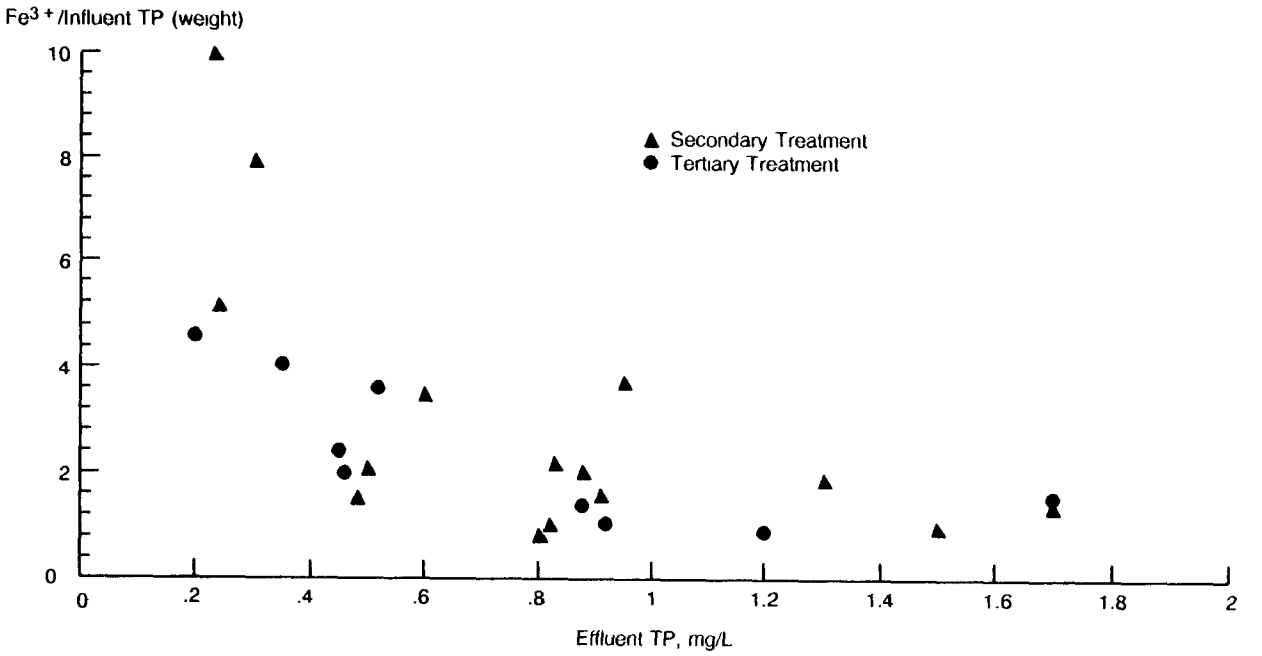


Figure 15. Relationship of Ferric Ion-to-Influent TP Weight Dosage Ratio to Effluent TP for Chemical Phosphorus Removal in the Chesapeake Bay and Great Lakes Drainage Basins (4).

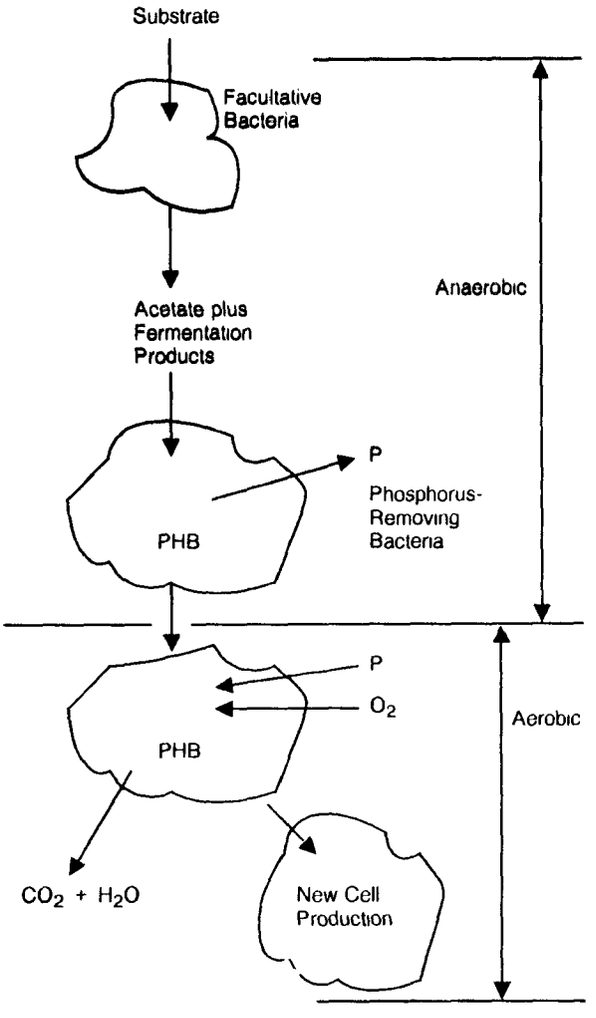


Figure 16. Schematic Representation of Biological Phosphorus Release/Uptake Cycle (3).

Zones where nitrate nitrogen but not DO is present are referred to as anoxic zones. First-step anaerobic (unaerated) zones may become anoxic zones when the subsequent second-step aerobic zone is operated to nitrify. Nitrate nitrogen produced in the nitrification reaction is introduced to the lead zone in the return sludge. Many investigators (3,4) believe that significant phosphorus release will not occur in an anoxic zone until the nitrate nitrogen present therein is reduced to near zero concentration via denitrification. Not only will this necessitate longer first-zone residence times for phosphorus release to occur, but the chemical oxygen released by the denitrification reaction will be used by obligate aerobes to oxidize readily biodegradable soluble substrate, making it unavailable to the phosphorus-leaching/storing organisms.

The impact of nitrate nitrogen in the return sludge on phosphorus release in the lead anaerobic zone can be minimized by intentionally denitri-

fying in other segments of the treatment train. This practice will reduce the concentration of nitrate nitrogen in the return sludge and decrease the detention time required in the lead zone to trigger the phosphorus release mechanism. Some proprietary dual nutrient control processes provide separate anoxic zones to accomplish nitrogen removal and help protect the lead anaerobic zone from unwanted nitrate.

Acinetobacter and other phosphorus-removing bacteria are slow growing organisms. Phosphorus release from these organisms in the anaerobic zone of a biological phosphorus removal system is dependent on the availability and concentration of volatile fatty acids (VFA's) in this zone. These substances are produced from fermentation of soluble substrate in the influent wastewater. VFA's are readily assimilated by the microorganisms capable of excess phosphorus uptake and converted to PHB for carbonaceous storage within the cell. Under subsequent aerobic conditions, PHB is oxidized to acetyl-COA, but only after the exogenous carbon supply is nearly exhausted.

The VFA produced in greatest abundance during anaerobic fermentation is acetic acid or acetate. Propionic acid and bytyric acid, and under the right environmental conditions several other fatty acids, are also products of fermentation but in much lesser quantities. Acetate entering bacterial cells under anaerobic conditions is converted in sequence into acetyl-COA, then acetoacetate, and finally PHB, provided energy is available. Release of stored polyphosphates provides the necessary energy for this series of reactions. The relationship of acetate assimilation vs. the increase in orthophosphorus concentration resulting from polyphosphate release in the anaerobic zone is shown in Figure 17 as a function of time. For the experiment depicted in this figure, the molar ratio of acetate assilimilation to phos-

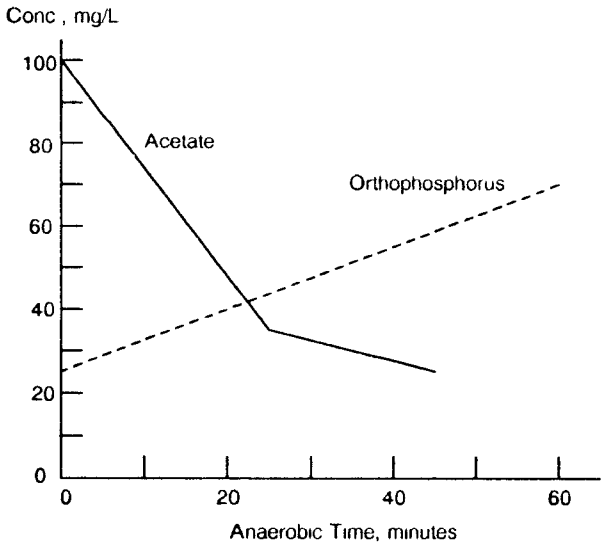


Figure 17. Typical Curve of Acetate Assimilation and Phosphorus Release as a Function of Anaerobic Zone Detention Time (3).

phorus release was 1.3 (3). Other reported ratios range from 0.6 to 1.0 (3).

Retrofit Considerations

The VFA's assimilated by phosphorus-removing bacteria in the anaerobic zone are derived from fermentation of soluble organics in the influent wastewater. It is generally considered to be uneconomical to provide sufficient time in the anaerobic zone for particulate organics to be hydrolyzed and converted to fermentation products (3). The exception to this is the PhoStrip process (to be described later) where the phosphorus release and VFA uptake reactions take place in a sidestream reactor (stripper) that receives and processes a portion of the return sludge flow. Unless supplemented by raw wastewater or primary effluent bypass, the only mechanisms by which VFA's are formed in the stripper are hydrolysis of particulate matter and bacterial cell lysis. The required stripper hydraulic detention time (HDT) for phosphorus release to occur is consequently several times that needed in an upfront mainstream anaerobic zone, but because only 15 to 30 percent of the influent flow (4) is being processed through the stripper, the economics of this unit do not become prohibitive. Accumulation of SP in the stripper supernatant occurs not only from phosphorus released by the phosphorus-removing organisms but also from the lysed bacteria.

The ratio of SBOD to SP in the secondary system influent of mainstream biological phosphorus removal processes directly affects attainable phosphorus removal efficiency. A SBOD-to-SP ratio of 10 to 15 is generally considered to be necessary to achieve an effluent TP limit of 1.0 mg/L in these processes (4). To achieve an effluent TP concentration of 0.5 mg/L, the required ratio increases to 20 to 25 (4). The influent SBOD-to-SP ratio is not nearly so critical to phosphorus removal efficiency in the PhoStrip process since the soluble organics for assimilation by the phosphorus-removing bacteria are provided not by fermentation of incoming organics but by solids hydrolysis and cell lysis in the return sludge stripper.

As discussed in the section on chemical phosphorus removal, effluent TSS levels determine the degree of phosphorus removal that is possible. The higher the phosphorus content of the effluent solids, the more the effluent solids will contribute to the effluent TP residual. The phosphorus content of mixed liquor suspended solids (MLSS) in biological phosphorus removal systems has been reported to vary from 2.3 to 5.8 percent. Because of the nature of their phosphorus-removal mechanisms, the mainstream processes tend to operate in the upper portion of this range and the PhoStrip process in the lower portion (3). Consequently, lower effluent TSS concentrations would generally be required with mainstream processes than with the PhoStrip process to achieve the same level of effluent TP. Fortunately, biological phosphorus removal systems generally produce improved sludge settling rates and liquid/solids separation performance.

The PhoStrip process is considered capable of meeting effluent TP limits down to 1 mg/L, and in some instances 0.5 mg/L if effluent TSS concentrations are low, without resorting to effluent polishing techniques. Mainstream processes may require occasional chemical polishing to meet 1 mg/L TP and almost certainly a continuous polishing dose of metal salt to meet a 0.5-mg/L

effluent TP residual (4). Similarly, effluent filtration should be seriously considered for mainstream processes wherever effluent TP limits of less than 1 mg/L are required, while the PhoStrip process would probably necessitate such a step only where TP residuals of less than 0.5 mg/L are specified.

With the PhoStrip process, two routes are possible for removal of phosphorus from the system, the waste sludge, which does not pass through the stripper, and the stripper overflow supernatant. The phosphorus-rich stripper supernatant is chemically treated with lime to precipitate soluble phosphorus. Approximately two-thirds of the phosphorus removed in the PhoStrip process is accomplished via the stripper route, one-third via the waste sludge route (4). Only that fraction of the removed phosphorus contained in the waste sludge is subject to resolubilization if anaerobic sludge digestion is practiced. The lime precipitated sludge will not resolublize during anaerobic digestion.

In contrast, all phosphorus removed from mainstream biological phosphorus removal processes ends up in the waste sludge. Resolubilization of a fraction of this removed phosphorus is likely if the waste sludge is digested either anaerobically or aerobically. The presence of naturally-occurring metal cations in some digesters may partially offset phosphorus released therein by precipitating it to insoluble forms. To avoid recycling a potentially large load of solubilized phosphorus back to the biological reactor, provision of dosing facilities to chemically treat anaerobic or aerobic digester supernatant return should be considered in any retrofit using a mainstream biological phosphorus removal process.

Biological phosphorus removal efficiency is directly related to the amount of phosphorus-removing organisms that can be grown. Fundamental studies (3) with pure cultures of Acinetobacter have indicated a maximum synthesis yield of 0.42 g solids/g acetate removed. The maximum growth rate of Acinetobacter will be approached only at low sludge retention times (SRT's). The longer the SRT or the lower the food-to-microorganism (F/M) loading, the lower the net yield of Acinetobacter and other phosphorus-removing bacteria. One pilot study noted that phosphorus uptake by the mixed liquor solids decreased by a factor of 2.6 as the F/M loading decreased from 0.2 to 0.1 kg TBOD/kg MLVSS/d (3). To maximize phosphorus removal efficiency, therefore, biological phosphorus removal systems should not be operated at SRT's in excess of overall treatment needs. Systems that are required to nitrify, and possibly also denitrify, will by definition operate at higher SRT's than carbonaceous removal systems. These systems cannot be expected to reduce SP to effluent concentrations of less than 1.0 mg/L unless the influent SBOD-to-SP ratio is very high, i.e., 25 or higher depending on the actual SRT.

Although no significant differences in secondary system sludge production are reported in the literature for biological phosphorus removal systems compared to typical sludge yields, some increase should be expected from associated chemical constituents that are co-transported into and stored in the sludge mass along with the phosphorus itself for the purpose of balancing the ionic charges of the polyphosphate compounds. An approximation of the mass of associated storage products is given in Table 5 (3).

TABLE 5. APPROXIMATE MASS OF PHOSPHORUS-ASSOCIATED STORAGE CONSTITUENTS IN THE MIXED LIQUOR SOLIDS OF BIOLOGICAL PHOSPHORUS REMOVAL SYSTEMS (3)

Constituent	Molecular Weight	Mole per Mole of P	g/g P
Mg	24.3	0.28	0.22
K	39.1	0.20	0.25
Ca	40	0.09	0.12
O	16	4	2.06
P	31	1	1.0
Total			3.65

As indicated in Table 5, the total expected additional stored mass per unit of phosphorus stored in the mixed liquor solids (or waste solids) is 3.65 units. At a typical net solids yield of 0.70 g TSS/g TBOD removed and a two-fold increase in the dry solids phosphorus content from 2 percent to 4 percent as a result of biological phosphorus removal, the increase in sludge production for a mainstream biological phosphorus removal process can be calculated to be 8.5 percent (3).

The same associated constituent storage mechanism will be operative in the PhoStrip process. Additional excess sludge will also be produced in significant amounts from the use of lime addition to precipitate soluble phosphorus in the stripper supernatant. The amount of excess lime sludge produced will depend on the percent of plant influent flow passing through the sidestream chemical reactor and the wastewater bicarbonate alkalinity. For example, if 20 percent of the influent flow receives chemical treatment and the bicarbonate alkalinity is 150 mg/L, the additional sludge produced from lime addition will approximate 70 mg/L, based on plant influent flow. Total excess sludge produced with the PhoStrip process can be expected to be equal to or greater than that resulting from conventional mineral addition.

The amount of volatile fatty acids (fermentation products) available in the anaerobic zone relative to the amount of phosphorus that must be removed can limit biological phosphorus removal rates, particularly in mainstream systems. Where influent BOD-to-phosphorus ratios are low, volatile fatty acid (VFA) production will also be low and the phosphorus release mechanism will be inhibited. In cases where VFA production from low influent BOD concentrations is limiting process efficiency, it may be cost effective to consider increasing the VFA concentration in the anaerobic zone. This can be accomplished by instituting one of several possible primary sludge fermentation schemes. These schemes depend on transferring VFA's released from primary sludge during extended fermentation periods to the primary effluent for transport into the lead anaerobic zone of the biological reactor. Two design schemes for achieving primary sludge fermentation, if needed, are presented in Figure 18 (3).

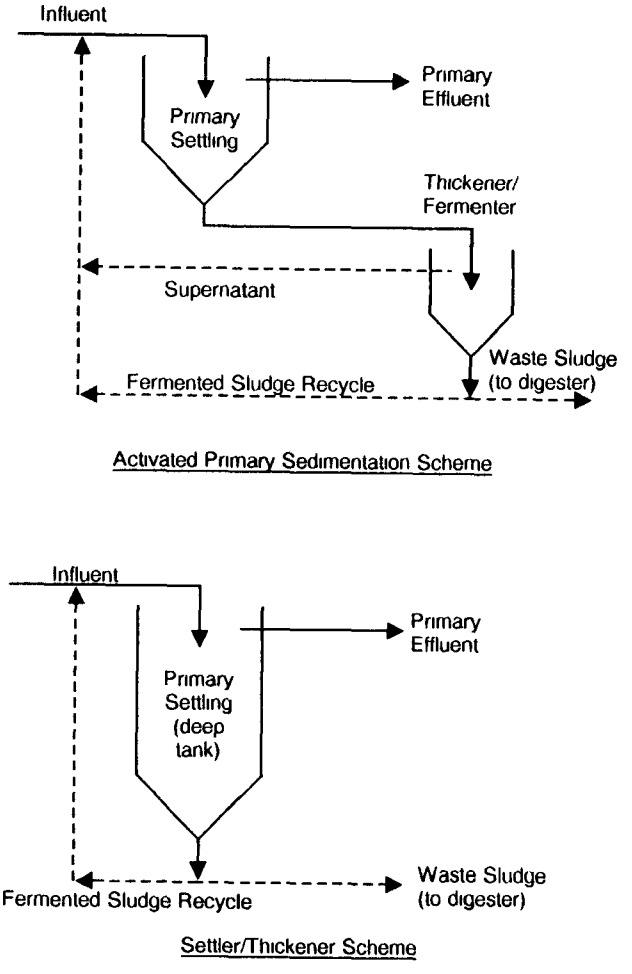


Figure 18. Primary Sludge Fermentation Design Schemes for Biological Phosphorus Removal (3).

As a general rule, high influent BOD-to-phosphorus ratios favor selection of a mainstream biological phosphorus removal process. Low influent BOD-to-phosphorus ratios normally favor the PhoStrip process, conventional mineral addition, or a mainstream process coupled with primary sludge fermentation (3).

Biological phosphorus removal is most applicable to wastewaters with relatively low influent TP concentrations in the range of 3 to 6 mg/L (4). Higher influent TP levels increase the probability that effluent chemical polishing and/or filtration will be required to meet effluent TP limits.

Only activated sludge plants are amenable to biological phosphorus removal retrofitting. The sidestream features of the PhoStrip process make it

suitable for retrofitting with almost any type of existing aeration tankage. Most activated sludge configurations are acceptable for the PhoStrip aerobic process segment with the possible exception of contact stabilization and extended aeration. If the HDT of the contact zone of a contact stabilization flow regime is too short to promote satisfactory phosphorus uptake, some or all of the stabilization (sludge reaeration) tankage may have to be appropriated for additional aerobic contact time. Conversely, if the HDT of an extended aeration reactor is so long that it is adversely affecting cell yield and phosphorus uptake, some of the tankage may have to be taken out of service to decrease the system SRT and increase biological growth rates.

Mainstream biological phosphorus removal retrofits are most easily accomplished in plug flow reactors. This preference arises from the need to delineate and provide separate anaerobic and aerobic (and anoxic if denitrification is desired) zones in a staged sequence within the existing tankage. If necessary, most other activated sludge flow configurations can be successfully adapted to mainstream biological phosphorus removal by creative engineering. Additional space is generally not needed for a mainstream process retrofit, while space for the new sidestream unit processes must be provided with PhoStrip. For this reason, a mainstream process retrofit is typically more easily and cheaply accomplished than a PhoStrip process retrofit.

Process Descriptions

General--

Simplified flow schematics for achieving biological phosphorus removal, reproduced directly from Reference 5, are presented in Figure 19 for mainstream process options and in Figure 20 for PhoStrip process options. Some of the process options shown are also designed to achieve nitrogen control, either nitrification or nitrification plus denitrification. The term EBPR used on some of the schematics refers to "enhanced biological phosphorus removal".

The original Bardenpho process was designed to remove nitrogen, but not phosphorus. The absence of an initial anaerobic zone dictated that any phosphorus removal achieved would be to satisfy incidental metabolic needs only. The recycle of mixed liquor from the first aerobic zone to the first anoxic zone, where soluble substrate is normally available from the secondary system influent in substantial quantities, is responsible for the majority of the denitrification obtained with this flow scheme. The second anoxic and aerobic zones are provided to ensure more complete denitrification and nitrification, respectively. Denitrification in the second anoxic zone will occur at reduced rates due to the relative lack of readily degradable soluble substrate. Nitrate nitrogen entering the lead anoxic zone in the return sludge will also be reduced rapidly in the presence of high soluble substrate and concentrations.

The modified Bardenpho process adds a fifth zone or stage to the original configuration to achieve dual biological phosphorus and nitrogen removal. An anaerobic zone is placed ahead of the first anoxic zone to promote phosphorus release therein. Although the major denitrification reaction is

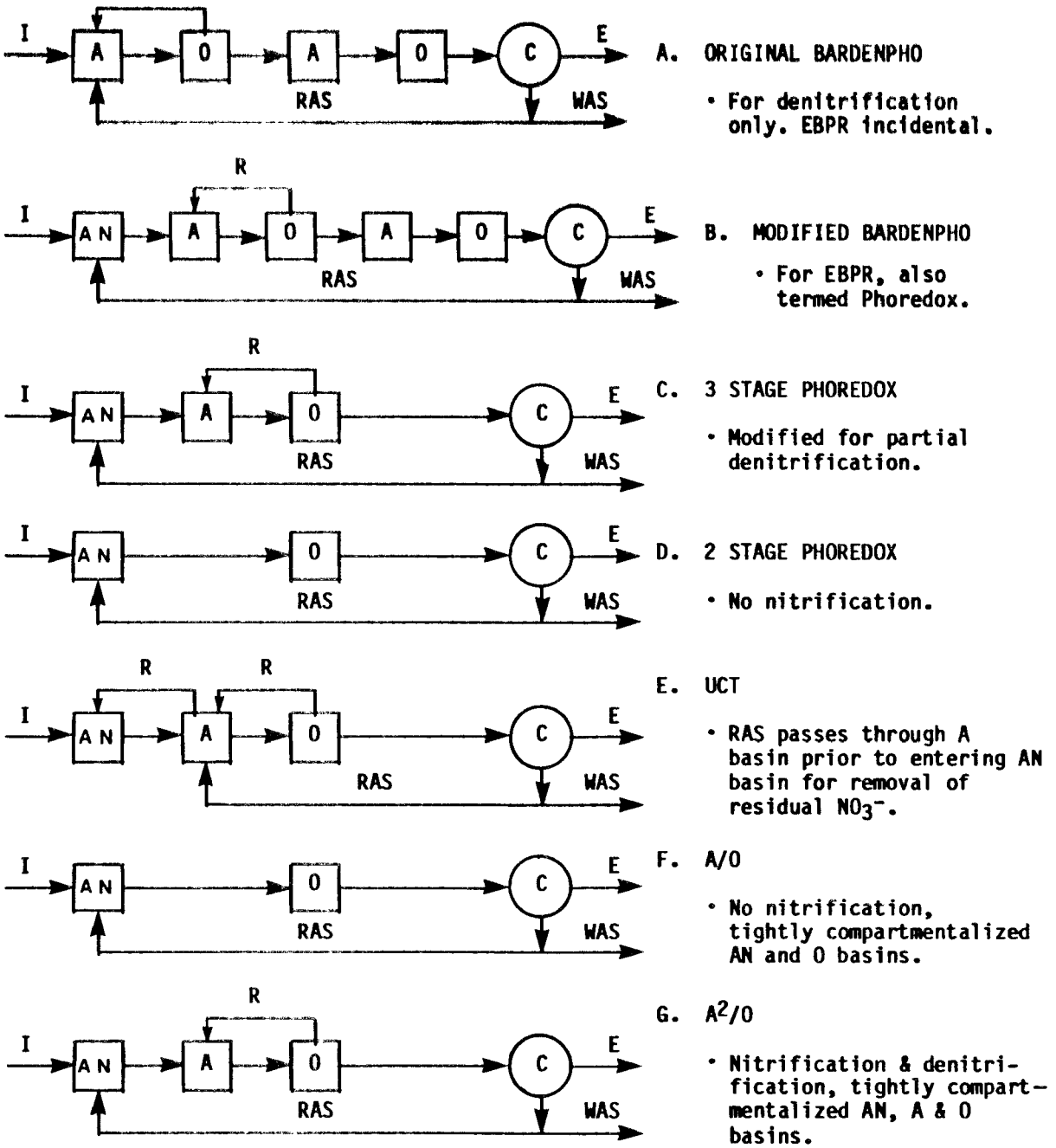
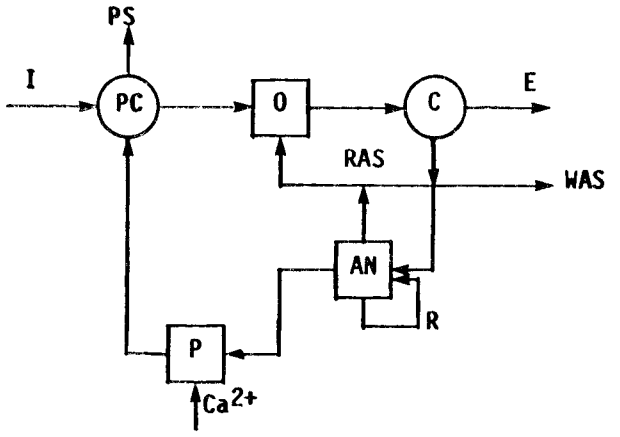
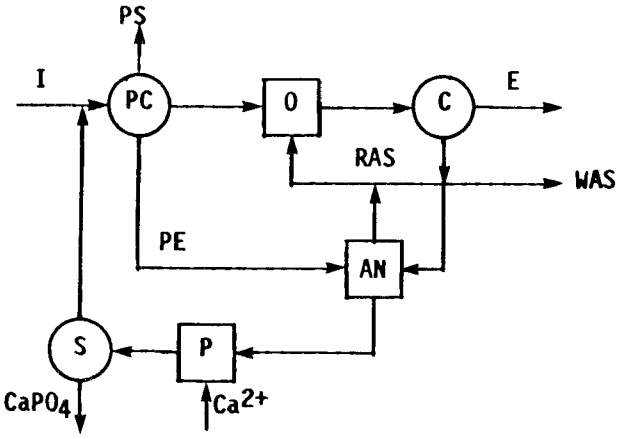


Figure 19. Basic Flow Diagrams for Mainstream Biological Nutrient Control Processes (5).



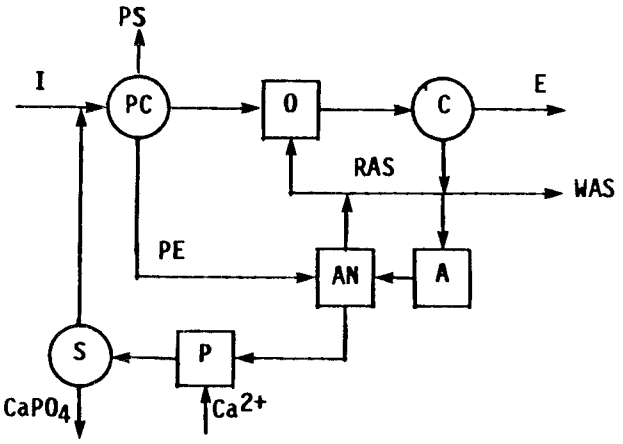
A. PHOSTRIP WITH RECYCLE STRIPPER (non-nitrifying system).

- Stripper (AN) overflow precipitated with lime & returned to PC for settling & removal with PC.



B. PHOSTRIP WITH ELUTRIATION STRIPPER (non-nitrifying system).

- Stripper (AN) "elutriated" with PE. Stripper overflow precipitated with lime (P) and settled. Chemical sludge disposed of separately. Overflow returned to PC.



C. PHOSTRIP FOR NITRIFYING ACTIVATED SLUDGE.

- Stripper (AN) is "protected" from NO_3^- in RAS by an anoxic denitrification basin (A).

RAS = Return Activated Sludge
WAS = Waste Activated Sludge
AN = Anaerobic Basin
A = Anoxic Basin
P = Reactor for Lime Addition
S = Lime Sludge Settler
R = Recycle Flow

I = Influent
E = Effluent
PC = Primary Clarifier
PS = Primary Sludge
PE = Primary Effluent
O = Aerobic Basin
C = Secondary Clarifier

Figure 20. Basic Flow Diagrams for Sidestream Biological Nutrient Control Processes (5).

taking place downstream of this added zone, any nitrate nitrogen contained in the return sludge will have a negative impact on phosphorus release in this zone. Some investigators believe that any nitrate nitrogen present in the anaerobic zone must first be reduced before fermentation of soluble substrate into the VFA's used by phosphorus-removing bacteria can begin. Others theorize that fermentation reactions can occur simultaneously with denitrification at some unspecified moderate level of nitrate nitrogen. It is universally acknowledged though that whichever of the above scenarios is correct, denitrification in the anaerobic zone does partially deplete the supply of soluble substrate that could otherwise be utilized in the fermentation products uptake/phosphorus release segment of the phosphorus removal cycle.

A three-zone or three-stage Bardenpho configuration, also known as the three-stage Phoredox process, is recommended by its proprietors where less stringent nitrogen removal is satisfactory. Whereas the five-stage Bardenpho process with effluent filtration has been designed to meet an effluent total nitrogen concentration of 3.0 mg/L (3), the three-stage alternative should not be expected to produce an effluent with less than 5 to 6 mg/L total nitrogen under the same conditions. Phosphorus removal efficiency with either alternative will depend, in large measure, on the SBOD-to-SP ratio in the influent wastewater.

A two-stage Phoredox process option is shown for situations where nitrification/denitrification is not desired. In this option, the second-stage aerobic zone is preceded only by a first-stage anaerobic zone with no intervening anoxic zone. The Anaerobic/Oxic (A/O) process is similar to the two-zone Phoredox concept except that the anaerobic and aerobic zones are divided into a number of equally-sized compartments (usually three in the anaerobic zone, four in the aerobic zone) to promote plug flow operation.

A modification of the basic A/O concept called the Anaerobic/Anoxic/Oxic (A^2/O) process is similar to the three-stage Phoredox process. The A^2/O process was conceived to provide dual phosphorus/nitrogen control. Like the A/O reactor, the several zones in the A^2/O reactor are tightly compartmentalized to create plug flow conditions.

The University of Capetown (UCT) process employs a three-stage reactor configuration with an added feature designed to protect the lead anaerobic zone from nitrate nitrogen intrusion. The return sludge is directed to the intermediate anoxic zone instead of the first-stage anaerobic zone. Nitrates contained in the return sludge are thereby reduced downstream of the anaerobic zone along with the mixed liquor recycled nitrates. Denitrified mixed liquor is then recycled from the anoxic zone to the anaerobic zone.

The above configurations are all considered mainstream biological phosphorus removal process options because the entire forward flow of a treatment plant must pass through both the phosphorus leaching and phosphorus uptake steps. In contrast, the PhoStrip process uses a sidestream flow regime for the phosphorus leaching (stripping) operation, with the forward flow passing only through the phosphorus uptake step.

The first two PhoStrip schematics in Figure 20 are designed for phosphorus removal only. The third schematic shows an anoxic basin incorporated in the return sludge feed line ahead of the sidestream stripper. The functions of the anoxic basin are to achieve partial denitrification in PhoStrip systems designed to nitrify and to protect the stripper from the introduction of nitrates via the return sludge.

The stripper supernatant is subjected to lime addition to precipitate the leached SP contained therein. The lime-phosphorus sludge can either be returned to the primary clarifier (Figure 20A) for co-settling and removal with the raw sludge or directed to a sidestream settler (Figures 20B and C) for subsequent separate disposal. In lieu of either a primary clarifier or a separate sidestream settler, a combination reactor-clarifier can be used to accomplish both lime coagulation and settling of the lime-phosphorus sludge in a single tank. Because the precipitated phosphorus is chemically rather than biologically insolubilized, it will not resolubilize during co-settling with raw sludge in the primary clarifier or during subsequent anaerobic or aerobic digester operations.

The return sludge solids in the stripper underflow, having been partly stripped of their intercellular-stored polyphosphates, are returned to the mainstream aeration tank along with that portion of the return sludge that was not processed through the stripper. This phosphorus-poor segment of the return sludge is now conditioned to biologically insolubilize enhanced quantities of SP from the mainstream flow.

Phosphorus release efficiency in the stripper can be increased by utilizing an elutriation stream. Possible elutriant sources include stripper tank underflow recycle (Figure 20A), primary effluent (Figures 20B and 20C), lime precipitation clarifier overflow, secondary effluent, and digester supernatant. The type and magnitude of elutriation flow selected is a site-specific decision based on wastewater characteristics and operational considerations. The decision sometimes is between a stream high in soluble organics and SP, such as or digester supernatant, and a stream low in both constituents, such as lime clarifier overflow or secondary effluent. In the first case, the presence of the soluble organics can expedite the release of phosphorus in the stripper through the production and assimilation of VFA's but at the risk of contaminating the return sludge with elutriant phosphorus. In the second case, the contamination risk is avoided, but the substrate-enhanced phosphorus release rate is forfeited.

North American Process Options--

In North America, the three process configurations from the above list employed to date are the PhoStrip process (without nitrificaiton/denitrification), the five-stage modified Bardenpho process, and the A/O process. More detailed flow diagrams of these three processes are given in Figure 21 as modified from Reference 3. Typical operating conditions, also modified from Reference 3, are summarized in Table 6 for all three processes plus the A/O process combined with nitrification/denitrification (A^2/O).

PhoStrip process--Key sidestream flow rates and other information are

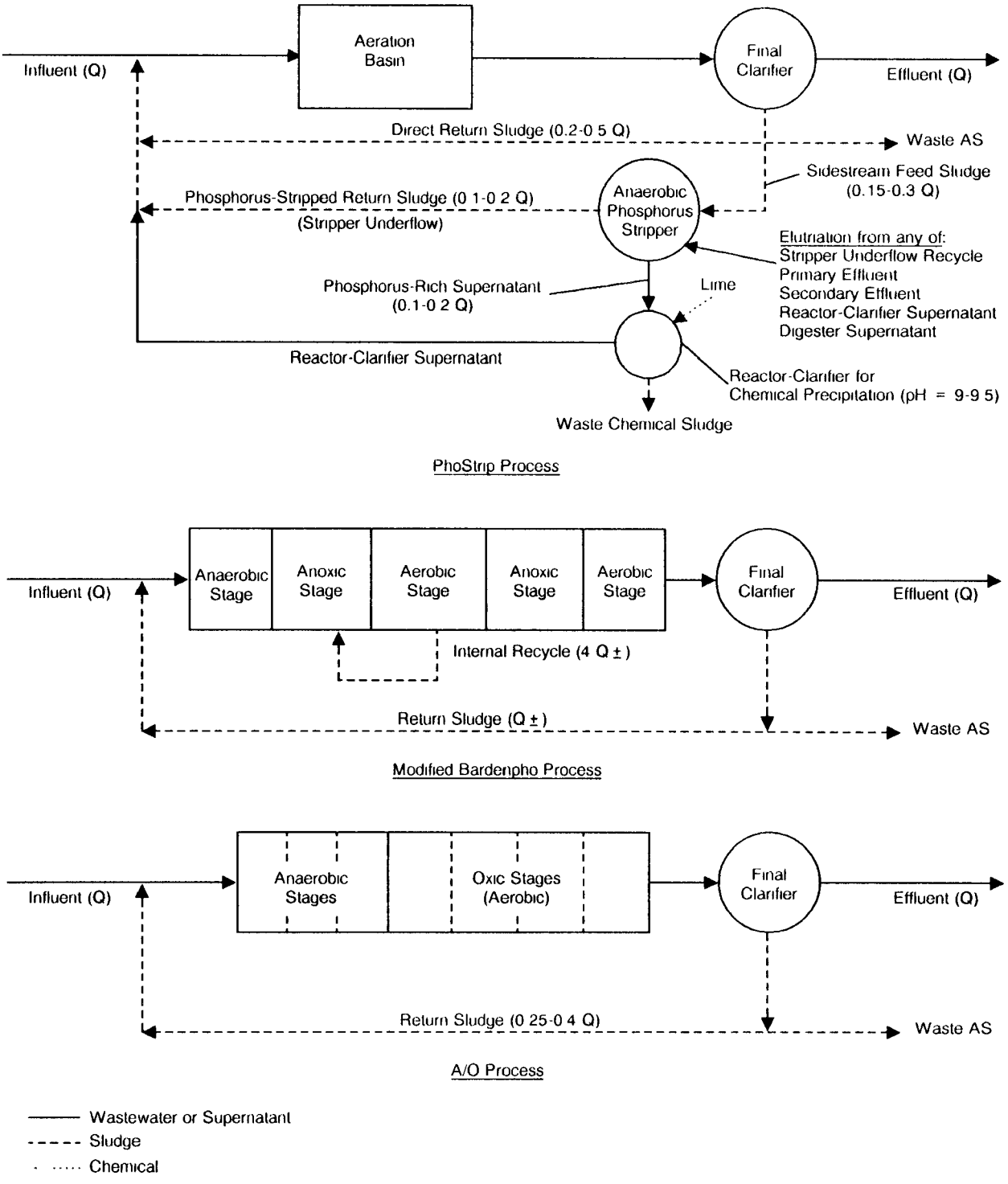


Figure 21. Flow Diagrams of Commercial Biological Phosphorus Removal Processes Marketed in North America (adapted from Reference 3).

TABLE 6. TYPICAL OPERATING CONDITIONS FOR COMMERCIAL BIOLOGICAL PHOSPHORUS REMOVAL PROCESSES MARKETED IN NORTH AMERICA (MODIFIED FROM REFERENCE 3)

PhoStrip		Modified Bardenpho		A/O		A^2/O	
Parameter	Value	Parameter	Value	Parameter	Value	Parameter	Value
AS System							
F/M, kg TBOD/ kg MLVSS/d	--[1]	F/M, kg TBOD/ kg MLVSS/d	0.1-0 2	F/M, kg TBOD/ kg MLVSS/d	0.2-0 7	F/M, kg TBOD/ kg MLVSS/d	0.15-0.25
SRT, days[2]	--[1]	SRT, days[2]	10-30	SRT, days[2]	2-6	SRT, days[2]	4-8
MLSS, mg/L	600-5,000	MLSS, mg/L	2,000-4,000	MLSS, mg/L	2,000-4,000	MLSS, mg/L	3,000-5,000
HDT, hr[3]	4-10	HDT, hr[3]		HDT, hr[3]		HDT, hr[3]	
		Anaerobic	1-2	Anaerobic	0 5-1.5	Anaerobic	0.5-1.5
		Anoxic 1	2-4	Aerobic	1.5-3	Anoxic	0 5-1.0
		Nitrification (Aerobic 1)	4-12			Nitrification	3.5-6 0
		Anoxic 2	2-4				
		Aerobic 2	0.5-1.0				
Stripper							
Feed, % of inf. flow	15-30	Return Sludge, % of inf. flow	100	Return Sludge, % of inf flow	25-40	Return Sludge, % of inf. flow	20-50
SRT, hr	5-20	Int. Recycle, % of inf. flow	400			Int Recycle, % of inf. flow	100-300
Sidewater Depth, m	6.1						
Elutriation Flow, % of stripper feed flow	50-100						
Underflow, % of inf. flow	10-20						
P Release, g P/g VSS	0.005-0.02						
Reactor-Clarifier							
Overflow Rate, $m^3/m^2/d$	48						
pH	9-9.5						
Lime Dosage, mg/L	100-300						

[1] Based on activated sludge system design.
[2] Average mass of solids in the system divided by average mass of solids wasted daily
[3] Nominal hydraulic detention time, volume divided by influent flow rate

superimposed on the PhoStrip process schematic (Figure 21). A key design feature is the SRT selected for the anaerobic stripper. The SRT, defined as the mass of solids in the stripper blanket divided by the mass of solids removed per day in the stripper underflow, can vary from 5 to 20 hr (3,4), with 8 to 12 hr being more typical (3). The sidestream feed sludge rate to the stripper controls the distribution of phosphorus that is removed chemically vs. that which is removed in the waste activated sludge. As previously mentioned, this distribution ratio typically approximates two-thirds via the chemical sludge route and one-third in the waste activated sludge.

Stripper supernatant is rich in SP with concentrations ranging from 15 to 100 mg/L (4). Lime dosages necessary to maintain reactor-clarifier pH values of 9.0 to 9.5 to effect precipitation of this SP load vary from 100 to 300 mg/L (3), depending on wastewater akalinity. From a chemical usage standpoint, lime is more economical than metal salts for phosphorus precipitation when the phosphorus stream is concentrated as it is in the stripper supernatant. In general, lime dosage is independent of phosphorus concentration, whereas iron or aluminum salts must be added on a metal ion-to-phosphorus molar basis. Consequently, lime consumption per unit of phosphorus removed decreases with increasing influent phosphorus concentration while metal salt consumption remains roughly the same (at least down to an effluent TP concentration of 1 mg/L) regardless of the influent phosphorus concentration. Metal salts, however, can be used in lieu of lime in the PhoStrip sidestream chemical reactor, if desired. Considering the extensive O&M problems often encountered with lime handling and feeding, metal salts may represent a more cost-effective choice in some cases despite their higher dose requirements.

Stripper surface area is sized to provide an SOR in the range of 16 to 24 $m^3/m^2/d$ (400 to 600 gpd/sq ft), a range representative of typical gravity thickener designs (4). In contrast, a much larger SOR can be used for the reactor-clarifier or separate chemical clarifier, normally about 48 $m^3/m^2/d$ (1200 gpd/sq ft). The stripper SOR must take into account the elutriation stream flow rate, which typically ranges from 50 to 100 percent of the stripper sludge feed rate.

This process should be able to reliably produce a 1-mg/L effluent TP without any type of effluent polishing (4). Attainment of an effluent TP concentration of 0.5 mg/L may require supplemental solids removal, such as effluent filtration (4). Meeting anything less than 0.5 mg/L effluent TP will probably involve some form of chemical polishing plus effluent filtration.

PhoStrip plants have been installed in activated sludge plants to date where the existing wastewater characteristics and operating conditions have fallen in the ranges listed below (4). Until additional information is developed to indicate otherwise, these characteristics and conditions should be considered prerequisites for retrofitting to the PhoStrip process.

Influent TBOD = 70 to 300 mg/L
Influent TP = 3 to 10 mg/L
Secondary clarifier oxidized nitrogen = 3 to 30 mg/L
Wastewater temperature = 10 to 30°C
Aeration tank HDT = 4 to 10 hr
MLSS = 600 to 5000 mg/L
F/M loading = 0.1 to 0.5 kg TBOD/kg MLVSS/d

The PhoStrip process is under license in North America to Biospherics, Inc., 4928 Wyaconda Road, Rockville, Maryland, 20852. A total of 13 PhoStrip systems have been constructed in North America to date, although five of them are no longer in operation. A fourteenth system is currently under construction. A list of these 14 PhoStrip systems along with their design capacities and current status is presented in Table 7.

Modified Bardenpho process--Because it is designed for highly efficient nitrogen removal as well as phosphorus removal, the range of total reactor HDT's shown in Table 6 for the modified Bardenpho process is much higher than for any of the other processes shown. The range of 9.5 to 23 hr is recommended by the vendor to cover the range of design wastewater temperatures encountered from the northern U.S. states and Canada to the southern U.S. states.

TABLE 7. PHOSTRIP SYSTEMS IN NORTH AMERICA AS OF SEPTEMBER 1987 (7)

Plant Site	Design Flow m^3/day	(mgd)
Operating Systems		
Lansdale, Pennyslvania	9,500	(2.5)
Adrian, Michigan	26,500	(7.0)
Tahoe-Truckee, California	28,000	(7.4)
Savage, Maryland	56,800	(15.0)
Southtowns, New York	60,600	(16.0)
Brockton, Massachusetts	68,100	(18.0)
Rochester, Minnesota	72,300	(19.1)
Reno-Sparks, Nevada	113,600	(30.0)
Systems No Longer in Operation		
Seneca Falls, New York	3,400	(0.9)
Lititz, Pennsylvania	13,200	(3.5)
Carpentersville, Illinois	18,900	(5.0)
Texas City, Texas	28,400	(7.5)
Amherst, New York	90,800	(24.0)
System Under Construction		
Ithaca, New York	37,900	(10.0)

The five-stage modified Bardenpho process is designed to achieve >90 percent nitrogen removal. Approximately 70 percent of the nitrate nitrogen produced in the system is removed in the first anoxic zone by instituting a 400-percent internal recycle of mixed liquor from the first aerobic zone to this zone. Any influent soluble substrate left over from the anaerobic zone aids the denitrification reaction (3). The remaining 20+ percent is removed in the second anoxic zone using endogenous destruction of mixed liquor solids to provide the necessary organic carbon to trigger denitrification (3,4).

The design SRT of the modified Bardenpho reactor (based only on the solids inventory in the aerobic and anoxic stages) is typically 10 to 20 days. Tank volumes in some designs have been increased to produce design SRT's of 20 to 30 days. The increased SRT results in increased in-process sludge stabilization, which may obviate the need for separate sludge digestion in the sludge handling cycle and avoid the possibility of phosphorus being recycled back to the reactor in digester supernatant (3).

Attainment of a 1-mg/L effluent TP with this process may require an occasional supplemental dose of metallic salt, particularly if the influent SBOD-to-SP ratio is low (4). Reliable production of an effluent with 0.5 mg/L TP or less will most likely necessitate a continuous polishing dose of metal salt combined in some instances with effluent filtration.

The license for marketing the modified Bardenpho process in North America is held by Eimco Process Equipment Co., 669 West Second South, Salt Lake City, Utah 84110. Nine modified Bardenpho systems are currently in operation in North America with six more under construction. These 15 systems are identified in Table 8 along with individual design capacities.

A/O and A^2/O processes--The key features of the A/O process are its short HDT and SRT and its relative high F/M loading rate compared to the other processes listed in Table 6. The A/O process produces more excess sludge than the modified Bardenpho process and higher phosphorus removal rates per unit of BOD removed. Unlike the 20- to 30day SRT modified Bardenpho process alternative, however, A/O systems must contend with the possibility of biologically-removed phosphorus reentering the reactor in digester supernatant recycle (3). A backup chemical feed system for adding polishing doses of metal salt to the aerobic zone, if needed, is recommended to meet effluent TP limits of 1 mg/L or less. Some locations may be able to attain a 1-mg/L effluent TP concentration most or all of the time without chemical polishing (4), but the element of risk will probably justify the backup feed system in most designs, particularly considering the relatively low capital cost of such a system.

The sequence of reactions that occur in the anaerobic and aerobic (oxic) stages of the A/O process is illustrated conceptually in Figure 22 (4). This conceptual schematic applies to any mainstream biological phosphorus removal approach provided nitrate nitrogen is not entering the anaerobic zone in the return sludge and/or via mixed liquor recycle. Figure 22 depicts concurrent SBOD uptake and SP release in the anaerobic zone followed by continued SBOD removal and cellular synthesis combined with "luxury" SP uptake in the aerobic zone.

TABLE 8. MODIFIED BARDENPHO SYSTEMS IN NORTH AMERICA AS OF SEPTEMBER 1987 (8)

Plant Site	Design Flow m^3/d	(mgd)
Operating Systems		
Orchard Development, Pennsylvania	800	(0.2)
Pluckemin, New Jersey	3,200	(0.85)
Palmetto, Florida	5,300	(1.4)
Payson, Arizona	6,400	(1.7)
Tarpon Springs, Florida	15,100	(4.0)
Easterly Orange County Sub-Regional, Florida	22,700	(6.0)
Kelowna, British Columbia	22,700	(6.0)
Fort Meyers (South Sub-Regional), Florida	45,400	(12.0)
Fort Meyers (Central Sub-Regional), Florida	45,400	(12.0)
Systems Under Construction		
Oldsmar, Florida	8,500	(2.25)
Cocoa, Florida	17,000	(4.5)
Easterly Orange County Sub-Regional (expansion), Florida	22,700	(6.0)
Rogers, Arkansas	25,400	(6.7)
Orlando, Florida	45,400	(12.0)
Springdale, Arkansas	59,000	(15.6)

When nitrogen removal is required, the A^2/O modification of the basic process is utilized. An anoxic zone, consisting of two or more baffled stages, is interposed between the anaerobic and aerobic zones. Nitrified mixed liquor is recycled from the last stage or compartment of the aerobic zone to the first stage of the anoxic zone. Nitrogen removal will not be as complete in this process as in the modified Bardenpho process due to the lack of second anoxic and aerobic zones. Using internal recycle rates of 100 to 300 percent of the influent flow, overall nitrogen removals of 40 to 70 percent can be expected (3).

The North American license holder of the A/O and A^2/O processes is Air Products & Chemicals, Inc., P.O. Box 538, Allentown, Pennsylvania 18105. Of the 15 full-scale A/O systems sold in North America to date, two are in operation and 13 are under construction. The list of North American A/O systems is summarized in Table 9 along with their design capacities.

Phosphorus Removal Performance

PhoStrip Process--

Basic design information and documented past phosphorus removal performance data for seven full-scale PhoStrip systems are summarized in Tables 10 and 11, respectively (3). Two of the systems, Seneca Falls, New York, and Amherst, New York, are among those PhoStrip units that are no longer in operation (see Table 7). The Seneca Falls project was a full-scale demonstration

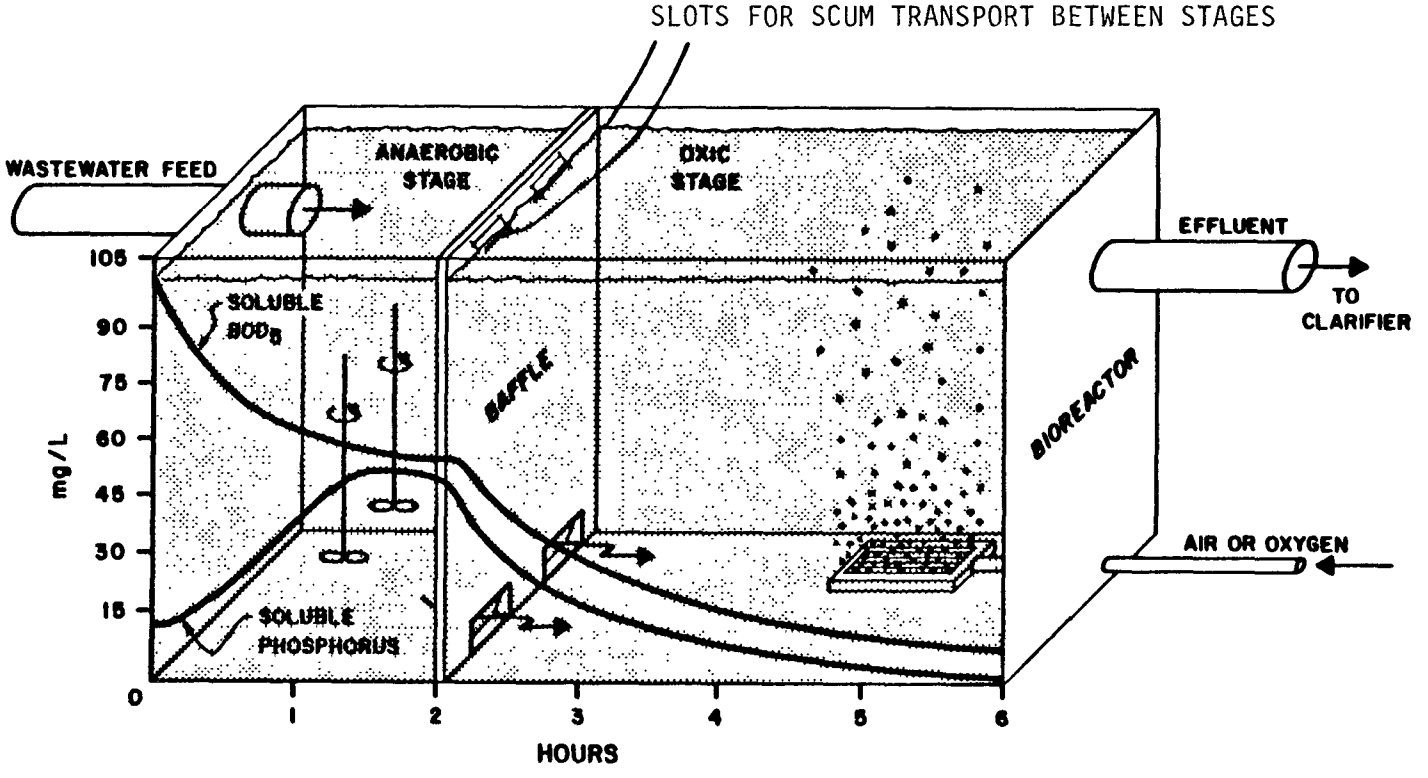


Figure 22. Conceptual Sequence of Anaerobic and Aerobic Reactions in the A/O Process (4).

TABLE 9. A/O SYSTEMS IN NORTH AMERICA AS OF SEPTEMBER 1987 (7)

Plant Site	Design Flow m^3/d	(mgd)
Operating Systems		
Pontiac, Michigan	13,200	(3.5)
Largo, Florida	56,800	(15.0)
Systems Under Construction		
Titusville, Florida	11,400	(3.0)
Montgomery, Pennsylvania	18,200	(4.8)
Warminster, Pennsylvania	30,300	(8.0)
Newark, Ohio	37,900	(10.0)
Fayetteville, Arkansas	41,600	(11.0)
Huron River Valley, Michigan	45,400	(12.0)
Rochester, New York	56,800	(15.0)
Springettsbury, Pennsylvania	56,800	(15.0)
York, Pennsylvania	98,400	(26.0)
Genesee County, Michigan	113,600	(30.0)
Lancaster, Pennsylvania	113,600	(30.0)
West Palm Beach, Florida	208,200	(55.0)
Baltimore, Maryland	265,000	(70.0)

that was scheduled to be terminated after completion of planned evaluation studies. At Amherst, where the influent TP concentration has dropped to lower-than-anticipated levels of 3 to 4 mg/L, two-point addition of ferric chloride has been found to be more effective than PhoStrip in removing phosphorus. Amherst also experienced mechanical problems with the PhoStrip lime feed system, which may have contributed to the City's decision to discontinue use of the process.

Four of the seven plants listed employ second-stage nitrification systems. Partial nitrification also occurs in the first-stage of the two-stage Savage, Maryland plant. With the exception of Savage, the nitrifying conditions did not impose nitrate nitrogen loads on the PhoStrip strippers during the data collection periods indicated in Table 11 because nitrification was taking place downstream of the stripper feed sludge takeoff point.

Primary clarification is provided at all of the plants shown except the Lansdale, Pennsylvania facility. Here, a 24-hr in-line flow equalization tank is used instead. Flow equalization is also provided at three other plants where primary clarification is practiced.

Four of the treatment plants were utilizing effluent filtration during their data collection periods. The effluent phosphorus data given in Table 11 for these four plants represent filtered effluent except for Savage. The effluent phosphorus data for Savage are for samples taken after the first-stage activated sludge system (3). The improvement noted in phosphorus

TABLE 10. BASIC DESIGN INFORMATION FOR FULL-SCALE PHOSTRIP SYSTEMS (3)

Parameter	Seneca Falls, N.Y.	Landsdale, Penn.	Adrian, Mich	Savage, Maryland	Southtowns, N.Y.	Amherst, N.Y	Reno-Sparks, Nevada
Design flow, m^3/d	3,400	9,500	26,500	56,800	60,600	90,900	113,700
Final eff. TP std , mg/L	1.0	2 0	1.0	0.3[1]	1.0	1.0	0.5[1]
Aeration by Oxygen or Air	A	A	A	A	O	O	A
Aeration mode	Complete Mix	Plug Flow	Conv.	Plug Flow or Step Feed	Plug Flow	High Rate	Plug Flow
1- or 2-stage sec. treatment	1	2	2	2	1	2	1
Equalization	No	Yes	Yes	Yes	No	Yes	No
Final filtration	No	No	Yes	Yes	Yes	Yes	Planned
Sludge handling	Thickening, Anaer. Dig.	Thickening, Vac Filt	Thickening, Anaer. Dig	Thickening, Anaer Dig	Filter Press, Incineration	DAF Thickening	Anaer. Dig.
Strippers, no	1	1	1	2	4	2	5
Reactor-Clarifiers or Mixer/Flocculators, no.	MF 1	RC 1	MF 1	RC 2	RC 4	RC 1	MF 2
Elutriation source[2]	SR	RC/SEC	PRI	RC	RC	RC	SR

[1] With final final filtration; chemical polishing available but not utilized
[2] Sludge Recycle elutriation; Reactor-Clarifier overflow elutriation; PRImary effluent supplement; SECondary effluent.

TABLE 11. PERFORMANCE DATA SUMMARY FOR FULL-SCALE PHOSTRIP SYSTEMS (3)

Plant	Design Flow	Startup Date	Data Period	Total Phosphorus, mg/L						Notes
				Influent Averages			Effluent Averages			
	m^3/d		mo.	min. mo.	ave. mo.	max. mo.	min. mo.	ave mo.	max. mo	
Seneca Falls, N.Y.	3,400	1973	1		6 3			0.6		Full-scale demonstration
Landsdale, Penn	9,500	1982	12	4.0	5 2	6 4	0.6	1 2	2.0	
Adrian, Michigan	26,500	1981	11	3.4	4.4	5.3	< 0 1	0 4	0.6	Excludes periods of upset due to other plant problems
Savage, Maryland	56,800	1982	6	5 7	8 1	9.3	0.5	1 2	1 7	
			1		6 6			1.7		July 1984
			1		7.0			0 5		April 1985
Southtowns, N Y.	60,600	1982	4	2.3	3 2	4.1	0.3	0 5	0.9	
Amherst, N.Y.	90,900	1982	12	2.9	5.2	14 3	0 4	1.3	2.5	
Reno-Sparks, Nevada	113,700	1981	4		7.0-7.3			0.8-1.1		9/82-12/82

removal performance for Savage in April 1985 resulted from changing the aeration tank operating mode from step feed to plug flow and replacing part of the reactor-clarifier overflow elutriation stream with stripper underflow recycle. The modified elutriation scheme was credited with increasing stripper overflow orthophosphorus concentration from 7.2 mg/L in July 1984 to 17.6 mg/L in April 1985.

As indicated in Table 11, effluent TP for all seven PhoStrip systems for the data periods considered ranged from 0.4 to 1.7 mg/L. The average effluent TP concentration for the seven plants was 0.9 mg/L. It should be remembered, though, that performance data for three of the plants were based on filtered effluents.

Modified Bardenpho Process--

Basic design parameters for the first two full-scale modified Bardenpho systems in North America, Palmetto, Florida (United States), and Kelowna, British Columbia (Canada), are given in Table 12 (3). A 1-year performance summary (April 1981 through March 1982) for Palmetto is presented in Table 13 (3). Two years of available performance data (January 1983 through December 1984) are summarized in Table 14 for Kelowna (3).

The total reactor design HDT for Kelowna is approximately twice as long as for Palmetto and the design SRT 1.5 to 2.0 times that of Palmetto because of the substantially colder wastewater temperatures experienced at Kelowna (3). The secondary clarifier design SOR and the polishing filter design application rate are also much more conservative for Kelowna.

Required effluent (filtered) limits for Palmetto are 5 mg/L for TBOD, 5 mg/L for TSS, 3 mg/L for total nitrogen, and 1 mg/L for TP (3). The corresponding filtered effluent limitations for Kelowna are 8, 7, 6, and 2 mg/L, respectively (3).

Palmetto more than met its required effluent limits for TBOD, TSS, and total nitrogen for the year of data shown in Table 13. For effluent TP, however, the 1-mg/L limit was met only during 4 of the 5 months when a small polishing dose of alum was added prior to secondary clarification. The somewhat limited phosphorus removal capacity exhibited was attributed to a relatively weak influent wastewater. The influent TBOD-to-TP ratio, which averaged 18.8 for the year, was not especially low. It is not known, however, what the primary effluent SBOD-to-SP ratio was during this period. This soluble ratio would be the more critical factor in determining the phosphorus release rate in the first-stage anaerobic zone.

The less stringent effluent limitations at Kelowna were reportedly met throughout the 2-year data period described in Table 14. This period included operation in a two-train mode, a single-train mode, with thickener supernatant feeding, and with the return sludge being split between the anaerobic zone and the first anoxic zone (3). With the influent load only at about 54 percent of the total plant design level during this period, the single-train operation resulted in a load equal to approximately 110 percent of design. Effluent requirements were also reportedly met during the 5-1/2

TABLE 12. BASIC DESIGN INFORMATION FOR FULL-SCALE MODIFIED BARDENPHO SYSTEMS (3)

Parameter	Palmetto, Florida	Kelowna, British Columbia
Startup date	10/79	5/82
Flow, m^3/d	5,300	22,700
Detention time, hr (no. cells)		
Anaerobic zone	1.0 (1)	2.0 (1)
Anoxic 1 zone	2.7 (1)	4.0 (4)
Nitrification zone	4.7 (1)	9.0 (9)
Anoxic 2 zone	2.2 (1)	4.0 (4)
Reaeration zone	1.1 (1)	2.0 (2)
Total	11.6 (5)	21.0 (20)
SRT, days	20	30-40
MLSS, mg/L	3,500	3,000
Temperature, °C	18-25	9-20
Sec. clarifier application rate, $m^3/m^2/d$	22.3	14.0
Polishing filter application rate, $m^3/m^2/d$	93.7	23.4
Primary treatment	No	Yes
Biological sludge handling	Drying Beds	DAF Thickening, Composting

month single-train period of operation (3).

Both plants utilize supplementary fermentation schemes to increase the concentration of fermentation products (VFA's) in the anaerobic zone (3). At Palmetto, return sludge is directed to the primary clarifier rather than the anaerobic zone to provide additional fermentation time. At Kelowna, this objective is accomplished by holding raw solids in the primary sludge gravity thickener for an extended period and recycling the thickener supernatant to the anaerobic and/or first anoxic zones.

Neither plant employs anaerobic or aerobic sludge digestion because of potential recycling of resolubilized phosphorus in the supernatant of either unit process back to the main plant flow (3). Sludge handling and disposal techniques are used that do not result in recycle sidestreams high in SP.

TABLE 13. PERFORMANCE DATA SUMMARY (1981-82) FOR PALMETTO, FLORIDA MODIFIED BARDENPHO SYSTEM (3)

Parameter	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	March
Influent												
Flow, m^3/d	3,200	3,000	3,500	3,600	5,500	5,900	3,700	3,300	3,200	3,600	3,700	4,500
TBOD, mg/L	164	159	124	104	74	67	113	157	182	160	163	150
TSS, mg/L	155	157	144	112	76	76	116	160	182	141	167	128
Temperature, °C	25	27	29.5	30.5	29 5	29	28	27	24	23	23	23
TKN, mg/L	31.8	40.8	30.1	25.0	19.7	21 9	28.1	40.0	38.2	37.7	42.4	32.4
NH_4-N, mg/L	25.0	25.2	20.4	18.7	12.7	12.7	17.8	22.6	27.2	28.0	25.8	23.7
Total P, mg/L	9.2	6.4	7.0	5.6	4.1	4.9	6.3	8.5	8.8	8.7	8.0	6.6
Ortho P, mg/L	6 5	6.1	5.3	4.5	2.8	3 5	4 7	5 9	5.9	5.4	5.2	4.4
Alkalinity, mg/L	174	169	156	154	143	140	144	171	198	191	201	187
Filtered effluent												
TBOD	2	1	1	1	1	1	1	1	1	1	1	1
TSS, mg/L	3	2	2	2	2	1	2	2	2	1	2	3
Total N, mg/L	2.1	2.1	1.9	2.8	2.0	1.7	1.9	2.1	2.5	2.7	2.6	2 8
NO_3-N, mg/L	1.0	1.3	1.0	1.9	1.2	1.1	1.1	1.3	1.5	1.9	1.8	2.1
NH_4-N, mg/L	0 4	0.3	0.3	0.2	0.2	0.2	0.2	0.2	0 4	0.3	0.1	0.2
Total P, mg/L	2.5	3.4	2.6	1.8	1.5	1.2	1.1	0.7	1.6	0.6	0.8	0.9
Ortho P, mg/L	2.2	1.4	2.5	1.7	1.1	1.3	0.9 *	0.7 *	1.0	0.5 *	0.7 *	0.8 *

* Minimal alum dose applied prior to secondary clarification.

Both systems use submerged turbines for aeration. Kelowna also uses them for mixing without air supply in the anaerobic and anoxic zones (3). Fine bubble diffusers have been employed for aeration at Payson, Arizona, and the Carrousel oxidation ditch process has been used in modified Bardenpho designs at Fort Meyers and Orange County, Florida.

A/O Process--

The only two full-scale A/O systems in operation to date are located at Largo, Florida, and Pontiac, Michigan (see Table 9). Both of these A/O systems are preceded by primary clarification. Neither is followed by effluent filtration. An existing contact stabilization system was retrofitted to an A/O system at the Largo facility (3). At Pontiac, two of four existing plug flow activated sludge trains were converted to A/O trains to afford a side-by-side comparison of biological phosphorus removal with conventional activated sludge treatment (3).

The Largo reactor was subdivided into 10 stages with a total design HDT of 4.1 hr (3). When operated to remove phosphorus, nitrify, and partially denitrify, the reactor sequence consists of three anaerobic stages, two anoxic stages, and five aerobic stages with mixed liquor recycled from the fifth aerobic stage to the first anoxic stage. When the City's objective is to remove phosphorus only, the SRT is lowered by increasing the activated

TABLE 14. PERFORMANCE DATA SUMMARY FOR KELOWNA, BRITISH COLUMBIA MODIFIED BARDENPHO SYSTEM BASED ON CUMULATIVE FREQUENCY PLOTS (3)

Parameter	Median	Lower 5%	Upper 5%
Influent (1/83-12/84)			
Flow, m^3/d	12,400	10,400	10,000
COD, mg/L	195	150	275
TKN, mg/L	24.5	19.0	33.5
NH_4-N, mg/L	17.5	15.0	21.1
Total P, mg/L	4.5	3.3	5.8
Ortho P, mg/L	3.8	3.0	4.3
Final Effluent (Filtered) (1/83-12/84)			
TKN, mg/L	1.5	0.2	1.8
NO_3-N, mg/L	1.8	0.8	4.2
NH_4-N, mg/L	0.1	<0.1	6.0
Total P, mg/L	0.8	0.2	1.8
Ortho P, mg/L	0.77	0.15	2.25
Final Effluent - 1 Train (5/22/84-11/9/84)			
Flow, m^3/d	14,000	12,000	17,000
NO_3-N, mg/L	2.0	1.2	3.4
NH_4-N, mg/L	<0.1	<0.1	0.75
Ortho P, mg/L	1.1	0.08	1.75

sludge wasting rate and the two anoxic stages are operated as additional anaerobic stages.

Table 15 summarizes 5 months of Largo A/O performance data for the non-nitrifying mode and 6 months when the system was nitrifying (3). As would be expected, effluent TP and orthophosphorus were higher for the nitrifying operational mode.

Average operating conditions for an 8-1/2 month period spanning parts of 1984 and 1985 are presented for the Pontiac A/O system in Table 16 (3). The system was operated at various times to fully nitrify and at other times to partially nitrify. Anoxic cells for denitrification were not utilized so the full impact of nitrate nitrogen in the return sludge was imposed on the anaerobic zone. Performance for four operating phases during the above 8-1/2 month period are summarized in Table 17 (3).

Anaerobic digester supernatant was returned to the A/O system during

TABLE 15. PERFORMANCE DATA SUMMARY FOR LARGO, FLORIDA A/O SYSTEM (3)

Parameter	Non-Nitrification*	Nitrification*
Data period	2/81 - 6/81	9/81 - 2/82
TBOD, mg/L	7 (6-8)	5 (4-7)
TSS, mg/L	10 (8-13)	18 (10-22)
Total P, mg/L	1.4 (1.2-1.5)	1.7 (1.3-2.2)
Ortho P, mg/L	0.6 (0.5-0.8)	1.0 (0.5-2.0)

*Data ranges are shown in ( ).

Phases II and IV. Minimal SP concentrations were measured in the digester supernatant; consequently, no adverse impact of the supernatant recycle on A/O system performance could be detected. The exact reason why more phosphorus was not solubilized during anaerobic digestion is not clear. One explanation offered by the investigators is the formation of a magnesium ammonium phosphate precipitate in the digesters, but more research is needed to verify this hypothesis (3,4).

Effluent TP at Largo was greater than 1.0 mg/L for both modes of operation, while at Pontiac, effluent TP was well below 1.0 mg/L for all four phases documented. Probable reasons for this difference were the lower influent TP levels at Pontiac [3.0 to 4.1 mg/L vs. 7.0 to 9.4 mg/L (3)] and Pontiac's higher influent TBOD-to-TP ratios [33 to 39 vs. approximately 12 (3)]. The high influent organics-to-phosphorus ratios at Pontiac were apparently sufficient to compensate for the soluble substrate depleted in the anaerobic zone by the return sludge nitrate nitrogen.

The Pontiac 13,200-m^3/day (3.5-mgd) A/O retrofit was accomplished at a capital expenditure of $57,000 in 1984 dollars (4). Wooden baffles were utilized to divide the aeration tanks into the desired stages. Mixing in the anaerobic stages was achieved with side-mounted submersible mixers after first plugging the existing diffuser lines in those stages (3). The normal license fee was waived for Pontiac beacause the project was a U.S. EPA-sponsored demonstration (4).

Operationally Modified Activated Sludge Plants--

Existing activated sludge plant operating patterns can be modified to simulate biological phosphorus removal performance achieved by proprietary mainstream processes. This modification is effected by turning off the aeration devices in the front segment of the aeration tank to create the anaerobic fermentation conditions essential to initial soluble organics uptake and SP release.

Modifications as described above were implemented at the Reedy Creek, Florida, and DePere, Wisconsin plants (3). At the Reedy Creek plant, which serves the Walt Disney World resort complex at Lake Buena Vista, the air supply to the first third of each of four plug flow aeration basins was

TABLE 16. OPERATING DATA SUMMARY FOR PONTIAC A/O SYSTEM (3)

Parameter	Value
Operating dates	7/13/84 - 3/31/85
Average flow, m^3/d	12,200
Average HDT, hr (no. cells)	
Anaerobic zone	1.8 (3)
Aerobic zone	6.7 (4)
SRT, days	16-24
Temperature, °C	10-17
Primary treatment	Yes
Biological sludge handling	Anaerobic Digestion

TABLE 17. PERFORMANCE DATA SUMMARY FOR PONTIAC A/O SYSTEM (3)

Parameter	Phase I	Phase II*	Phase III	Phase IV*
Influent				
Flow, m^3/d	11,300	10,800	12,070	14,680
TBOD, mg/L	110	137	143	112
SBOD, mg/L	65	65	87	65
NH_4-N, mg/L	15.2	17.8	16.1	18.5
Total P, mg/L	3.2	4.1	3.7	3.0
Soluble P, mg/L	1.9	2.2	2.2	1.6
Temperature, °C	17	16	11	10
Reactor				
MLSS, mg/L	2,820	2,410	2,340	2,360
MLVSS, mg/L	1,800	1,670	1,640	1,590
SRT, days	24	21	19	16
Effluent				
TBOD, mg/L	6.2	9.4	12.9	12.7
SBOD, mg/L	1.8	3.0	2.6	2.0
NH_4-N, mg/L	0.9	2.8	5.9	4.5
NO_3-N, mg/L	10.4	11.6	6.7	8.8
Total P, mg/L	0.8	0.7	0.4	0.7
Soluble P, mg/L	0.7	0.6	0.3	0.5
TSS, mg/L	6	7	8	10
VSS, mg/L	4	4	5	6

*Anaerobic digester supernatant returned during these phases.

turned off. No interstage baffles were installed, and backmixing from the aerated section was adequate to keep solids in suspension in the anaerobic zone. The stabilization basin of a contact stabilization system was converted to an anaerobic zone at DePere. Existing submerged turbine aerators without air supply were utilized for mixing. The complete mix contact tank was used for the second-stage aerobic zone.

Both plants were completely nitrifying during a 3-month test period from June through August 1984. Denitrification of return sludge nitrates was also occurring in both plant's anaerobic zones. Mixed liquor recycle from the aerobic zone to the anaerobic zone to increase nitrogen removal was not practiced at either plant. Operating and performance data for the above 3-month test period are summarized in Table 18. Excellent phosphorus removal was obtained in both plants.

COMPARISON OF CHEMICAL AND BIOLOGICAL PHOSPHORUS REMOVAL

Economics Standpoint

Cost estimates for retrofitting existing wastewater treatment plants vary widely depending on the assumptions made in developing the estimates. For chemical phosphorus removal, the handling and disposing of the additional sludge generated by mineral addition is potentially the most significant cost in the entire estimate and, at the same time, the most difficult to generalize. If an existing plant is operating below design load and the sludge processing facilities have excess capacity available, only minor increases in retrofit capital and operation and maintenance (O&M) costs may be involved. Conversely, if the existing plant is fully loaded or overloaded, considerable increases in capital and O&M costs may result from retrofitting to chemical phosphorus removal.

For biological phosphorus removal, the degree to which existing plant treatment facilities can be incorporated in a retrofit design can significantly affect the costs of conversion. This factor is usually of more importance in retrofitting to mainstream biological phosphorus removal schemes than to the PhoStrip process. On the other hand, the PhoStrip process will produce significantly more additional sludge to handle and dispose of than the mainstream process options. License fees for proprietary biological nutrient control processes are generally determined on a case-by-case basis and can have a substantial impact on overall cost effectiveness.

The applicable effluent TP limitation is another important consideration in estimating costs for a phosphorus removal retrofit. For the mainstream biologically-based processes, achieving any effluent TP concentration below 1.0 mg/L, and in some cases including 1.0 mg/L, will typically require effluent polishing, either in the form of a small dose of metal salt supplement or granular media effluent filtration or both. The PhoStrip process would not normally be expected to require effluent polishing except for effluent TP requirements of 0.5 mg/L or less. Likewise, conventional chemical addition should not require effluent polishing (filtration) except at TP residual levels of 0.5 mg/L or less.

TABLE 18. OPERATING AND PERFORMANCE DATA SUMMARY FOR OPERATIONALLY MODIFIED ACTIVATED SLUDGE PLANTS (3)

Parameter	Reedy Creek	DePere
Operating Data		
Design flow, m^3/d	22,700	53,750
HDT, hr		
Unaerated zone	3.0	7.5
Aerated zone	6.0	15.0
Sec. clar. overflow rate, $m^3/m^2/d$	14.7	17.9
SRT, days	7.2	10.6
MLSS, mg/L	2,100	3,000
Return sludge ratio	0.59	0.81
Primary treatment	Yes	No
Sludge handling	DAF Thickening, Aerobic Digestion, Land Spreading	DAF Thickening, Filter Press, Incineration
Performance Data		
Influent		
TBOD, mg/L	155	150
SBOD, mg/L	85	86
Total P, mg/L	6.7	5.1
Ortho P, mg/L	5.3	1.9
Effluent		
TBOD, mg/L	3	7
TSS, mg/L	13	7
Total P, mg/L	0.9	0.3
Ortho P, mg/L	0.4	0.1
NH_4-N, mg/L	0.7	1.4

The authors of the CBDB Handbook (4) prepared cost estimates for retrofitting to chemical phosphorus removal (1986 U.S. dollars). They chose not to develop generalized cost estimates for retrofitting to biological phosphorus removal because of the uncertainties involved. The chemical retrofit estimates are based on the addition of alum and polymer and are specific for price quotations in the CBDB. These estimates were generated for four different effluent TP limitations (2.0, 1.0, 0.5, and 0.2 mg/L) and two influent TP ranges (6 to 10 mg/L and 3 to 6 mg/L). The costs of handling and disposing of the chemical phosphorus sludge produced and for possible effluent polishing at the lower effluent TP limits were not considered, i.e., the estimates include the costs associated with the dosing of chemicals only.

Capital cost estimates for dosing alum and polymer in the CBDB are shown in Table 19 (4). Estimated chemical costs are given in Table 20 (4). As facilities is a small fraction of the total capital cost of a conventional activated sludge plant. Estimated chemical costs escalate dramatically as the required effluent TP level becomes more stringent. The ranges of estimated chemical costs also become increasingly broad with decreasing allowable effluent TP, reflecting the higher level of dependence of chemical dose requirements on wastewater characteristics and other site-specific conditions at low effluent TP limitations.

More extensive phosphorus removal retrofit cost estimates were made by the authors of Reference 4 to define the technical and economic feasibilities of modifying existing plants in Canada for this purpose. Estimates were prepared for metal salt addition using ferric chloride and the modified Bardenpho, UCT, A/O, and PhoStrip biological processes. Three existing plant configurations were considered: primary treatment, conventional activated sludge (CAS), and extended aeration. Estimates were generated for two different effluent TP limitations (1.0 and 0.3 mg/L).

For brevity, only the results of the estimates dealing with the CAS configuration will be summarized in this paper. In addition, since no full-

TABLE 19. ESTIMATED CAPITAL COSTS FOR STORAGE AND DOSING EQUIPMENT FOR CHEMICAL PHOSPHORUS REMOVAL IN THE CBDB (4)#

	Effluent TP			
Plant Size	2.0 mg/L	1.0 mg/L	0 5 mg/L	0 2 mg/L
	$	$	$	$
Influent TP: 6-10 mg/l				
< 380 m^3/d	34,000	34,000	34,000	41,000
380-3,800 m^3/d	54,000	54,000	54,000	87,000
> 3,800-18,900 m^3/d	130,000	130,000	145,000	185,000
> 18,900-37,800 m^3/d	170,000	170,000	170,000	200,000
> 37,800 m^3/d *	213,000	213,000	213,000	250,000
Influent TP: 3-6 mg/L				
< 380 m^3/d	34,000	34,000	34,000	41,000
380-3,800 m^3/d	34,000	54,000	54,000	79,000
> 3,800-18,900 m^3/d	115,000	115,000	120,000	185,000
> 18,900-37,800 m^3/d	160,000	160,000	170,000	200,000
> 37,800 m^3/d *	200,000	200,000	213,000	250,000

To put these capital costs in perspective, the approximate capital costs for conventional secondary treatment plants without phosphorus removal utilizing the activated sludge process, vacuum filter sludge dewatering, and landfilling of dewatered sludge are.

Design Flow	Capital Cost
380 m^3/d	$2,400,000
3,800 m^3/d	$8,000,000
18,900 m^3/d	$18,600,000
37,800 m^3/d	$29,300,000

* Capital cost estimates of 25 percent above the 18,900-37,800 m^3/d estimates are recommended across the board for plant sizes above 37,800 m^3/d

TABLE 20. ESTIMATED CHEMICAL COSTS FOR CHEMICAL PHOSPHORUS REMOVAL IN THE CBDB (4)

Influent TP (mg/L)	Effluent TP Limitation (mg/L)	Unit Chemical Cost ($/1000 m^3)	Annual Chemical Cost ($/yr/1000 m^3)
6 - 10	2.0	18 - 22	6550 - 8050
	1.0	22 - 27	8050 - 9850
	0.5	27 - 38	9850 - 13,850
	0.2	36 - 92	13,150 -33,600
3 - 6	2.0	11 - 14	4000 - 5100
	1.0	13 - 17	4750 - 6200
	0.5	16 - 23	5850 - 8400
	0.2	22 - 57	8050 - 20,800

scale UCT systems have yet been installed in North America, only the modified Bardenpho, A/O, and PhoStrip systems are included in the CAS cost estimate summaries presented herein.

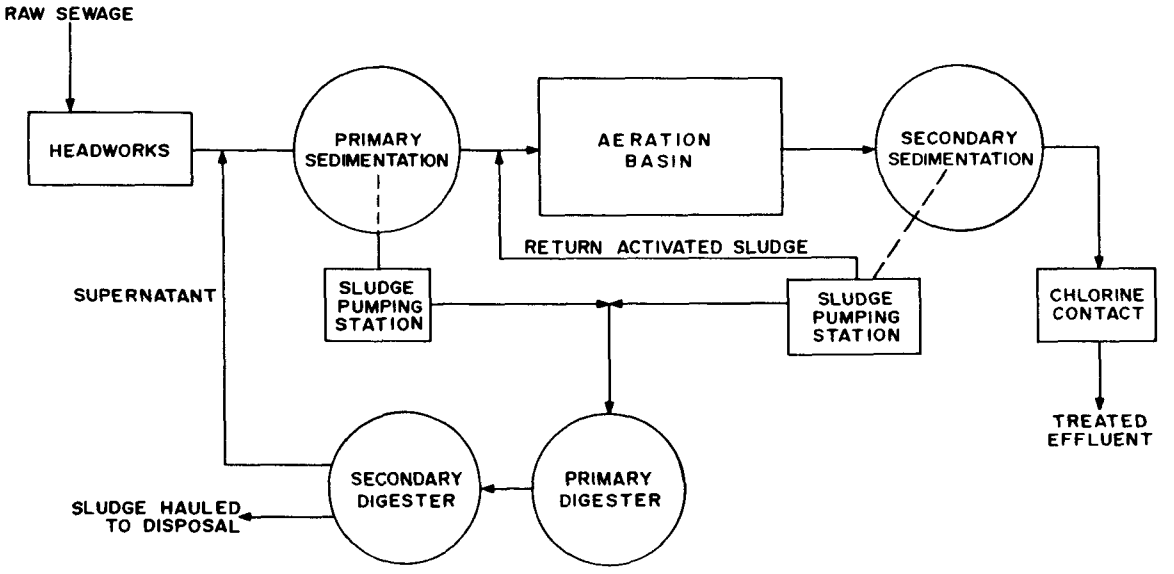
A flow diagram of the model CAS plant chosen for the Canadian estimates is illustrated in Figure 23 along with selected design criteria (5). This was the basic flowsheet used for preparing chemical addition and biological phosphorus removal retrofit estimates. Superimposition of the modified Bardenpho, A/O, and PhoStrip retrofit alternatives on this flowsheet is shown in Figures 24, 25, and 26, respectively (5).

For the mineral addition option, Reference 5 assumes ferric chloride doses of 8 and 14 mg Fe^{3+}/L, respectively, for the 1.0- and 0.3-mg/L TP effluent limits. Increases in total plant sludge generation of 26 and 42 percent were also assumed for the 1.0- and 0.3-mg/L TP effluent objectives, respectively. To meet the 0.3-mg/L TP effluent limit, effluent filtration was assumed to be necessary for the mineral addition alternative. No effluent polishing provisions were incorporated in the estimates to attain the 1.0-mg/L TP effluent limit.

For the mainstream modified Bardenpho and A/O biological phosphorus removal retrofit estimates, either supplemental ferric chloride dosing or dual-media effluent filtration was assumed to be necessary to meet the 1.0-mg/L TP residual and both supplemental chemical dosing and dual-media effluent filtration to achieve an effluent of 0.3 mg/L TP. For the 1.0-mg/L TP effluent goal, the assumption was made that ferric chloride would be added (dosing rate unspecified in Reference 5) only during effluent excursions above 1 mg/L TP. Ferric chloride at a dosing rate of 6 mg Fe^{3+}/L, equivalent to the difference in the selected chemical dosages to achieve effluent TP limits of 1.0 and 0.3 mg/L for the mineral addition option, was assumed for the 0.3-mg/L TP effluent goal. No effluent polishing requirements were assumed to be necessary for the PhoStrip process to meet

PLANT TYPE : CONVENTIONAL ACTIVATED SLUDGE

PROCESS FLOW SHEET



Design Criteria

Headworks:
- Manual Bar Screen
- Aerated Grit Chamber
- Comminutor
- Parshall Flume

Primary Clarifier:

Surface Overflow Rate @ Peak Flow ($m^3/m^2 \cdot d$)	80-120
Sidewater Depth (m)	3.0-4.6

Aeration System:

Organic Loading Rate (g BOD/$m^3 \cdot h$)	13-30
F/M (d^{-1})	0.2-0.5

Secondary Clarifier:

Surface Overflow Rate @ Peak Flow ($m^3/m^2 \cdot d$)	35
Sidewater Depth (m)	3.6-4.6

Anaerobic Digestion:

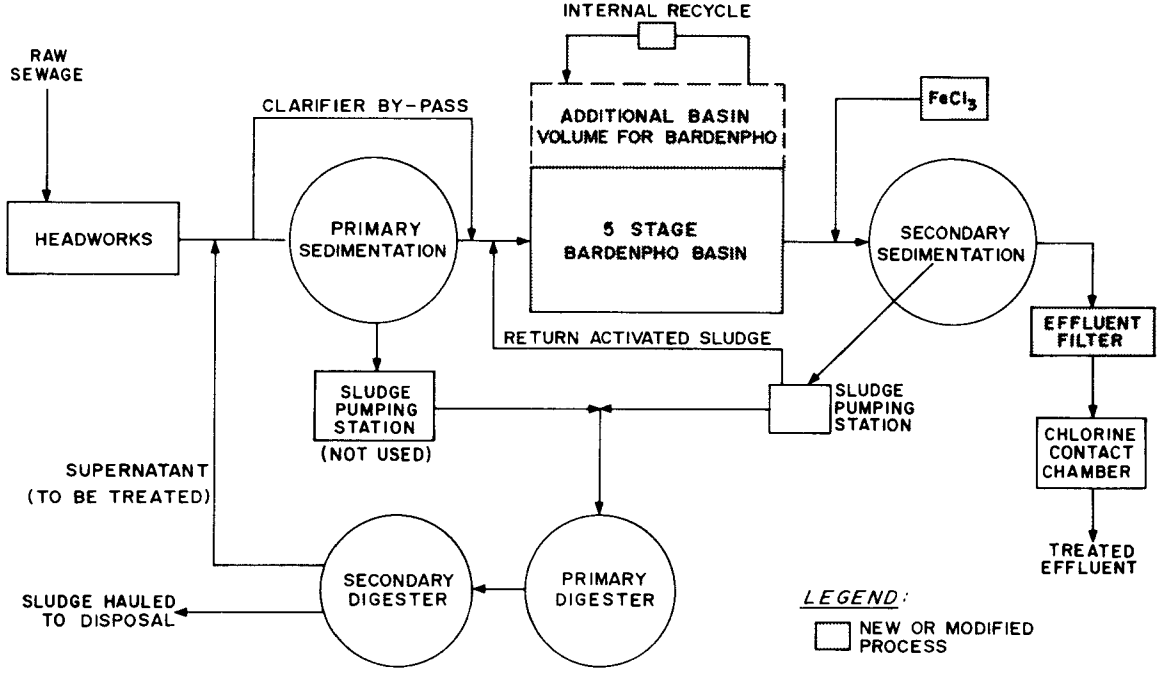
Volatile Solids Loading (kg VS/$m^3 \cdot d$)	0.65-1.6
Volatile Solids Destroyed (%)	50

Chlorine Contact:
- 30 min HRT
- 8-15 mg/L Chlorine Dosage

Figure 23. Flow Diagram and Design Criteria for Model CAS Plant Selected For Canadian Biological Phosphorus Removal Retrofit Cost Estimates (5).

PLANT TYPE : CONVENTIONAL ACTIVATED SLUDGE

PROCESS FLOW SHEET



Retrofit Design Criteria

Bardenpho Basin - 5 Stage

Stage 1:	Anaerobic	-	1.0 h HRT
Stage 2:	Anoxic	-	3.4 h HRT
Stage 3:	Aerobic	-	4.5 h HRT
Stage 4:	Anoxic	-	0.5 h HRT
Stage 5:	Aerobic	-	0.5 h HRT
	Total		9.9 h HRT

Stage 2 HRT equals model plant HRT.

Internal Recycle Pumping Station

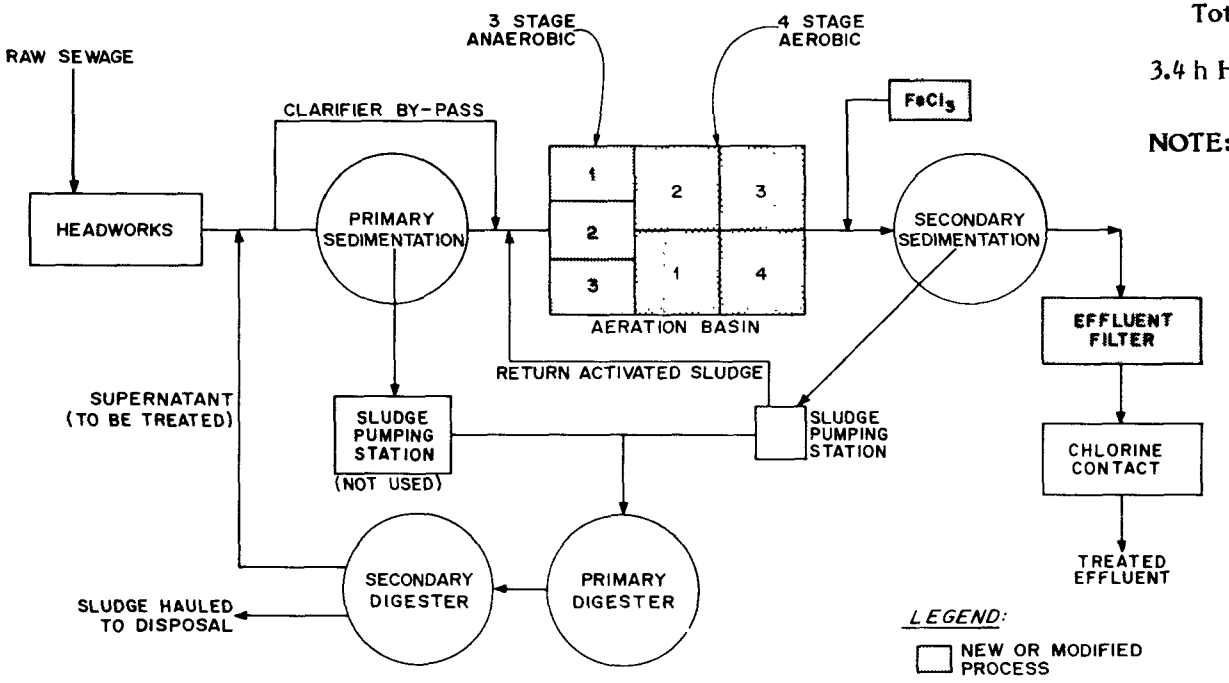
- Stage 3 to Stage 2 @ 400% of Average Flow

NOTE: $FeCl_3$ or filter required to achieve 1.0 mg/L TP. $FeCl_3$ and filter required to achieve 0.3 mg/L TP.

Figure 24. Flow Diagram and Design Criteria for Retrofitting Modified Bardenpho Process to Canadian Model CAS Plant (5).

PLANT TYPE : CONVENTIONAL ACTIVATED SLUDGE

PROCESS FLOW SHEET



Retrofit Design Criteria

A/O Basin

- 3 Stage Anaerobic HRT	- 1.2 h HRT
4 Stage Aerobic HRT	- 2.2 h HRT
Total	3.4 h HRT

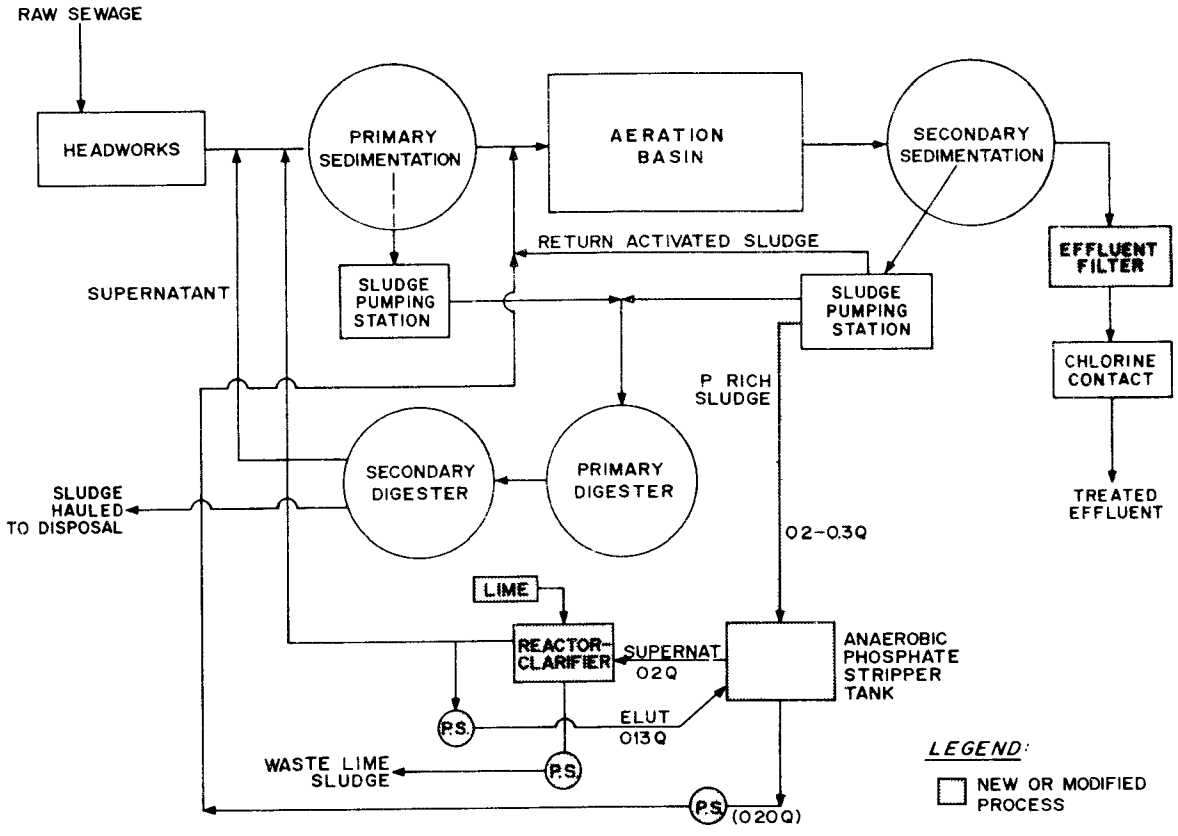
3.4 h HRT equals HRT of model plant.

NOTE: $FeCl_3$ or filter required to achieve 1.0 mg/L TP. $FeCl_3$ and filter required to achieve 0.3 mg/L TP.

Figure 25. Flow Diagram and Design Criteria for Retrofitting A/O Process to Canadian Model CAS Plant (5).

PLANT TYPE : CONVENTIONAL ACTIVATED SLUDGE

PROCESS FLOW SHEET



Retrofit Design Criteria

Existing Activated Sludge Pumping Station Modifications

- Modifications as required to transfer sludge to anaerobic phosphate stripper tank.

Anaerobic Phosphate Stripper Tank

- 10 h HRT
- Inflow @ 20-30% Average Flow

Reactor-Clarifier

- Surface Overflow Rate @ Average Flow ($m^3/m^2 \cdot d$) = 49
- Underflow Pumping Capacity @ 10-20% Average Flow
- Elutriate Pumping Capacity @ 10-30% Average Flow

Chemical Feed System

- 240 mg/L Hydrated Lime

NOTE: Effluent filter only required to achieve 0.3 mg/L TP.

Figure 26. Flow Diagram and Design Criteria for Retrofitting PhoStrip Process to Canadian Model CAS Plant (5).

the 1.0-mg/L TP effluent limit and only dual-media filtration to reach the 0.3-mg/L TP effluent objective.

Elutriation by recycle of stripper underflow was factored into the PhoStrip estimates along with a dedicated reactor-clarifier for precipitating and settling the phosphorus-rich stripper supernatant. Sludge production from the reactor-clarifier operation was assumed to be 0.5 kg dry solids/m^3 (4.2 lb/Kgal) of stripper supernatant. Submerged turbine aerators at a mixing requirement of 10 W/m^3 (0.4 hp/1000 ft^3) were selected for the anaerobic, anoxic, and aerobic zones of the modified Bardenpho process and the anaerobic and aerobic zones of the A/O process.

The estimated total annual costs in 1984 Canadian dollars of the above four options for retrofitting CAS plants to phosphorus removal are summarized in Table 21 (5). Estimates are shown for design flows of 4500 m^3/d (1.2 mgd), 13,600 m^3/d (3.6 mgd), and 36,400 m^3/d (9.6 mgd), which correspond to the 25, 50, and 75-percentile frequency distributions of actual CAS plant design flows in the province of Ontario.

An annual interest rate of 12 percent over 20 years was assumed for capital amnortization. Operator and maintenance labor rates of $(Can.) 15.60/hr and $(Can.) 18.00/hr, respectively, were used as were $(Can.) 0.04/kWh for the cost of electricity, $(Can.) 187/ton for the cost of chlorine, $(Can.) 0.92/kg Fe^{3+} for the cost of ferric chloride, and $(Can.) 77/ton for the cost of lime. No license fees were included for the biological phosphorus removal options.

The data in Table 21 clearly depict that effluent filtration is a much more expensive polishing step to meet a 1.0-mg/L TP effluent requirement than supplemental chemical dosing with the modified Bardenpho and A/O processes. Based on these estimates, none of the biological process alternatives are competitive with iron addition for the 1.0-mg/L effluent TP case using effluent filtration. With supplemental chemical dosing, retrofitting with the A/O process becomes more cost-effective than chemical addition retrofitting at plant sizes above about 14,000 m^3/d (3.7 mgd)(5). Retrofitting with the PhoStrip process, without any type of assumed effluent polishing add-on, is shown to be more costly than the A/O retrofit option with effluent filtration to attain a 1.0-mg/L TP effluent limitation and more costly than both the A/O and modified Bardenpho process options when supplemental chemical dosing is used for effluent polishing.

All of the biological phosphorus removal alternatives become more competitiive with chemical phosphorus removal in attempting to meet a 0.3-mg/L effluent goal. However, retrofitting with A/O is still the only option that is actually more cost effective than chemical addition, again at plant sizes above approximately 14,000 m^3/d (3.7 mgd).

The above cost estimates are presented here as guides only. Extreme caution should be used in applying them to individual situations. For example, depending on wastewater characteristics, effluent polishing may not be required with a modified Bardenpho or A/O retrofitted system to produce an effluent with 1.0 mg/L TP. It also must be remembered that with a five-stage

TABLE 21. SUMMARY OF ESTIMATED TOTAL ANNUAL COSTS (1984 CANADIAN $/kg TP REMOVED)# FOR RETROFITTING CAS TREATMENT PLANTS TO BIOLOGICAL PHOSPHORUS REMOVAL IN ONTARIO, CANADA (5)

Effluent TP Limit (mg/L)	Retrofit Option	Plant Hydraulic Capacity (m^3/) 4500	13,600	36,400
1.0	Chemical	8.27	6.14	5.17
	Modified BardenPho*	46.83	28.12	18.46
	A/O*	31.31	18.51	11.81
	PhoStrip	33.84	22.45	16.74
1.0	Chemical	8.27	6.14	5.17
	Modified Bardenpho†	28.84	16.28	10.28
	A/O†	13.34	6.68	3.62
	PhoStrip	33.84	22.45	16.74
0.3	Chemical*	28.03	19.22	14.46
	Modified Bardenpho*†	45.60	27.80	19.08
	A/O*†	32.38	19.63	13.39
	PhoStrip*	46.74	30.10	21.63

#The average exchange rate in 1984 was $1.00 Canadian = $0.772 U.S.
*With dual-media effluent filtration.
†With 6 mg/L Fe^{3+} supplemental chemical addition.
*†With both effluent filtration and supplemental chemical addition.

modified Bardenpho flow regime, efficient nitrogen removal will be achieved, which may have an economic value in some cases.

As further cases-in-point of the site-specificity of phosphorus removal cost estimating, the following examples are offered: 1) the Pontiac, Michigan retrofit to a 13,200-m^3/d (3.5-mgd) A/O system at a cost of $57,000 in 1984 U.S. dollars (4) is over eight times less than the estimated capital cost of a similarly-sized, more-permanently constructed A/O retrofit system based on the Canadian estimates (5), and 2) the engineer's recommendation to select PhoStrip for the 113,700-m^3/d (30-mgd) Reno-Sparks, Nevada phosphorus removal retrofit was based on an estimate that this option would produce a total annual cost savings of $500,000 compared to mineral addition.

Process Considerations Standpoint

A comparison of the key process factors affecting retrofitting with chemical addition vs. retrofitting with either the A/O or PhoStrip biological phosphorus removal processes is given in Table 22, as adapted from Reference

TABLE 22. COMPARISON OF KEY FACTORS AFFECTING CHEMICAL AND BIOLOGICAL PHOSPHORUS REMOVAL RETROFIT SYSTEMS

Items	Chemical	Biological
Attainable Effluent TP Concentration	0.2 mg/L	1.0 mg/L (0.5 mg/L or less requires chemical addition).
Amenable Unit Processes	All secondary treatment processes can be retrofitted.	Amenable only to activated sludge-type secondary processes.
Reliability	Reliable, proven technology.	Long-term reliability not proven.
Costs	Low capital cost. High O&M costs for purchase of chemicals and handling of additional sludge produced. Chemical feed system O&M costs can be significant if feeding corrosive chemicals (such as ferric chloride) and/or if utilizing sophisticated instrumentation.	Capital cost can be high depending on degree to which existing tankage can be utilized and amount of license fee. Small increase in excess sludge handling O&M costs with A/O, large increase with PhoStrip. Low equipment O&M costs except for PhoStrip lime feeding system.
Sludge Quantities	A considerable amount of chemical sludge is produced for which additional sludge handling facilities may be required.	Sludge quantities will increase only slightly with the A/O process but significantly with the PhoStrip process.
Operator Training	Relatively simple to operate. Some training will be required for operation of chemical feed pumps and systems.	More difficult to operate and control. Some training will be required for monitoring and operating the chemical feed system for the PhoStrip process.
Additional Staffing Requirements	An additional person may be required depending on the size of the plant.	For the PhoStrip process, an additional person may be required depending on the size of the plant. Additional staff should not be necessary for the A/O process.
Infiltration/ Inflow	No major effect on retrofit design.	Can significantly affect retrofit design.
Potential Technical Advances	None anticipated.	Several anticipated.

4. As indicated, chemical addition is more reliably proven, has a lower achievable effluent TP concentration without polishing, is amenable to more types of existing secondary treatment systems, is less affected by infiltration/inflow, and is easier to operate than the biolgical processes. On the other hand, the biological processes generally produce less excess sludge, have lower O&M costs, and have greater potential for technical advancement than chemical addition.

SUMMARY

The following synoptic statements are offered based on the information presented in this paper:

1. Over 15 years of experience with chemical phosphorus removal has produced a high degree of confidence in North America in the technology's ability to meet a wide range of effluent standards.

2. Biological phosphorus removal is relatively new in North America. In some areas, it is still considered to be an emerging technology. Although some biological phosphorus removal approaches possess inherent potential advantages over mineral addition in the areas of O&M requirements and sludge handling costs, their eventual acceptance and usage on equal terms with the chemical alternative will depend on their ability to provide long-term reliable operation and performance.

3. Lower effluent soluble phosphorus concentrations can be achieved with chemical (mineral salt) addition than with biological phosphorus removal.

4. Lower effluent phosphorus concentrations can more consistently be attained with the sidestream PhoStrip biological phosphorus removal process than with the mainstream biological phosphorus removal processes. This is especially true at low influent SBOD-to-SP ratios.

5. Recycle of significant quantities of solubilized phosphorus in digester supernatants back to the main plant flow is a distinct possibility that must be considered with some biological phosphorus removal options.

6. Lower mineral salt dosages appear to be required to reach effluent TP concentrations of 1 mg/L or less with activated sludge plants than with plants employing fixed film biological systems. Part of the higher dosages required with the fixed film systems may be attributable to the generally shallower and less-conservatively designed final clarifiers provided in many older North American trickling filter plants.

7. Biological phosphorus removal technology offers the engineer a variety of options for effectively combining nitrogen control with phosphorus removal, if desired. However, combining nitrogen and phosphorus control in a single biological system requires careful design to avoid upsetting the operation and efficiency of the anaerobic zone.

REFERENCES

1. Process Design Manual for Phosphorus Removal. Prepared by Black & Veatch, Consulting Engineers for U.S. EPA, Technology Transfer, Washington, D.C., October 1971.

2. Process Design Manual for Phosphorus Removal. EPA 625/1-76-001a, Prepared by Shimek, Roming, Jacobs, & Finklea, Consulting Engineers for U.S EPA, Technology Transfer, Washington, D.C., April 1976.

3. Design Manual - Phosphorus Removal. EPA/625/1-87/001, Prepared by J. M. Smith & Associates, PCS for U.S EPA, CERI & WERL, Cincinnati, Ohio, September 1987.

4. Handbook - Retrofitting POTW's for Phosphorus Removal in the Chesapeake Bay Drainage Basin. EPA/625/6-87/017, Prepared by McNamee, Porter & Seeley, Consulting Engineers for U.S. EPA, WERL & CERI, Cincinnati, Ohio, September 1987.

5. Retrofitting Municipal Wastewater Treatment Plants for Enhanced Biological Phosphorus Removal. Report EPS 3/UP/3, Prepared by CANVIRO Consultants Ltd. with Norbert W. Schmidtke & Associates Ltd. and David I. Jenkins and Associates Inc. for Environment Canada, Environmental Protection Programs Directorate, Burlington, Ontario, October 1986.

6. Letter report from John N. English, U.S. EPA, WERL, Cincinnati, Ohio, to James W. Wheeler, U.S. EPA, OWPO, Washington, D.C., March 28, 1983.

7. Personal communication from S. Joh Kang, McNamee, Porter & Seeley, Ann Arbor, Michigan, to Richard C. Brenner, U.S. EPA, WERL, Cincinnati, Ohio, September 10, 1987.

8. Personal communication from David DiGregorio, Eimco Process Equipment Co., Salt Lake City, Utah, to Richard C. Brenner, U.S EPA, WERL, Cincinnati, Ohio, September 10, 1987.

STATUS OF FINE PORE AERATION IN THE UNITED STATES

by

Richard C. Brenner
Environmental Engineer
Wastewater Research Division
Water Engineering Research Laboratory
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

and

William C. Boyle, Ph.D.
Professor, Civil and Environmental Engineering
University of Wisconsin
Madison, Wisconsin 55706

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

Prepared for Presentation at:

Eleventh United States/Japan Conference
on Sewage Treatment Technology
Tokyo, Japan

October 12-14, 1987

STATUS OF FINE PORE AERATION IN THE UNITED STATES

Richard C. Brenner and William C. Boyle

INTRODUCTION

With its potential for enhanced energy efficiency, fine pore diffused aeration (FPDA) has become a major force in the United States aeration market in the 1980's. Virtually every aeration system design undertaken in the United States today, new or retrofit, includes consideration of one or more types of FPDA devices before a final decision is made. More often than not, an FPDA system is being selected for installation.

The renewed interest in FPDA in the Unites States can be traced to escalating energy costs in the mid- and late-1970's. Prior to that, coarse bubble aeration had been the standard of U.S. municipal wastewater treatment for several decades. Coarse bubble aeration, although relatively energy intensive, is reliable and requires little maintainence. The first FPDA systems to make a significant impact in the United States following the energy crisis were the ceramic grid configurations used successfully in the United Kingdom for many years. Today, a wide array of FPDA alternatives is available, including ceramic plates, domes, discs, and tubes; plastic plates, discs, and tubes; and flexible membrane tubes and discs.

Because of the expanding number of FPDA choices that are becoming available, the sometimes exaggerated claims made for them, the inherent potential for greater maintenance requirements than with other devices, and a general lack of reliable performance data on which to base the design and operation of these systems, the U.S. Environmental Protection Agency (EPA) decided to initiate a major R&D effort to expand and improve the technical data base for this aeration technology. A $1.2 million cooperative agreement was entered into between EPA and the American Society of Civil Engineers (ASCE) in 1985 for this purpose.

The ultimate goal of the EPA/ASCE project is the publication of a Comprehensive Design and Operating Manual (CDOM) on Fine Pore Aeration in 1989.

Richard C. Brenner is an Environmental Engineer with the U.S. Environmental Protection Agency, Cincinnati, Ohio, and Project Officer of the EPA/ASCE project on fine pore aeration.

William C. Boyle is Professor of Civil and Environmental Engineering, University of Wisconsin, Madison, Wisconsin; Principal Investigator of the EPA/ASCE project on fine bubble aeration; and Chairman of the ASCE Committee on Oxygen Transfer.

It is intended that the first distribution of this manual will be at the 1989 Water Pollution Control Federation Conference. The project is being technically administered and supervised by the ASCE Committee on Oxygen Transfer. The Committee's first action following consummation of the cooperative agreement was to prepare a Summary Report on Fine Pore Aeration (1)*. This report discusses the status of the technology in 1985 and identifies perceived technical gaps. It is recommended to the reader as an excellent background source in the subject area.

Since mid-1985, the Committee has, under the cooperative agreement, funded a number of subcontracts to address the gaps in the FPDA data base mentioned above. For the most part, these subcontracts are in the form of full-scale field studies. Several of the studies will continue well into 1988 to provide as much long-term data as possible for incorporation in the CDOM. Primary areas of study include in situ evaluation of oxygen transfer performance of several generic FPDA systems, assessment of the impact of process operating conditions on FPDA oxygen transfer efficiency, investigation of the incidence and severity of fouling of fine pore devices and the impacts on performance resulting therefrom, and evaluation of the effectiveness of several methods used for cleaning fine pore diffusers (either for preventive or restorative purposes).

This paper will describe interim results in two major areas of study on this project: the effect of process operating conditions and the impact of diffuser cleaning on FPDA performance and design. The reader is urged to use any relationships and trends shown herein with caution, as additional data are being generated.

DESCRIPTION OF EPA/ASCE PROJECT

Background

The ASCE Committee on Oxygen Transfer is an outgrowth of the ASCE Oxygen Transfer Standards Committee, which was formed in 1977 to research and develop consensus standard methods for conducting oxygen transfer tests and evaluating oxygen transfer test data. The Standards Committee, through two previous EPA/ASCE cooperative agreements, produced a "Standard" for clean water oxygen transfer testing and evaluation (2) and refined and field evaluated several procedures for process water oxygen transfer testing (3). The methodology produced on these forerunner projects is being used extensively on the fine pore aeration project.

Committee Structure and Operating Procedures

The ASCE Committee on Oxygen Transfer is composed of 50 volunteer engineers and scientists, primarily from the United States, but with representa-

*In this report, the ASCE Committee on Oxygen Transfer defines fine pore aeration as diffused aeration by a porous device that typically produces a headloss due to surface tension in clean water of greater than about 5 cm (2 in.) water gauge.

tion from Canada, Sweden, the United Kingdom, and West Germany. The Committee roster includes representatives from academia, local and federal government, POTW users, consultant engineering, and manufacturing.

Administrative responsibilities are handled by ASCE staff. Technical direction is provided by a 10-member Steering Subcommittee, selected from the Committee-at-large. The Committee Chairman/Principal Investigator is a non-voting member of the Steering Subcommittee. The Subcommittee and Committee are chaired by different individuals.

Subcontractors were chosen on the basis of competitive proposals (in response to broad general criteria provided by the Steering Subcommittee) from the Committee membership. Steering Subcommittee members were not eligible to receive subcontracts. Each Steering Subcommittee member is responsible for monitoring one or more subcontract field studies.

The Steering Subcommittee meets four to five times a year to review progress on all phases of the project and to make mid-course changes in direction as needed. The entire Committee membership meets once a year, is invited throughout the year to review and comment on all Subcontractor reports, and is periodically advised of Steering Subcommittee actions. Preparation of the CDOM on Fine Pore Aeration will be the responsibility of the Steering Subcommittee using authors from the general membership and from within the Subcommittee itself.

Project Field Studies

Fifteen (15) major field studies have been funded by the Steering Subcommittee to date to generate the data necessary to carry out the objectives of this project. A brief description of each field study along with a list of the study sites is given in Table 1. Two of the studies (I and O) have been completed; the other 13 are ongoing. The diffuser cleaning methods referred to in some of the study descriptions are discussed in more detail later in the paper.

TABLE 1. FIELD STUDIES FUNDED BY EPA/ASCE FINE PORE AERATION PROJECT

	Study Site	Description
A.	Whittier Narrows (LACSD)*, California	Evaluation of acid gas cleaning on ceramic disc and dome fine pore diffusers
B.	Terminal Island (City of Los Angeles), California	Retrofit comparison of two types of flexible membrane tube fine pore diffusers vs. coarse bubble diffusers
C.	Green Bay, Wisconsin	Side-by-side performance comparison of ceramic disc and flexible membrane tube fine pore diffusers with preventive acid gas cleaning of the disc diffusers and preventive cleaning of the tube diffusers by membrane flexing

(continued)

TABLE 1. (continued)

	Study Site	Description
D.	Frankenmuth, Michigan	Design, operating, and cleaning (acid gas) case history of ceramic disc fine pore diffuser system comparing two side-by-side cleaning frequencies
E.	Monroe, Wisconsin	Side-by-side performance evaluation of ceramic disc fine pore diffusers of several different permeabilities with possible comparison of acid gas and Milwaukee cleaning methods if diffusers become sufficiently fouled
F.	Madison, Wisconsin	Side-by-side comparison of several cleaning methods (to be selected from Milwaukee, hosing, and steam methods) on ceramic disc and dome fine pore diffusers
G.	Hartford, Connecticut	Design, operating, and cleaning (hosing) case history of ceramic dome fine pore diffuser system
H.	Glastonbury, Connecticut	Performance evaluation of porous plastic tube fine pore diffuser system
I.	Ridgewood, New Jersey	Design, operating, and cleaning (hosing) case history of ceramic dome fine pore diffuser system
J.	North Texas Municipal Water District (Dallas), Texas	Laboratory evaluation of six cleaning methods applied to ceramic dome fine pore diffusers
K.	Jones Island (Milwaukee), Wisconsin	Performance evaluation of ceramic fine pore plate diffusers
L.	South Shore (Milwaukee), Wisconsin	Side-by-side performance comparison of ceramic plate and disc fine pore diffusers
M.	Valencia (LACSD)*, California	Performance evaluation of porous plastic disc fine pore diffusers
N.	Milwaukee, Wisconsin	Long-term case history write-ups of ceramic plate fine pore diffuser performance and O&M on Milwaukee's Jones Island and South Shore plants (predates ongoing Studies K and L)

(continued)

TABLE 1. (continued)

Study Site	Description
O. Northern Europe	Survey of Scandanavian O&M and performance experience with porous plastic tube and disc fine pore diffusers

*LACSD - Los Angeles County Sanitation Districts

Test Methods

A substantial clean water oxygen transfer data base exists for FPDA systems. Where additional clean water data have been necessary or desirable, the ASCE "Standard" clean water test method (2) has been utilized.

The great majority of data needed for this project, however, are in the area of process water or in-process oxygen transfer performance evaluation. Two methods, the off-gas analysis and inert gas tracer procedures, that do not require steady process loads, a positive aeration basin dissolved oxygen (DO) concentration, or oxygen uptake rate measurements have been found to be very effective and reliable for process water testing of diffused aeration systems (1).

All of the process water oxygen transfer data on this project have been generated using the off-gas procedure because it is considerably cheaper to implement than the inert gas tracer methods. A portable gas collection hood is utilized to sample off gas at different points around an aeration basin. An integrated oxygen transfer rate for the entire basin can be developed, if desired. Off-gas testing is also useful in defining differences in oxygen transfer efficiency at various points in the basin.

In addition to off-gas oxygen transfer measurements, the following tests have been devised to evaluate fine pore diffuser characteristics. Air flow profiles are measured across the surfaces of selected diffusers removed from aeration basins. These removed diffusers are also subjected to dynamic wet pressure (DWP) and bubble release vacuum (BRV) tests. DWP is defined as the operating headloss across diffuser media submerged in water at a specified air flow rate per diffuser (1). BRV is defined as the applied headloss required to induce air flow through the diffuser media at a given point (1). Benchmark values have been established for these tests for new or clean ("like new") ceramic diffusers. Corresponding values determined on dirty diffusers help to assess the degree to which fouling has progressed. DWP tests can also be conducted on in-place, permanently-installed diffusers for in-situ prediction of foulant buildup.

Samples of foulants are scraped from the surfaces of the test diffusers for further characterization. Laboratory tests include determinations of acid solubles and percent volatile fractions, among others. The Committee is

currently attempting to develop a fundamental study using Scanning Electron Microscopy technology that will shed additional light on the nature and progression of fine pore diffuser fouling.

To better characterize fine pore diffuser fouling patterns and to coordinate and interpret fouling data from one plant to another, an interplant fouling control study was initiated early in the project. Specially constructed removable units called "four lungers" have been installed in eight of the key project plants. The "four lungers" are each equipped with four ceramic disc diffusers to which air flow can be individually controlled. The "four lungers" are immersed in the mixed liquor and operated as conventional diffuser units. Periodically, diffusers are removed and shipped to a central laboratory facility for extensive testing and foulant characterization.

Although the data developed through the individual plant evaluations and interplant fouling control study are still too preliminary at this juncture to present detailed summaries, they do indicate that fine pore diffuser fouling is apparently not as universal and severe as initially suspected. Rather, fouling is encountered on a random basis with little or none experienced at some plants and extensive problems experienced at others. The variables influencing measurable fouling of fine pore diffusers are still being delineated at this time.

It has also been determined that less-than-acceptable oxygen transfer efficiency at some facilities may not be due to diffuser fouling or clogging, but rather to local wastewater characteristics and/or secondary process operating conditions. The presence of undegraded surfactants especially has a negative impact on fine pore oxygen transfer performance. The effects of process operating conditions on oxygen transfer performance will be discussed later in the paper.

Terminology

A number of terms are used in this paper, the definition of which are important to a full understanding of the data presented. SOTE, SOTR, and SAE refer to standard oxygen transfer efficiency, standard oxygen transfer rate, and standard aeration efficiency in clean water, respectively. Standard conditions are defined as: DO = 0.0 mg/L, water temperature = 20°C, pressure = 1.00 atmosphere, alpha (α) = 1.0, and beta (β) = 1.0.

Alpha is a factor that describes the ratio of the apparent volumetric mass transfer coefficient (K_La) in dirty or process water to that of the same coefficient in clean water. As such, α describes the negative (usually) impact of a particular wastewater on the oxygen transfer rate of a specific type of aeration device operating in a specific aeration system configuration under specific environmental conditions and with a specific basin geometry. By definition, α is a term that measures this impact on new or clean ("like new") diffusers.

The random and unknown effects of fine pore diffuser fouling will normally reduce the alpha factor with time. It is difficult, if not impossible, to separate the effects of wastewater characterisitics from fouling on diffuser

performance. To account for this situation, the Committee has coined a new term called the apparent alpha factor (α').* The apparent alpha factor combines the above two impacts into one term that defines the real-life oxygen transfer situation under process conditions at any one time. Theoretically, α and α' are only equal at the moment new diffusers are placed in service or immediately after full restorative diffuser cleaning.

When describing in-process oxygen transfer performance in this paper, the terms α'SOTE, α'SOTR, and α'SAE will denote standard oxygen transfer efficiency, standard oxygen transfer rate, and standard aeration efficiency under process or dirty water conditions.

PERFORMANCE OF FINE PORE DIFFUSERS UNDER PROCESS CONDITIONS

Summary of Selected Performance Data

The performance of fine pore diffusers under process conditions is affected by myriad factors. Some of the better-documented factors appear in Table 2. Not included in this table, but of great concern to design engineers and operators, are the additional factors of flow regime, operating conditions, and diffuser fouling. One of the objectives of this research program is to better quantify the influence of these variables on performance. Over the past 2-1/2 years, a substantial amount of field data have been collected relative to these factors. Table 3 presents an abbreviated tabulation of selected data generated to date. Several interesting observations suggested by these data are discussed below.

TABLE 2. SELECTED FACTORS THAT AFFECT OXYGEN TRANSFER OF FINE PORE AERATION DEVICES

Wastewater Characteristics
Diffuser Submergence
Diffuser Type and Characteristics
Gas Flow per Diffuser
Diffuser Density (No. of diffusers per unit floor area)
Diffuser Layout (Configuration)
Basin Geometry
Mixed Liquor Temperature and DO

Effect of Operating Conditions on Performance

For some time, it has been suspected that process operation affects the oxygen transfer performance of fine pore diffusers. The impacts of volumetric organic loading, solids retention time (SRT), food-to-microorganism (F/M) loading, and mixed liquor suspended solids (MLSS) concentration on performance

*In Reference 1, the term α^d was used instead of α' to denote the apparent alpha factor.

TABLE 3. FINE PORE DIFFUSER PERFORMANCE SUMMARY - PROCESS WATER INTERIM DATA BASE

System No.§	Diffuser Type	Diffuser Age*	Diffuser Subm. (m)	Diffuser Density (m^2/diff.)	Air Flow/Diff. (L/sec)	F/M Loading (day^{-1})†	SRT (days)	$\overline{\alpha' SOTE}$** (%)	$\overline{\alpha' SOTE}$** (%/m)	$\overline{\alpha'}$**	Flow Regime
1	Cer. Dome	U	4.6	0.39	0.9-1.4	1.26	~1.0	6.86	1.61	0.27	SF
2	Cer. Dome	N	4.2	0.31	0.42-0.61	0.55	1.98	7.92	1.89	0.29	PF
3	Cer. Disc	N	4.2	0.35	0.24-0.71	0.61	1.63	8.18	1.95	0.28	PF
4	Cer. Disc	1 yr	5.0	0.57	0.42	0.18	5.4	14.32	2.86	0.44	PF
5	Cer. Disc	1 yr	5.0	0.57	0.42	0.10	10.3	17.88	3.58	0.60	PF
6	Cer. Dome	1 yr	4.6	0.37	0.28-0.52	0.75	1.5	10.75	2.34	0.34	PF
7	Cer. Dome	1 yr	4.6	0.37	0.28-0.52	0.30	4.8	12.70	2.76	0.40	PF
8	Cer. Disc	N	3.8	0.41	0.47	0.51	2.2	10.3	2.71	0.33	PF
9	Cer. Dome	1 yr	4.1	0.22	0.57	0.4	-	9.0	2.20	-	SF
10	Cer. Dome	1 yr	4.1	0.22	0.35	0.2-0.3	-	13.77	3.36	-	SF
11	Cer. Dome	U	4.3	0.40	0.38-1.1	0.25	4.3	9.5	2.21	0.40	PF
12	Cer. Dome	U	4.3	0.40	0.8-0.9	0.19	1.6	10.7	2.49	0.47	PF
13	Cer. Dome	U	3.2	0.36	0.42	0.1	11.7	10.5	3.28	-	PF
14	Cer. Dome	U	3.2	0.46	0.42	0.44	2.7	7.0	2.19	-	PF
15	Flex. Memb. Tube	2 yr	4.0	0.57	1.4-1.9	0.22	4.5	7.6	1.90	0.36	PF

§System nos. do not correspond to the sequence of survey sites presented in Table 1.
*Age: U = unknown; N = "like new"
†Expanded units: kg total BOD_5 ($TBOD_5$)/day/kg MLSS
**Mean weighted values
††SF = step feed; PF = plug flow

are being examined in the field studies that are a major part of the EPA/ASCE project. Although it is too early to develop reliable quantitative information, the effects of SRT and F/M loading on α'SOTE are indicated in Figures 1 and 2, respectively, developed from the data in Table 3. As SRT increases (or F/M loading decreases), the value of α'SOTE (expressed as %/m of diffuser submergence) increases. As much as a two-fold increase in α'SOTE appears to occur within the range of SRT's and F/M loadings studied. It should be emphasized that this information is preliminary. As evident in Table 2, there are other variables that influence α'SOTE that were not "controlled" in presenting these data. Of particular interest in Figures 1 and 2 are the data from Rye Meads, United Kingdom, which were reported by the Water Research Centre (WRC) from studies conducted in 1983 (4) and will be discussed later.

In more controlled studies at the Madison (Wisconsin) Metropolitan Sewerage District (MMSD) facility, the influence of F/M loading on α'SOTE is illustrated in Figure 3. These data, representing the first two ceramic disc diffuser grids of a six-grid aeration system, were collected over a narrow range of F/M loadings wherein nitrification occurred. Nevertheless, the advantage of operating at higher MLSS concentrations (lower F/M loadings for the same volumetric loadings) is indicated.

The independent effect of SRT (or F/M loading) on α'SOTE values at constant volumetric organic loading is depicted in Table 4 for the MMSD facility. Over a 2-month period, the first-pass tanks of two parallel three-pass aeration trains were operated at two different SRT's (approximately 11 and 6 days) at the same volumetric organic loading rate. As seen in Table 4, the average first-pass (comprising two ceramic disc grids) α'SOTE value for the 11-day SRT train was approximately one percentage point higher (10.52% vs. 9.62%) than for the 6-day SRT train.

TABLE 4. EFFECT OF SRT ON α'SOTE FOR CONSTANT VOLUMETRIC ORGANIC LOADING AT MADISON, WISCONSIN (FIRST-PASS TANKS)

Test No.	System A			System B		
	SRT (days)	α'SOTE (%)	Vol. Org. Load (kg $TBOD_5$/day/m^3)	SRT (days)	α'SOTE (%)	Vol. Org. Load (kg $TBOD_5$/day/m^3)
1	11.6	14.39	0.15	7.6	13.06	0.15
2	11.6	10.13	0.19	6.4	9.37	0.19
3	10.4	9.89	0.15	5.8	7.99	0.15
4	12.1	10.07	0.16	6.0	9.29	0.16
5	10.7	10.51	0.14	6.3	9.74	0.14
6	10.9	9.61	0.17	5.6	9.20	0.17
7	11.1	9.85	0.17	5.9	8.91	0.17
8	11.1	9.71	0.18	5.6	9.40	0.18
Avg.	11.2	10.52		6.1	9.62	

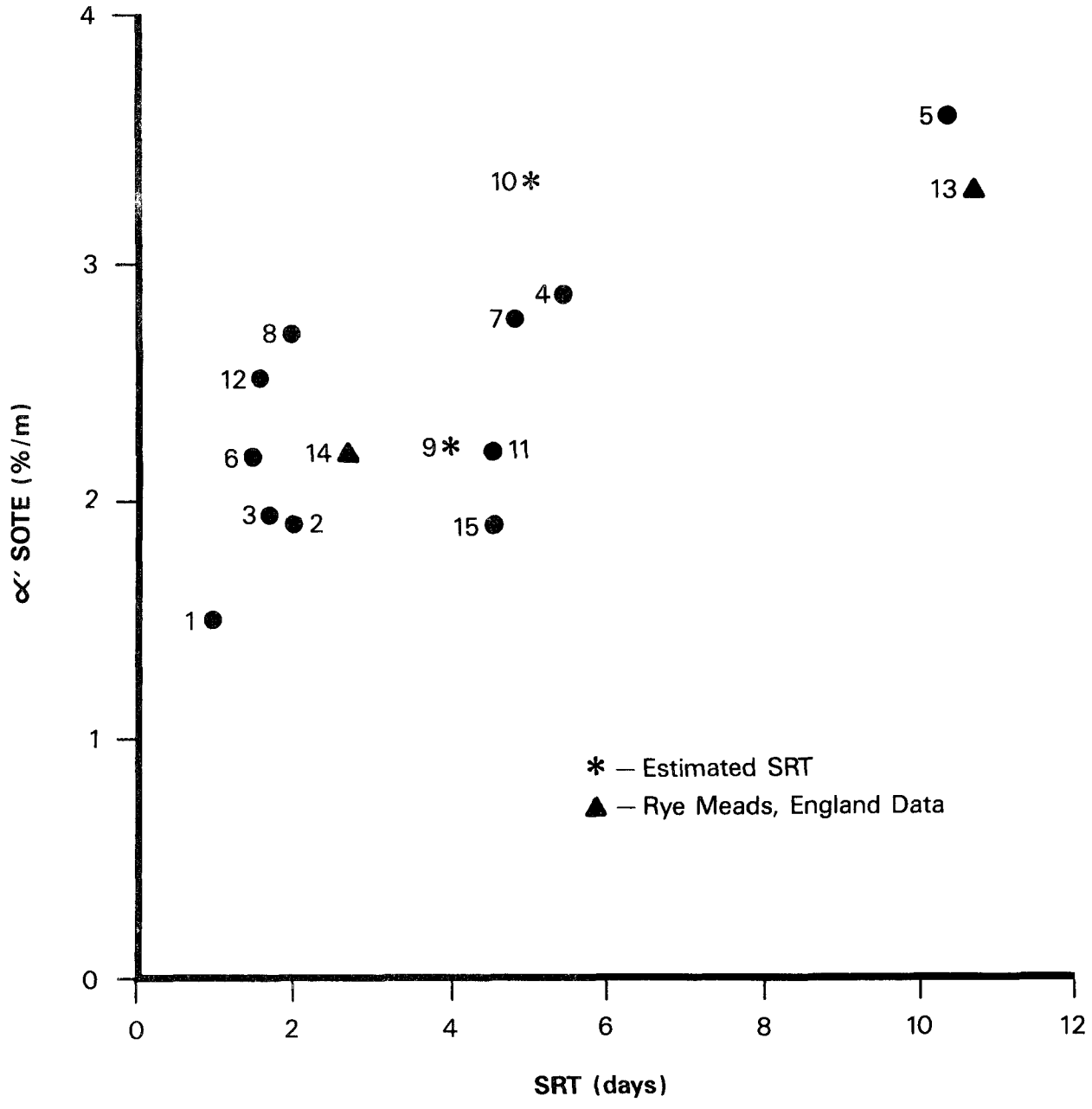


FIGURE 1. Effect of SRT on α′ SOTE of Fine Pore Diffusers.

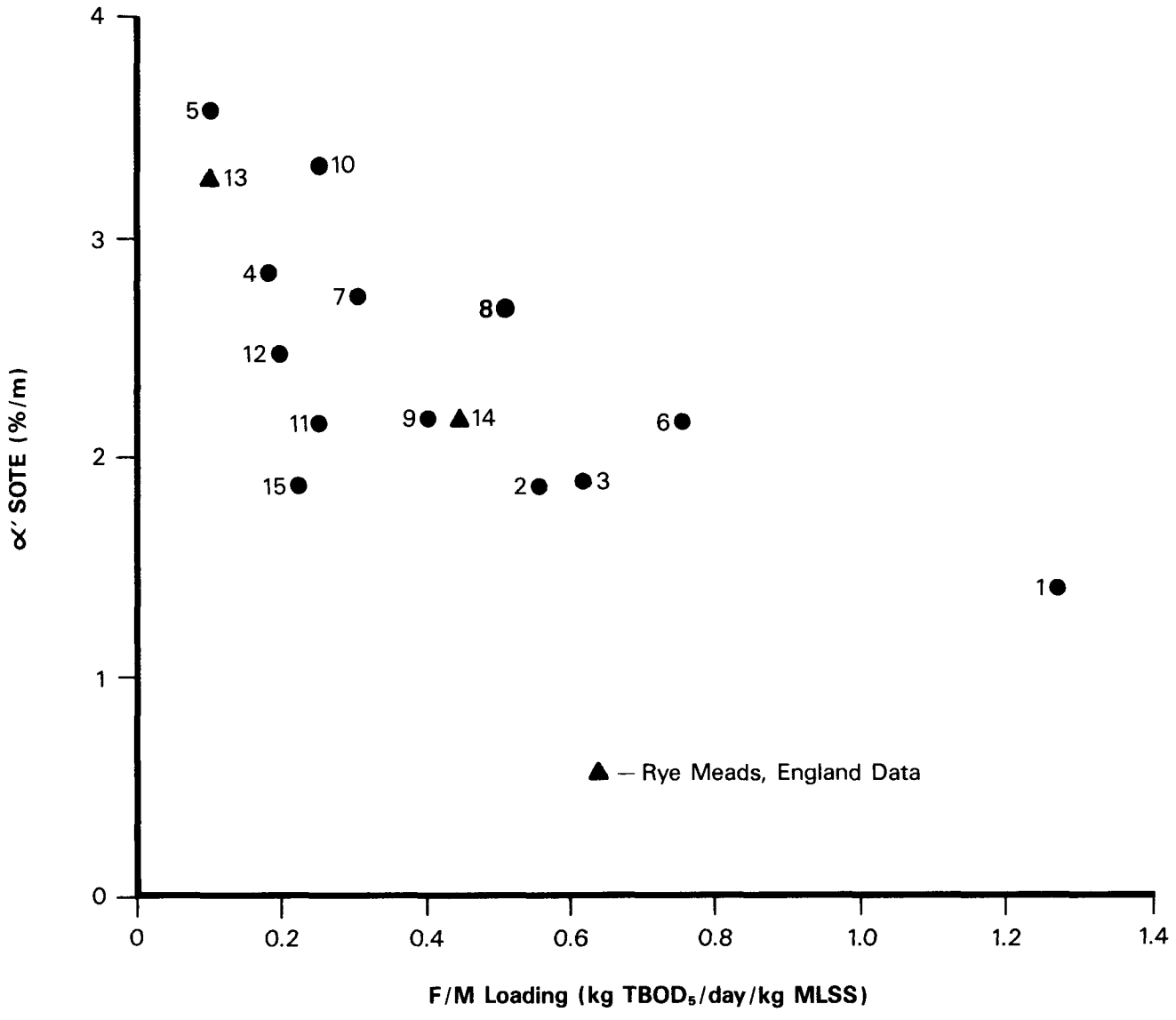


FIGURE 2. Effect of F/M Loading on α′ SOTE of Fine Pore Diffusers.

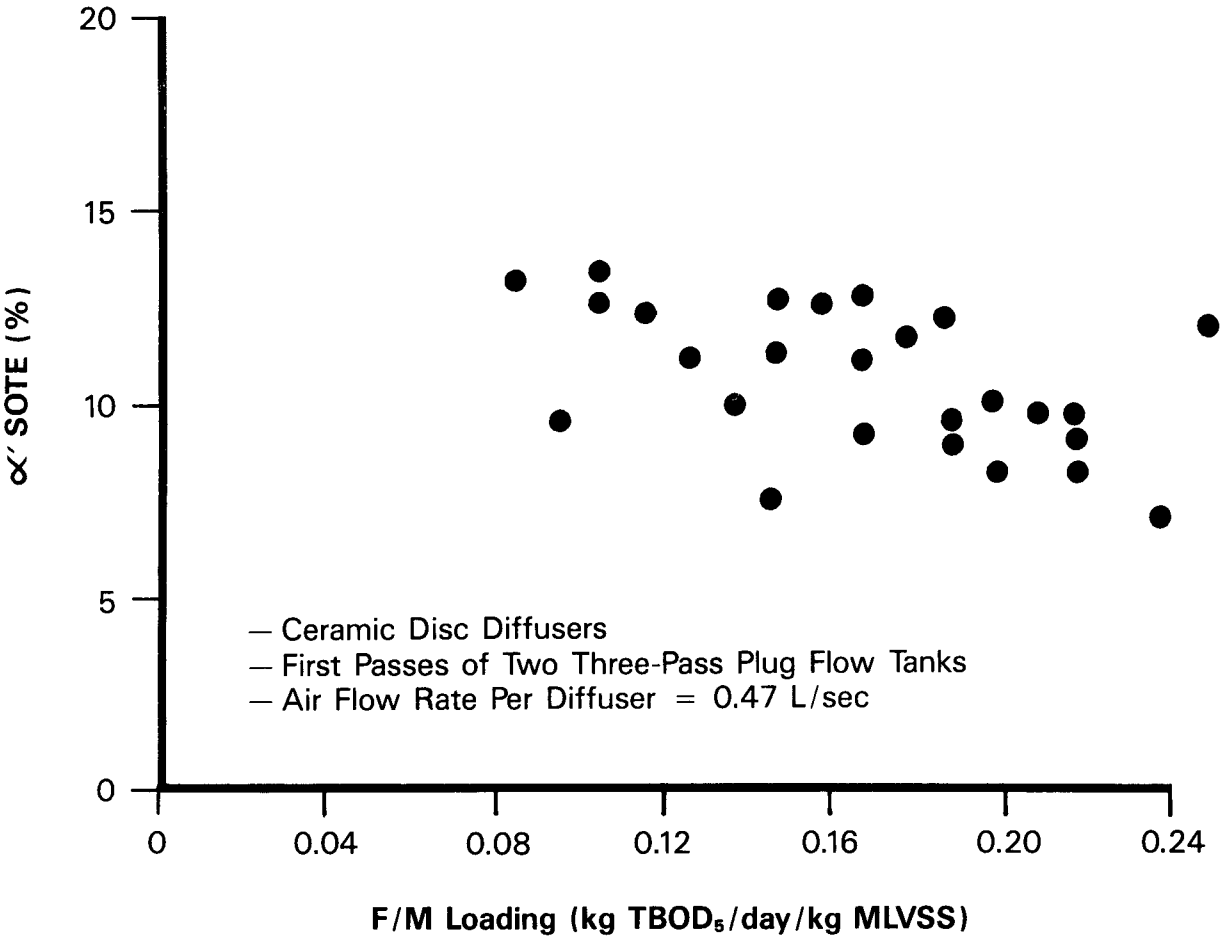


FIGURE 3. Effect of F/M Loading on ∝′ SOTE of Nitrifying Fine Pore Aeration Systems at Madison, Wisconsin.

The reasons for the increases noted in α'SOTE values with increasing SRT (or decreasing F/M loading) can only be speculated on at this time. Since surfactants play an important role in depressing oxygen transfer in many wastewater treatment systems, factors that affect surfactant concentrations in these systems may directly impact values of alpha. The higher SRT systems may produce biomass that will more effectively sorb or biodegrade surfactants, resulting in an overall higher value of alpha than in lower SRT systems. More research is needed, however, to substantiate this hypothesis.

Figure 4 depicts the impact of SRT on α'SOTE down the length of two plug flow aeration tanks at MMSD equipped with fine pore ceramic disc diffusers. Recovery of α'SOTE (due to rapid increases in alpha) with tank length is greatly enhanced in the longer sludge age system. For the conditions of this test, alpha values ranged from 0.22 to 0.30 for the low-SRT system and from 0.25 to 0.50 for the higher-SRT system. A review of Table 3 suggests that low-SRT systems must be designed with significantly lower alpha values than higher-SRT systems.

The value of operating an aeration system at high SRT's (low F/M loadings) is well documented in the literature. The benefits of high-SRT operation include greater biological process stability, production of a nitrified effluent, lower SVI's, lower sludge yields, a more stabilized waste activated sludge, and improved waste activated sludge thickening properties. Limitations to high-SRT operation include clarifier and aeration tank capacities, unwanted (in some cases) nitrification, floating sludge in final clarifiers, and supposedly higher aeration power requirements. Other site-specific considerations may also make operation at high SRT's infeasible and/or undesirable.

Preliminary results of the EPA/ASCE field studies suggest that higher aeration power costs may not necessarily result from operation at high SRT's since α'SOTE appears to increase more rapidly with increasing SRT than does oxygen demand. Figure 5 diagrams in a simplistic qualitative fashion what might occur as SRT increases with respect to the system oxygen balance and the power required to achieve sufficient oxygen transfer to satisfy oxygen demand. For this hypothetical example, a low-SRT system was assumed. As shown on the diagram, if α'SOTE increases more rapidly than oxygen demand increases, a net power savings will be realized according to the equation:

$$\text{Power} = f\ \frac{\text{Oxygen Demand}}{\alpha'\text{SOTE}}$$

When incipient nitrification occurs, power consumption increases momentarily (Figure 5) because of the rapid increase in oxygen demand and standard oxygen uptake rate (SOUR). As SRT continues to increase past the point of incipient nitrification, overall power consumption may decrease to a level below that for non-nitrifying operation, again depending on the relative slopes of the oxygen demand and oxygen transfer curves. An additional advantage of a nitrifying mode of operation is the potential positive impact on the oxygen balance in the form of denitrification credits with the use of a lead anoxic stage.

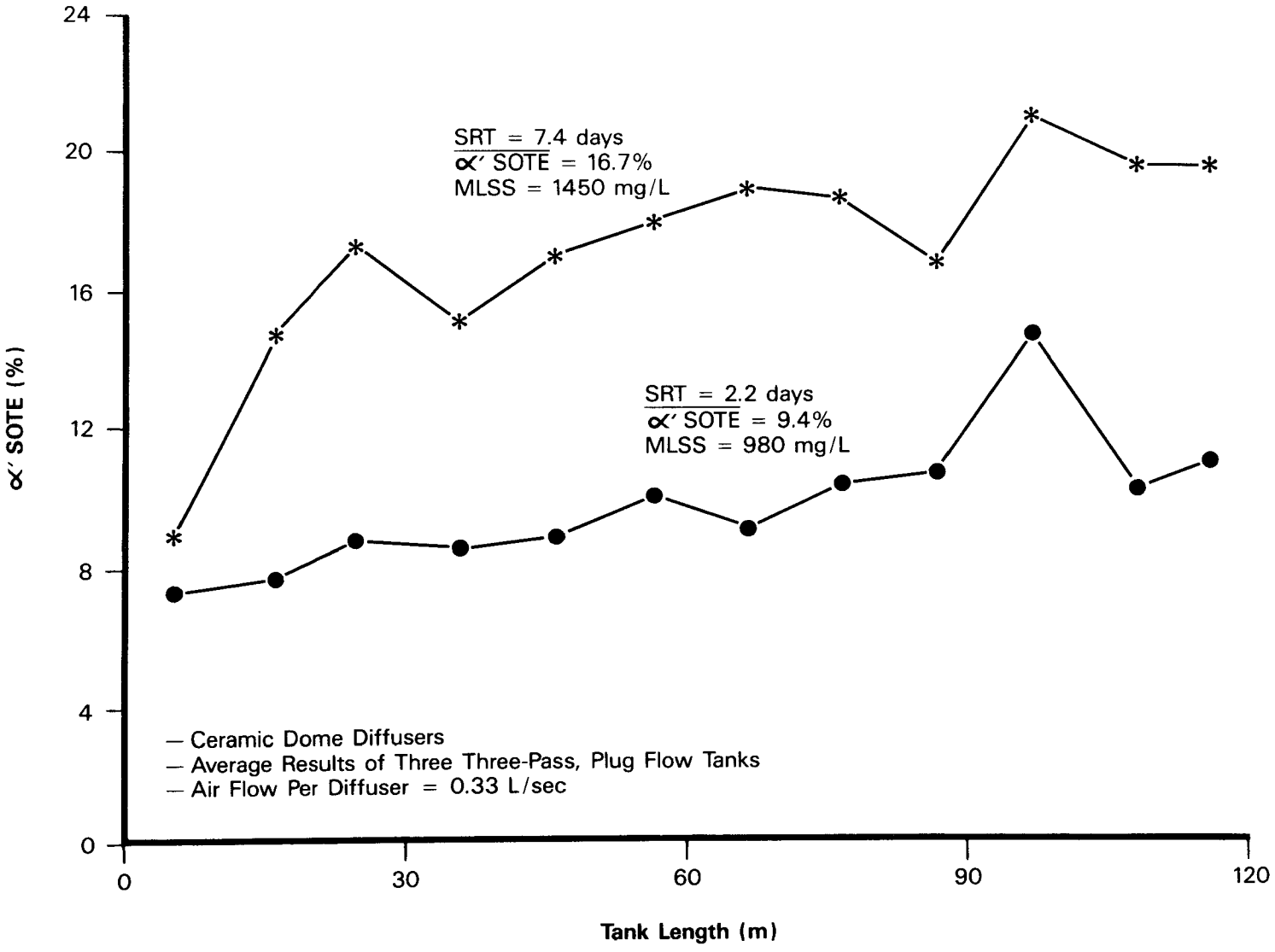


FIGURE 4. Effect of SRT on ∝′ SOTE Tank Profile Values of Fine Pore Aeration Systems at Madison, Wisconsin.

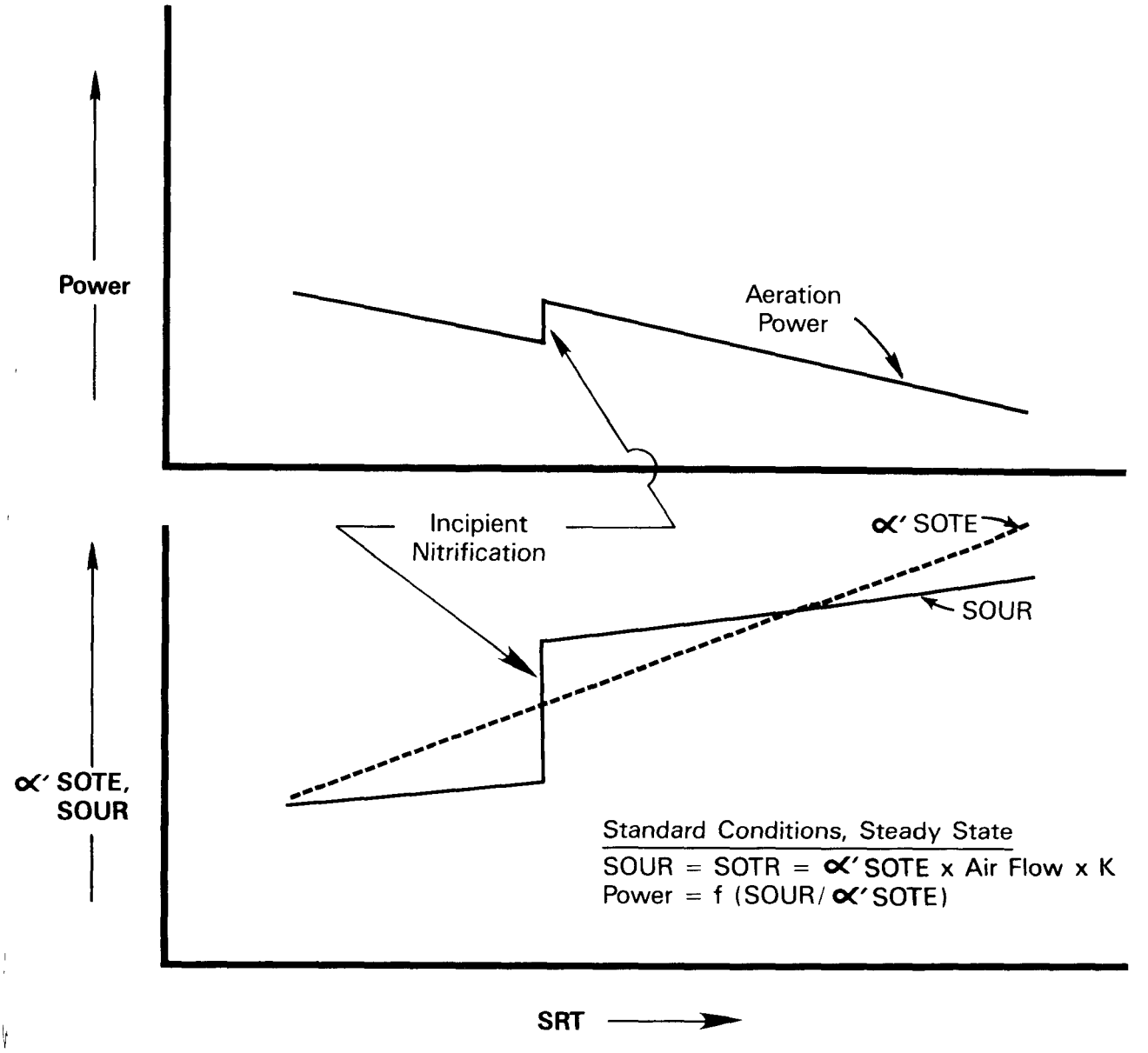


FIGURE 5. Hypothetical Diagram of the Impact of SRT on ∝′ SOTE, Oxygen Uptake, and Aeration Power Requirements of Fine Pore Aeration Systems.

The potential for achieving aeration power savings via selection of a nitrifying operational mode was demonstrated in the WRC study at Rye Meads. Two parallel ceramic dome diffuser systems were operated at SRT's of 2.7 (non-nitirifying) and 11.7 (nitrifying) days. The nitrifying system consumed 1726 kg O_2/day (3805 lb/day) at a power expenditure of 964 kWh/day (1293 wire hp-hr/day), while the non-nitrifying system utilized 1170 kg O_2/day (2597 lb/day) at a power consumption of 981 kWh/day (1316 wire hp-hr/day). The field aeration efficiency for the 11.7-day SRT system was 1.79 kg O_2/kWh (2.94 lb/wire hp-hr) vs. 1.19 kg O_2/kWh (1.96 lb/wire hp-hr) for the 2.7-day SRT system.

Effect of Flow Regime on Performance

Several field studies have been conducted to evaluate the impact of flow regime on α'SOTE. Figure 6 compares the effect of a step aeration flow regime vs. that of a plug flow configuration on the oxygen transfer performance of ceramic dome diffusers at Madison, Wisconsin (MMSD). In both instances, SRT was approximately 2.2 days. Clearly, the plug flow configuration produced a superior mean weighted α'SOTE value compared to that of the step feed mode of operation (9.44% vs. 7.15%). Addition of primary effluent at several points along the basin of the step aeration system resulted in depressed α'SOTE values at each feed point. Apparently, reduced alpha values associated with each feed point overrode the benefits of load balancing, such that α'SOTE was negatively impacted from the second feed point through the remaining length of the aeration tank. These data suggest again that sorption (and/or biodegradation) of surfactants may be favored in the plug flow configuration, at least in short-SRT systems.

EFFECT OF CLEANING ON FINE PORE DIFFUSER PERFORMANCE

Cleaning Techniques

Although data collected to date indicate that fine pore diffuser fouling is not as prevalent and severe as first presumed, some installations do experience rapid fouling rates. Rapid fouling situations appear to be closely tied to wastewater characteristics that either 1) promote deposition or precipitation of solids onto diffuser surfaces and/or penetration of those solids into the outer pore structure of the diffusers or 2) encourage profuse biological slimes to grow on the surfaces of diffusers. Solids deposition is often associated with wastewaters that do not practice primary treatment or have poor grit removal. Solids precipitation may occur under certain conditions with wastewaters that have high natural concentrations of metal cations or introduce metal salts to the wastewater for phosphorus removal and/or improved settling. High rates of biological growth on diffusers are usually related to municipal wastewaters that contain industrial waste components with high soluble carbohydrate concentrations. These types of waste exhibit abnormally high C/N ratios, and the resulting nutrient imbalance may be the primary causative factor in accelerated biofouling.

Fine pore systems that do not experience rapid fouling can generally rely on long-term rigorous cleaning every 1 or 2 years to restore α'SOTE to acceptable levels. When severe fouling is encountered, however, α'SOTE can be

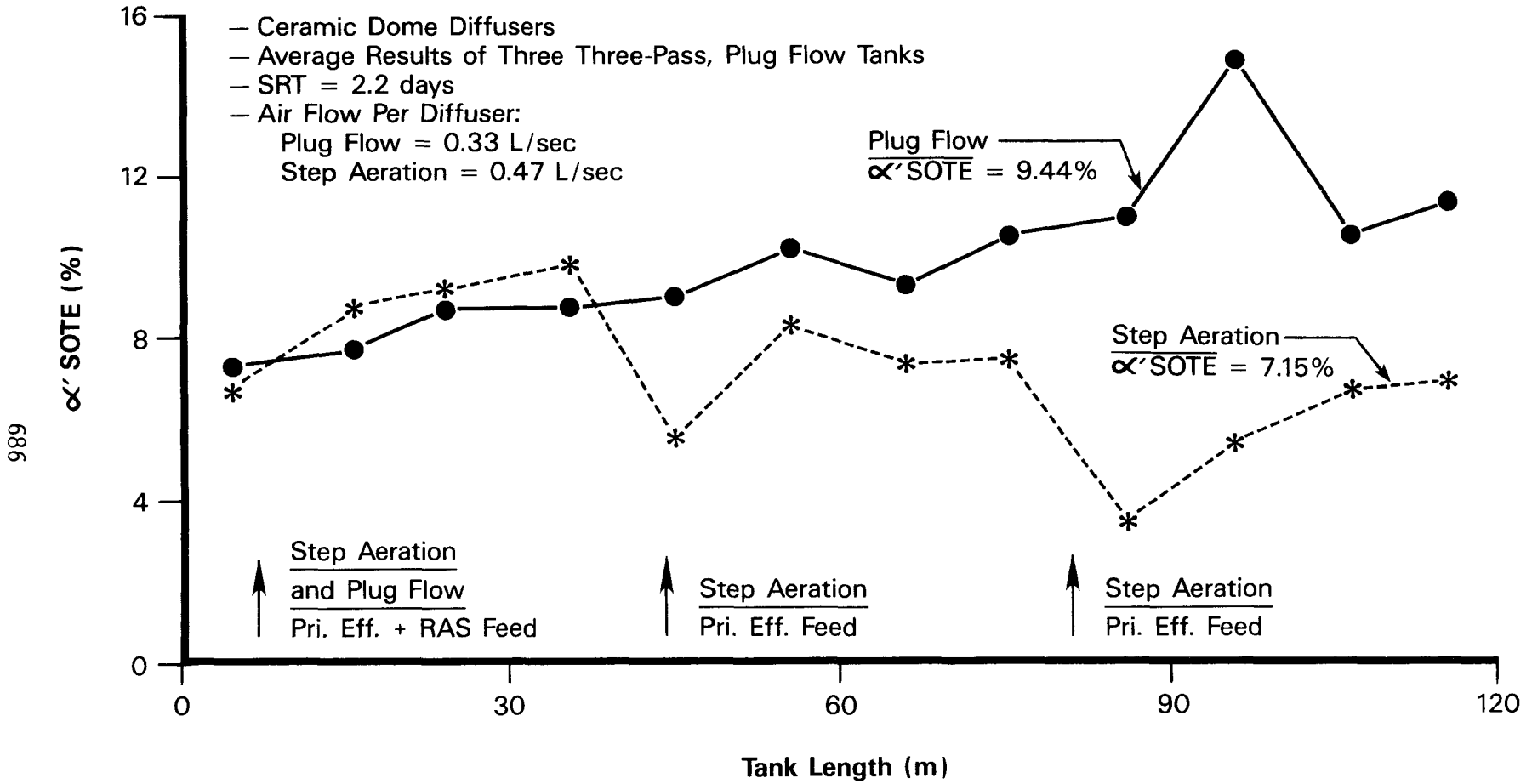


FIGURE 6. Effect of Flow Regime on α′ SOTE of Fine Pore Diffusers at Madison, Wisconsin.

expected to decrease at a sufficient rate to warrant more frequent rigorous cleaning or implementation of a preventive cleaning strategy. Such strategies may include regular usage (e.g., monthly) of techniques that only partially restore α'SOTE but do not require aeration basin dewatering. These techniques are combined with periodic, less frequent utilization of rigorous cleaning methods that restore diffuser performance to "like new" or nearly new conditions but do necessitate basin dewatering.

Cleaning techniques that require the aeration basin to be taken out of service to gain access to the diffusers have been broadly classified by the Committee as process interruptive techniques (1). Those that do not require such access are referred to as process noninterruptive techniques (1). A further distinction can be made in the process interruptive category between those techniques that require the diffusers to be removed for cleaning once the basin is dewatered (ex situ) and those that do not require removal (in situ)(1). All process noninterruptive techniques are by definition in situ procedures.

In-situ techniques in use today include high and low pressure water hosing, brushing, steam cleaning, externally-applied liquid acid soaking, internally injected acid gas or liquid, and flexing. The first four are all process interruptive methods that are applied to the external surfaces of diffusers. Internally-injected gas or liquid and flexing are the only process noninterruptive techniques to be commercially developed to date.

Hosing, brushing, and steam cleaning are all predicated on the use of physical action to dislodge loosely adherrent, liquid-side biological foulants. Biological foulants consist of an initial slime layer of primarily biological origin that subsequently entraps inert materials (sand, grit, etc.) that impinge on the diffuser surface. As the slime layer grows, a gelatinous mixture forms whose nonvolatile content can vary from 10 to 90 percent, depending on the character of the wastewater. Steam cleaning may provide an additional benefit in some cases of dissolving greases and oils in the gelatinous mixture.

Externally-applied liquid acid cleaning is used primarily to dissolve inorganic scale that forms on the surfaces and within the interstices of ceramic diffusers from chemical precipitation. Hosing, brushing, and steam cleaning are not effective for this purpose. A solution of 14-percent HCl is applied to the surface of each diffuser with a portable spray applicator and then hosed off after 20 to 30 minutes. Externally-applied liquid acid cleaning is not appropriate for flexible membrane diffusers and also not for porous plastic diffusers unless they are made of acid-resistant materials.

Some operators have found that a combination of rigorous cleaning techniques is necessary to cope with the variety of foulants that may exist at a given plant. For example, the sequence of hosing or steam cleaning followed by a 20- to 30-minute external application of 14-percent liquid HCl followed by rehosing of the spend acid has been reported to be effective in removing both organic and inorganic foulants from ceramic diffusers and restoring their transfer efficiencies to the original "like new" condition (1). This sequence using hosing as the first step is known as the Milwaukee method,

acknowledging one of the first cities that employed it.

Internal acid gas cleaning utilizes injection of an aggressive gas (HCl for porous ceramic diffusers; formic acid for porous plastic diffusers) into the air supply for passage through the diffuser media from the air side. Since the technique does not require basin dewatering, it can be readily applied at frequent intervals as needed. An approximate cost for internal HCl gas application is 6 to 7¢ per diffuser per cleaning (5). Initially developed by Sanitaire, Water Pollution Control Corp. (U.S. Patent No. 4,382,867) to combat chemical scale formation, there is now evidence that HCl gas cleaning may also be effective against some biofoulants (6). It will not remove inert granular material such as silica deposited or entrapped in gelatinous slimes adhering to the liquid side of ceramic diffusers. Its principal use is envisioned as a preventive procedure to extend the interval between more rigorous cleaning.

An alternative in-situ technique proposed by the Norton Company for cleaning ceramic diffusers utilizes liquid HCl instead of gaseous HCl. With this technique (7), liquid HCl is first injected into the air supply piping followed in succession by 1) turning off the air supply, 2) mixing the liquid acid with backflowing or added water, 3) forcing the solution into the diffuser media pores by briefly turning on the air supply, 4) turning off the air supply and allowing the acidic solution to saturate and contact the clogged pores for sufficient time to react with the foulants, and 5) forcing the spent solution into the basin liquid by turning the air supply on again. This sequence is generally repeated at least once to promote the maximum obtainable level of cleaning, consistent with the character of the foulants. Although a recently installed full-scale prototype liquid HCl cleaning system has reportedly been successfully demonstrated at La Crosse, Wisconsin (8), published results were not yet available at the time of this writing. The technique is claimed to be effective in dissolving and removing internal foulants (primarily carbonate scale) and causing the release of biological surface-adherring foulants (7). Effectiveness is reportedly enhanced in a high hardness, high alkalinity environment.

Flexing is applicable only to flexible membrane diffusers and involves periodically shutting off the air to collapse the flexible sheath onto its frame, then increasing the air flow to approximately twice its normal rate to balloon the sheath, and finally returning to the normal air flow rate. The premise of flexing is that alternate collapsing and ballooning of the membrane sheath causes foulants to break away from the surface and from around the orifices. Like internal gas cleaning, flexing is an inexpensive preventive cleaning strategy that can be used in an attempt to decrease the frequency of or need for rigorous cleaning.

Diffuser refiring (applied only to ceramic diffusers) is the principal ex situ cleaning technique being used today. It involves heating the diffusers in a kiln in the same fasion and to the same temperature used in their manufacture to thermally remove essentially all foulants from the surfaces and/or incorporated within the pores. Diffusers removed for refiring are typically replaced immediately with other diffusers that have already been refired to minimize downtime. The removed diffusers are transferred to a "diffuser bank" for later use once they are refired. Refiring, although

highly effective in restoring diffusers to their original condition, involves extensive labor and a certain degree of diffuser element breakage as well as the refiring operation itself. It is the most expensive of the methods currently in use and is practiced primarily in the United Kingdom.

Another ex situ cleaning technique used only in the United Kingdom is acid bath soaking of ceramic diffusers. This method is not widely practiced. The Screiber Corporation utilizes an ex situ jet washing machine to clean its fine pore tube diffusers.

The Green Bay Project

As described earlier in this paper, the Committee is assisting in sponsoring studies at seven municipal treatment plants that will evaluate all of the above diffuser cleaning methods except steam cleaning. A study of considerable magnitude is being undertaken at the time of this writing by the Green Bay (Wisconsin) Metropolitan Sewerage District (GBMSD). The District operates an activated sludge plant with an average design capacity of 2.3 m^3/sec (52.5 mgd). Currently, the plant is receiving a flow of about 1.8 m^3/sec (42 mgd) with an aeration system influent $TBOD_5$ concentration that averages approximately 440 mg/L. A combination of paper mill, meat packing, and canning wastes contributes significantly to the plant load. All of the influent wastewater receives primary treatment except the paper mill fraction of the industrial wastes.

The Green Bay aeration tankage is comprised of four contact stabilization quadrants (9). Each quadrant has a contact basin with dimensions of 74.4 m x 22.3 m x 6.2 m SWD (244 ft x 73.3 ft x 20.5 ft) and a stabilization basin with dimensions of 74.4 m x 11.1 m x 7.8 m SWD (244 ft x 36.3 ft x 22.5 ft). Both the contact and stabilization basins are operated in the plug flow mode. The Green Bay plant has excess treatment capacity and, therefore, does not need to keep all of its aeration basins in service at any one time. Typically, only enough contact and stabilization basins are operated to prevent the contact basin volumetric organic loading from exceeding 4.0 kg $TBOD_5$/day/m^3 (250 lb/ day/ 1000 cu ft).

Prior to the study, the Green Bay plant was equipped entirely with submerged turbine aerators: 93-kW (125-hp) units that actually drew 78 kw (105 hp) in the contact basins and 56-kW (75-hp) units that actually drew 48 kW (65 hp) in the stabilization basins (9). An analysis conducted in 1984 indicated that to transfer 81,700 kg O_2/day (180,100 lb/day) to satisfy an average design load of 81,700 kg $TBOD_5$/day (180,100 lb/day) would require a total power draw of 3500 kW (4695 hp) with the existing turbine aerators. These figures equate to a field aeration efficiency (α'SAE) of 1.02 kg O_2/kWh (1.67 lb/wire hp-hr). Estimated power requirements were split 39% for the turbines and 61% for the blowers. This power estimate was based on an assumed field OTE (at 2 mg/L DO) of 19% and an assumed alpha factor of 1.0.

The same analysis suggested that considerable power might be saved by retrofitting the Green Bay aeration basins with fine pore aeration (9). To treat the same 81,700-kg $TBOD_5$/day (180,100-lb/day) design load with fine pore ceramic disc diffusers, it was estimated that the total power draw could be

reduced to 1665 kW (2230 hp), a 52.5% savings in aeration energy. This projection, which was based on assumed field OTE's (at 2 mg/L DO) of 15.8-19.6% in the contact basins and 23.8% in the stabilization basins and assumed alpha factors of 0.6-0.75 in the contact basins and 0.9 in the stabilization basins, would result in an α'SAE of 2.14 kg O_2/kWh (3.51 lb/wire hp-hr). Based on a present worth analysis approach and the above energy projection, the payback period for a ceramic disc system was estimated at 1.5 years.

Likewise, an energy projection was also prepared based on retrofitting with fine pore flexible membrane tube diffusers (9). The assumed alpha factors were the same as for the ceramic disc diffusers, but the assumed contact and stabilization basin field OTE's (at 2 mg/L DO) were somewhat lower at 12.4-15.4% and 18.7%, respectively. The estimated aeration energy requirement to treat the aforementioned design load was 2125 kW (2850 hp), a reduction of 39.3% compared with the existing turbines and equivalent to an α'SAE of 1.67 kg O_2/kWh (2.75 lb/wire hp-hr). The projected payback period was 1.8 years.

To more accurately define the potential for saving aeration energy, the District decided to conduct a side-by-side comparison of the above two fine pore diffusion systems. Ceramic discs were installed in one quadrant and flexible membrane tubes in a second quadrant. The submerged turbines were left intact in the other two quadrants. Since the beginning of the evaluation in May 1986, the two fine pore diffuser quadrants have handled most of the plant load. A third contact basin (either one of the two remaining submerged turbine contact basins) is brought on line as needed to reduce the contact basin volumetric organic loading to 4.0 kg $TBOD_5$/day/m^3 (250 lb/day/1000 cu ft) or less.

GBMSD and its consultant postulated that the key to maintaining the desired high field OTE's and realizing maximum power savings was a preventive diffuser cleaning program that would routinely interrupt and reverse the inherent diffuser fouling pattern before extensive fouling occurred. Significantly high fine pore media fouling rates were anticipated due to the nature of the combined municipal/industrial wastewater. A program was initiated to investigate the effects of regular internal acid gas cleaning on the performance of the ceramic disc diffusers. Similarly, a program to evaluate the efficacy of regular membrane flexing on the performance of the flexible tube diffusers was also implemented. The contact and stabilization basin air diffusion systems were divided into six plug flow grids or bays per basin (Figure 7)(10). In the contact basins, the north grids were cleaned and the south grids were not, providing a control against which to measure cleaning effectiveness. In the stabilization basins, only every other grid or bay was subjected to cleaning.

Using Figure 7 as a reference, grids C-1N, C-2N, and R-1 of the ceramic disc quadrant were gas cleaned on 5/29/86, 7/10/86, 8/19-8/20/86, 9/24/86, and 10/21/86. Grids C-3N, C-4N, and R-3 were gas cleaned on 7/10/86, 8/19-8/20/86, and 9/30/86, while grids C-5N, C-6N, and R-5 were cleaned only on 8/19-8/20/86. As indicated by the numbering sequence, the more heavily loaded grids (nearer the influent end) were cleaned more frequently. In the flexible member diffuser quadrant, the north grid diffusers in the contact

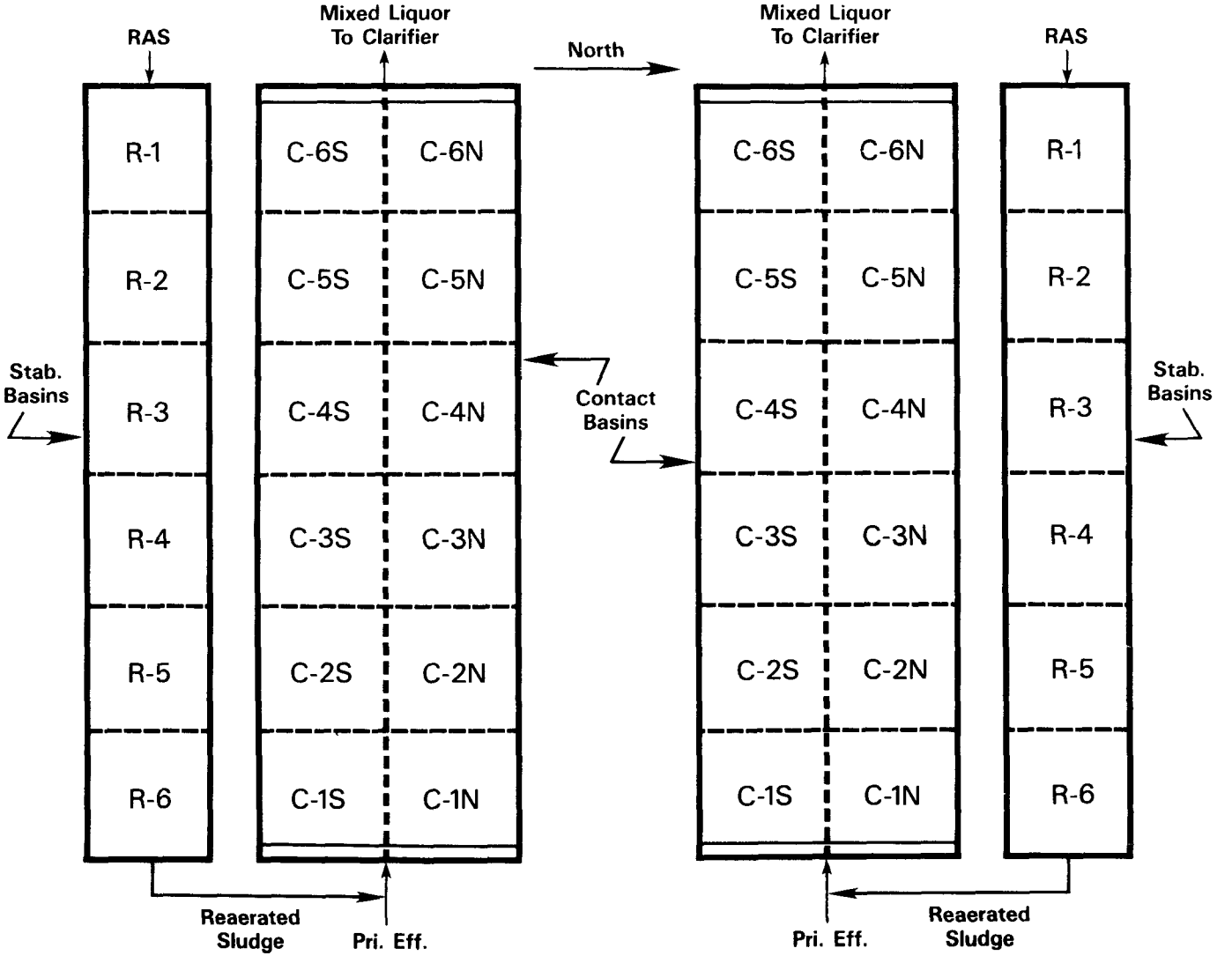


FIGURE 7. Schematic Diagrams of Flexible Tube and Ceramic Disc Aeration Basins at Green Bay, Wisconsin (10).

basin (C-1N through C-6N) and every other stabilization basin grid (R-1, R-3, and R-5) were air flexed every 3 weeks during the May-October 1986 period. For this period, the SRT of the ceramic disc system averaged 2.65 days compared to 2.61 days for the flexible tube system (10).

Off-gas OTE results for five tests are summarized in Table 5 comparing performance of the cleaned vs. the uncleaned grids for the two contact basins. As indicated, the tests span the period prior to the initiation of acid gas cleaning and membrane flexing (i.e., when the diffusers were new) through October 1986.

In Table 6, the oxygen transfer performance of the cleaned and uncleaned grids is combined for each quadrant. α'SOTE results and estimates of apparent alpha are given for both the contact and stabilization basins. As indicated, an extra contact basin equipped with existing submerged turbine aerators was in service on three of the five test dates to reduce the organic loading on the ceramic dome and flexible tube contact basins.

The two test systems were placed in side-by-side operation (the ceramic discs new and the flexible tubes after one process interruptive rigorous cleaning) on 4/26/86. The data in Tables 5 and 6 show that process noninterruptive preventive cleaning was apparently not required to maintain oxygen transfer efficiency at original "like new" levels in either system for the first two months (through 7/2/86). Sometime in July, oxygen transfer efficiencies began to drop as noted by the 7/30/86 off-gas test results. The substantial drops in α'SOTE values during this period, indicative of rapid fouling rates, may have been stimulated by the onset of the local canning season. Continued regular preventive cleaning through October 1986 failed to fully restore oxygen transfer performance to its original level. When compared with uncleaned diffuser test results (Table 5), it is questionable whether preventive cleaning had any significant effect on diffuser oxygen transfer performance in the contact basin of either system.

Comparing the average of the clean and uncleaned diffuser test data in Table 6 indicates little difference in process water performance between the two systems. Overall, oxygen transfer performance for the two diffuser systems through October 1986 was judged to be equivalent by the project engineer (10).

Because of deterioriating effluent quality in October 1986 (presumably due in part to reduced oxygen transfer performance), Green Bay plant personnel requested that rigorous cleaning of both systems be considered. The project engineer concurred, and it was decided to drain and clean both aeration basin systems in November 1986 before severe winter conditions arrived.

The rigorous cleaning procedure used for the ceramic discs consisted of low pressure hosing with fire hoses from the basin walkways, partially filling the basins with water and gas cleaning the diffusers using 0.09 kg (0.2 lb) HCl gas per disc, draining the basin again, and rehosing from the tank top. The flexible membrane tubes were first subjected to high pressure hosing from 1-1/2 to 3 m (5 to 10 ft) using a jet rodding machine with a discharge pressure at the nozzle of about 9 kN/m^2 (100 psig) followed by

TABLE 5. COMPARISON OF CONTACT BASIN OXYGEN TRANSFER PERFORMANCE FOR PREVENTIVELY CLEANED VS. UNCLEANED DIFFUSER GRIDS, GREEN BAY, WISCONSIN (10).

	α'SOTE (%)			
	Ceramic Disc Quadrant		Flexible Tube Quadrant	
Date	North Grid (Cleaned)	South Grid (Uncleaned)	North Grid (Cleaned)	South Grid (Uncleaned)
5/13/86	14.1	15.8	16.6	16.7
5/15/86	14.9	14.8	16.7	16.2
7/2/86	16.2	17.1	16.9	16.6
7/30/86	10.4	9.1	9.8	14.9
10/30/86	11.9	12.5	14.1	14.6

TABLE 6. COMPARISON OF CERAMIC DISC AND FLEXIBLE MEMBRANE TUBE OXYGEN TRANSFER PERFORMANCE DURING PREVENTIVE CLEANING FOR CONTACT AND STABILIZATION BASINS, GREEN BAY, WISCONSIN (10).

		Ceramic Disc Quadrant			Flexible Tube Quadrant			
Date	Basin	Air Flow Rate per Diffuser (L/sec)	α'SOTE (%)	α'	Air Flow Rate per Diffuser (L/sec)	α'SOTE (%)	α'	Extra Contact Basin in Service
5/12/86	Stab.	0.73	18.5	0.50	1.58	17.8	0.53	
5/13/86	Contact	0.86	15.0	0.45	1.19	16.7	0.48	Yes
5/16/86	Stab.	0.80	17.2	0.50	1.65	17.2	0.52	
5/15/86	Contact	1.03	14.9	0.47	1.47	16.4	0.50	No
7/1/86	Stab.	1.14	21.4	0.65	1.65	18.6	0.57	
7/2/86	Contact	0.98	16.9	0.57	1.30	16.7	0.53	Yes
7/29/86	Stab.	0.94	14.6	0.42	1.89	11.5	0.35	
7/30/86	Contact	1.53	9.8	0.32	1.77	12.2	0.40	No
10/29/86	Stab.	1.09	11.6	0.34	1.62	13.4	0.41	
10/30/86	Contact	1.25	12.2	0.39	1.52	14.3	0.44	Yes
Avg.	Stab.	0.94	16.7	0.48	1.68	15.7	0.48	
Avg.	Contact	1.13	13.8	0.44	1.45	15.3	0.47	

scrubbing with a nylon brush and low pressure hosing or rinsing. The above cleaning procedures were applied to all diffusers in the two test contact basins and two test stabilization basins, not just the diffuser grids to which preventive cleaning had previously been applied.

The effect of rigorous cleaning on the two systems was assessed in early December 1986 using off-gas analysis. The results of this testing are summarized in Table 7. α'SOTE data for this test plus the five off-gas tests conducted during the preventive cleaning cycle are plotted in Figure 8 for the contact basins and in Figure 9 for the stabilization basins.

TABLE 7. EFFECT OF RIGOROUS CLEANING ON CERAMIC DISC AND FLEXIBLE MEMBRANE TUBE OXYGEN TRANSFER PERFORMANCE, GREEN BAY, WISCONSIN (10)

		Ceramic Disc Quadrant			Flexible Tube Quadrant			
Date	Basin	Air Flow Rate per Diffuser (L/sec)	α'SOTE (%)	α'	Air Flow Rate per Diffuser (L/sec)	α'SOTE (%)	α'	Extra Contact Basin in Service
12/2/86	Stab.	0.81	19.6	0.55	1.15	13.1	0.38	
12/3/86	Contact	0.75	19.1	0.57	0.96	16.3	0.46	Yes

It is apparent that rigorous cleaning had greater positive impact on the ceramic discs than on the flexible tubes. Ceramic disc α'SOTE's were restored to values in both the contact and stabilization basins that were higher than the May 1986 ("like new" condition) values. On the other hand, the flexible tube stabilization basin α'SOTE did not improve at all over the October 1986 value while the contact basin exhibited a modest improvement over the October 1986 value but not quite back to the May 1986 ("like new" condition) value.

Further work is required to determine why the rigorous scrubbing and hosing technique failed to fully restore flexible membrane diffuser oxygen transfer performance to "like new" conditions. Another 10 months of study were scheduled for Green Bay beyond December 1986 to further evaluate preventive and rigorous cleaning on both types of fine pore diffuser systems. These data were not available to the authors during the preparation of this paper.

It is interesting to note the effect of diffuser fouling on air flow rate per diffuser. Initially, both types of diffusers were operating within their recommended specific air flow rates: 0.47-0.94 L/sec (1-2 scfm) for the ceramic discs and 1.18-1.65 L/sec (2.5-3.5 scfm) for the flexible tubes. As fouling progressed, particularly in July 1986, specific air flow rates rose to levels outside the recommended range for the discs and near the upper end or outside the recommended range for the tubes. Increasing specific air flow rates produce higher headlosses, larger bubbles, and lower OTE's (Table 6). After rigorous cleaning was implemented, the specific air flow rates for

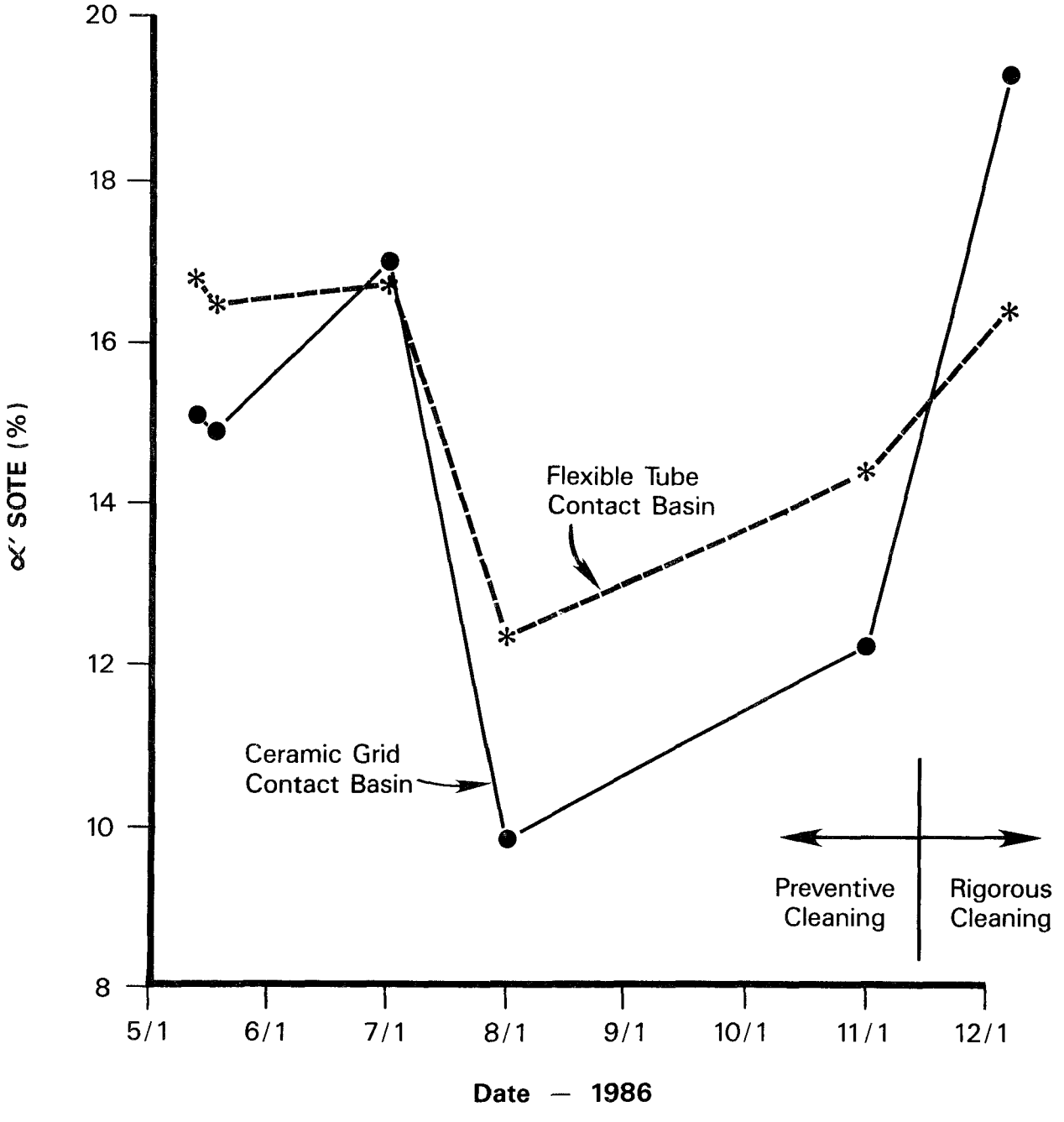


FIGURE 8. Effect of Diffuser Cleaning on Contact Basin Fine Pore Aeration Systems at Green Bay, Wisconsin (10).

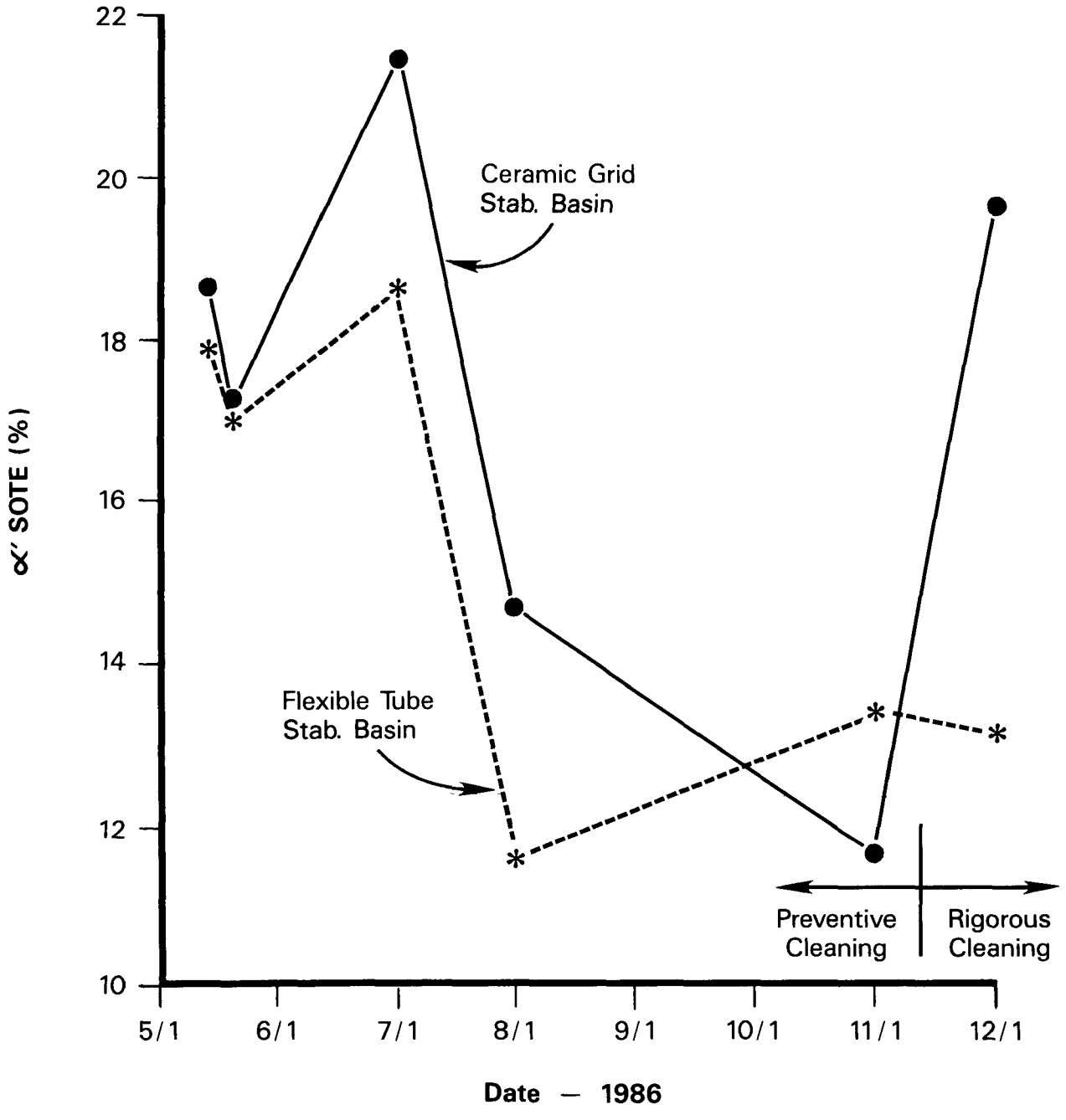


FIGURE 9. Effect of Diffuser Cleaning on Stabilization Basin Fine Pore Aeration Systems at Green Bay, Wisconsin (10).

both diffuser types decreased back into or to lower than their recommended ranges (Table 7). Since α' values for the flexible membrane diffusers did not change dramatically after rigorous cleaning, it appears that the reduced specific air flow rates for these units were due in part, at least, to a decrease in oxygen demand. This is plausible since the Green Bay canning season usually ends in November.

A similar effect of fouling on α' can also be seen in Tables 6 and 7. When the diffusers were first installed, α' and α were theoretically the same with values of 0.45 to 0.50 for the ceramic discs and 0.48 to 0.53 for the flexible tubes. While the diffusers were still relatively clean through early July 1986, α' remained at or slightly above the original values in all four basins. With the onset of the canning season, however, diffuser fouling rates increased rapidly and α' dropped to 0.4 or lower in all basins. Rigorous cleaning restored α' to better than the original conditions for the ceramic discs, but the flexible tube α' values continued to decrease. This discrepancy provides further evidence that while the drop-off in oxygen transfer performance for the discs was due to fouling, the performance deterioration for the flexible tubes was caused by other factors, such as possible changes in diffuser characteristics.

The results of early testing at Green Bay suggest that the District will have to modify its original planned operating procedure for a retrofitted plant. It was estimated that if α' could be maintained between 0.6 and 0.75, the plant could meet all expected oxygen demand situations with only two of four available quadrants in service. Not only were the "like new" α' values lower than this range, but preventive cleaning failed to significantly retard decreases in these values once the canning season load began to be received at the plant.

Based on results to date and the assumption that more diffusers will not be installed in the two fine pore quadrants, the following operating strategies, among others, appear to be potential options for GBMSD: 1) utilize more frequent rigorous cleaning to maintain α' values as high as possible, 2) operate a third contact basin (equipped with submerged turbine aerators) as needed to alleviate the organic load on the retrofitted basins, 3) operate the fine pore diffusers at higher specific air flows than recommended by the manufacturer, or 4) a combination of two or more of the above. Regardless of what choice is made, the District will probably have to accept lower field OTE's and a somewhat longer payback period than originally hoped for. It should be emphasized, however, that despite the lesser performance, the District will still save substantial aeration energy compared with the existing submerged turbine equipment.

The Whittier Narrows Project

In contrast to Green Bay where, to date, internal acid gas cleaning has been ineffective in maintaining consistent oxygen transfer performance, the Whittier Narrows Water Reclamation Plant of the Los Angeles (California) County Sanitation Districts (LACSD) aappears to be maintaining oxygen transfer performance at consistent levels with an acid gas cleaning program. Whittier Narrows is a high-rate plant with an F/M loading of about 1.3 kg

total chemical oxygen demand (TCOD)/day/kg MLVSS, or 0.7 kg $TBOD_5$/day/kg MLVSS, and an SRT of 1.7 to 2.9 days (11). The plant has three parallel aeration basins currently being operated in the plug flow configuration. Two of the basins are equipped with ceramic dome diffusers and the other with ceramic disc diffusers.

Historically, these diffusers have tended to slime fairly rapidly possibly due to such factors as wastewater characteristics and low-SRT and low-DO modes of operation. Prior to May 1986, the α'SOTE of the disc aeration system was being maintained at an average value of approximately 8.5% by periodic tank draining followed by hosing of the diffusers from the tank top.

In May 1986, the diffusers in all three basins were subjected to the Milwaukee cleaning method (high pressure hosing/external liquid HCl spray application/high pressure hosing). Just prior to the rigorous Milwaukee method cleaning, α' for the discs was 0.28. After the cleaning, apparent alpha values for the disc diffuser system were elevated to about 0.33 and α' SOTE values to about 10.3 percent by this technique (11).

Since May 1986, the ceramic disc basin has been internal acid gas cleaned on a schedule of every 3 months for the first diffuser grid, every 6 months for the second grid, and every 9 months for the third grid. This procedure has so far maintained the α' and α'SOTE values, respectively, at approximately the 0.33 and 10.3 percent levels established by the Milwaukee method restoration (11). Gas cleaning also appears to be effective in preventing a buildup in DWP.

A similar cleaning program was instituted on one of the ceramic dome basins. Unfortunately, gas leakage between the diffuser media and the base plates on some of the dome diffusers has made a comprehensive evaluation impossible. The leakage may be partially due to atypical gaskets and/or bolts. Negotiations are currently underway with the diffuser manufacturer to replace the diffusers, gaskets, and bolts so that an effective gas cleaning evaluation program on domes can be implemented at Whittier Narrows.

At this point in time, acid gas cleaning appears to be a technically feasible alternative to rigorous manual cleaning for maintaining consistent α' SOTE values at Whittier Narrows. The cost effectiveness of this cleaning method, however, needs to be further evaluated under varying plant operating conditions.

Acid gas cleaning studies on ceramic fine pore diffusers are also underway at Frankenmuth, Michigan. Acid gas cleaning is utilized whenever the DWP of Frankenmuth's ceramic discs reaches 46 cm (18 in.). Gas cleaning results in a lowering of DWP to 18 to 20 cm (7 to 8 in.). At the time of this writing, off-gas test data were not yet available to determine whether the reduction in DWP is also accompanied by an increase in α'SOTE. Acid gas cleaning effectiveness at all three sites discussed above will be updated in the CDOM in 1989.

SUMMARY

Significant progress has been made in filling the technical gaps on fine pore diffuser design and operation identified by the ASCE Committee on Oxygen Transfer in 1985. Much still remains to be learned. While it is unreasonable to expect that all questions will be answered by 1989, it is believed sufficient new information and sound engineering data will have been generated to permit the preparation of a truly comprehensive design and operating guide document.

The progress made to date prompts the authors to offer the following interim summary observations:

1. While of concern with some plants, fine pore diffuser fouling in the United States does not appear to be as prevalent or severe as first contemplated.

2. Other factors, most notably wastewater characteristics and process operating conditions, play a larger role than fouling in lowering field OTE's for some fine pore diffuser installations.

3. A correlation appears to exist between fine pore diffuser performance and SRT (or F/M loading). Increasing SRT promotes improved diffuser performance; as much as a two-fold increase in α'SOTE has been noted over the range of SRT's studied.

4. It may be possible to save aeration energy by increasing process SRT from a non-nitrifying to a nitrifying range. This can happen when the resulting increase in α'SOTE is more rapid than the increase in oxygen demand/oxygen uptake caused by nitrification and greater endogenous respiration.

5. Preliminary information indicates that plug flow operation may produce higher α'SOTE values than the step aeration mode of operation, at least in short-SRT systems.

6. Rigorous cleaning, while necessitating tank draining and process interruption, is effective in restoring ceramic diffuser field OTE to "like new" conditions.

7. Internal acid gas cleaning effectiveness in restoring ceramic fine pore diffuser performance and/or retarding the rate of field OTE loss appears to be site specific.

REFERENCES

1. Summary Report - Fine Pore (Fine Bubble) Aeration Systems. Editor - W.C. Boyle, EPA/625/8-85/010, U.S. EPA, Technology Transfer and Water Engineering Research Labortory, Cincinnati, Ohio, October 1985.

2. American Society of Civil Engineers. ASCE Standard - Measurement of Oxygen Transfer in Clean Water. ISBN 0-87262-430-7, New York, New York, July 1984.

3. American Society of Civil Engineers. Oxygen Transfer Under Process Conditions. Editor - W.C. Boyle, Final report for U.S. EPA Cooperative Agreement No. CR808840, Water Engineering Research Laboratory, Cincinnati, Ohio, In press.

4. Robertson, P., V.K. Thomas, and B. Chambers. Energy Saving-Optimization of Fine-Bubble Aeration. Final report for U.S. EPA Cooperative Agreement No. CR808855, Water Engineering Research Laboratory, Cincinnati, Ohio, Prepared by Water Research Centre, Stevenage, England, In press.

5. Survey report from M.J. Pierner, Green Bay Metropolitan Sewerage District, Green Bay, Wisconsin, to J.A. Heidman, U.S. EPA, Cincinnati, Ohio, April 15, 1987.

6. Wren, J.D. Gas Cleaning of Ceramic Diffusers. Biofouling Seminar, New York Water Pollution Control Federation Annual Meeting, Hyatt Regency, New York, New York, January 22, 1985.

7. In-Situ Acid Cleaning System for Porous Diffuser Systems. Product information bulletin, Norton Company, Northboro, Massachusetts, 1986.

8. Personal communication from G. Rushton, Norton Company, Northboro, Massachusetts, to R.C. Brenner, U.S. EPA, Cincinnati, Ohio, July 27, 1987.

9. Letter report from J.J. Marx, Donohue & Associates, Inc., Sheboygan, Wisconsin, to P. Thormodsgard, Green Bay Metropolitan Sewerage District, Green Bay, Wisconsin, November 30, 1984.

10. Letter report from J.J. Marx, Donohue & Associates, Inc. Sheboygan, Wisconsin, to P. Thormodsgard, Green Bay Metropolitan Sewerage District, Green Bay, Wisconsin, March 4, 1987.

11. Personal communication from M.K. Stenstrom, University of California at Los Angeles, California, to R.C. Brenner, U.S. EPA, Cincinnati, Ohio, July 10, 1987.

ACKNOWLEDGMENTS

In addition to recognizing the subcontractors and Steering Subcommittee members who have worked on this project over the past 2 years, the authors wish to especially acknowledge the cooperation of the Green Bay (WI) Metropolitan Sewerage District, the Madison (WI) Metropolitan Sewerage District, and the Los Angeles County (CA) Sanitation Districts. In addition, we wish to thank Dr. Michael K. Stenstrom, Mr. James J. Marx, Dr. Hugh J. Campbell, Jr., Dr. James A. Heidman, Dr. James J. McKeown, Mr. Fred W. Yunt, Mr. Lloyd Ewing, Mr. David T. Redmon, Dr. Henryk Melcer, Mr. Gordon Speirs, and Mr. Jerome D. Wren for their review of this manuscript.

THE USE OF BIOMONITORING IN MICHIGAN'S SURFACE WATER QUALITY PROGRAM

by

Paul D. Zugger, Chief
Surface Water Quality Division
Michigan Department of Natural Resources
P.O. Box 30028
Lansing, Michigan 48909

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

Prepared for Presentation at:

Eleventh United States/Japan Conference
on Sewage Treatment Technology
Tokyo, Japan

October 12-14, 1987

CONTENTS

Page

INTRODUCTION.....705

SURVEILLANCE BIOMONITORING.....706
Fish Contaminant Monitoring.....706
Biosurveys.....708

COMPLIANCE BIOMONITORING.....709
Acute Toxicity Test Methods.....710
Chronic Toxicity Test Methods.....711
Quality Assurance/Control Tests.....712
Biouptake Assessments.....712

BIOMONITORING IN SURFACE WATER DISCHARGE PERMITS.....713

SUMMARY.....714

ACKNOWLEDGEMENTS.....715

FIGURES AND TABLES.....716

Figure 1. Procedure for Preparation of "Standard fillets" analyzed in this study.....717

Table 1. Standard Edible Portions of Michigan's Sport and Commercial Fishes.....718

Table 2. Fish Analysis Parameters List.....719

APPENDICES.....720

A. Physical, Chemical and Biological Monitoring Results from the Kalamazoo River, Comstock to Plainwell, 1984, dated February, 1986.....721

B. A Biological Investigation of Prairie River and Prairie River Lake, St. Joseph County, August 12, 1986.....744

C. Water, Sediment, and Macroinvertebrate Survey of the South Branch Raisin River, and Eastside Drain, vicinity of Adrian, 1982-85.....752

D. Department of Natural Resources, Water Resources Commission, General Rules, Part 4. Water Quality Standards, filed with Secretary of State November 14, 1986.....768

INTRODUCTION

Biomonitoring is the assessment of water quality through the observation of impacts of contaminants in water and wastewater on biological organisms. Biomonitoring has been a key aspect of Michigan's water pollution control program for many years. Michigan has used biological techniques to assess the quality of surface waters of the State and wastewater effluents since the early 1950's. Biological techniques which have been employed consist of acute fish and macroinvertebrate toxicity tests on effluents, fish contaminant monitoring, taste and odor studies, biouptake studies and the traditional biosurveys which evaluate instream benthic macroinvertebrate and fish communities.

Many of the early biomonitoring activities were conducted to evaluate gross pollution problems caused by excessive discharges of solids, oxygen consuming substances, oils, and heavy metals. Prior to the major pollution control programs of the last 25 years, severe impacts on biological communities, including frequent fish kills, were prevalent. Pollution-tolerant aquatic organisms dominated the benthic community for long distances downstream of wastewater discharges. The elimination of unacceptable acute toxicity was a primary concern of the water pollution control programs of that time period.

Today water pollution problems in Michigan are quite different. As a result of efforts of municipal and industrial dischargers, and a strong regulatory program, gross problems have been essentially eliminated. More subtle impacts to the biological communities of our surface waters, however, remain and are of concern. Chronic toxicity to aquatic organisms, including reproductive effects, and the bioaccumulation of toxic substances by aquatic organisms with the eventual impact on fish-eating birds and mammals are especially significant. In response to the changing types of problems, biological techniques for documenting and defining the extent of these concerns have to be modified and refined.

Today Michigan's biomonitoring program consists of both a surveillance aspect and a compliance monitoring aspect. Surveillance monitoring is conducted in ambient waters while compliance monitoring is conducted primarily on wastewater discharges. Major activities under both of these aspects are summarized in this paper. The use of biomonitoring in Michigan's discharge permit program, through which Michigan administers the federal National Pollutant Discharge Elimination System (NPDES) permit program, is also discussed.

SURVEILLANCE BIOMONITORING

Surveillance monitoring is comprised of two primary activities: Fish Contaminant Monitoring, and Biological Lake and Stream assessments.

Fish Contaminant Monitoring

Michigan's first comprehensive fish contaminant monitoring program was conducted from 1969 through 1978. In 1986, Michigan reinstituted and expanded the fish contaminant monitoring program at an annual cost of approximately $400,000. Under this program, fish are collected from approximately 40 sites, totaling approximately 700 samples.

The goal of the fish contaminant monitoring program is to collect and analyze data on contaminant levels in fish so that fisheries managers, water quality administrators, public health officials, and the public can make informed management decisions and be apprised of the contaminant levels in fish from various geographical areas of Michigan. The specific objectives of the program are:

- To collect and analyze fish to determine their contaminant status;
- To develop a computerized data base for storing and statistically analyzing fish contaminant data;
- To evaluate the overall quality of surface waters through analysis of fish contaminant data;
- To evaluate the effectiveness of surface water quality regulatory programs to control persistent, bioaccumulative chemicals;
- To develop and implement an effective means of communicating fish contaminant data to the public on a timely basis;
- To identify "new" chemical contaminants in fish and determine long-term trends of toxic substances concentrations in fish.

Several of the objectives noted above will take many years of effort and refinement to fully accomplish. The program consists of the sampling and analyzing of sport fish in the Great Lakes and nearshore areas (Elements 1 and 2), in inland rivers and streams (Elements 3 and 4), and in inland lakes (Elements 5 and 6), and the performance of limited special studies (Element 7).

Element 1 - Great Lakes Sport Fish Monitoring: Sport fish from the Great Lakes and connecting channels within Michigan's political boundaries are collected for contaminant analyses. Standard edible portions are analyzed and the data compared to Michigan Department of Public Health (MDPH) action levels or other criteria. Based upon the results, public health advisories may be issued or revised for sport anglers by the MDPH; management information can be transmitted to appropriate pollution control authorities; and long-term trends can be analyzed.

Element 2 - Great Lakes Nearshore Problem Identification: River mouths and embankments, particularly those which receive significant municipal and/or industrial discharges, have been identified as locations with potential toxic materials contamination. Whole fish of species and sizes likely to accumulate contaminants are collected and analyzed, or caged fish studies are conducted to detect new or previously unidentified contamination, identify geographic areas of concern for a broad range of contaminants, and provide long-term trend data which can be used to evaluate pollution control programs.

Element 3 - Stream and River Sport Fish Monitoring: Sport fish from rivers and streams are collected for contaminant analyses. Standard edible portions are analyzed and compared with MDPH action levels. Based upon the results, public health advisories may be issued or revised for sport anglers; management information is transmitted to appropriate pollution control authorities; and evaluations are made regarding the feasibility of fish restoration projects.

Element 4 - Stream and River Problem Identification: Rivers and streams that receive significant municipal and/or industrial discharges or are impacted by non-point source pollution are included in this element. Whole fish of species and sizes likely to accumulate contaminants are collected and analyzed to detect new and/or previously unidentified contamination. This information is used to identify geographic areas of concern for a broad range of contaminants and as timely management information by appropriate pollution control authorities.

Element 5 - Inland Lakes Sport Fish Monitoring: Sport fish from inland lakes are collected for contaminant analyses. Standard edible portions are analyzed and compared to MDPH action levels and other criteria. Based upon the results, public health advisories may be issued or revised and timely management information is transmitted to appropriate fisheries managers and pollution control authorities.

Element 6 - Inland Lakes Problem Identification: Fish from inland lakes are collected for contaminant analyses. Whole fish are analyzed to detect new and/or previously unidentified contamination and identify geographic areas of concern for a broad range of contaminants, and as management information by appropriate pollution control authorities.

Element 7 - Special Studies: This element is intended to cover those fish monitoring activities which do not directly fit into any of the above mentioned elements. Thus, fish collection and analytical details will depend on the needs of the particular study being undertaken.

Fish are collected using standard fish census techniques as appropriate for the water body and analytical method. The methods include electrofishing, trap nets, gill nets, seizing, trawling, and hook-and-line fishing. Some of the fish collected from the Great Lakes are taken by sport anglers and donated to the Department for use in this effort.

Fillet samples are generally processed (Figure 1) on site in the field to obtain the "standard edible portion" for the species as shown in Table 1. The specimens are measured (total length), weighed and sexed prior to filleting. Each sample is individually wrapped in aluminum foil, placed in a separate polyethylene bag and kept on ice until it can be placed in freezers until time of analysis.

Samples are analyzed by laboratories using acceptable methods of digestion, extraction, and quantification and having adequate quality assurance programs. In general, the fish are analyzed for all or a portion of the parameters listed in Table 2. Additional parameters are included based on site specific concerns.

Biosurveys

Biological lake and stream assessments, or biosurveys, are the traditional means of assessing the quality of ambient water. These assessments have been used in Michigan for decades as indicators of the quality of streams. One of the principal means of assessing the progress toward achieving the goals of state and federal water pollution control laws and the effectiveness of water pollution control efforts is the use of biosurveys. Biological community surveys are those related to determining if the biological community present at a site is adversely impacted by water pollution. Water bodies in which the biological communities are not healthy are evaluated by follow-up work to identify the causes of the impacts. Necessary corrective programs and remedial actions are identified and required to be implemented through the state regulatory program. After the remedial actions are completed, the water body is reassessed to determine the success of the corrective programs.

There are two basic types of biosurveys - those which involve a survey of an entire river or lake system as a whole, and those oriented toward a specific problem evaluation. An example of a river system study is the study performed on the Kalamazoo River in 1984 by the Department of Natural Resources (Appendix A). Comprehensive studies such as these are very staff intensive and expensive.

The majority of our biosurveys are the problem-evaluation type. Problem evaluation surveys are directed at assessing a particular problem (such as a point source wastewater discharge), evaluating the success of a remedial program, or investigating a more general concern such as nonpoint source effects. These surveys range from one to many stations but are generally confined to a few problems in a limited area.

There are two types of problem evaluation surveys: Site Investigations and Intensive Studies. The difference between these surveys is the level of effort involved. Site Investigations are limited in effort, generally involving only a few stations. Intensive Studies are more comprehensive and usually include five or more stations.

An example of a Site Investigation survey is the Prairie River and Prairie River Lake study performed in August, 1986 (Appendix B). An

example of an Intensive Study survey is the Adrian vicinity study of 1982-1985 (Appendix C).

In general, these studies entail a description of the physical nature of the stream and its indigenous biological communities at selected sites along a reach of stream. Fish, aquatic insects, and plants are identified and on-site estimates of their abundance and distribution are made. Quantitative and/or qualitative samples of the biota are collected and evaluated in the field or laboratory as necessary.

One of the principal reasons for these studies is to support the discharge permit program, Michigan's principal regulatory tool. This program is managed so that ideally all 1,400 discharges have current permits, reissued on a five-year cycle on a river basin basis. The basin approach provides for all the permits in a basin to be considered together, thereby facilitating waste load allocations and assuring that interactions among various discharges are addressed.

The priorities for water body investigations are determined from the river basins which are scheduled for NPDES permit reissuance during a subsequent fiscal year. For permit reissuance, the data on sites must be available one-to-two years in advance of permit reissuance. Therefore, necessary water body investigations are conducted on a priority basis one-to-two years ahead of the fiscal year in which the permits are due for reissuance. Ideally, all discharges with potential for environmental damage are monitored prior to permit reissuance, and the results of necessary biological studies are available at the time of permit processing.

Problem evaluation surveys are also initiated upon r quest from district field staff or to support enforcement actions. For en' >rcement purposes it is usually necessary to document resource damages resulting from the illegal activity and to quantify the extent of damage. Problem evaluation surveys are also used to determine the appropriate designated use for a waterbody and to identify necessary follow-up studies such as toxicity testing or dissolved oxygen studies.

Currently, Michigan is conducting approximately 30 site investigations and 10 intensive studies per year. It is MDNR's goal to increase biosurvey capabilities to 50 site investigations and 15 intensive studies per year.

COMPLIANCE BIOMONITORING

Compliance Biomonitoring is the biomonitoring program which assures that discharges do not cause unacceptable toxicity. These activities are performed under the Department's Aquatic Toxicity Evaluation Program. These aquatic toxicity tests examine the toxic effects of different concentrations of a test substance (e.g., an industrial or municipal effluent) on aquatic organisms. Replicate vessels of either individual or groups of aquatic organisms are maintained under static, static/renewal or flow-through conditions. Aquatic toxicity tests include short-term exposure (usually less than one week) to measure acute toxicity and long-term exposure tests to measure

chronic toxicity. Effluent aquatic toxicity tests are conducted in a permanent-based laboratory as well as on-site at the actual discharge location using a mobile laboratory. The goals of the Aquatic Toxicity Evaluation Program are to:

- Measure the acute and/or chronic toxicity of industrial and municipal effluents, receiving waters, and other water matrices;
- Assess whether effluents and receiving waters are in compliance with the aquatic toxicity-related requirements of the Michigan Water Quality Standards;
- Assess dischargers' compliance with whole effluent toxicity limits in NPDES permits;
- Screen wastewater effluents for acute and/or chronic toxicity for purposes of identifying potential problem discharges;
- Generate quality toxicological data for effluents/receiving waters to assist enforcement actions against violators of environmental laws;
- Verify the appropriateness of aquatic toxicity-based chemical specific effluents limits in a facility's NPDES permit, taking into account combined effects of the various pollutants in discharges; and
- Develop and validate new and innovative methods for assessing aquatic toxicity of effluents and receiving waters.

The following situations receive priority attention for compliance monitoring activities:

- Wastewater discharges with NPDES permits scheduled for reissuance in two years for which concern exists for aquatic toxicity;
- Industrial/municipal facilities which discharge wastewater to watersheds targeted for special attention in the Michigan Department of Natural Resources - Surface Water Quality Division Management Plan;
- Wastewater discharges which have whole effluent toxicity limits and/or toxicity testing requirements in their NPDES permits; and
- Industrial/municipal effluents or leachates, known or suspected of being toxic to aquatic life, which are targeted for special regulatory enforcement action.

The methods listed below are currently run in the Aquatic Toxicity Evaluation Program.

Acute Toxicity Test Methods

- Daphnia static acute toxicity test.

This test is relatively inexpensive and requires 48 hours to complete. The test endpoint is "immobilization" of the test animal. Results can be used to assess whether the aquatic toxicity-related requirements of Rule 82 and, in some cases, Rule 57 of the Michigan Water Quality Standards are being satisfied (see Appendix D).

° Ceriodaphnia static acute toxicity test.

This test is similar in design and value to the Daphnia static acute toxicity test. The test organism in Ceriodaphnia dubia.

° Larval fathead minnow (Pimphates promelas) static/renewal acute toxicity test.

This test is relatively inexpensive and requires 48 or 96 hours to complete. The test endpoint is death of the larval fish. Results can be used to assess whether the aquatic toxicity-related requirements of Rule 82, and in some case, Rule 57 are being satisfied.

° Fathead minnow (Pimphates promelas) rainbow trout (Sahno gairdnerii) flow-through acute toxicity test.

This test must be performed on-site using a mobile laboratory. Consequently, staffing cost is considerably higher than that required for a static test. Test duration is 96 hours and the primary test endpoint is death of the test organism. In the mobile laboratory, test fish can be exposed to actual effluent concentrations which exist in the receiving stream or lake after mixing with allocated design flow. Test results can be used to assess whether the aquatic toxicity-related requirements of Rule 82 and, in some cases, Rule 57 are being met.

Chronic Toxicity Test Methods

° Daphnia magna 21 day chronic toxicity test.

This test is a static/renewal test which, due to its duration, is rather costly. Test endpoints usually are death and reproductive impairment. Test results can be used to assess whether the aquatic toxicity-related requirements of Rule 57 are being satisfied.

° Embryo/larval fish partial life cycle toxicity test.

This test can be performed under flow-through or static/renewal exposure conditions and requires 28 days to complete. Fathead minnows and rainbow trout are the most commonly used test species. Because of the 28 day exposure period, the test is also rather costly and time consuming. Test results can be used to assess whether the aquatic toxicity-related requirements of Rule 57 are being met.

° Fathead minnow larval survival and growth test.

This newly developed test is a static/renewal experiment which requires seven days to complete. The reduced exposure period and relatively low cost represent its primary advantages compared to the more traditional chronic test methods. Test endpoints are death and growth (weight gain). Test results can be used to assess whether the aquatic toxicity-related requirements are being satisfied.

° Ceriodaphnia survival and reproduction test.

This newly developed test is a static/renewal experiment which usually requires seven days to complete. The reduced exposure period and relatively low cost represent its primary advantages. Test endpoints are death and reproductive impairment. Test results can be used to assess whether the aquatic toxicity-related requirements of Rule 57 are being met.

Quality Assurance/Control Tests (QA/QC)

For purposes of demonstrating acceptable test organism culture health, adequate laboratory water quality, and consistency in test method data, the following QA/QC tests are conducted with standard reference toxicants:

a) Daphnia static acute toxicity test (monthly).
b) Ceriodaphnia static acute toxicity test (monthly).
c) Larval fathead minnow static acute toxicity test (monthly).
d) Ceriodaphnia survival and reproduction test (quarterly).
e) Fathead minnow larval survival and growth test (quarterly).

The data from the toxicity tests are used in various applications. Staff reports are prepared for each aquatic toxicity evaluation which describe test objectives, methods, conclusions and recommendations. Reports are distributed to the MDNR-Surface Water Company Division Compliance Section and permit development staff, in addition to the discharger. Test data are compared with existing or theoretical whole effluent toxicity limits established for the discharge. If an exceedence is detected, appropriate regulatory action is taken. Test data are also used to determine whether whole effluent toxicity limits or aquatic toxicity testing requirements need to be included in a facility's NPDES permit. In-lab, static, acute toxicity test data are also used to identify candidates for additional advanced biomonitoring.

MDNR staff are presently conducting five flow-through acute tests, three chronic static/renewal tests and 30 to 40 static acute screening tests on a yearly basis. These numbers are lower than what is expected to be achieved on a regular basis because of the large amount of time devoted to setting up the new permanent laboratory in Lansing and developing staff expertise and capability to run new short-term chronic tests.

Biouptake Assessments

These studies are used to evaluate the discharge of bioaccumulative substances. The test generally involves caged fish placed in the

effluent for 28 days. Test species are usually channel catfish (Ictalurus punctatus), six-to-ten inches long. A select number of fish (usually six) are removed and analyzed on days zero, two, four, eight, sixteen and twenty-eight. Individual fish are analyzed for pesticides, PCBs, heavy metals, mercury or other bioaccumulative contaminants.

This test has only been performed infrequently by the Department because of the extensive time required on site. A more common practice is to require that the test be conducted by the discharger where bioaccumulative substances are present as a condition of the discharge permit for facilities.

BIOMONITORING IN SURFACE WATER DISCHARGE PERMITS

Michigan is currently evaluating the expanded use of toxicity testing in its surface water discharge program to address aquatic toxicity. Whole effluent toxicity testing requirements, using biological organisms to assess the acute and chronic toxicity of various concentrations of an effluent, have been placed in permits in the past to some extent, but an increased emphasis in this area is needed. Although Michigan has had the capability to conduct acute toxicity tests on fish for many years, the data have been used primarily to identify problems. The use of whole effluent testing in discharge permits will enable early detection and correction of toxicity concerns before they become significant problems.

In 1985, Michigan revised Rule 57 of its Water Quality Standards to establish a clear process for setting effluent limitations for toxic chemicals in NPDES permits. Under the revised rule, limits are established using detailed guidelines for calculating acceptable levels in the receiving stream after mixing to protect aquatic life as well as public health and welfare.

Since the Rule 57 package was adopted in January 1985, water quality-based effluent limitations in permits have been primarily developed on a chemical-specific basis. Criteria are developed and limits are established for specific chemicals which assure compliance with the acute and chronic aquatic toxicity requirements of the Water Quality Standards. These Standards prohibit unacceptable acute toxicity in the mixing zone and require adequate protection against chronic toxicity after mixing. However, there are some limitations to the chemical specific approach. It is impossible to fully identify all chemicals in an effluent and the interaction of chemicals cannot be directly evaluated. Therefore, to complement the chemical-specific approach, Rule 57 also provides for the use of biological methods or whole effluent testing techniques to address aquatic toxicity concerns. Although whole effluent techniques have been used only sparingly in the past, Michigan's goal is to use an integrated approach of both chemical-specific and whole effluent techniques to control aquatic toxicity. The recent development of short-term methods for assessing chronic toxicity has the potential of making the use of whole effluent techniques much more acceptable from a cost standpoint. Michigan has been enhancing its toxicity testing program by improving its laboratory facility and staff expertise to where routine implementation of an integrated approach is now possible. At present, whole effluent toxicity techniques are only applicable to aquatic life. Effluent

limitations to protect human health or to control the discharge of bioaccumulative substances can only be addressed on a chemical-specific basis.

Michigan is currently developing a strategy to implement the integrated approach to toxics regulation. For purposes of addressing the whole effluent aspect of this approach, the strategy will address the following areas:

- The data needed to evaluate whether a Toxicity Reduction Evaluation (TRE) and/or whole effluent toxicity limits are needed in a discharge permit;
- The frequency of toxicity testing that is appropriate to assess compliance with a whole effluent toxicity limits;
- The appropriateness of short-term toxicity testing to better characterize the effluent;
- The process for the development of whole effluent toxicity limits; and
- The type and number of species which should be tested to adequately assess effluent toxicity.

Using professional judgment, site specific factors such as the known level of toxicity, variability of toxicity, and quality and quantity of the available data will be reviewed to determine appropriate discharge permit requirements. The range of permit requirements includes monitoring, whole effluent limitations, and toxicity reduction evaluation programs. The actual permit condition for any given discharger will vary depending upon the site-specific circumstances.

Michigan is currently receiving public comment on the strategy. Meetings with the regulated community, environmental groups and other interested parties are scheduled to discuss issues and concerns with the proposed integrated approach. It is anticipated that a number of future permits will be issued with whole effluent testing requirements. These requirements will assure better protection of Michigan's waters against the threat of toxic chemicals.

SUMMARY

Biomonitoring is a valuable tool in assessing the effectiveness of water pollution control efforts and targeting remedial actions. The various aspects of biomonitoring, including fish contaminant monitoring, biosurveys, compliance monitoring, bioassays, and biouptake assessments are all necessary components of Michigan's program. Also, biomonitoring is now being used as a regulatory tool to assure mixtures of pollutants do not exceed water quality criteria. Fully intergrading the biological and engineering sciences will ensure a strong and effective water pollution control program.

ACKNOWLEDGEMENTS

The contributions of the staff of the Great Lakes and Environmental Assessment Section of the Surface Water Quality Division were essential to the preparation of this paper and are greatly appreciated.

FIGURES AND TABLES

Number

Figure 1. Procedure for Preparation of "Standard fillets" analyzed in this study.

Table 1. Standard Edible Portions of Michigan's Sport and Chemical Fishes.

Table 2. Fish Analysis Parameters List.

Figure 1: Procedure for preparation of "standard fillets" analyzed in this study.

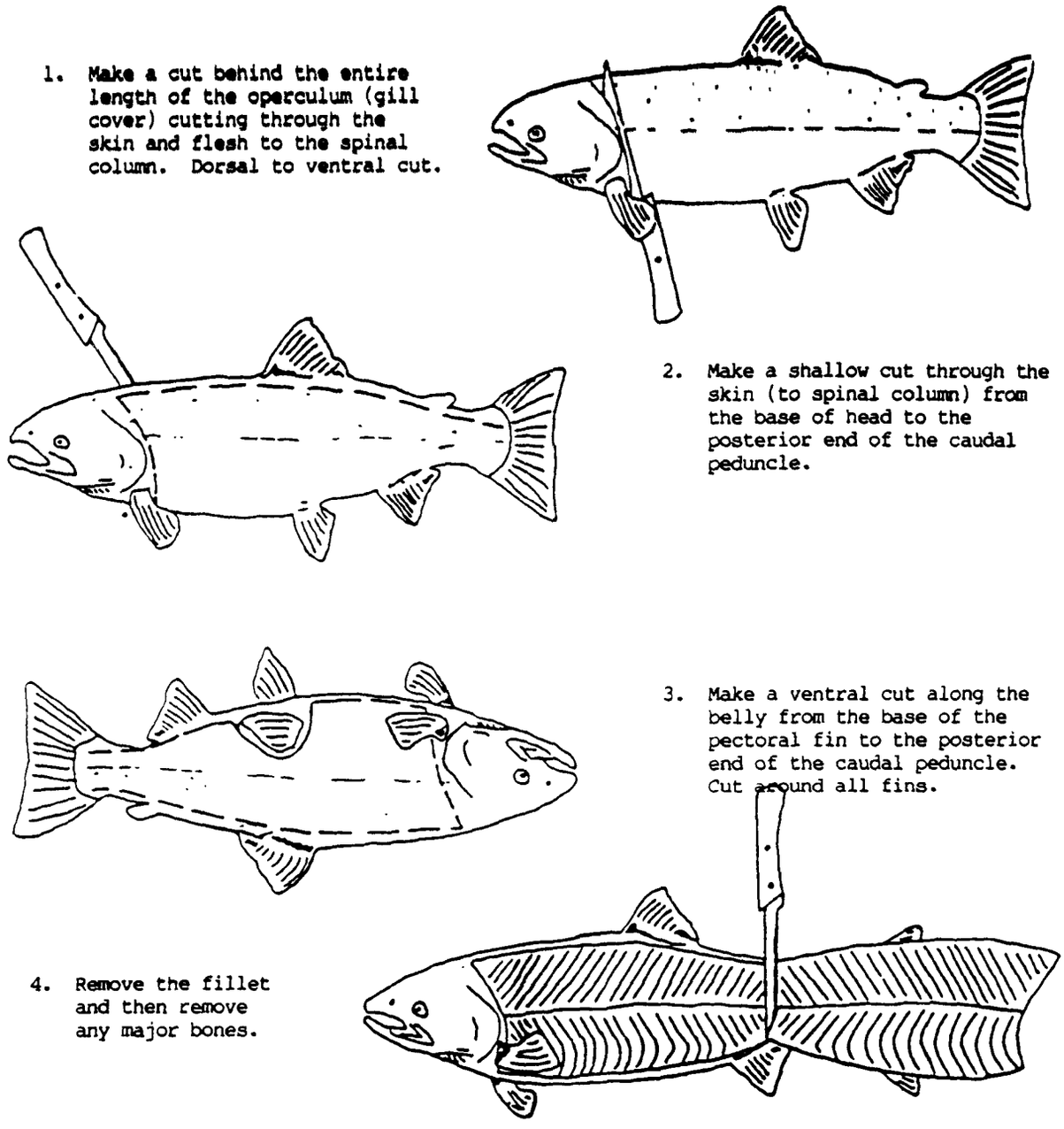


TABLE 1. STANDARD EDIBLE PORTIONS OF MICHIGAN'S SPORT AND COMMERCIAL FISHES

Listed below are the "standard edible portions" for Michigan fishes. The "standard edible portion" will be used for preparing fish for contaminant analyses. The "standard edible portion" is that portion of the listed species of fish that most people eat.

Standard Edible Portion	Common Name	Scientific Name
Skin-on Fillet	Yellow Perch	Perca flavenscens
	Walleye	Stizostedion vetreum
	Sauger	Stizostedion canadense
	Largemouth Bass	Micropterus salmoides
	Smallmouth Bass	Micropterus dolomieui
	Bluegill	Lepomis macrochirus
	Pumpkinseed	Lepomis gibbosus
	Rock Bass	Ambloplites rupestris
	White Bass	Morone americana
	Black Crappie	Pomoxis nigromaculatus
	White Crappie	Pomoxis annularis
	Green Sunfish	Lepomis cyanellus
	Longear Sunfish	Lepomis megalotis
	Warmouth	Lepomis gulosus
	Sucker Family	Catastomidae
	Lake Whitefish	Coregonus clupeaformis
	Lake Trout (lean & siscowet)	Salvelinus namaycush
	Steelhead (Rainbow Trout)	Salmo gairdneri
	Brown Trout	Salmo trutta
	Brook Trout	Salvelinus fontinalis
	Splake	Salvelinus fontinalis X Salvelinus namaycush
	Atlantic Salmon	Salmo salar
	Coho Salmon	Oncorhynchus kisutch
	Chinook Salmon	Oncorhynchus tshawytscha
	Pink Salmon	Oncorhynchus gorbuscha
Skin-off Fillet	Black Bullhead	Ictalurus melas
	Brown Bullhead	Ictalurus nebulosus
	Yellow Bullhead	Ictalurus netalis
	Channel Catfish	Ictalurus punctatus
	Muskellunge	Esox masquinongy
	Northern Pike	Esox lucius
	Round Whitefish (Menominee)	Prosopium cylindraceum
	Lake Herring	Coregonus artedii
	Chubs	Coregonus hoyi
	Carp	Cyprinus carpio
	Sheepshead	Aplodinotus grunniens
	Buffalo	Ictiobus cyprinellus
	Burbot	Lota lota
	Quillback	Carpiodes cyprinus
	Sturgeon	Acipenser fulvescens
Headless, Gutted	Rainbow Smelt	Osmerus mordax

Table 2. Parameter List

METALS	ORGANICS
Mercury	Aldrin
Cadmium	g-BHC (lindane)
Chromium	a-chlordane
Copper	g-chlordane
Lead	4,4'-DDD
Nickel	4,4'-DDE
Zinc	4,4'-DDT
	Dieldrin
	Octachlorostyrene
	Heptachlor
	Heptachlor Epoxide
	Hexachlorobenzene
	cis-Nonachlor
	trans-Nonachlor
	Oxychlordane
	PBB (BP-6)
	PCB (as Arochlor 1254)
	Toxaphene

APPENDICES

A. Physical, Chemical and Biological Monitoring Results from the Kalamazoo River, Comstock to Plainwell, 1984, dated February, 1986.

B. A Biological Investigation of Prairie River and Prairie River Lake, St. Joseph County, August 12, 1986.

C. Water, Sediment, and Macroinvertebrate Survey of the South Branch Raisin River, and Eastside Drain, vicinity of Adrian, 1982-85.

D. Department of Natural Resources, Water Resources Commission, General Rules, Part 4. Water Quality Standards, filed with Secretary of State November 14, 1986.

APPENDIX A

Physical, Chemical and Biological Monitoring Results From the Kalamazoo River, Comstock to Plainwell 1984

by
John D. Suppnick and William Creal
Michigan Department of Natural Resources
Environmental Protection Bureau
Surface Water Quality Division
February, 1986

Table of Contents

		Page
I	Introduction	723
II	Summary and Conclusions	723
III	Background	723
IV	Methods	724
V	Results	729
VI	Discussion	737
VII	References Cited	741
VIII	Appendix *	743

*Due to the length of the Appendix of this report, it has not been attached. Copies are available from the author by writing to the address shown on the front cover page of this entire report.

Introduction

During April - October, 1984, benthic plant and animal communities and related physical/chemical stream parameters were measured in the Kalamazoo River between Comstock and Plainwell. A major objective of this study was to document the nuisance aquatic weed growths in the Kalamazoo River at a high nutrient load for comparison to a follow-up study at a low nutrient load. The results of these studies will be used to determine future NPDES permit limits.

Summary and Conclusions

1. Benthic macroinvertebrate communities, aquatic plants and water chemistry were sampled in the Kalamazoo River from Comstock to Plainwell during the 1984 growing season.

2. Related measurements of channel geometry, stream flow, and reaeration rates were made to provide data for a water quality model.

3. The stream life indicated good water quality upstream of the Kalamazoo wastewater treatment plant (WWTP) and poor quality downstream. Downstream communities were dominated by sowbugs, blackflies, midges, and nuisance growths of an aquatic weed (Cladophora).

4. Cladophora standing crops reached nuisance levels in only 15 days of growing time downstream of the Kalamazoo WWTP, while nuisance levels were never attained upstream of the Kalamazoo WWTP. These nuisance growths persisted from June through September and were due to the phosphorus loads discharged by the Kalamazoo WWTP.

5. The sloughing and downstream transport of Cladophora persisted throughout the growing season. This sloughed Cladophora represents a significant potential demand on the oxygen resources of the River.

6. An apparent zone of Cladophora growth inhibition was observed for about 4 km immediately downstream of the Kalamazoo WWTP.

7. A follow-up study is planned for 1986 to document the improvement in stream conditions after the upgraded Kalamazoo WWTP is operating. After the follow-up study, an evaluation of the Cladophora growth levels should be done to determine if further reductions in phosphorus discharges are needed to prevent nuisance Cladophora growths.

Background

The Kalamazoo River is a cool water stream which drains over 5,000 km^2 in southwest Michigan. This river has suffered severe water quality degradation during the region's industrial development and growth in the first half of this century. In 1956 the waste loadings from 14 paper mills in

the Kalamazoo vicinity resulted in a 32 km reach of river that was anaerobic (1). However, a large clean-up effort costing over $500 million for the construction of public treatment facilities alone has resulted in improved stream quality.

Michigan Department of Natural Resources (MDNR) stream dissolved oxygen (D.O.) modeling studies indicate that aquatic plant respiration will be the main cause of low D.O. after all discharges are in compliance with their waste discharge permits (2,3). Field investigations (4,5) have identified nuisance growths of Cladophora as the dominant aquatic plant in the Kalamazoo River. Because of this finding, the MDNR initiated this study of the aquatic plant problem in the Kalamazoo River. The objective of this study is to document the change in water quality before and after the City of Kalamazoo implements advanced waste treatment. Of particular interest is to document the change in Cladophora growth as phosphorus removal is implemented at the Kalamazoo wastewater treatment plant.

Methods

The overall sampling strategy for this study is shown in table 1. Biological data were collected during boat cruises approximately every 2 weeks from April to October.

Cladophora Monitoring

Nine river stations were sampled to determine the relative standing crop of Cladophora (Figure 1). At each station, three representative rocks were selected, the Cladophora was harvested from each rock and the surface area of the colonized portion of the rock was estimated in the field with a ruler. Stream velocity at the water surface was determined with a stopwatch and float, and the water depth was recorded. At some locations, Cladophora was confined to a small isolated patch of growth. In these cases, representative rocks from the patch of growth were selected.

The Cladophora sloughing rate was measured by placing a Surber sampling net with a 1 square foot opening in the stream for a measured time period. The Cladophora collected was returned to the lab for drying and weighing. The corresponding stream velocity was determined using a float and stopwatch. Sloughing measurements were made in triplicate at three stations: Patterson Avenue, D Avenue and M-89.

All Cladophora samples were air dried on newspapers in a metal shed. Drying took 23-60 days. Samples were then weighed using a Mettler analytical balance. Twenty randomly selected samples were allowed to dry an additional 28 days or more. The additional weight loss averaged 1.5% with no sample losing more than an additional 5%.

Benthic Macroinvertebrate Sampling

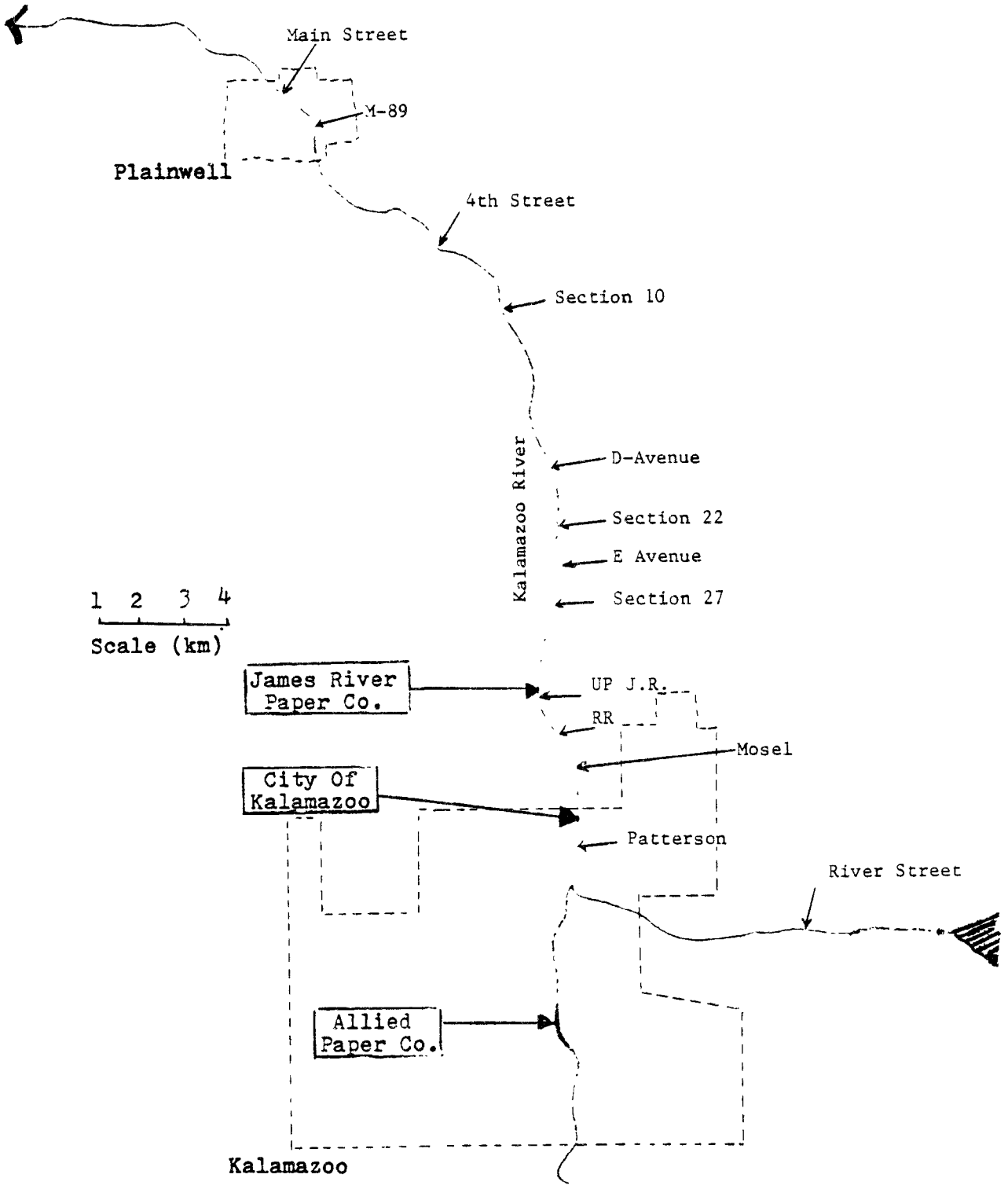
Qualitative benthic macroinvertebrate samples were collected using a triangular dip net and by handpicking available substrate. Sampling was

Table 1. Kalamazoo River Station Description and Sampling Strategy

Wastewater and Tributary Inputs	Station Description	River Km	STORET #	Reaeration Measurement	Cross Sectioning	Weekly Nutrient Monitoring	Monthly Diurnal Water Quality Monitoring	Biweekly* Cladophora Biomass Monitoring	Biweekly* Cladophora Sloughing Measurement	Daily Flow Monitoring	Biweekly* Transparency Monitoring
Allied Paper Co Via Portage Creek →	River St.	109	390079					X		X	
	Patterson	102.2	390081			X	X	X	X		X
Kalamazoo WWTP →	Mosel	100.6	390082			X					X
	Penn RR	99.5	390114					X			
	Above James R	98.3	--					X			
James River Co →	McKinley St	95.8	390380				X				
	Section 27	95.2	--					X			
	E Ave	94.5	--								
	Section 22	93.6	--					X			
	D Ave	92.9	390058			X	X	X	X		
	Section 10	90.2	--					X			
	4th St	87.8	390381				X				
	M - 89	80.4	--					X	X		
	Main St	80.0	030209				X				

* Once every 2 weeks

Figure 1. Kalamazoo River Sampling Locations, 1984.



continued until no new forms were found. Taxa were identified on-site to the lowest taxonomic level possible. Relative abundance was noted.

Periphyton Sampling

Qualitative periphyton samples were collected by scraping rocks. Samples were placed on ice and identified the following day using 400X magnification. No estimate of relative abundance was made. Periphyton sampling was discontinued when Cladophora growth became present.

Water Chemistry Monitoring

River samples for nutrients and other chemical analysis were collected from the D Avenue Bridge weekly by Surface Water Quality Division District staff. Samples were also collected at D Avenue during each boat cruise. Additional water samples were collected upstream (Paterson Rd.) and downstream (Mosel Ave.) of the Kalamazoo WWTP during each of the boat cruises and analyzed for nutrients.

Light Monitoring

The light extinction coefficient for the Kalamazoo River was measured at Paterson Road and at McKinley on September 18, 1984, using an underwater irradiometer (Kahlsico model 268WA310) by Kahl Scientific Instrument Corporation. The Secchi depth transparency was measured during each boat cruise at Paterson Avenue (upstream of the Kalamazoo wastewater treatment plant) and at 1 or more locations downstream of the WWTP. Incident light intensity over the visible spectrum was measured daily by the Michigan State University Gull Lake Biological Station with a Li-cor Pyrenometer. This weather station is about 16 km east of the study reach.

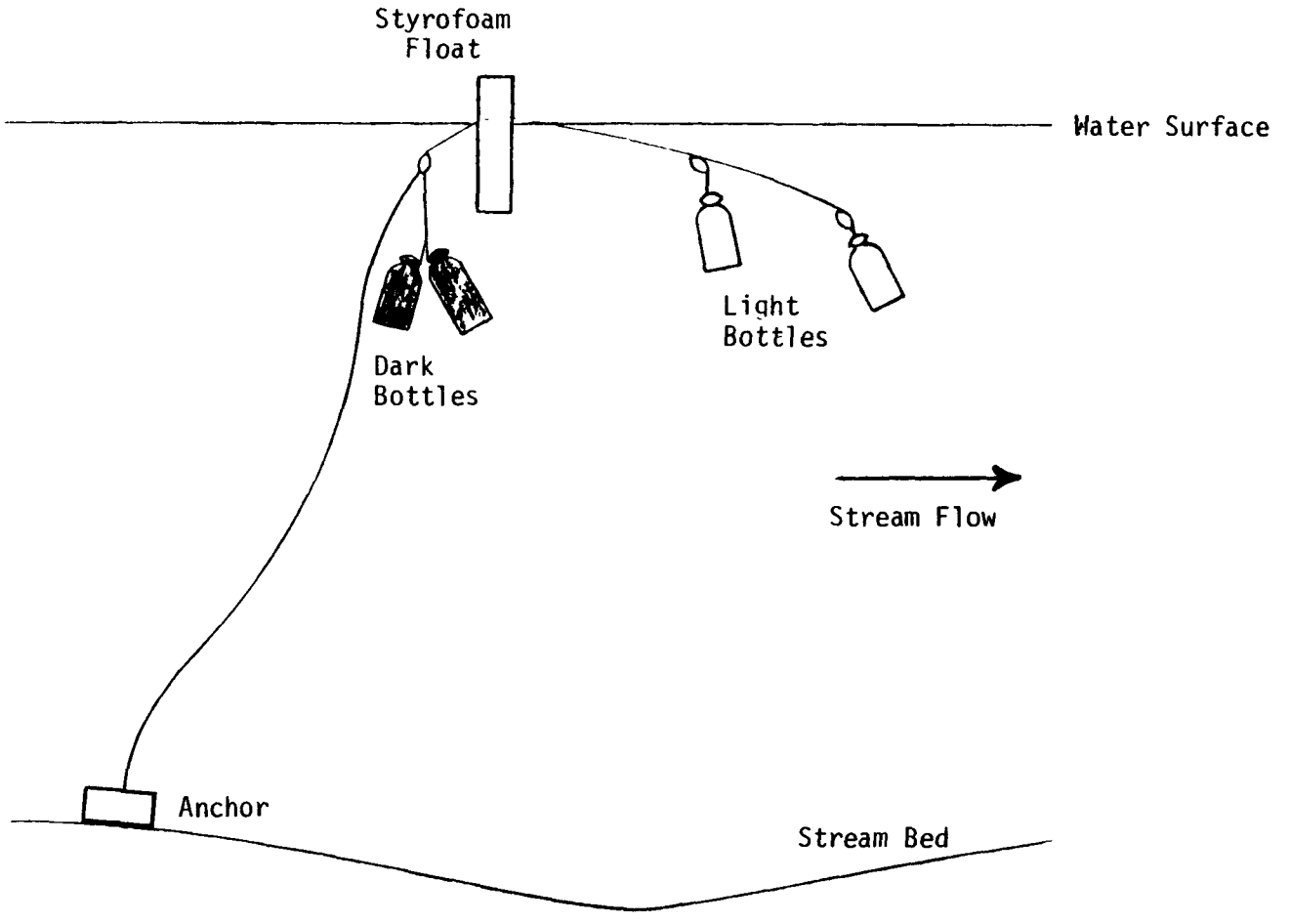
Stream Flow Monitoring

A United States Geological Survey (USGS) gage is located on the Kalamazoo River at the River Street Bridge in Comstock. This gage is a telemark gage and river stage can be determined by phone. River stage was determined daily from April to October by phone. In addition, stage was continuously monitored using a Stevens strip chart recorder at the gage during 3 of the 4 intensive diurnal monitoring studies. Water Management Division personnel made several miscellaneous measurements at various locations downstream of the gage using standard USGS techniques.

Plankton Productivity Measurements

Photosynthesis and respiration of planktonic algae were measured using a standard light/dark bottle method (6). Light bottles were clear, 250 ml BOD bottles. Dark bottles were 250 ml BOD bottles covered with a double layer of black electrical tape. Bottles were suspended from styrofoam floats with wire (Figure 2). All bottles were filled by skimming water from the surface of the stream. Samples for BOD, nutrients, chlorophyll, initial DO and other chemical analysis were collected simultaneously with the filling of the light and dark bottles. Dissolved oxygen was determined using the Winkler method. Samples were fixed in the field and

Figure 2. Kalamazoo River Light/Dark Bottle Apparatus



titrated in the Laboratory in Lansing. Gross photosynthesis and respiration were determined from the following equations:

$$R = (C_I - C_D) - [UBOD\ (1-\exp-(t\ K_1\ 1.047^{T-20}))]$$

$$P = C_L - C_D$$

Where:

R = Planktonic respiration (mg/l)
C_I = Initial DO concentration (mg/l)
C_D = Final DO concentration in dark bottles (mg/l)
UBOD = Ultimate carbonaceous BOD of the stream (mg/l)
K_1 = Lab determined bottle CBOD decay coefficient at 20°C base e ($days^{-1}$)
T = Average stream temperature during the test (°C)
t = Duration of the light dark bottle test (days)
P = Gross photosynthesis during the test (mg/l)
C_L = Final DO concentration in the light bottle (mg/l)

The duration of the tests varied from 0.8 days to 1.02 days.

Diurnal Stream Chemistry Surveys

Four diurnal stream chemistry sampling surveys were done. Samples were collected and analyzed by Michigan DNR staff. The three major discharges (Kalamazoo WWTP, Allied Paper Co. and James River Paper Co.) to the study reach were sampled during the September diurnal study by the MDNR Point Source Studies Section. (7,8,9)

Reaeration Coefficients

Reaeration rate coefficients were measured by MDNR personnel between E Avenue and the 4th Street sampling station (Figure 1). The procedure used was the ethylene tracer method as developed by the MDNR on the Clam River (10).

Stream Geometry

Stream channel geometry and substrate type was determined on September 6 and 7, 1984. Stream depth and width were determined at representative transects at approximately 500 meter intervals. Some portions of the study reach were surveyed more intensively to provide data for a habitat evaluation model. River depth was determined with a surveying rod graduated in 1 foot increments and river width was determined with a steel cable graduated in 20 foot increments. Substrate type was determined for each depth measurement either visually, by probing with a surveying rod or by sampling with a dredge.

Results

Cladophora Monitoring

Three different zones were apparent in the River based on three distinct seasonal growth patterns of Cladophora. Upstream of the Kalamazoo WWTP

growths were sparse; immediately downstream of the WWTP growths were inhibited; and further downstream growths were profuse. Relative standing crop measurements for the 1984 growing season are summarized in Figure 3, and the raw data is listed in Table 2 of the appendix.

In the upstream zone (Paterson and River Street sampling stations), growths were sparse and never reached nuisance levels. The maximum standing crop observed was 177 grams of dry weight/square meter (gdw/m^2) at River Street and 60 gdw/m^2 at Patterson Avenue. Growth began about mid May and persisted throughout the summer.

Immediately downstream (0-4 km) of the Kalamazoo WWTP, Cladophora growth did not begin until late July and only persisted through August. Growth occurred in isolated patches and most available substrate was not colonized.

The zone of nuisance growths began about 7 km downstream of the Kalamazoo WWTP and extended to the end of the study reach in Plainwell. Growth began in early June and increased rapidly for three weeks. Standing crops peaked in late June or early July and gradually declined thereafter.

The most rapid growth was measured at the D Avenue station where the standing crop grew from $4g/m^2$ to 6000 g/m^2 during 21 days in June. On June 5th, the longest Cladophora stringer observed at D avenue was 4 inches. By June 20th, 15 foot stringers were observed.

The sloughing measurements from the D Avenue station and the M-89 station are summarized in Figure 4 and the raw data is listed in Table 3 of the appendix. Sloughing measurements were also made at Patterson Avenue but only a trace of sloughed Cladophora was detected there.

The D Avenue sampling station is at the downstream end of the largest riffle in the study area. The sloughed Cladophora load reached a peak of 4600 kg dw/day at D Avenue in mid July. This peak followed the period of rapid growth in June. The loading of sloughed Cladophora carried by the stream was reduced by about 90% at M-89 in Plainwell, about 22 km downstream of D Avenue. This indicates that 90% of the sloughed Cladophora was either deposited, decomposed or broken into pieces smaller than the mesh of the sample net between these two stations. However, no large depositional areas developed between D Avenue and Plainwell because of the relatively high stream velocity. Therefore, we believe that most of the sloughed Cladophora load decayed and was broken into small pieces between D Avenue and M-89 in Plainwell.

The oxygen demand potential of sloughed Cladophora tissue was determined in the lab to be 0.65g O_2/gm dry weight Cladophora. BOD samples were prepared using River water and a known weight of Cladophora and incubated until oxygen demand was 95% satisfied. The results are summarized below:

Figure 3. Cladophora Biomass Monitoring Results from the Kalamazoo River - 1984

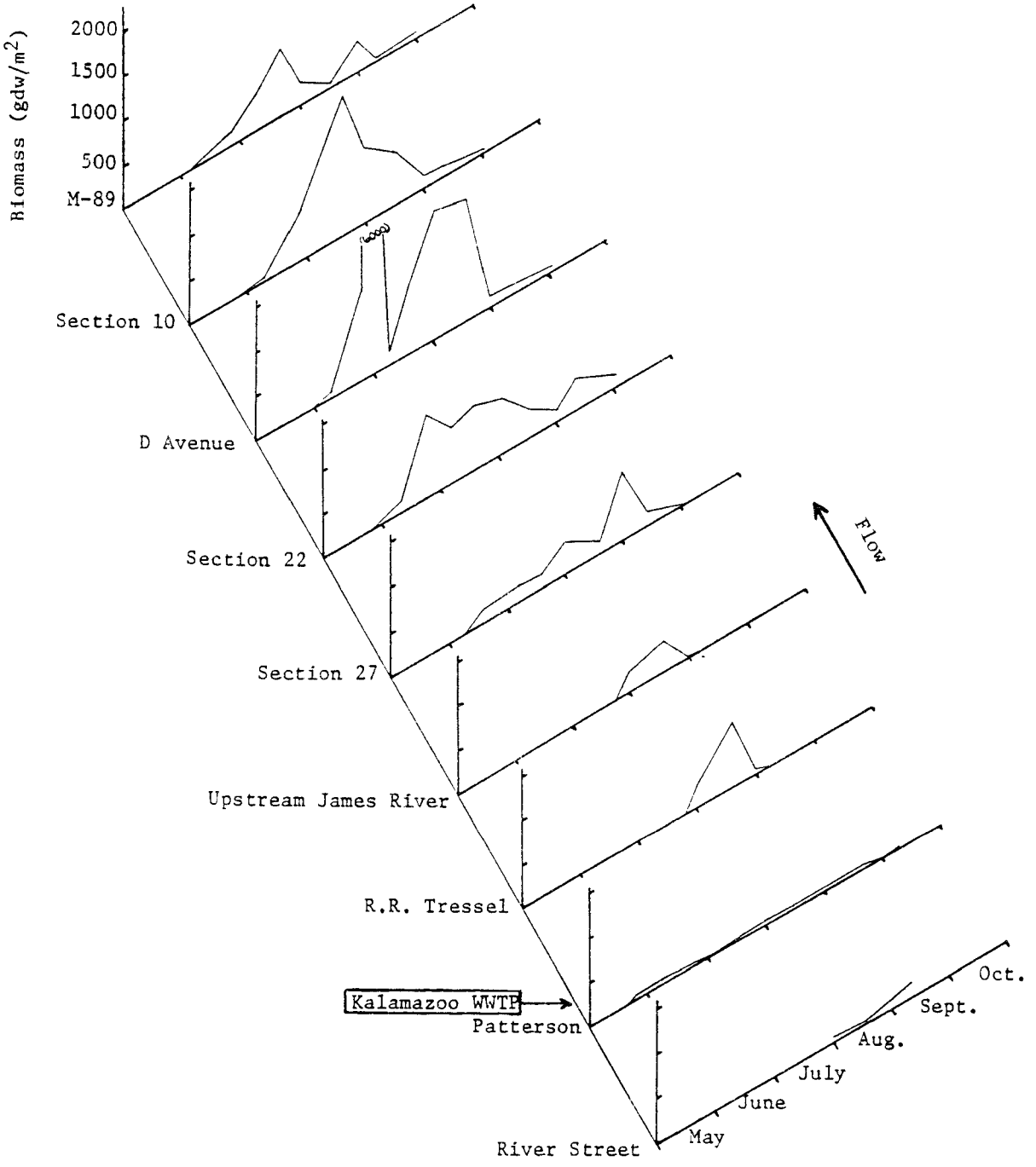
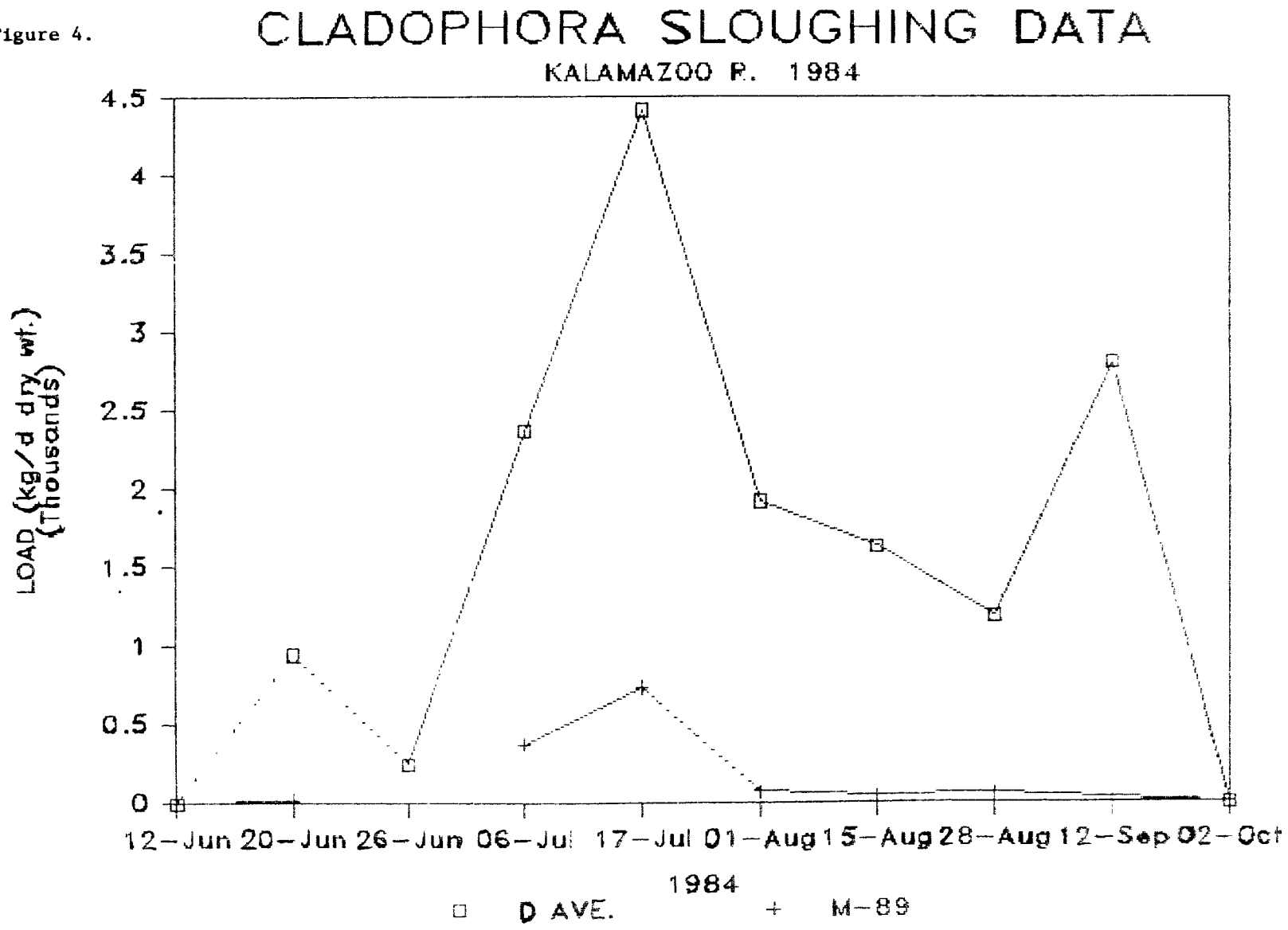


Figure 4.


CLADOPHORA SLOUGHING DATA
KALAMAZOO R. 1984
LOAD (kg/d dry wt.)
(Thousands)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
12-Jun 20-Jun 26-Jun 06-Jul 17-Jul 01-Aug 15-Aug 28-Aug 12-Sep 02-Oct
1984
D AVE.
M-89

Sample	Dry Weight Cladophora mg/l	Ultimate CBOD mg/l Undiluted	1:5 dilution
1	0	7.18	8.55
2	0	6.34	-
3	0	6.63	-
4	5.2	7.83	-
5	13.0	15.0	17.6
6	26.0	-	24.0
7	39.0	-	32.0

The annual production of Cladophora upstream of D avenue was estimated by multiplying the average daily sloughed Cladophora load at D avenue during this period (June 12 - October 2) by the number of days. Based on these calculations, about 240,000 Kg dw of Cladophora was produced in the reach between the McKinley sampling station (which was just upstream of where Cladophora growth began) and the D Avenue station. This reach has a surface area of 184,000 square meters. Therefore, if the Cladophora was evenly distributed over the entire reach, the annual production was 1.36kg dry weight/square meter.

Benthic Macroinvertebrates

Upstream of the Kalamazoo WWTP, benthic macroinvertebrate communities were diverse (8-15 taxa) and dominated by scuds, mayflies and caddisflies (Table 4 of the appendix). At the three stations sampled downstream of the WWTP, the communities were restricted in the number of taxa present (usually 4-5) and dominated by midges, sowbugs and leeches.

Periphyton

Results of the periphyton sampling are presented in Table 5 of the appendix. Diatoms were the most represented group by number of taxa.

Plant Growth Parameter Monitoring

Figure 5 presents the results of stream temperature, transparency and incident light monitoring compared to a summary of the standing crop measurement results. Stream nutrient data at the D Avenue station and stream flow at the River Street gage are plotted in Figure 6, and the raw chemistry data is listed in Table 6 of the appendix.

The results of light extinction coefficient measurements are shown below:

Location	% Light Reflection at Surface	Secchi Depth (m)	Light Extinction Coefficient (1/m)
Patterson Rd.	23	1.4	1.28
McKinley	31	1.0	2.21

Stream Flow Measurements

Several stream flow measurements were made by Water Management Division Personnel and are listed below.

Figure 5. Summary of Selected Kalamazoo River Monitoring Data from Downstream of the Kalamazoo WWTP - 1984

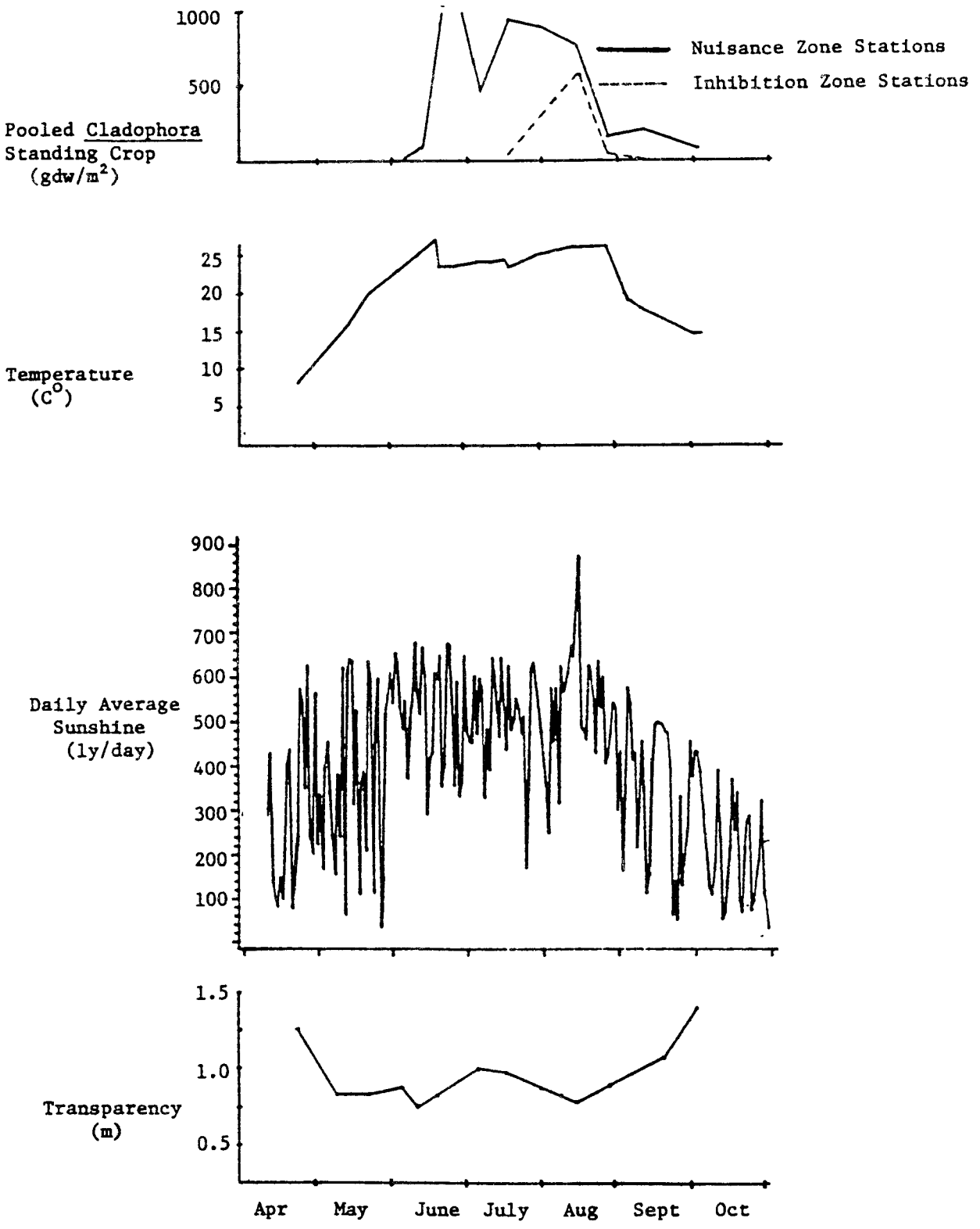
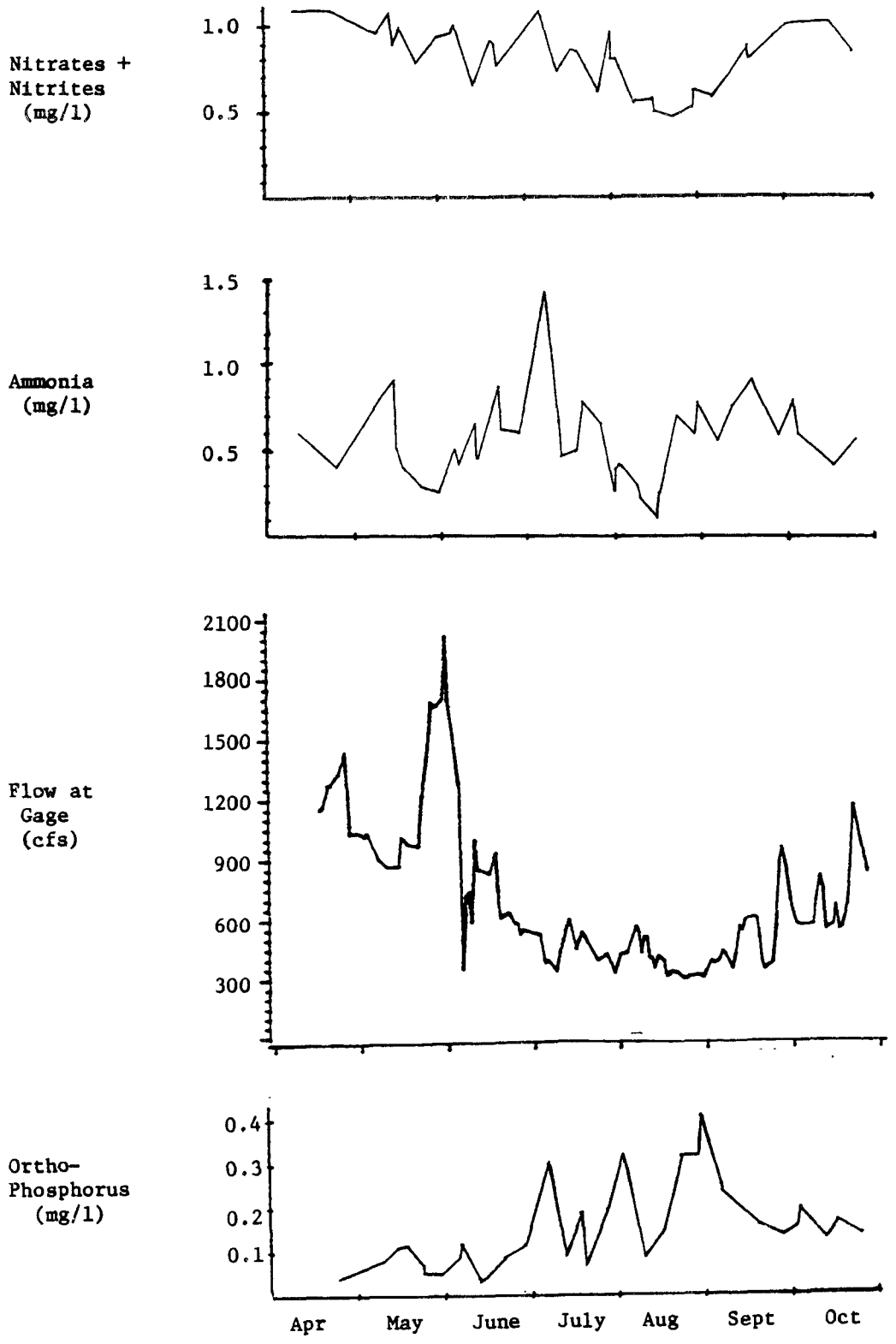


Figure 6. Summary of Selected Kalamazoo River Monitoring Data at D Avenue Downstream of the Kalamazoo WWTP - 1984.



Location	Date	Flow (cfs)
Kalamazoo R. at Comstock gage	June 20, 1984	622
	July 31, 1984	414
Kalamazoo R. above James River Paper Co.	June 27, 1984	688
	July 31, 1984	503
	Sept. 19, 1984	617
Spring Brook at Riverview Rd.	June 27, 1984	15.7
	July 31, 1984	14.1
	Sept. 19, 1984	14.6
Kalamazoo River above Plainwell WWTP	Sept. 19, 1984	663

Diurnal Stream Chemistry Surveys

The results of the 4 diurnal water chemistry surveys are listed in Table 6 of the appendix. The river hydrographs were monitored continuously during 3 of the 4 surveys and are plotted in Figure 9 of the appendix. The dissolved oxygen standard was met at all stations during the July, August and September surveys but was not met between 4th Street and Main Street during the June survey. Diurnal variations in DO at D Avenue during the June, July, August and September surveys were 5.5 mg/1, 6.7 mg/1, 3.8 mg/1 and 3.7 mg/1 respectively. All surveys were done during sunnier than average days for their respective months. The August survey was done on the sunniest day of the summer. These sunny conditions contributed to the relatively high DO levels found in the stream during the surveys by stimulating a high level of photosynthetic oxygen production.

Reaeration Measurements

The results of stream reaeration measurements using the ethylene tracer technique are summarized below.

Reach	Length (Km)	Temp (°C)	Flow at Gage (cfs)	Time of Travel (days)	Average Depth (M)	Reaeration Rate Coefficient at 25.5°C (base e) (day^{-1})
E Ave to D Ave	1.6	25.5	403	.053	0.39	15.0
D Ave to 4th St	5.1	25.5	403	.127	0.70	3.3

The tracer monitoring data are presented graphically in Figure 10 of the appendix.

Plankton Productivity Measurements

Results of light/dark bottle productivity measurements are presented in Table 7 of the appendix. Gross productivity of planktonic algae varied from 12.5 mg O_2/1/d at 0.15 meters depth in July; to 1.8 mg O_2/1/d at 0.5 meters depth in June. Similarly, BOD corrected respiration of planktonic organisms varied from 3.1 mg O_2/1/d in July to 1.7 mg O_2/1/day in June.

Channel Geometry

Results of the stream channel cross sectioning are summarized below and the raw field data are presented graphically in Figure 11 of the appendix.

	Average depths in feet for various gage flows			
Reach	260 cfs	440 cfs	608 cfs	810 cfs
McKinley to D Ave	1.33	1.71	2.07	2.46
D Ave to 4th St	2.14	2.46	2.80	3.19
4th St to Main St	2.71	3.07	3.29	3.84

Effluent Monitoring

The results of self monitoring data for Allied Paper Co., Kalamazoo WWTP and the James River Corp. - Parchment, during the intensive survey periods are shown in Table 8 of the appendix.

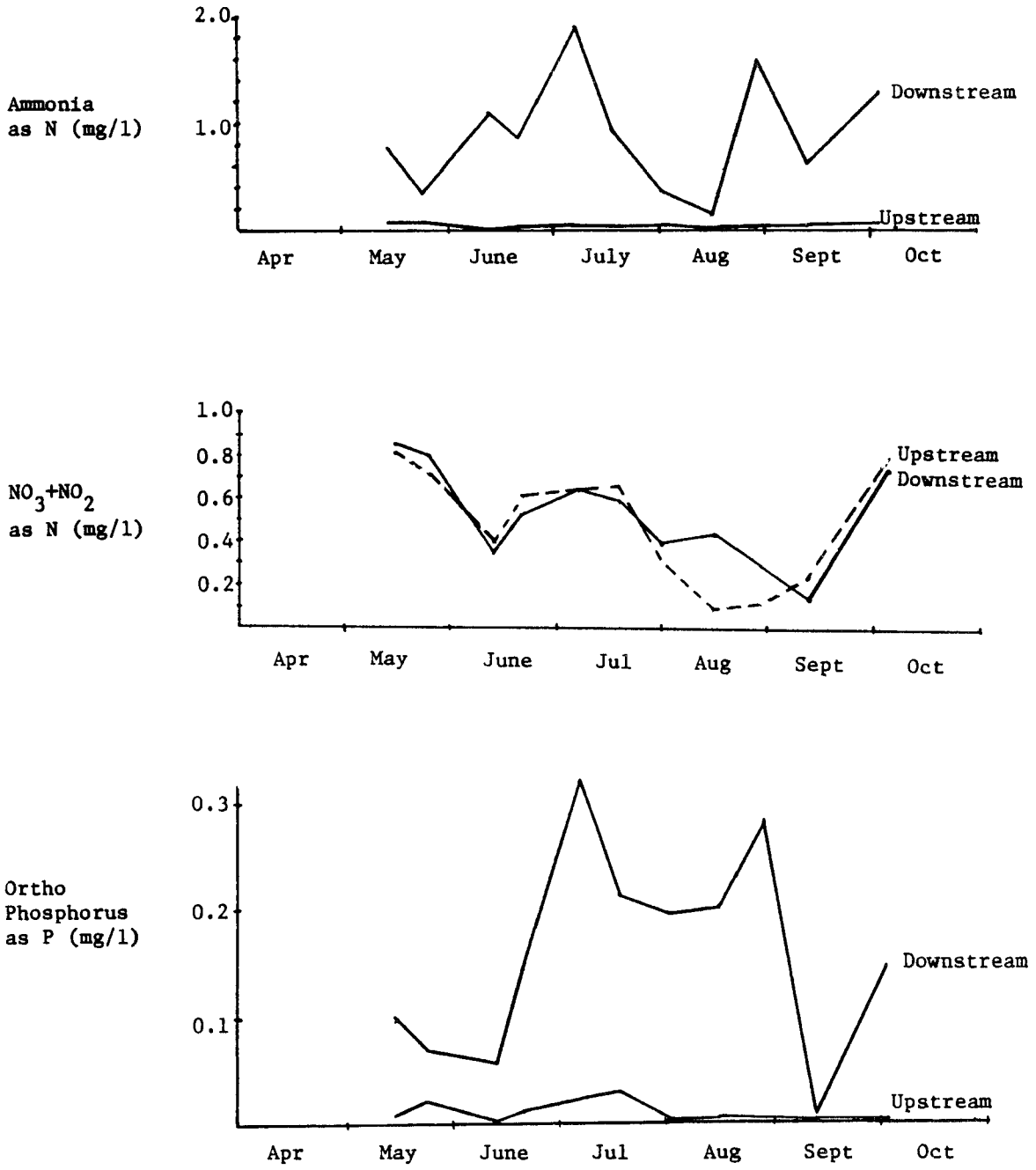
Discussion

The aquatic life downstream of the Kalamazoo WWTP was dominated by Cladophora, sowbugs, blackflies, midges and leeches, and is typical of that found in organically enriched streams downstream of municipal WWTP's (16). This degraded biological community has persisted even though a significant reduction in pollutant loads from some sources has occurred. The cause of the continued poor quality in 1984 was the Kalamazoo WWTP. Significant improvement in the stream quality is expected to occur following the implementation of advanced treatment at the Kalamazoo WWTP. A follow-up study is planned to document this improvement.

Cladophora standing crops were much greater downstream of the Kalamazoo WWTP than upstream. Nuisance growths of Cladophora are commonly associated with phosphorus enrichment in lakes and streams (11,12). Stream ortho-phosphorus concentrations averaged 11 ug/l upstream of the Kalamazoo WWTP and 177 ug/l downstream of the WWTP during the Cladophora growing season (Figure 7). All other growth factors for Cladophora (light, substrate, velocity) were similar at upstream and downstream stations. Therefore, we conclude that phosphorus enrichment from the Kalamazoo WWTP caused the nuisance Cladophora growth downstream. We expect the downstream orthophosphorus concentration to decrease to an average of 50 ug/l or less after advanced treatment is implemented at the Kalamazoo WWTP. A follow-up study is planned to determine what effect this nutrient reduction will have on Cladophora growth in the Kalamazoo River.

Immediately downstream of the Kalamazoo WWTP there was a zone where Cladophora did not appear until late July and only persisted through August. This was in spite of favorable nutrients, light, velocity and substrate for growth. The late onset of growth may be due to a growth inhibiting substance in the Kalamazoo WWTP effluent. This hypothesis is

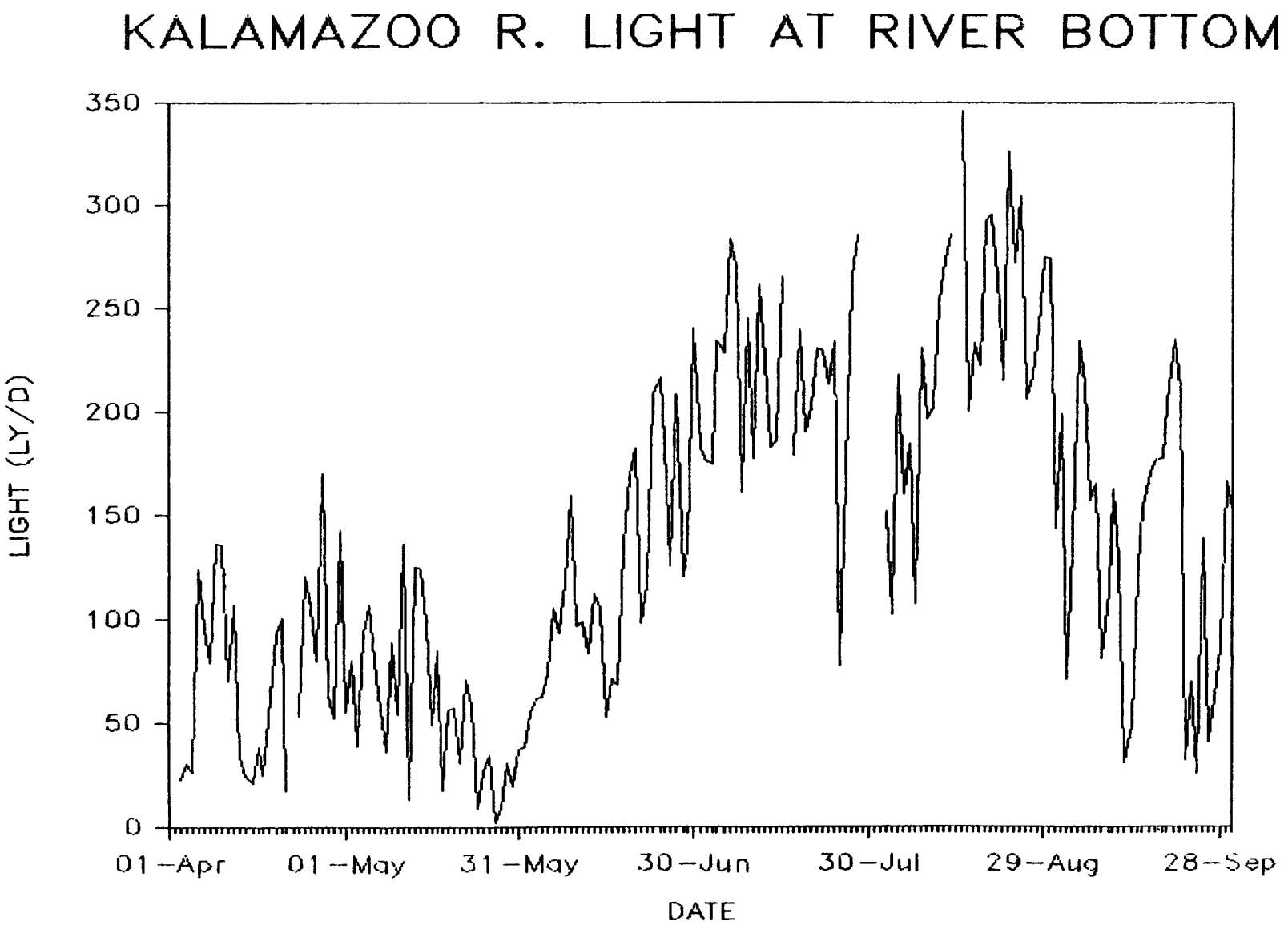
Figure 7. Kalamazoo River Nutrient Monitoring Results Upstream and Downstream of the Kalamazoo Wastewater Treatment Plant.



supported by the observation that, when growth did occur in this zone, the Kalamazoo WWTP was providing the best treatment performance of the summer (table 8 of appendix). If some pollutant in the effluent caused the growth inhibition, then the discharge of that pollutant would likely be minimized during periods of good treatment performance. This apparent inhibition should be investigated further if it persists after the improved treatment is provided at the Kalamazoo WWTP.

The period of intense Cladophora growth from June 6 - June 26 was associated with stream temperatures of 23-27°C, incident light levels of 300-680 ly/d and ortho-phosphorus concentrations in the stream of 0.036-0.22 mg/l. Cladophora is a summer algae with growth generally thought to be initiated by temperatures of 10-15°C (12,14). It is unclear why Cladophora did not become established earlier in the growing season when temperature was in the range of 10-20°C. One possible explanation is that light levels at the stream bottom were limiting. In Figure 8, the daily incident light intensity was corrected for stream depth, and transparsency to compute light intensity at the stream bottom of the Kalamazoo River at the Section 22 sampling station. This figure shows that the late May to early June period was characterized by unusually low light levels and could have prevented the growth of Cladophora even though favorable temperatures were present.

Figure 8.


KALAMAZOO R. LIGHT AT RIVER BOTTOM
LIGHT (LY/D)
350
300
250
200
150
100
50
0
01-Apr
01-May
31-May
30-Jun
30-Jul
29-Aug
28-Sep
DATE

References Cited

1. Michigan Water Resources Commission, March 1958. "Report on Self Purification Capacities Kalamazoo River 1956 Survey Comstock to Trowbridge". Lansing, Michigan.

2. Beck M. and S. Buda, April 1978. "Kalamazoo River Study Comstock to Plainwell August 16-18, 1976". Department of Natural Resources. Lansing, Michigan.

3. Suppnick John D., February 13, 1984. "Dissolved Oxygen Impacts of Hydropower Dam Restoration at Plainwell, Ostego and Trowbridge Dams on the Kalamazoo River." Department of Natural Resources. Lansing, Michigan.

4. Wuycheck John, 1984. Personal Communication of Unpublished Data, Department of Natural Resources. Lansing, Michigan.

5. Creal William, November 1982. "Macroinvertebrate and Sediment Chemistry Survey of the Kalamazoo River, Kalamazoo to Allegan, July 22 and October 7, 1982." Department of Natural Resources. Lansing, Michigan.

6. Vollenweider Richard A. 1971. "A Manual on Methods for Measuring Primary Production in Aquatic Environments." Blackwell Scientific Publications. Oxford, England.

7. Reznick Ralph and Chris Little. Undated. "Report o an Industrial Wastewater Survey Conducted at Allied Paper Co pany September 18 and 19, 1984." Department of Natural Resource , Lansing, Michigan.

8. Boersen Gary and Joe Hey. Undated. "Report of a Municipal Wastewater Survey Conducted at Kalamazoo Wastewater Treatment Plant September 18-19, 1984." Department of Natural Resources, Lansing, Michigan.

9. Boersen Gary, William Long and John Eckland. Undated. "Report of an Industrial Wastewater Survey Conducted at James River Corporation, September 18-19, 1984." Department of Natural Resources, Lansing, Michigan.

10. Suppnick John D. July 1984. "Clam River Reaeration Measurements Using the Ethylene Tracer Method." Department of Natural Resources, Lansing, Michigan.

11. Neil John H. and Glenn E. Owen. 1964. "Distribution, Environmental Requirements and Significance of Cladophora in the Great Lakes." Proceedings 7th Conference on Great Lakes Research 113-121.

12. Pitcairn Carole E.R. and H.A. Hawkes. 1973. "The Role of Phosphorus in the Growth of Cladophora." Water Research 7: 159-171.

13. Bellis V. D. and D.A. McLarty. 1967. "Ecology of Cladophora Glomerata (L.) Kutz. In Southern Ontario." Journal of Phycology 4: 19-23.

14. Moore, Lawrence F. 1979. "Attached Algae at Thermal Generating Stations - the effect of temperature on Cladophora." Verh. Internat. Verein. Limnol. 20: 1727-1733.

15. Storr, J.F. and R.A. Sweeney. 1971. "Development of a Theoretical Seasonal Growth Response Curve of Cladophora Glomerata to Temperature and Photoperiod" Proc. 14th Conf. Great Lakes Research. 119-127.

16. Hynes, H. 1960. The Biology of Polluted Waters. University Press, Liverpool England. 202 P.

The appendix to this report on the Kalamazoo River is available from the author of this paper, Mr. Paul Zugger, by writing to him at the address on the cover sheet of this report.

Michigan Department of Natural Resources
Surface Water Quality Division
November, 1986
Staff Report

A Biological Investigation of Prairie River and Prairie River Lake, St. Joseph County, August 12, 1986

Benthic macroinvertebrates, sediment and water were collected on August 12, 1986 from the Prairie River and Prairie River Lake in Burr Oak Township, St. Joseph County (Figure 1). Benthic macroinvertebrates were sampled to determine present river quality conditions. Sediments and water were collected and analyzed for the presence of toxicants. These surveys were requested by Kay Brower, Environmental Enforcement Division, to evaluate the effect of suspected increased releases of leachate from the Ford/Young landfill. A similar study had been conducted in 1983 (Creal, 1983).

Summary and Conclusions

1. Benthic macroinvertebrate communities in the Prairie River indicated good to excellent stream quality. Slight changes in the benthic macroinvertebrate communities occurred between McKale Road and Prairie River Lake. These changes were likely due to differences changes in current speed and stream substrate. These findings are consistent with those of the 1983 study. There was no indication of an adverse biological effect in the Prairie River due to the Ford/Young landfill leachate.

2. There was no change in water or sediment quality in Prairie River or Prairie River Lake due to the Ford/Young landfill.

3. The leachate flowing to the Prairie River through the landfill drain contained elevated levels of certain constituents. These included total organic carbon, ammonia nitrogen, total phosphorus, and nickel, which may be of concern if the loadings increase over present levels.

Methods

Benthic macroinvertebrate samples were taken from the Prairie River using a triangular dip net with a 1.0 mm mesh. Organisms were also handpicked from all available substrates. Sampling was conducted at each location until no new forms were found. Macroinvertebrates were identified as collected and relative abundance estimated.

Surface water grab samples were preserved, placed on ice and returned to the Environmental Laboratory in Lansing for analysis. Sediment samples from depositional areas were also collected, placed on ice and returned

to the Enviornmental Laboratory in Lansing for analysis. General stream observations were also made at each station.

Results and Discussion

Macroinvertebrates

The benthic macroinvertebrate community at McKale Road (Station 1), upstream of any possible influence from the Ford/Young Landfill or materials dumped from the McKale Road Bridge, was indicative of excellent stream quality. Large numbers of fingernail clams, scuds, stoneflies, mayflies, and caddisflies were present and comprised the majority of a diverse macroinvertebrate community (Table 1).

Relative to McKale Road, slight shifts in the benthic macroinvertebrate community were found at Stations 2 and 3, 1.2 and 2.0 km downstream of McKale Road, respectively. Declines in the numbers of fingernail clams, scuds, mayflies and stoneflies were observed. The likely cause of these changes was a reduction in current speed and macroinvertebrate substrate (especially logs) and an increase in stream depth. Other factors such as temperature and canopy may be influencing this shift, since McKale Road is the downstream boundary for the designated trout stream portion of the Prairie River. Even with these changes from Station 1, the macroinvertebrate communities present at Stations 2 and 3 were still indicative of good stream quality. A diverse macroinvertebrate community was found which included stoneflies, mayflies and caddisflies. These findings are consistent with the 1983 findings, even though some minor changes in communities were found. For example, burrowing mayflies were present at all three stations in 1983, but were only found at McKale Road in 1986. This difference was likely due to a major effort in 1983 to find the mayflies. Overall, stream quality has not changed.

Water Chemistry

The water chemistry results for Stations 1, 3, and 4 on the Prairie River were very similar, indicating negligible effects on water chemistry constituent from any inputs between McKale and Prairie River Roads (Table 2).

Station B (the downstream channel) was similar to the Prairie River in water chemistry. However, Station A (the upstream channel) was noticeably different than the river. Concentrations of total organic carbon, ammonia nitrogen, total phosphorus, sodium, zinc and cadmium were elevated, while total dissolved solids, calcium, conductivity, iron, potassium, magnesium, pH, and sulfate were depressed. These differences may be due to the very sluggish flow at this station.

The landfill drain (Station C) was markedly different in quality than the other five stations sampled. The landfill drain discharge contained elevated levels of total dissolved solids, total organic carbon, ammonia nitrogen, total phosphorus, alkalinity, calcium, chloride, conductivity, iron, potassium, magnesium, sodium and nickel. The discharge of total organic carbon, ammonia nitrogen, phosphorus and nickel may be a concern to the quality of Priaire River and Prairie River Lake if loadings

increase over present levels. At the rate of discharge observed on August 12, these concentrations would have a negligible effect on river/lake quality.

Elevated nickel and iron results in the landfill drain were consistent with the 1983 results. However, contrary to the 1983 results, no volatile organic compounds were detected in the landfill drain.

Sediments

Sediment sampling results at McKale Road (Station 1), Prairie River Lake inlet (Station 5) and Prairie River Lake outlet (Station 6) were generally similar (Table 3). There was no evidence of sediment contamination with substances, such as mercury, polychlorinated biphenyls (PCBs) or pesticides, which could bioaccumulate fish to unacceptable levels. Based on the sediment results, fish contamination in the Prairie River or Prairie River Lake is not expected.

Sediment zinc concentrations were lower in non-river stations (Station A,B,C), indicating that zinc was not migrating from the landfill. Overall, sediment results at stations B and C did not indicate any source of contaminants from the landfill. Concentrations were usually similar to or less than those found in the river or lake.

The sediment results at Station A exhibited a pattern similar to that found in the water results. Sodium and cadmium concentrations were elevated while nickel, calcium, iron, potassium, magnesium and manganese were depressed.

Report by: William Creal, Aquatic Biologist
Field Work by: Brenda Sayles, Aquatic Biologist
William Creal, Aquatic Biologist

Referenced Cited

Creal, W. 1983. A fish, benthic macroinvertebrate, sediment, and water chemistry survey of Prairie River and Prairie River Lake, St. Joseph County, March 15 and June 17, 1983. MDNR report.

Figure 1. Sampling station locations on the Prairie River, St. Joseph County, August, 1986.

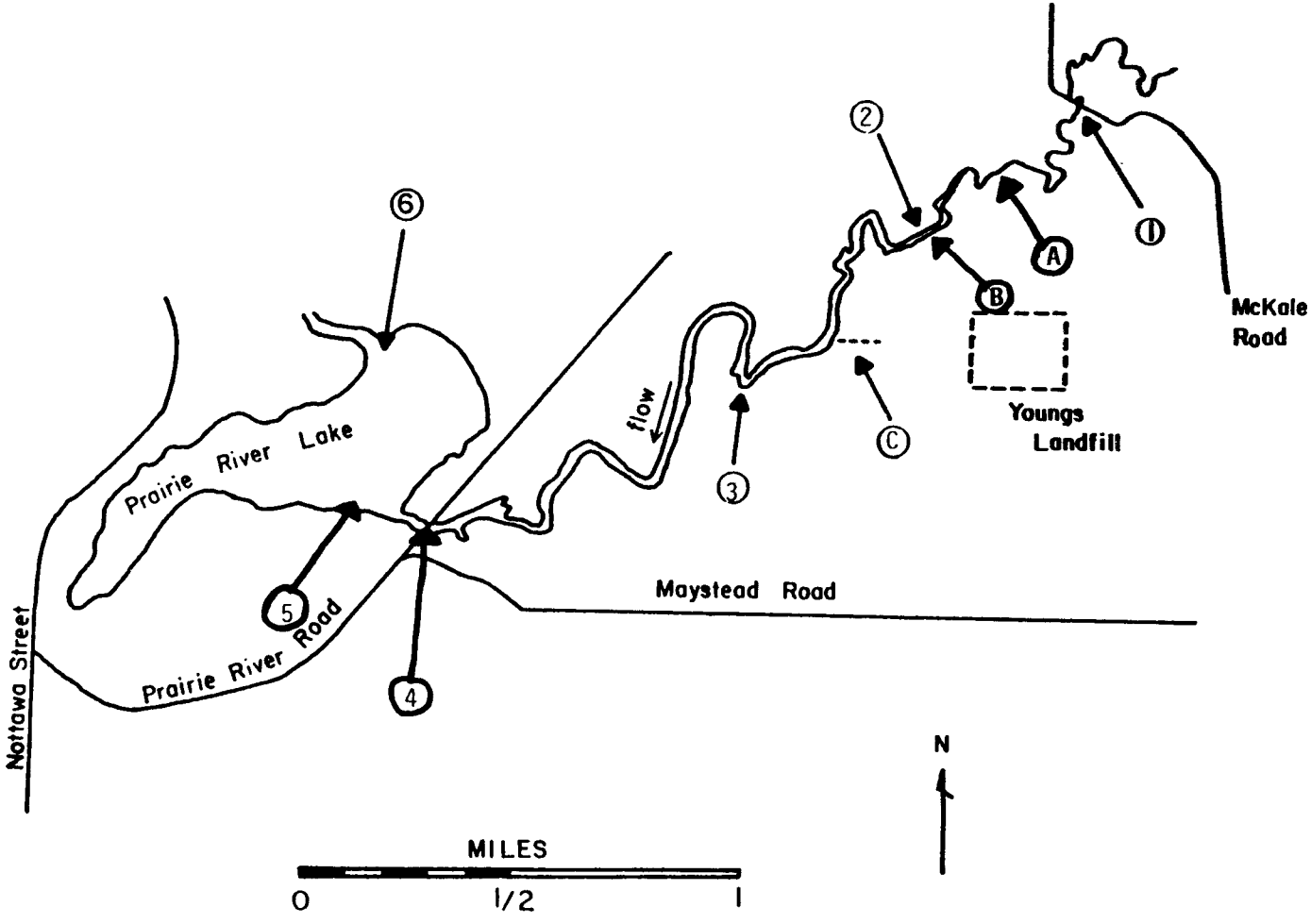


TABLE 1. PRAIRIE RIVER MACROINVERTEBRATE SAMPLING RESULTS

STATION:	1	2	3
LOCATION:	MCKALE ROAD	ACROSS FROM LANDFILL	DOWNSTREAM OF LANDFILL
TAXON			
PORIFERA (sponges)		X	
ISOPODA (sowbugs)		X	X
AMPHIPODA (scuds)	X	X	X
GASTROPODA (snails)			
LYMNAEA	X		
SPHAERIIDAE (clams)	X	X	
OLIGOCHAETA (worms)	X		
ANISOPTERA (dragonflies)	X	X	
HEMIPTERA (bugs)			
NOTONECTIDAE	X		
COLEOPTERA (beetles)			
ELMIDAE(a)	X		X
ELMIDAE(l)			X
DYTISCIDAE	X		
DIPTERA (flies)			
SIMULIIDAE	X	X	X
CHIROMIDAE	X	X	X
PLECOPTERA (stoneflies)			
PARAGNETINA		X	X
PTERONARCYS	X	X	
PERLESTA	X		
PHASGONOPHORA	X		
EPHEMEROPTERA (mayflies)			
BAETIDAE	X	X	X
HEPTAGENIIDAE	X		X
HEXAGENIA	X		
ISONYCHIA	X		
TRICHOPTERA (caddisflies)			
BRACHYCENTRUS	X		
HYDROPSYCHIDAE	X	X	X
LIMNEPHILIDAE	X		X
PHRYGANEIDAE	X		X
RHYACOPHILIDAE		X	
TOTAL TAXA	22	12	12

AUGUST 12, 1986

TABLE 2. PRAIRIE RIVER WATER RESULTS, AUGUST 12, 1986

STATION:	1	A	B	C	3	4
LOCATION:	MCKALE ROAD	UPSTREAM CHANNEL	DOWNSTREAM CHANNEL	LANDFILL DRAIN	DOWNSTREAM LANDFILL	PRAIRIE ROAD
LAB NO.:	61693	61692	61691	61690	61689	61688
SUSPENDED SOLIDS	<4	<4	4	4	5	9
DISSOLVED SOLIDS	412	288	336	736	380	352
ORGANIC CARBON	3.83	8.09	6.34	14.00	4.13	4.44
NITRATE NITROGEN	1.60	0.09	0.92	1.10	1.50	1.20
AMMONIA NITROGEN	0.014	0.280	0.100	9.200	0.026	0.050
KJELDAHL NITROGEN	0.39	0.93	0.62	14.00	0.41	0.45
TOTAL PHOSPHORUS	0.015	0.040	0.032	0.060	0.015	0.018
ALKALINITY	230	230	240	390	230	210
ALKALINITY, $CO_3^=$	<5	<5	<5	<5	<5	<5
ALKALINITY, HCO_3^-	230	230	240	390	230	210
CALCIUM	83.2	60.7	78	115	82.8	78.5
CHLORIDE	12	9.8	11	140	12	12
CONDUCTIVITY (UMHOS/CM)	509	470	563	1315	509	588
IRON	0.412	0.284	0.424		0.492	0.682
MERCURY	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
POTASSIUM	1.3	0.6	1.3	16.5	1.3	1.2
MAGNESIUM	21.8	16.1	20.7	36.4	21.8	21
SODIUM	6.3	10.7	6.1	94.8	6.1	6.2
PH (SU)	8.0	7.8	8.0	8.1	8.2	8.3
SULFATE	56	5.7	41	18	55	51
ZINC	<0.004	0.0044	<0.004		<0.004	<0.004

RESULTS ARE IN MG/L UNLESS INDICATED

TABLE 2. PRAIRIE RIVER WATER RESULTS, AUGUST 12, 1986

CADMIUM	<0.0002	0.0004	<0.0002	<0.0002	<0.0002	<0.0002
CHROMIUM	<0.003	<0.003	<0.003	<0.003	<0.003	<0.003
HEX CHROMIUM	<0.005	<0.005	<0.005	<0.005	<0.005	<0.005
COPPER	<0.001	0.0028	0.0015	0.0019	0.0016	0.0024
NICKEL	<0.004	<0.004	<0.004	0.0083	<0.004	<0.004
LEAD	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
METHYLENE CHLORIDE	<0.005	<0.005	<0.005	<0.005	<0.005	<0.005
CHLOROBENZENE	<0.005	<0.005	<0.005	<0.005	<0.005	<0.005
SCAN 1	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
PCB	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001
SCAN 3	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001

RESULTS ARE IN MG/L UNLESS INDICATED

TABLE 3. PRAIRIE RIVER SEDIMENT RESULTS, AUGUST 12, 1986

STATION:	1	A	B	C	5	6
LOCATION:	MCKALE ROAD	UPSTREAM CHANNEL	DOWNSTREAM CHANNEL	LANDFILL DRAIN	PRAIRIE L. INLET	PRAIRIE L OUTLET
LAB NO.:	61699	61698	61697	61696	61695	61694
TOTAL SOLIDS(%)	35	12	21	66	15	18
CALCIUM	41000	46300	133000	24200	92500	213000
CADMIUM	<2	2.5	<2	<2	<2	<2
COBALT	6.75	<5	<5	<5	6	<5
CHROMIUM	12	<5	6.5	8.5	11	12
COPPER	10	8.5	12	29	19	19
IRON	37000	11500	26000	14500	43000	33000
MERCURY	<0.5	<0.5	<0.5	<0.2	<0.5	<0.2
POTASSIUM	253	181	228	350	260	330
LITHIUM	5.25	<2	3	6	5	4.5
MAGNESIUM	5750	2200	41000	8550	4900	4900
MANGANESE	2150	775	1500	470	4200	3900
SODIUM	140	250	130	170	110	130
NICKEL	12	<5	6.5	10	10	8
LEAD	15.5	15	<5	11	19	29
ZINC	77.5	23	44	42	84	72.5
PCB	<1.5	<2.8	<2.9	<0.77	<4.4	<3.1
SCAN 3	<0.15	<0.28	<0.29	<0.077	<0.44	<0.31

SCAN 3 = chlorinated hydrocarbons, and organochlorine pesticides

RESULTS ARE IN MG/KG UNLESS INDICATED

APPENDIX C

Michigan Department of Natural Resources
Surface Water Quality Division
April, 1986

Water, Sediment, and Macroinvertebrate Survey of the South Branch Raisin River, and Eastside Drain, vicinity of Adrian, 1982-85.

Water, sediment and macroinvertebrate sampling was conducted on the Raisin River near Adrian and its tributaries from 1982 to 1985. The initial purpose of the study was to evaluate the effect of the Adrian Watewater Treatment Plant (WWTP) discharge on the Raisin River and the quality of Eastside Drain.

Summary and Conclusions

1) Moderate river quality exists upstream of the Adrian WWTP (Table 1).

2) In 1983, impacts from the Adrian WWTP on the South Branch Raisin River were present but not considered severe.

3) In 1983-85, a drastic decline in river quality downstream of Howell Road was detected due to an unpermitted discharge of rendering wastes from Adrian Tankage into the river. Extremely foul odors eminated from this section of the river. Large fat globules, blood and hair floated on the surface and lodged behind logs and debris. Invertebrate species composition declined dramatically in this river reach.

4) Poor stream quality was found in Eastside Drain, especially in the area of Parr Highway. Sediment chemistry analysis of Eastside Drain showed low levels of organic compounds. In 1972 and 1982 the upper reach of the drain (Parr Highway) was virtually devoid of animal life; stream quality improved from Parr Highway to the confluence with the South Branch Raisin River.

Methods

Water and sediment samples were taken and preserved following procedures in the Quality Assurrance Manual and sent to the Environmental Laboratory in Lansing for analysis. Chlorine was analyzed in the field using an amperometric titrator.

Qualitative benthic macroinvertebrate samples were collected using a triangular dip net and hand picking and identified in the field. Relative abundance was estimated. Stream Problem Assess-

ment cards were completed and observations recorded.

Results and Discussion

Upstream of the Adrian WWTP moderate river quality and moderate numbers of aquatic insects and fish were found (Tables 1, 2 & 3). From 1983 to 1985 an increase in abundance of isopods and a decline in species diversity occurred (Table 3).

Downstream of the Adrian WWTP aquatic biota differed (Tables 2, 3 & 5). At Howell Road these differences may have been due to a change in stream velocity. Chlorine was detected in the water to Howell Road (Table 4). Overall, the impact from the Adrian WWTP was not considered severe. Several metals were detected in the WWTP effluent. Due to the nature of the metals and their concentrations their impact on the system was concluded to be insignificant (Table 6).

In 1983, an unpermitted discharge of rendering wastes by Adrian Tankage was rediscovered downstream of Howell Road; this discharge was known to exist prior to 1972. There was a gross degradation of stream quality at this discharge with foul sewage odors eminating from the water and sediments. The Adrian Tankage discharge exceeded 120 F and contained white globules which were presumably oils and fats and high concentrations of BOD, TOC and ammonia (Table 6). An increase of 6 F in river temperature was recorded (68 to 74 F) downstream of the discharge. The discharge caused a reduction in invertebrate species (Table 5), as well as abundant growths of bacterial slimes. Immediately downstream of the discharge, species tolerant of organic pollution dominated, namely midges and worms. Fisheries Division reported similar degraded river conditions in a 1983 survey (Towns, 1985).

In 1984 and 1985, the discharge of rendering waste was still occurring with sludge, blood, and fats present at the point of discharge and downstream. Bacterial slime growths were present in the Raisin River at least to the Beaver Creek confluence. Conditions in the river had not improved from the 1983 observations.

Sediment sampling on upper reaches of Eastside Drain showed the presence of organic compounds and heavy metals (Tables 7 & 8). Biological investigations of Eastside Drain in 1972 reported no living aquatic organisms (Evans, 1973). Investigations in 1983 showed only sparse numbers of tolerant invertebrate species and in 1984 an abundance of bacterial slimes (Table 2 & 9). Stream quality improved from Parr Highway to the confluence with the South Branch Raisin River.

Bibliography

Evans, E. 1973. A Biological Survey of the Raisin River in the Vicinity of Adrian, Lenawee County, Michigan, July 13, 1972

Towns, G.L. 1985. A Fisheries Survey of the River Raisin, August, 1984. MDNR Fisheries Division Technical Report No# 85-3.

Report by: Stuart Kogge, Student Aide
William Creal, Aquatic Biologist

Figure 1.--South Branch Raisin River and tributaries sampled for water, sediment and macroinvertebrate analysis, 1982-85.

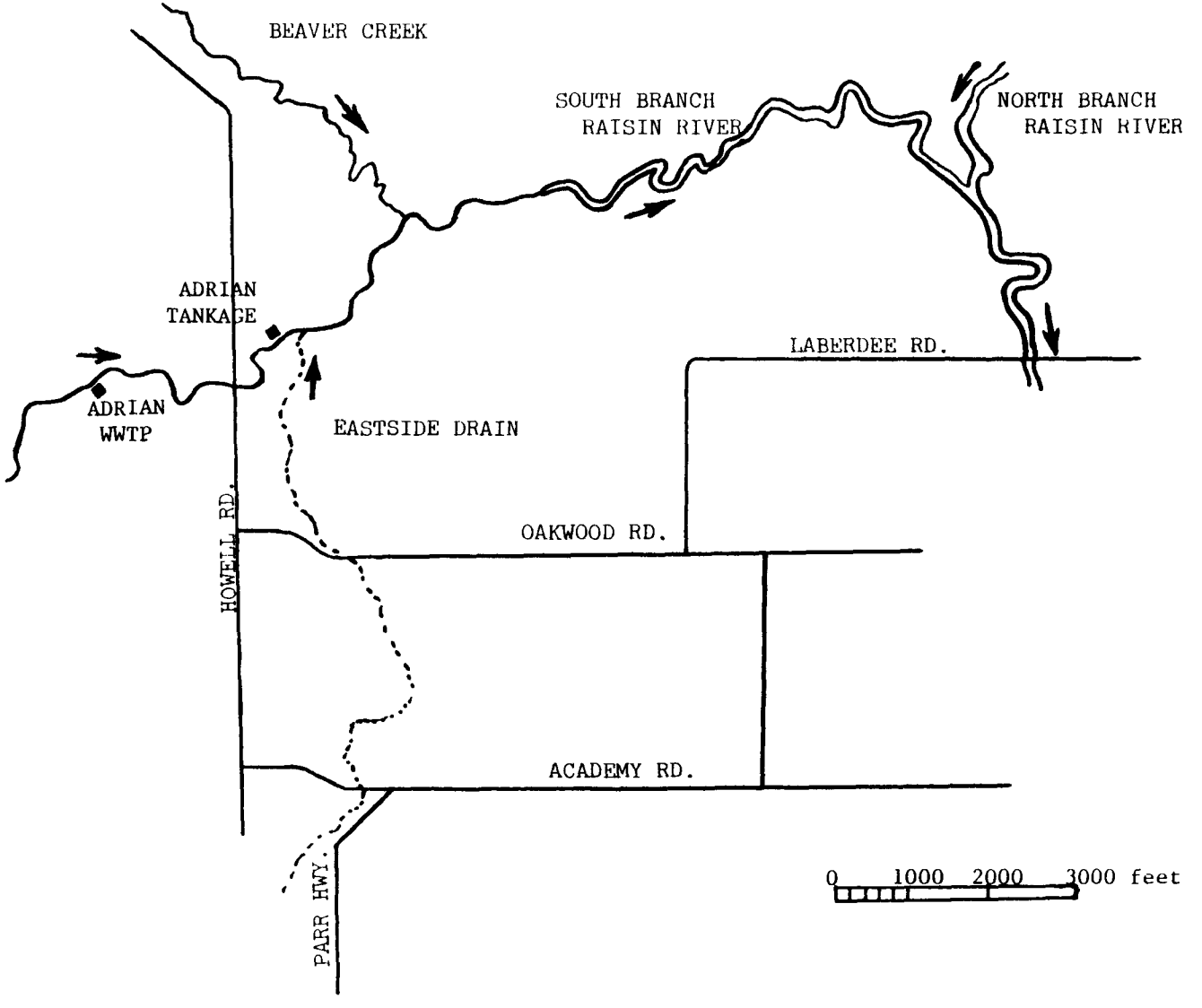


Table 1.--Observations on the Raisin River, vicinity of Adrian, 1983-85.

Upstream of Adrian WWTP	water odors	surface oils	sediment odors	sediment oils	sediment deposits	substrate undersides
9-12-83	normal	none	normal	absent	silt	normal
6-28-84	normal	none	normal	absent		normal
8-21-85	normal	none	normal	absent		
Downstream of Adrian WWTP						
9-12-83	chemical-chlorine	none		slight		normal
Downstream of Howell Rd.						
9-12-83	normal	none	petroleum	moderate		
Upstream of Adrian Tankage						
9-28-83	normal	none	normal-fishy	absent		
Adrian Tankage						
9-12-83	sewage	globs	sewage	absent	sludge	black
6-28-84				greases		
Upstream of East Side Drn.						
9-28-83	sewage-rancid	none	anaerobic-rancid	absent	sludge-rancid	black
East Side Drn. Oakwood Xing						
9-12-83	chemical	none	chemical	absent		normal
East Side Drn. Pan Hwy Xing						
6-28-84	petroleum	sheen	petroleum	profuse		black

East Side Drn. River Confluence	water odors	surface oils	sediment odors	sediment oils	sediment deposits	substrate undersides
9-28-83	normal	none	normal	absent	sand	normal
Beaver Creek River Confluence						
9-28-83	normal	none	normal	absent	sand	normal
Confluence of Beaver Creek						
9-28-83	rancid	none	very anaerobic	absent	sludge	black
6-28-84	fats & grease	flecks fat globules	petroleum anaerobic	moderate		
N. & S. Branch Confluence						
9-28-83	normal-sewage	globs	anaerobic	absent	sludge	black
N. Branch Wilmouth Rd.						
9-28-83	normal	none	normal	absent	sand-silt	normal
Raisin R. Laberdee Rd.						
9-28-83	normal	none	normal	absent		black

* only in small depositional zones

Table 1.--Biological observations on the Raisin River, vicinity of Adrain, 1983-85.

Upstream of Adrian WWTP	phyto-plankton	periphyton	filamentous algae	macro-phytes	bacterial slimes	zoo-plankton	macroin-vertebrates	fish	comments
9-12-83	absent	sparse	absent	absent	sparse	absent	moderate	moderate	Carp, minnows, sculpins present reddish iron ppt. along shore silty deposits on rocks
6-28-84	absent	none-sparse	sparse	absent	absent		moderate		spotty distribution of
8-21-85	absent	sparse	absent	absent	absent		sparse		invertebrates still sparse
Downstream of Adrian WWTP									
9-12-83	absent	sparse	sparse		absent	absent	moderate-abundant		200' d/s of discharge animals sparse, *no cladophora*
Downstream of Howell Rd.									
9-12-83	absent	sparse	absent	absent					Carp, minnows, 1 dead creek chub sediments with many bubbles
Upstream of Adrian Tankage									
9-28-83	absent	sparse	absent	absent	absent	absent	moderate	sparse	
Adrian Tankage									
9-12-83					abundant		sparse		Foul rancid odor Hot turbid water in discharge White globules in discharge
6-28-84					abundant-profuse in the plume				Odors putrid/nauseating Discharge channel full of slimes, blood, hair Fats on surface
8-21-85					abundant-profuse in the plume				Odors putrid/nauseating Discharge channel full of slimes, blood, hair Fats on surface
Upstream of East Side Drn.									
9-28-83	absent	absent	absent	absent	abundant	absent	sparse	absent	Stagnant pools, bubbling sludge black sediments

East Side Drn. Oakwood Xing	phyto-plankton	periphyton	filamentous algae	macro-phytes	bacterial slimes	zoo-plankton	macroin-vertebrates	fish	comments
9-12-83	absent	absent	absent	absent	absent	absent	sparse	absent	
East Side Drn. Pan Hwy Xing									
6-28-84			sparse	absent	abundant		absent-sparse		Some sediments approx. 4' deep with oils
East Side Drn. River Confluence									
9-28-83	absent	sparse	absent	absent	absent	absent	sparse	sparse	Considerable loose sand bedload
Beaver Creek River Confluence									
9-28-83	absent	absent	absent	absent	absent	absent	sparse	absent	Considerable loose sand bedload
Confluence of Beaver Creek									
9-28-83	absent	absent	sparse	absent	profuse	absent	sparse	absent	Sludge beds with slimes black, anaerobic, bubbling
6-28-84			absent	absent	abundant				Fat globules on water surface max. river depth 2' slimes present on all sticks and logs less black sludge present more fats on water surface large gatherings of reddish brown globules behind log jams
N. & S. Branch Confluence									
9-28-83	absent	sparse	sparse	absent	abundant	absent	sparse	absent	Very small globs of grease on surface, may indicate discharge from Adrian Tankage approx. 12 hours prior - time of passage from Howell Rd.

N. Branch Milsouth Rd.	phyto-plankton	periphyton	filamentous algae	macro-phytes	bacterial slimes	zoo-plankton	macroin-vertebrates	fish	comments
9-28-83	absent	sparse	sparse	absent	absent	absent	sparse-moderate	moderate	Habitat limited largely to sticks and logs considerable bedload of sand and silt most animals beneath logs, if on top sometimes covered with sediment
Raisin R. Laberdee Rd.									
9-28-83	absent	sparse	absent	absent	sparse	absent	sparse	sparse-moderate	Deep water made collections difficult limited habitat steep clay banks

Table 3.--Qualitative macroinvertebrate sampling results from the South Branch Raisin River and Eastside Drain near Adrian, September, 1983, June, 1984 and August, 1985. P = Present, AB = Abundant

Station Location	Upstream of Adrian WWTP 1983	Upstream of Adrian WWTP 1984	Upstream of Adrian WWTP 1985
Porifera (sponges)	P		P
Oligochaeta (earthworms)	P		
Turbellaria (flatworms)	P		
Gastropoda (snails)	P		
Ephemeroptera (mayflies)			
Baetid	P	P	
Caenidae	AB		
Heptageniidae	AB	P	P
Trichoptera (caddisflies)			
Hydropsychidae	P	P	P
Anisoptera (dragonflies)		P	
Zygoptera (damselflies)	P	P	
Coleoptera (beetles)			
Gerridae	P	P	
Elmidae	P	P	P
Chironomidae (midges)	P	P	
Tipulidae (craneflies)	P		
Decapoda (crayfish)	P	P	P
Isopoda (sow bugs)			AB
Amphipoda (scuds)	P		P
Total Number of Taxa	15	9	7

Table 4.--Total chlorine sampling results in the South Branch Raisin River, vicinity of Adrian WWTP, September 12, 1983. Results in mg/l.

Location	South Side	Middle	North Side
Upstream of Adrian WWTP discharge	<0.02	<0.02	<0.02
Adrian WWTP discharge	0.03, 0.39 0.57	-	-
10 ft. downstream	0.30	0.20	0.31
50 ft. downstream	0.07	0.11	0.06
120 ft. downstream	0.03	0.04	0.04
200 ft. downstream	0.03	-	0.03
300 ft. downstream	0.03	-	0.03
400 ft. downstream	0.05	-	-
Howell Rd.	0.02	-	0.02

Table 5.--Qualitative macroinvertebrate sampling results from the Raisin River Adrian, September 12 and 28, 1983.
P = Present, AB = Abundant

Station Location	Downstream Adrian WWTP	Howell Road	Upstream Adrian Tankage	Upstream Eastside Drain	Downstream Eastside Drain	Upstream North Branch Confluence	Laberdee Road
Oligochaeta (earthworms)		P	P	P	P	P	P
Isopoda (sowbugs)							
Gastropoda (snails)		P		P		P	P
Pelecypoda (clams)	P						
Amphipoda (scuds)		P					
Decapoda (crayfish)	P	P	P				P
Epheneroptera (mayflies)							
Baetidae	P		P				AB
Caenidae		AB					
Heptageniidae	P	AB	P				P
Trichoptera (caddisflies)	AB						
Hydropsychidae			AB				P
Anisoptera (dragonflies)		P			P		P
Zygoptera (damselflies)		P					P
Coleoptera (beetles)							
Gerridae	P	P	P				P
Elmidae	P						
Corixidae (boatmen)							P
Chironomidae (midges)	P	P	P	P	P	P	P
Simuliidae (blackflies)	P						
Tipulidae (craneflies)	P						
Total Number of Taxa	10	10	7	3	3	3	11

Table 6.--Water chemistry results of the South Branch Raisin River, September, 1983.

Station Number *(Number attributed to site location)	1	2	3	4	5	6	7	8	9
Cadmium (ug/l)	<0.2	<0.2	<0.2	<0.2	-	-	-	-	-
Chromium (ug/l)	2.0	2.0	2.4	1.9	-	-	-	-	-
Copper (ug/l)	1.8	4.0	2.5	3.0	-	-	-	-	-
Iron (ug/l)	600	40.0	580	466	-	-	-	-	-
Lead (ug/l)	3.4	4.1	2.9	2.7	-	-	-	-	-
Nickel (ug/l)	<4.0	4.9	<4.0	<4.0	-	-	-	-	-
Zinc (ug/l)	4	12	4	6	-	-	-	-	-
Total Organic Carbon (mg/l)	-	-	-	-	600	-	-	-	-
Nitrate Nitrogen (mg/l)	-	-	-	-	4	-	-	-	-
Ammonia Nitrogen (mg/l)	-	-	-	-	44	0.071	0.057	0.059	0.083
Total Kjeldahl Nitrogen (mg/l)	-	-	-	-	90	-	-	-	-
Total Phosphorous (mg/l)	-	-	-	-	6.6	-	-	-	-
5-day BOD (mg/l)	-	-	-	-	1600	1.2	1.6	4.7	1.2
Temperature	68	-	-	-	<120	-	-	-	-
Dissolved Oxygen	-	-	-	-	-	8.8	-	-	9.2

Staion Number - Location

September 12, 1983 1 - 300m upstream of Adrian WWTP
. 2 - Adrian WWTP discharge
. 3 - 70m downstream of Adrian WWTP, south side
. 4 - 70m downstream of Adrian WWTP, north side
. 5 - Adrian Tankage discharge
September 28, 1983 6 - Upstream of Adrian Tankage
. 7 - Upstream of Beaver Creek
. 8 - Upstream of confluence with North Branch
. 9 - Eastside Drain near confluence

Table 7.--Sediment Sampling Results from Parr Highway, Eastside Drain, Adrian, June 28, 1984.

Compound	mg/kg
Polychlorinated biphenyls *	
1254	2.10
1260	4.70
Napthalene	1.60
Phenanthrene	7.00
Anthracene	0.84
Fluoranthene	13.00
Pyrene	1.20
Benzo (A) Anthracene	6.70
Chrysene	10.00
Benzo (B) Fluoranthene	14.00
Benzo (K) Fluoranthene	2.80
Benzo (A) Pyrene	12.00
Dibenzo (A*H) Anthracene	2.40
1*12 - Benzoperylene	4.60
Indeno (1*2*3 - CD) Pyrene	5.90

All other Scan 3 and 6 compounds were undetectable.
* detection limit 250 ug/kg for Aroclors, all other Scan 3 compounds = 130
** detection limit polar pesticides = 250
detection limit phthalates = 5000

Table 8.--Eastside Drain sediment analysis, May 5, 1982 (mg/kg). See map for location of sampling stations.

Station	Aluminum	Cadmium	Chromium	Cobalt	Copper	Lead	Mercury	Nickel	Antimony	Zinc	‡Curene 442	Firemaster 680
1	5600	K2	76	7	350	220	K0.5	64	NAV	680	2300	2700
2	5700	4	130	K5	390	200	K0.5	95	NAV	1200	750	6200
3	5700	5	80	K5	280	110	K0.2	56	NAV	720	7800	1800
4	1500	4	57	7	120	79	K0.2	40	NAV	810	3800	2200
5	8400	K2	54	8	59	54	K0.2	36	NAV	310	480	940
6	2800	K2	27	K5	28	22	K0.5	13	NAV	130	370	330
7	3500	K2	16	K5	19	13	K0.2	12	NAV	98	140	190
9	3900	K2	25	K5	23	15	K0.2	14	NAV	110	K50	58
10	9800	K2	51	K5	64	53	K0.2	34	NAV	380	200	1100

‡ Possible low recovery in the exchange of solvents for HPLC analysis due to the "oiliness" of the extract.
K Element present at levels below the given value
NAV Not available

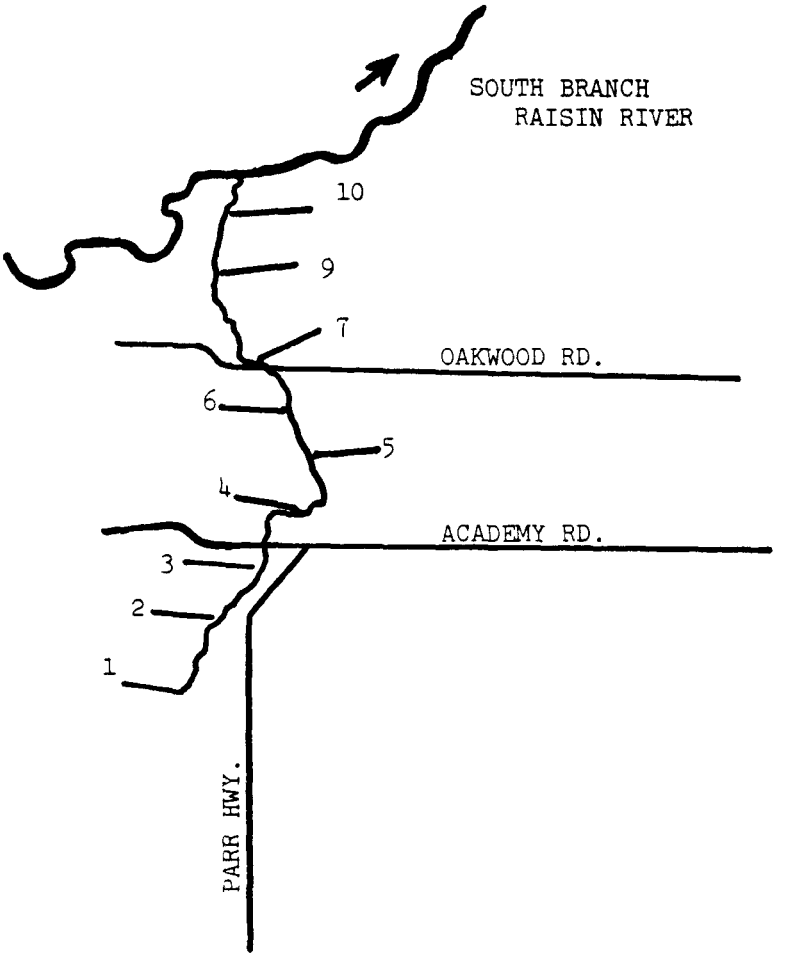


Table 9.--Qualitative macroinvertebrate sampling results from Eastside Drain. P = Present

Station Location	Oakwood Road (9/12/83)	Upstream Raisin River Confluence (9/28/83)	Parr Highway (6/28/84)
Oligochaeta (earthworms)	P	P	
Isopoda (sowbugs)	P		
Gastropoda (snails)	P		
Pelecypoda (clams)			
Amphipoda (scuds)			
Decapoda (crayfish)			
Ephemeroptera (mayflies)			
Baetidae			
Caenidae			
Heptageniidae			
Trichoptera (caddisflies)			
Hydropsychidae	P		
Anisoptera (dragonflies)	P	P	
Zygoptera (damselflies)	P	P	
Coleoptera (beetles)			
Gerridae	P	P	
Elmidae			
Corixidae			
Veliidae		P	
Chironomidae (midges)	P	P	P
Simuliidae (blackflies)			
Tipulidae (craneflies)	P		
Total Number of Taxa	9	5	1

APPENDIX D

DEPARTMENT OF NATURAL RESOURCES

WATER RESOURCES COMMISSION

GENERAL RULES

Filed with the Secretary of State on November 14, 1986
These rules take effect 15 days after filing with the Secretary of State

(By authority conferred on the water resources commission by sections 2 and 5 of Act No. 245 of the Public Acts of 1929, as amended, being §§323.2 and 323.5 of the Michigan Compiled Laws)

R 323.1041 to R 323.1050, R 323.1053, R 323.1055, R 323.1058 to R 323.1065, R 323.1070, R 323.1075, R 323.1082, R 323.1092 to R 323.1098, R 323.1100, and R 323.1116 of the Michigan Administrative Code, appearing on pages 1630 and 1632 to 1639 of the 1979 Administrative Code and pages 162 to 164, 166, and 167 of the 1984 Annual Supplement to the Code, are amended, and R 323.1099 is added, to read as hereinafter set forth.

R 323.1074, R 323.1080, R 323.1091, R 323.1110, and R 323.1115 of the Michigan Administrative Code, appearing on pages 1636 to 1644 of the 1979 Michigan Administrative Code, are rescinded.

PART 4. WATER QUALITY STANDARDS

R 323.1041 Purpose.

Rule 41. The purpose of the water quality standards as prescribed by these rules is to establish water quality requirements applicable to the Great Lakes, the connecting waters, and all other surface waters of the state, to protect the public health and welfare, to enhance and maintain the quality of water, to protect the state's natural resources, and serve the purposes of Public Law 92-500, as amended, 33 U.S.C. §466 et seq., Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws, and the Great Lakes water quality agreement enacted November 22, 1978. These standards may not reflect current water quality in all cases, but are minimum water quality requirements for which the waters of the state are to be managed.

R 323.1043 Definitions; A to N.

Rule 43. As used in this part:

(a) "Agricultural use" means a use of water for agricultural purposes, including livestock watering, irrigation, and crop spraying.

(b) "Anadromous salmonids" means those trout and salmon which ascend streams to spawn.

(c) "Carcinogen" means a substance which causes an increased incidence of benign or malignant neoplasms or a substantial decrease in the latency period between exposure and onset of neoplasms through oral or dermal exposure or through inhalation exposure when the cancer occurs at nonrespiratory sites, in at least 1 mammalian species, or man through epidemiological or clinical studies, unless the commission, on the basis of credible scientific evidence, determines that there is significant

uncertainty regarding the credibility, validity, or significance of such study or studies, in which case it shall refer the question of carcinogenicity to experts on carcinogenesis and shall consider the recommendations of those experts in making its final determination.

(d) "Coldwater fish" means those fish species whose populations thrive in relatively cold water, including trout, salmon, whitefish, and cisco.

(e) "Commission" means the Michigan water resources commission established pursuant to Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws.

(f) "Connecting waters" means any of the following:

(i) The St. Marys river.

(ii) The Keweenaw waterway.

(iii) The Detroit river.

(iv) The St. Clair river.

(v) Lake St. Clair.

(g) "Designated use" means a use of the waters of the state as established by these rules, including use for any of the following:

(i) Industrial, agricultural, and public water supply.

(ii) Recreation.

(iii) Fish, other aquatic life, and wildlife.

(iv) Navigation.

(h) "Dissolved oxygen" means the amount of oxygen dissolved in water and is commonly expressed as a concentration in terms of milligrams per liter.

(i) "Dissolved solids" means the amount of materials dissolved in water and is commonly expressed as a concentration in terms of milligrams per liter.

(j) "Effluent" means a wastewater discharged from a point source to the waters of the state.

(k) "Fecal coliform" means a type of coliform bacteria found in the intestinal tract of humans and other warm-blooded animals.

(l) "Final acute value" means the level of a chemical or mixture of chemicals that does not allow the mortality of important fish or fish food organisms to exceed 50% when exposed for 96 hours, except where a shorter time period is appropriate for certain species.

(m) "Fish, other aquatic life, and wildlife use" means the use of the waters of the state by fish, other aquatic life, and wildlife for any life history stage or activity.

(n) "Industrial water supply" means a water source intended for use in commercial or industrial applications or for noncontact food processing.

(o) "Inland lake" means an inland body of standing water of the state situated in a topographic depression other than an artificial agricultural pond less than one acre, unless it is otherwise determined by the commission. The commission may designate a dammed river channel or an impoundment as an inland lake based on aquatic resources to be protected.

(p) "Keweenaw waterway" means the entire Keweenaw waterway, including Portage lake, Houghton county.

(q) "MATC" means the maximum acceptable toxicant concentration obtained by calculating the geometric mean of the lower and upper chronic limits from a chronic test. A lower chronic limit is the highest tested concentration which did not cause the occurrence of a specified adverse effect. An upper chronic limit is the lowest tested concentration which

did cause the occurrence of a specified adverse effect and above which all tested concentrations caused such an occurrence.

(r) "Mixing zone" means that portion of a water body wherein a point source discharge is mixed with the receiving water.

(s) "Natural water temperature" means the temperature of a body of water without an influence from an artificial source or a temperature as otherwise determined by the commission.

(t) "NOAEL" means the highest level of toxicant which results in no observable adverse effects to exposed test organisms.

(u) "Non-point source" means a source of material other than a source defined as a point source.

R 323.1044 Definitions; P to W.

Rule 44. As used in this part:

(a) "Palatable" means the state of being agreeable or acceptable to the sense of sight, taste, or smell.

(b) "Plant nutrients" means those chemicals, including nitrogen and phosphorus, necessary for the growth and reproduction of aquatic rooted, attached, and floating plants, fungi, or bacteria.

(c) "Point source" means a discernible, confined, and discrete conveyance from which wastewater is or may be discharged to the waters of the state, including the following:

(i) A pipe.
(ii) A ditch.
(iii) A channel.
(iv) A tunnel.
(v) A conduit.
(vi) A well.
(vii) A discrete fissure.
(viii) A container.
(ix) A concentrated animal feeding operation.
(x) A boat or other watercraft.

(d) "Public water supply sources" means a surface raw water source which, after conventional treatment, provides a source of safe water for various uses, including human consumption, food processing, cooking, and as a liquid ingredient in foods and beverages.

(e) "Raw water" means the waters of the state before any treatment.

(f) "Receiving waters" means the waters of the state into which an effluent is or may be discharged.

(g) "Sanitary sewage" means treated or untreated wastewaters which contain human metabolic and domestic wastes.

(h) "Standard" means a definite numerical value or narrative statement promulgated by the commission to maintain or restore water quality to provide for, and fully protect, a designated use of the waters of the state.

(i) "Suspended solids" means the amount of materials suspended in water and is commonly expressed as a concentration in terms of milligrams per liter.

(j) "Total body contact recreation" means any activity where the human body may come into direct contact with water to the point of complete submergence, including swimming, waterskiing, and skin diving.

(k) "Toxic substance" means a substance, except heat, when present in sufficient concentrations or quantities which are or may become harmful to plant life, animal life, or designated uses.

(l) "Warmwater fish" means those fish species whose populations thrive in relatively warm water, including any of the following:

(i) Bass.

(ii) Pike.

(iii) Walleye.

(iv) Panfish.

(m) "Wastewater" means storm water runoff which could result in injury to a use designated in R 323.1100; liquid waste resulting from commercial, institutional, domestic, industrial, and agricultural activities, including cooling and condensing waters; sanitary sewage; and industrial waste.

(n) "Waters of the state" means all of the following, but does not include drainage ways and ponds used solely for wastewater conveyance, treatment, or control:

(i) The Great Lakes and their connecting waters.

(ii) All inland lakes.

(iii) Rivers.

(iv) Streams.

(v) Impoundments.

(vi) Open drains.

(vii) Other surface waterbodies within the confines of the state.

R 323.1050 Physical characteristics.

Rule 50. The waters of the state shall not have any of the following unnatural physical properties in quantities which are or may become injurious to any designated use:

(a) Turbidity.

(b) Color.

(c) Oil films.

(d) Floating solids.

(e) Foams.

(f) Settleable solids.

(g) Suspended solids.

(h) Deposits.

R 323.1051 Dissolved solids.

Rule 51. (1) The addition of any dissolved solids shall not exceed concentrations which are or may become injurious to any designated use. Point sources containing dissolved solids shall be considered by the commission on a case-by-case basis and increases of dissolved solids in the waters of the state shall be limited through the application of best practicable control technology currently available as prescribed by the administrator of the United States environmental protection agency pursuant to section 304(b) of Public Law 92-500, as amended, 33 U.S.C. §466 et seq., except that in no instance shall total dissolved solids in the waters of the state exceed a concentration of 500 milligrams per liter as a monthly average nor more than 750 milligrams per liter at any time, as a result of controllable point sources.

(2) The waters of the state designated as a public water supply source shall not exceed 125 milligrams per liter of chlorides as a monthly average, except for the Great Lakes and connecting waters, where chlorides shall not exceed 50 milligrams per liter as a monthly average.

R 323.1053 Hydrogen ion concentration.

Rule 53. The hydrogen ion concentration expressed as pH shall be maintained within the range of 6.5 to 9.0 in all waters of the state. Any artificially induced variation in the natural pH shall remain within this range and shall not exceed 0.5 units of pH.

R 323.1055 Taste- or odor-producing substances.

Rule 55. The waters of the state shall contain no taste-producing or odor-producing substances in concentrations which impair or may impair their use for a public, industrial, or agricultural water supply source which impair the palatability of fish as measured by test procedures approved by the commission.

R 323.1057. Toxic substances.

Rule 57. (1) Toxic substances shall not be present in the waters of the state at levels which are or may become injurious to the public health, safety, or welfare; plant and animal life; or the designated uses of those waters. Allowable levels of toxic substances shall be determined by the commission using appropriate scientific data.

(2) All of the following provisions apply for purposes of developing allowable levels of toxic substances in the surface waters of the state applicable to point source discharge permits issued pursuant to Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws:

(a) Water quality-based effluent limits developed pursuant to this subrule shall be used only when they are more restrictive than technology-based limitations required pursuant to R 323.2137 and R 323.2140.

(b) The toxic substances to which this subrule shall apply are those on the 1984 Michigan critical materials register established pursuant to Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws; the priority pollutants and hazardous chemicals in 40 C.F.R. §122.21, appendix D (1983); and any other toxic substances as the commission may determine are of concern at a specific site.

(c) Allowable levels of toxic substances in the surface water after a discharge is mixed with the receiving stream volume specified in R 323.1082 shall be determined by applying an adequate margin of safety to the MATC, NOAEL, or other appropriate effect end points, based on knowledge of the behavior of the toxic substance, characteristics of the receiving water, and the organisms to be protected.

(d) In addition to restrictions pursuant to subdivision (c) of this subrule, a discharge of carcinogens, not determined to cause cancer by a threshold mechanism, shall not create a level of risk to the public health greater than 1 in 100,000 in the surface water after mixing with the allowable receiving stream volume specified in R 323.1082. The commission may require a greater degree of protection pursuant to R 323.1098 where achievable through utilization of control measures already in place or where otherwise determined necessary.

(e) Guidelines shall be adopted pursuant to Act No. 306 of the Public Acts of 1969, as amended, being §24.201 et seq. of the Michigan Compiled Laws, setting forth procedures to be used by staff in the development of recommendations to the commission on allowable levels of toxic substances and the minimum data necessary to derive such recommendations. The commission may require the applicant to provide the minimum data when

otherwise not available for derivation of allowable levels of toxic substances.

(f) For existing discharges, the commission may issue a scheduled abatement permit pursuant to R 323.2145 upon a determination by the commission that the applicant has demonstrated that each of the following conditions is met:

(i) Immediate attainment of the allowable level of a toxic substance is not economically or technically feasible.

(ii) No prudent alternative exists.

(iii) During the period of scheduled abatement, the permitted discharge will be consistent with the protection of the public health, safety, and welfare.

(iv) Reasonable progress will be made toward compliance with this rule over the term of the permit, as provided for in a schedule in the permit.

R 323.1058 Radioactive substances.

Rule 58. The control and regulation of radioactive substances discharged to the waters of the state shall be pursuant to the criteria, standards, or requirements prescribed by the United States nuclear regulatory commission in 10 C.F.R. §20.1 et seq. and by the United States environmental protection agency.

R 323.1060 Plant nutrients.

Rule 60. (1) Consistent with Great Lakes protection, phosphorus which is or may readily become available as a plant nutrient shall be controlled from point source discharges to achieve 1 milligram per liter of total phosphorus as a maximum monthly average effluent concentration unless other limits, either higher or lower, are deemed necessary and appropriate by the commission.

(2) In addition to the protection provided under subrule (1) of this rule, nutrients shall be limited to the extent necessary to prevent stimulation of growths of aquatic rooted, attached, suspended, and floating plants, fungi or bacteria which are or may become injurious to the designated uses of the waters of the state.

R 323.1062 Microorganisms.

Rule 62. (1) All waters of the state shall contain not more than 200 fecal coliform per 100 milliliters. This concentration may be exceeded if such concentration is due to uncontrollable non-point sources. The commission may suspend this rule from November 1 through April 30 upon determining that designated uses will be protected.

(2) Compliance with the fecal coliform standards prescribed by subrule (1) of this rule shall be determined on the basis of the geometric average of any series of 5 or more consecutive samples taken over not more than a 30-day period.

(3) Protection of the waters of the state designated for total body contact recreation and public water supply source by the water quality standards prescribed by this rule may be subject to temporary interruption during or following flood conditions, accidents, or emergencies which affect a sewer or wastewater treatment system. In the event of such occurrences, notice shall be served to those affected in accordance with procedures established by the commission. Prompt corrective action shall be taken by the discharger to restore the designated use.

R 323.1064 Dissolved oxygen in Great Lakes, connecting waters, and inland streams.

Rule 64. (1) A minimum of 7 milligrams per liter of dissolved oxygen in all Great Lakes and connecting waterways shall be maintained, and, except for inland lakes as prescribed in R 323.1065, a minimum of 7 milligrams per liter of dissolved oxygen shall be maintained at all times in all inland waters designated by these rules to be protected for coldwater fish. In all other waters, except for inland lakes as prescribed by R 323.1065, a minimum of 5 milligrams per liter of dissolved oxygen shall be maintained. These standards do not apply for a limited warmwater fishery use subcategory or limited coldwater fishery use subcategory established pursuant to R 323.1100(10) or during those periods when the standards specified in subrule (2) of this rule apply.

(2) Waters of the state which do not meet the standards set forth in subrule (1) of this rule shall be upgraded to meet those standards. For existing point source discharges to these waters, the commission may issue permits pursuant to R 323.2145 which establish schedules to achieve the standards set forth in subrule (1) of this rule. If existing point source dischargers demonstrate to the commission that the dissolved oxygen standards specified in subrule (1) of this rule are not attainable through further feasible and prudent reductions in their discharges or that the diurnal variation between the daily average and daily minimum dissolved oxygen concentrations in those waters exceeds 1 milligram per liter, further reductions in oxygen-consuming substances from such discharges will not be required, except as necessary to meet the interim standards specified in this subrule, until comprehensive plans to upgrade these waters to the standards specified in subrule (1) of this rule have been approved by the commission and orders, permits, or other actions necessary to implement the approved plans have been issued by the commission. In the interim, all of the following standards apply:

(a) For waters of the state designated for use for coldwater fish, except for inland lakes as prescribed in R 323.1065, the dissolved oxygen shall not be lowered below a minimum of 6 milligrams per liter at the design flow during the warm weather season in accordance with R 323.1090(3) and (4). At the design flows during other seasonal periods, as provided in R 323.1090(4), a minimum of 7 milligrams per liter shall be maintained. At flows greater than the design flows, dissolved oxygen shall be higher than the respective minimum values specified in this subdivision.

(b) For waters of the state designated for use for warmwater fish and other aquatic life, except for inland lakes as prescribed in R 323.1065, the dissolved oxygen shall not be lowered below a minimum of 4 milligrams per liter, or below 5 milligrams per liter as a daily average, at the design flow during the warm weather season in accordance with R 323.1090(3) and (4). At the design flows during other seasonal periods as provided in R 323.1090(4), a minimum of 5 milligrams per liter shall be maintained. At flows greater than the design flows, dissolved oxygen shall be higher than the respective minimum values specified in this subdivision.

(c) For waters of the state designated for use for warmwater fish and other aquatic life, but also designated as principal migratory routes for anadromous salmonids, except for inland lakes as prescribed in R 323.1065, the dissolved oxygen shall not be lowered below 5 milligrams per liter as a minimum during periods of migration.

(3) The commission may cause a comprehensive plan to be prepared to upgrade waters to the standards specified in subrule (1) of this rule taking into consideration all factors affecting dissolved oxygen in these waters and the cost effectiveness of control measures to upgrade these waters and, after notice and hearing, approve the plan. After notice and hearing, the commission may amend a comprehensive plan for cause. In undertaking the comprehensive planning effort the commission shall provide for and encourage participation by interested and impacted persons in the affected area. Persons directly or indirectly discharging substances which contribute towards these waters not meeting the standards specified in subrule (1) of this rule may be required after notice and order to provide necessary information to assist in the development or amendment of the comprehensive plan. Upon notice and order, permit, or other action of the commission, persons directly or indirectly discharging substances which contribute toward these waters not meeting the standards specified in subrule (1) of this rule shall take the necessary actions consistent with the approved comprehensive plan to control these discharges to upgrade these waters to the standards specified in subrule (1) of this rule.

R 323.1065 Dissolved oxygen; inland lakes.

Rule 65. (1) The following standards for dissolved oxygen shall apply to lakes designated as trout lakes by the natural resources commission or lakes listed in the publication entitled "Coldwater Lakes of Michigan":

(a) In stratified coldwater lakes which have dissolved oxygen concentrations less than 7 milligrams per liter in the upper half of the hypolimnion, a minimum of 7 milligrams per liter dissolved oxygen shall be maintained throughout the epilimnion and upper 1/3 of the thermocline during stratification. Lakes capable of sustaining oxygen throughout the hypolimnion shall maintain oxygen throughout the hypolimnion. At all other times, dissolved oxygen concentrations greater than 7 milligrams per liter shall be maintained.

(b) Except for lakes described in subdivision (c) of this subrule, in stratified coldwater lakes which have dissolved oxygen concentrations greater than 7 milligrams per liter in the upper half of the hypolimnion, a minimum of 7 milligrams per liter of dissolved oxygen shall be maintained in the epilimnion, thermocline, and upper half of the hypolimnion. Lakes capable of sustaining oxygen throughout the hypolimnion shall maintain oxygen throughout the hypolimnion. At all other times, dissolved oxygen concentrations greater than 7 milligrams per liter shall be maintained.

(c) In stratified coldwater lakes which have dissolved oxygen concentrations greater than 7 milligrams per liter throughout the hypolimnion, a minimum of 7 milligrams per liter shall be maintained throughout the lake.

(d) In unstratified coldwater lakes, a minimum of 7 milligrams per liter of dissolved oxygen shall be maintained throughout the lake.

(2) For all other inland lakes not specified in subrule (1) of this rule, during stratification, a minimum dissolved oxygen concentration of 5 milligrams per liter shall be maintained throughout the epilimnion. At all other times, dissolved oxygen concentrations greater than 5 milligrams per liter shall be maintained.

R 323.1069. Temperature; general considerations.

Rule 69. (1) In all waters of the state, the points of temperature measurement normally shall be in the surface 1 meter; however, where turbulence, sinking plumes, discharge inertia or other phenomena upset the natural thermal distribution patterns of receiving waters, temperature measurements shall be required to identify the spatial characteristics of the thermal profile.

(2) Monthly maximum temperatures, based on the ninetieth percentile occurrence of natural water temperatures plus the increase allowed at the edge of the mixing zone and in part on long-term physiological needs of fish, may be exceeded for short periods when natural water temperatures exceed the ninetieth percentile occurrence. Temperature increases during these periods may be permitted by the commission, but in all cases shall not be greater than the natural water temperature plus the increase allowed at the edge of the mixing zone.

(3) Natural daily and seasonal temperature fluctuations of the receiving waters shall be preserved.

R 323.1070 Temperature of Great Lakes and connecting waters.

Rule 70. (1) The Great Lakes and connecting waters shall not receive a heat load which would warm the receiving water at the edge of the mixing zone more than 3 degrees Fahrenheit above the existing natural water temperature.

(2) The Great Lakes and connecting waters shall not receive a heat load which would warm the receiving water at the edge of the mixing zone to temperatures in degrees Fahrenheit higher than the following monthly maximum temperature:

(a) Lake Michigan north of a line due west from the city of Pentwater.

J	F	M	A	M	J	J	A	S	0	N	D
40	40	40	50	55	70	75	75	75	65	60	45

(b) Lake Michigan south of a line due west from the city of Pentwater.

J	F	M	A	M	J	J	A	S	0	N	D
45	45	45	55	60	70	80	80	80	65	60	50

(c) Lake Superior and the St. Marys river:

J	F	M	A	M	J	J	A	S	0	N	D
38	36	39	46	53	61	71	74	71	61	49	42

(d) Lake Huron north of a line due east from Tawas point:

J	F	M	A	M	J	J	A	S	0	N	D
40	40	40	50	60	70	75	80	75	65	55	45

(e) Lake Huron south of a line due east from Tawas point, except Saginaw bay.

J	F	M	A	M	J	J	A	S	O	N	D
40	40	40	55	60	75	80	80	80	65	55	45

(f) Lake Huron, Saginaw bay:

J	F	M	A	M	J	J	A	S	O	N	D
45	45	45	60	70	75	80	85	78	65	55	45

(g) St. Clair river:

J	F	M	A	M	J	J	A	S	O	N	D
40	40	40	50	60	70	75	80	75	65	55	50

(h) Lake St. Clair:

J	F	M	A	M	J	J	A	S	O	N	D
40	40	45	55	70	75	80	83	80	70	55	45

(i) Detroit river:

J	F	M	A	M	J	J	A	S	O	N	D
40	40	45	60	70	75	80	83	80	70	55	45

(j) Lake Erie:

J	F	M	A	M	J	J	A	S	O	N	D
45	45	45	60	70	75	80	85	80	70	60	50

R 323.1075 Temperature of rivers, streams, and impoundments.

Rule 75. (1) Rivers, streams, and impoundments naturally capable of supporting coldwater fish shall not receive a heat load which would do either of the following:

(a) Increase the temperature of the receiving waters at the edge of the mixing zone more than 2 degrees Fahrenheit above the existing natural water temperature.

(b) Increase the temperature of the receiving waters at the edge of the mixing zone to temperatures greater than the following monthly maximum temperatures:

J	F	M	A	M	J	J	A	S	O	N	D
38	38	43	54	65	68	68	68	63	56	48	40

(2) Rivers, streams, and impoundments naturally capable of supporting warmwater fish shall not receive a heat load which would warm the receiving water at the edge of the mixing zone more than 5 degrees Fahrenheit above the existing natural water temperature.

(3) Rivers, streams, and impoundments naturally capable of supporting warmwater fish shall not receive a heat load which would warm the receiving water at the edge of the mixing zone to temperatures greater than the following monthly maximum temperatures:

(a) For rivers, streams, and impoundments north of a line between Bay City, Midland, Alma and North Muskegon:

J	F	M	A	M	J	J	A	S	O	N	D
38	38	41	56	70	80	83	81	74	64	49	39

(b) For rivers, streams, and impoundments south of a line between Bay City, Midland, Alma, and North Muskegon, except the St. Joseph river:

J	F	M	A	M	J	J	A	S	O	N	D
41	40	50	63	76	84	85	85	79	68	55	43

(c) St. Joseph river:

J	F	M	A	M	J	J	A	S	O	N	D
50	50	55	65	75	85	85	85	85	70	60	50

(4) Non-trout rivers and streams that serve as principal migratory routes for anadromous salmonids shall not receive a heat load during periods of migration at such locations and in a manner which may adversely affect salmonid migration or raise the receiving water temperature at the edge of the mixing zone more than 5 degrees Fahrenheit above the existing natural water temperature.

R 323.1082 Mixing zones.

Rule 82. (1) A mixing zone to achieve a mixture of a point source discharge with the receiving waters shall be considered a region in which the response of organisms to water quality characteristics is time dependent. Exposure in mixing zones shall not cause an irreversible response which results in deleterious effects to populations of aquatic life or wildlife. As a minimum restriction, the final acute value for aquatic life shall not be exceeded in the mixing zone at any point inhabitable by these organisms, unless it can be demonstrated to the commission that a higher level is acceptable. The mixing zone shall not prevent the passage of fish or fish food organisms in a manner which would result in adverse impacts on their immediate or future populations. Watercourses or portions thereof which, without 1 or more point source discharge, would have no flow except during periods of surface runoff may be considered as a mixing zone for a point source discharge. The area of mixing zones should be minimized. To this end, devices for rapid mixing, dilution, and dispersion are encouraged where practicable.

(2) For toxic substances, not more than 25% of the receiving water design flow, as stated in R 323.1090, shall be utilized when determining effluent limitations for surface water discharges, unless it can be demonstrated to the commission that the use of a larger volume is acceptable. The commission shall not base a decision to grant more than 25% of the receiving water design flow for purposes of developing effluent limitations for discharges of toxic substances solely on the use of rapid mixing, dilution, or dispersion devices. However, where such a device is or may be employed, the commission may authorize the use of a design flow greater than 25% if the effluent limitations which correspond to such a design flow are shown, based upon a site-specific demonstration, to be consistent with Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws, and other applicable law.

(3) For substances not included in subrule (2) of this rule, the design flow, as stated in R 323.1090, shall be utilized when determining effluent limitations for surface water discharges if the provisions in subrule (1) of this rule are met, unless the commission determines that a more restrictive volume is necessary.

(4) For all substances, defined mixing zone boundaries may be established and shall be determined on a case-by-case basis.

(5) Mixing zones in the Great Lakes, their connecting waters, and inland lakes shall be determined on a case-by-case basis.

R 323.1090. Applicability of water quality standards.

Rule 90. (1) The water quality standards prescribed by these rules shall not apply within mixing zones, except for those standards prescribed in R 323.1082(1) and R 323.1050.

(2) Water quality standards prescribed by these rules are minimally acceptable water quality conditions. Water quality shall be equal to or better than such minimal water quality conditions not less than 95% of the time.

(3) Water quality standards shall apply at all flows equal to or exceeding the design flow. The design flow is equal to the most restrictive of the 12 monthly 95% exceedance flows, except where the commission determines that a more restrictive design flow is necessary or where the commission determines that seasonal design flows may be granted pursuant to R 323.1090(4). The 95% exceedance flow is the flow equal to or exceeded 95% of the time for the specified month.

(4) A maximum of 4 seasonal design flows may be granted when determining effluent limitations for a surface water discharge if it is determined by the commission that the use of such design flows will protect water quality and be consistent with the protection of the public health, safety, and welfare. The seasonal design flows shall be the most restrictive of the monthly 95% exceedance flow for the months in each season. Seasonal design flows shall not be granted for toxic substances which, on the basis of credible scientific evidence, may bioaccumulate in biota inhabiting or using the waters of the state unless, taking into account the receiving water characteristics the persistence and environmental fate characteristics of the substance or substances and the presence of other discharges of bioaccumulative toxic substances into the same receiving waters, the commission determines that the increased mass loading of the substance or substances resulting from granting seasonal design flows is consistent with Act No. 245 of the Public Acts of 1929, as amended, being §323.1 et seq. of the Michigan Compiled Laws, and other applicable law.

R 323.1092 Applicability of water quality standards to dredging or construction activities.

Rule 92. Unless the commission determines, after consideration of dilution and dispersion, that such activities result in unacceptable adverse impacts on designated uses, the water quality standards prescribed by these rules shall not apply to dredging or construction activities within the waters of the state where such activities occur or during the periods of time when the aftereffects of dredging or construction activities degrade water quality within such waters of the state, if the dredging operations or construction activities have been authorized by the United States army corps of engineers or the department of natural resources. The

water quality standards shall apply, however, in nonconfined waters of the state utilized for the disposal of spoil from dredging operations, except within spoil disposal sites specifically defined by the United States army corps of engineers or the department of natural resources.

R 323.1096 Determinations of compliance with water quality standards.

Rule 96. Analysis of the waters of the state to determine compliance with the water quality standards prescribed by these rules shall be made pursuant to procedures outlined in 40 C.F.R. §136, as amended by F.R. pp. 43234 to 43442 October 26, 1984, and F.R. pp. 690 to 697 January 4, 1985, or pursuant to other methods prescribed or approved by the commission and the United States environmental protection agency.

R 323.1097 Materials applications not subject to standards.

Rule 97. The application of materials for water resource management projects pursuant to state statutory provisions is not subject to the standards prescribed by these rules, but all projects shall be reviewed and approved by the commission before application.

R 323.1098 Antidegradation.

Rule 98. (1) This rule applies to waters of the state in which the existing water quality is better than the water quality standards prescribed by these rules or than needed to protect existing uses.

(2) These waters shall not be lowered in quality by action of the commission unless it is determined by the commission that such lowering will not do any of the following:

(a) Become injurious to the public health, safety, or welfare.

(b) Become injurious to domestic, commercial, industrial, agricultural, recreational, or other uses which are or may be made of such waters.

(c) Become injurious to the value or utility of riparian lands.

(d) Become injurious to livestock, wild animals, including birds, fish, and other aquatic animals, or plants, or their growth or propagation.

(e) Destroy or impair the value of game, fish, and wildlife.

(f) Be unreasonable and against the public interest in view of the existing conditions.

(3) All of the following waters are designated as protected waters:

(a) All Michigan waters of the Great Lakes, except as these waters may be affected by discharges to the connecting waters and tributaries.

(b) Trout streams south of a line between Bay City, Midland, Alma, and North Muskegon.

(c) Inland lakes.

(d) Reaches of country-scenic and wild-scenic rivers designated under Act No. 231 of the Public Acts of 1970, being §281.761 et seq. of the Michigan Compiled Laws.

(e) Scenic and recreational rivers designated under the wild and scenic rivers act of 1968, 16 U.S.C. §1721 et seq.

(4) In addition to the requirements of subrule (2) of this rule, the waters specified in subrule (3) of this rule shall not be lowered in quality unless, after opportunity for public hearing, it has been demonstrated by the applicant to the commission that a lowering in quality will not be unreasonable, is in the public interest in view of existing conditions, is necessary to accommodate important social or economic

development, and that there are no prudent and feasible alternatives to lowering water quality.

(5) Reaches of the following rivers have been designated pursuant to Act No. 231 of the Public Acts of 1970, being §281.761 et seq. of the Michigan Compiled Laws:

(a) Jordan river - October, 1972, natural river plan.
(b) Betsie river - July, 1973, natural river plan.
(c) Rogue river - July, 1973, natural river plan.
(d) White river - May, 1975, natural river plan.
(e) Boardman river - December, 1975, natural river plan.
(f) Huron river - May, 1977, natural river plan.
(g) Pere Marquette river - July, 1978, natural river plan
(h) Flat river - October, 1979, natural river plan.
(i) Rifle river - May, 1980, natural river plan.
(j) Kalamazoo river - June, 1981, natural river plan.
(k) Pigeon river - June, 1982, natural river plan.

Designated reaches of these rivers are provided in the department of natural resources natural river plan for each respective river.

(6) Reaches of the AuSable river - November, 1984, have been designated pursuant to the wild and scenic rivers act of 1968, 16 U.S.C. §1721 et seq.

(7) Michigan's waters of the Great Lakes are of special significance and are designated as outstanding state resource waters. In addition to the protection specified under subrules (2), (3) and (4) of this rule, mixing zones shall not be used for new or increased discharges to the Great Lakes of toxic substances, as defined by R 323.1057(2)(b), which would result in a lowering of water quality. However, the commission may grant a mixing zone for certain toxic substances on a case-by-case basis, taking into account credible scientific evidence, including persistence and environmental fate characteristics of the substances. Mixing zones for existing discharges of these toxic substances to the Great Lakes and for all discharges of these toxic substances to the connecting waters shall be minimized.

(8) Before authorizing a new or increased discharge of wastewater directly to the Great Lakes or connecting waters, the commission shall provide, in addition to the public notice required by commission rules, supplemental notice of its intent to authorize such discharge, of its proposed determination with respect to the applicable factors set forth in subrule (4) of this rule, and the proposed national pollutant discharge elimination system permit terms and conditions, to the administrator of the United States environmental protection agency, the director of the state or provincial water pollution control agency for all states or provinces which border the lake or connecting waters which receive the new or increased discharge, the United States fish and wildlife service, and the international joint commission. The commission shall allow not less than 30 days for comments from the recipients of the supplemental notice and shall carefully consider all comments received in making its determination.

(9) Wild rivers designated under the wild and scenic rivers act of 1968, 16 U.S.C. §1721 et seq., rivers flowing into, through, or out of national parks or national lakeshores, and wilderness rivers designated under Act No. 231 of the Public Acts of 1970, being §281.761 et seq. of the Michigan Compiled Laws, shall not be lowered in quality. Reaches of

the Two Hearted river - December, 1973, natural river plan - are designated under Act No. 231 of the Public Acts of 1970 as a wilderness river.

R 323.1099 Waters which do not meet standards.

Rule 99. Waters of the state which do not meet the water quality standards prescribed by these rules shall be improved to meet those standards. Where the water quality of certain waters of the state does not meet the water quality standards as a result of natural causes or conditions, further reduction of water quality is prohibited.

R 323.1100 Designated uses.

Rule 100. (1) As a minimum, all waters of the state are designated for, and shall be protected for, all of the following uses:

(a) Agriculture.
(b) Navigation.
(c) Industrial water supply.
(d) Public water supply at the point of water intake.
(e) Warmwater fish.
(f) Other indigenous aquatic life and wildlife.
(g) Partial body contact recreation.

(2) All waters of the state are designated for, and shall be protected for, total body contact recreation from May 1 to October 31 in accordance with R 323.1062. The commission will annually publish a list of those waters of the state located immediately downstream of municipal sewage system discharges where total or partial body contact recreation is contrary to prudent public health practices.

(3) All inland lakes identified in the publication entitled "Coldwater Lakes of Michigan," as published in 1976 by the department of natural resources, are designated for, and shall be protected for, coldwater fish.

(4) All Great Lakes and their connecting waters, except the entire Keweenaw waterway, including Portage lake, Houghton county, and Lake St. Clair, are designated for, and shall be protected for, coldwater fish.

(5) All lakes designated as trout lakes by the natural resources commission under the authority of Act No. 165 of the Public Acts of 1929, as amended, being §301.1 et seq. of the Michigan Compiled Laws, are designated for, and shall be protected for, coldwater fish.

(6) All waters of the state designated as trout streams by the director of the department pursuant to section 8 of Act No. 165 of the Public Acts of 1929, as amended, being §301.8 et seq. of the Michigan Compiled Laws, shall be protected for coldwater fish.

(7) All waters of the state which are designated by the Michigan department of public health as existing or proposed for use as public water supply sources are protected for such use at the point of water intake and in such contiguous areas as the commission may determine necessary for assured protection.

(8) Water quality of all waters of the state serving as migratory routes for anadromous salmonids shall be protected as necessary to assure that migration is not adversely affected.

(9) Discharges to wetlands, as defined by Act No. 203 of the Public Acts of 1979, being §281.701 of the Michigan Compiled Laws, that result in quality less than that prescribed by these rules may be permitted after a use attainability analysis shows that designated uses are not and cannot be attained and shows that attainable uses will be protected.

(10) After completion of a comprehensive plan developed pursuant to R 323.1064(3), upon petition by a municipality or other person, and in conformance with the requirements of 40 C.F.R. §131.10 (1983), the commission may determine that attainment of the dissolved oxygen standards of R 323.1064(1) is not feasible and designate, by amendment to this rule, a limited warmwater fishery use subcategory of the warmwater fishery use or a limited cold water fishery use subcategory of the cold water fishery use. For waters so designated, the dissolved oxygen standards specified in R 323.1064(2) and all other applicable standards of these rules apply. For waters so designated, the dissolved oxygen standards specified in R 323.1064(1) do not apply. Not less than sixty days before filing a petition under this subrule by a municipality or other person, a petitioner shall provide written notice to the executive secretary of the water resources commission and the clerk of the municipalities in which the affected waters are located of its intent to file such petition.

R 323.1105. Multiple designated uses.

Rule 105. When a particular portion of the waters of the state is designated for more than 1 use, the most restrictive water quality standards for one or more of those designated uses shall apply to that portion.

R 323.1116 Availability of documents.

Rule 116. Documents referenced in R 323.1057, R 323.1058, R 323.1065, R 323.1096, and R 323.1100 may be obtained at current costs as follows:

(a) "EPA Priority Pollutants and Hazardous Substances," 40 C.F.R. §122.21, appendix D (1983); copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost, or from the Office of Water Enforcement, United States Environmental Protection Agency, Washington, D.C. 20460, at no cost.

(b) "1984 Michigan Critical Materials Register;" copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(c) "Guidelines Establishing Test Procedures for Analysis of Pollutants," 40 C.F.R. §136 as amended by F.R. pp 43234 to 43442, October 26, 1984, and F.R. pp. 690 to 697, January 4, 1985; copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(d) "Designated Trout Lakes," a publication of the department of natural resources; copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(e) "Coldwater Lakes of Michigan," August, 1976, a publication of the department of natural resources, fisheries division, copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(f) "Designated Trout Streams for the State of Michigan," April, 1975, a publication of the department of natural resources; copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(g) "Standards for Protection Against Radiation," 10 C.F.R. §20, January 1, 1985. Copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

(h) "Designation of uses," 40 C.F.R. §131.10, as published in November 8, 1983 F.R. pp. 51406 and 51407; copies may be obtained from the Department of Natural Resources, P.O. Box 30028, Lansing, Michigan 48909, at no cost.

SELECTED INNOVATIVE AND ALTERNATIVE TECHNOLOGY DEVELOPMENTS

by

John J. Convery, James F. Kreissl,
Dr. James A. Heidman, Richard C. Brenner,
And Dr. Richard A. Dobbs
Wastewater Research Division
Water Engineering Research Laboratory
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

And

Richard Field
Land Pollution Control Division
Hazardous Waste Engineering Research Laboratory
U.S. Environmental Protection Agency
Edison, New Jersey 08837

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

Prepared for presentation at:

Eleventh United States/Japan Conference
on Sewage Treatment Technology
Tokyo, Japan

October 12-14, 1987

INTRODUCTION

The 1986 Needs Survey[1] has identified a capital cost funding requirement to build publicly owned municipal wastewater treatment facilities of $76.2 billion to satisfy all categories of documented need for the design year (2005) population of 244 million people. The number of facilities will increase from 15,438 to 16,980. The percent of population treated will increase from 73% to 87%. Seventy-three percent of the 1,542 new facilities to be constructed will be minor facilities with flows of less than 1.0 MGD (0.043 m^3s). Seventy-seven percent of all existing facilities are minor facilities. Currently, 2,000 existing minor facilities and 800 existing major facilities require construction worth $15.5 billion dollars to be brought into compliance with their current effluent permit limits.

Several of the technologies discussed in this paper are particularly well suited for upgrading the performance of currently overloaded small treatment plants. The trickling filter/solids contactor process can be used to economically upgrade the performance of many of the 1,700 existing trickling filter plants. The Two-Zone wastewater treatment process evaluation results at Norristown, Pennsylvania illustrate an approach to upgrading existing activated sludge treatment facilities which are organically overloaded. Vacuum assisted sludge drying bed technology offers a capability to retrofit existing facilities which will permit production of a liftable sludge cake within 24 hours. This technology will aid small treatment facilities with inadequate sludge drying bed capacity; particularly during the cold, wet winter months. The rubber tide gate technology will prevent excess inflows and hydraulic overloading and preserve usable treatment plant capacity.

Twenty-nine percent of the total facilities in the year 2005 will provide treatment levels better than secondary treatment. Many will require the application of control technology which can achieve small residual concentrations of nitrogen and phosphorus such as the Rim-Nut Process.

TRICKLING FILTER/SOLIDS CONTACT PROCESS

The information presented on the trickling filter/solids contact process (TF/SC) is summarized from references (2) and (3) and is based upon the performance of four full-scale TF/SC facilities which were evaluated for a total of 29 weeks. Plant locations included Oconto Falls, Wisconsin; Tolleson, Arizona; Medford, Oregon and Chilton, Wisconsin. Operating records from Corvallis, Oregon and Norco, California were also analyzed.

The TF/SC was first successfully demonstrated in Corvallis, Oregon in 1979. A flow schematic of one mode of the process is shown in Figure 1. The SC feature provides a short aerobic contact period of one hour or less

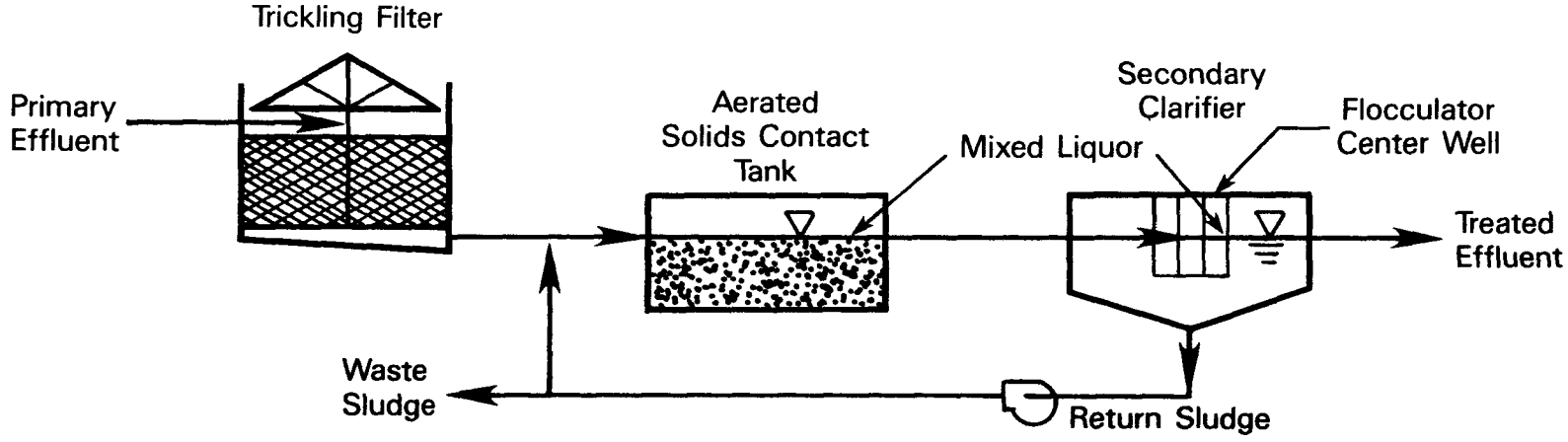


Figure 1. Trickling Filter/Solids Contact Process (TF/SC)

(based upon total flow) between the TF effluent and recycled underflow solids from the secondary clarifier to promote solids capture and produce a final effluent with a low suspended solids concentration.

The TF/SC process is a biological and physical process that includes (1) a TF, (2) an aerobic solids contact basin, (3) a flocculation zone, and (4) a secondary clarifier. Two operating features are particularly important: solids must be maintained in an aerobic flocculant state; and solids are recycled from the secondary clarifier to combine with TF effluent as a mixed liquor. Other modes of operation for the TF/SC process include: return sludge reaeration either[1] an alternative to or [2] in addition to the aeration of TF effluent and sludge recycle.

The primary function of the first element in the TF/SC process, the TF, is to reduce the majority of the soluble BOD in the wastewater. The aerobic solids contact period is then used to provide contact between finely divided solids in the TF effluent and recycled biological solids and to provide additional soluble BOD removal, if necessary. The solids retention time (SRT) of the SC tank is less than two days. The contact opportunity provides for initial flocculation of dispersed solids into floc. The length of the aerobic solids contact period is governed by the requirements for particulate and soluble BOD removal. Figure 2 illustrates the soluble BOD_5 profile along the aerated solids contact tank at Medford, Oregon. The third element in the TF/SC process is the flocculation period. Flocculation, which is initiated in the contact tank, continues in the clarifier, preferably in a mildly stirred environment of a center well. The flocculation step promotes clear effluent and growth of large, settleable floc that are removed in secondary clarification.

Additional characteristics that distinguish TF/SC from other processes are that return sludge solids are mixed with TF effluent rather than primary effluent, and the aerated solids contact tank is not designed to nitrify, although nitrification may occur in the TF.

PERFORMANCE OF EXISTING FACILITIES

The Oconto Falls and Tolleson plants were originally rock TF plants. Tolleson was originally a two-stage rock TF plant with intermediate clarification treating industrial and residential flows. The first-stage rock filter was replaced with a 20 ft. (6.1 m) deep plastic media filter with intermediate clarification and the remaining rock filter was followed by aerated solids contact and a flocculator clarifier. The Medford plant was originally an activated sludge (AS) plant that was converted to a coupled TF/AS plant. This plant presently operates in the TF/SC mode since its flows and loads are significantly below design levels.

Tables 1 and 2 summarize the design and operating performance results for four of these facilities including the Corvallis plant. The effluent quality was excellent with effluent BOD_5 of <21 mg/L and TSS of <13 mg/L.

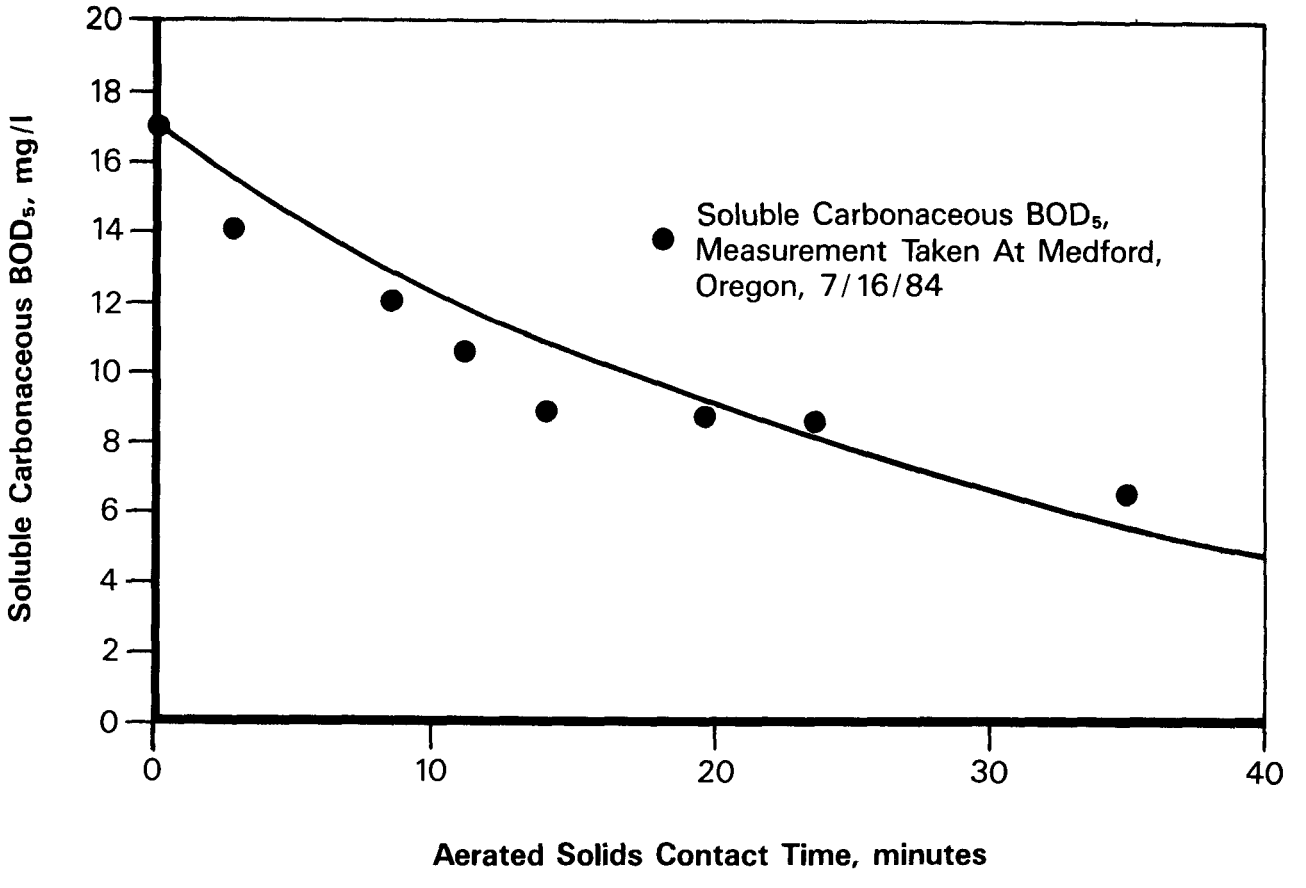


Figure 2. Soluble BOD_5 Profile Along The Aerated Solids Contact Tank At Medford, Oregon

TABLE 1. SUMMARY OF DESIGN INFORMATION FOR TF/SC FACILITIES

PARAMETER	TOLLESON	OCONTO FALLS	CORVALLIS	MEDFORD
AVERAGE (D.W.) DESIGN FLOW, MGD	8.3	0.38	9.7	18.0
DESIGN LOADING, 1000 lb/day				
BOD (mg/L)	24	0.67	10.9	35.0
SS (mg/L)	21.6	0.79	11.5	38.0
PRIMARY CLARIFIER OVERFLOW RATE, (gpd/sq. ft.)	970	370	980	1030
TRICKLING FILTER				
MEDIA TYPE	PLASTIC/ROCK	ROCK	ROCK	PLASTIC
BOD LOADING, (lb/day/ 1000 cu. ft.)	55/9.1	35	24	115
RETURN SLUDGE AERATION TIME @33% RETURN, (MINS)	-	-	9	-
AERATED SOLIDS CONTACT TIME (TOTAL FLOW, MINS)	9	8	2	39*
FLOCCULATOR CENTER WELL DETENTION TIME, (MINS)	25	38	25	5
SECONDARY CLARIFIER OVERFLOW RATE (gpd/sq. ft.)	440	300	470	480

1 MGD = 0.0438 m^3/s
1 lb/day = 0.45 kg/day
1 gpd/ft^2 = 0.041 $m^3/m^2 \cdot d$
1 lb/day/1000 cu. ft. = 16 $g/m^3 \cdot d$

* AT CURRENT FLOW OF 8.8 MGD PLUS RECYCLE

TABLE 2. SUMMARY OF ANNUAL AVERAGE PERFORMANCE AT OPERATING TF/SC FACILITIES

PARAMETER	TOLLESON	OCONTO FALLS	CORVALLIS	MEDFORD
FLOW, MGD	6.1	0.36	10.5	8.9
INFLUENT CHARACTERISTICS				
BOD, mg/L	277	146	108	157
SS, mg/L	224	118	154	138
TEMPERATURE, °C	-	13	17	19
PRIMARY EFFLUENT				
BOD, mg/L	173	-	70	81
SS, mg/L	121	-	66	34
TRICKLING FILTER EFFLUENT				
BOD, mg/L	22.8*	-	30	66
SS, mg/L	23.6*	-	59	71
RETURN SLUDGE SS, g/L	-	-	11.3	-
MLSS, mg/L	1040	-	3130	1620
SECONDARY EFFLUENT				
BOD, mg/L	7	21	7	19
SS, mg/L	9	13	9	8

* INTERMEDIATE CLARIFIER EFFLUENT

RESULTS OF SPECIAL TF/SC STUDIES

Special studies were conducted during the field evaluation program to determine the following:

1. Assess the influence of cosettling waste secondary solids with raw sewage solids on primary sedimentation tank performance.

2. Assess soluble BOD removal kinetics with TF depth.

3. Assess the effect of TF loading on TF/SC performance.

4. Assess the effect of media type on aerated solids contact tank performance.

5. Assess the effect of aerated solids contact tank operating parameters on TF/SC performance.

6. Assess soluble BOD removal in the aerated solids contact tank.

7. Assess the effect of aeration rate on TF/SC performance.

8. Assess the effect of secondary clarifier overflow rate on final effluent quality.

9. Assess the effect of coagulant addition for phosphorus removal on TF/SC performance.

Based upon the special study results and a review of historical operating records, the following conclusions could be reached.

1. Cosettling - Primary treatment suspended solids (SS) removal averaged between 53 and 62 percent at three TF/SC plants that cosettle and 74 percent at Medford, which did not cosettle. The Medford results were exceptional. Primary sludge concentrations were 3.7 and 5.3 percent at the two plants practicing cosettling where samples could be obtained for analysis. Concentrations of 5 to 7 percent are common without cosettling.

2. TF Soluble BOD Removal - The Velz equation successfully modeled soluble carbonaceous BOD_5 removal with TF depth at Tolleson.

3. TF Loading - In the range of average TF BOD_5 loadings studied under this project (5.8 to 29 pounds per day per 1,000 cu. ft.)(93 to 464 $g/m^3 \cdot d$), there was a statistically significant but weak correlation between BOD_5 loading and final effluent SS. The correlation coefficient (R) was in the range of (R = .29 - .46) for the Corvallis and Tolleson data. Effluent SS were more highly correlated (R = .39 - .76) with the effluent SS concentration. At Corvallis the relation was (TF effluent SS) = 26.4 + 0.5 (primary effluent SS). The results show the need for reliable primary treatment.

4. Media Type - Microscopic examination of the TF effluent indicate that the floc from the rock media was more compact than the floc from plastic media filters.

5. Solids Contact Operating Parameters

a. SRT - Correlations between SRT in the aerated solids contact tank and final effluent SS were not statistically significant at Corvallis and Tolleson. A statistically significant but weak correlation was observed at Medford.

b. Mixed Liquor Suspended Solids - Mixed liquor suspended solids (MLSS) concentrations between 900 and 2,300 mg/L at Medford and Tolleson did not affect final effluent SS significantly and only produced an average increase of about 2 mg/L at Corvallis where the MLSS concentration varied from 1500 to 7,000 mg/L. The insensitivity to mixed liquor level means simplification of operation, since less attention needs to be given to sludge inventory management.

c. Sludge Volume Index - Sludge volume index values varied from 60 to 130 mL/g at Medford and increasing values were correlated with reduced final effluent SS. No correlation was observed at Tolleson or Corvallis. Corvallis and Tolleson had large flocculator center wells whereas those at Medford were much smaller.

6. Solids Flocculation - Field test results at Medford suggest that the majority of flocculation in the aerated solids contact channel occurred within the first 12 minutes of aerated solids contact time in a channel that had a total hydraulic retention time of 39 minutes.

7. Contact Tank Soluble BOD Removal - Although the primary function of the contact tank is to flocculate SS and particulate BOD, a significant fraction of the filter effluent soluble BOD can be removed. The Medford contact tank removed an average of

75 percent of the residual soluble carbonaceous BOD from the filter in 39 minutes of contact time. First-order reaction kinetics adequately described the removal as follows:

$$\ln \left(\frac{C}{C_o}\right) = [-K_{20}\ \theta^{(T-20)}\ X_V]t \qquad (1)$$

where: C_o = mixed liquor soluble carbonaceous BOD at the channel beginning, mg/L

C = soluble carbonaceous BOD after time t, mg/L

K_{20} = first-order reaction rate coefficient at 20°C, L/mg • min.

θ = temperature correction coefficient (assume $\theta = 1.035$)

T = wastewater temperature, °C

X_V = MLVSS, mg/L and

t = contact time, minutes, based upon total flow in the channel.

Figure 3 shows an example plot of the data from Medford, Oregon. The slope of the line of best fit which passes through the origin for a plot of $\ln (C/C_o)$ versus time is equal to the bracketed term in Equation 1. First-order removal rate coefficients (K_{20}) ranged from 2.0×10^{-5} to 3.3×10^{-5} L/mg. min.

8. Secondary Clarifier Overflow - Secondary clarifiers that included inboard launders, high sidewater depths, and flocculator center wells were insensitive up to 1300 gpd/sq. ft. (53 $m^3m^2 \cdot d$) at Corvallis and up to 700 gpd/sq. ft. (29 $m^3/m^2 \cdot d$) at Tolleson. These were the maximum overflow rates at these respective plants.

9. Coagulant Addition - Ferric chloride addition in the aerated solids contact tank for phosphorus removal at Oconto Falls did not adversely affect TF/SC operation.

Most of the operating trickling filter plants use rock media. To test TF/SC performance with rock filters at high organic loadings, U.S. EPA sponsored full-scale studies at the Morro Bay-Cayucos TF/SC plant. The studies also included an assessment of trickling filter performance with flocculator-clarifiers and reaction rate coefficients for soluble carbonaceous BOD_5 ($SCBOD_5$) removal in rock trickling filters.

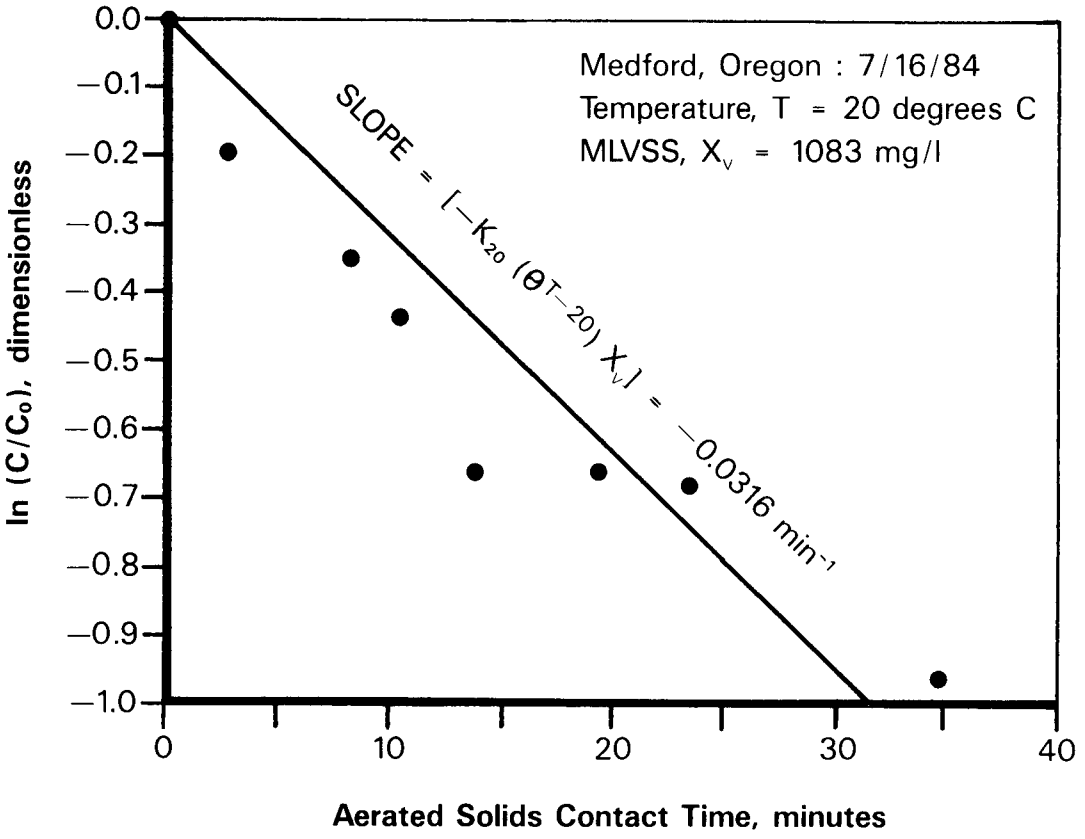


Figure 3. Linear Plot of Soluble BOD Profile For First-Order Equation at Medford, Oregon

The studies included nine weeks of field investigations at the Morro Bay-Cayucos facility in Morro Bay, California. The field investigations data were supplemented with operating records from the Morro Bay-Cayucos plant and from plants in Coeur d'Alene, Idaho; Corvallis, Oregon and Oconto Falls, Wisconsin.

The Morro Bay-Cayucos studies showed that TF/SC could produce high quality effluent with rock filters even up to loadings as high as 960 60 lb/day/1000 cu. ft. (g/m^3-d). The results indicated that if the trickling filter could operate satisfactorily at this high load, the TF/SC process would produce its typically high quality effluent.

These loadings were significantly higher than the previously tested 25 lb/day/1000 cu. ft. (400 g/m^3-d) loadings at the Corvallis TF/SC plant. They provide a wide margin of potential increased capacity at existing rock filter plants and indicate that these plants can be expanded merely by adding solids contact features without constructing new trickling filters. Each plant should be evaluated individually, since all rock filters may not operate effectively at such high loadings. The possibility of such economical expansion is particularly important in view of the nearly 1,700 rock filter plants in the U.S.A. that may need upgrading by the year 2005.

Work at the Morro Bay-Cayucos plant and long-term data at the Coeur d'Alene plant showed that effluent quality from rock trickling filter plants could be improved significantly simply by replacing conventional secondary clarifiers with flocculator-clarifiers. The flocculator-clarifier at Coeur d'Alene reduced average effluent TSS from 25 mg/L to 16 mg/L.

TWO-ZONE WASTE TREATMENT PROCESS

The information presented on the Two-Zone Process, which is a product of the Canadian Liquid Air Company, is summarized from the Project Summary and Project Final Report [4] on the U.S. EPA 0.5 MGD (0.022 m^3/s) plant evaluation in the Borough of Norristown, Pennsylvania. The process shown schematically in Figure 4 can be retrofitted to upgrade existing aerators or clarifiers at volumetric requirements of 40-50% less than conventional secondary treatment. Details of a 30 ft. (9.15 m) X 30 ft. (9.15 m) section of the aeration tank used as a reactor/clarifier are shown in Figure 5. The lower portion of the tank serves as a biological reactor section and the upper section of the tank serves a clarification function. Recycled biomass at flow rates of 3 to 6 times the influent flow is oxygenated with pure oxygen in an external transfer device and blended with the influent wastewater in a baffled inlet chamber. The combined flow then passes upward through the sludge blanket to the clarification zone and effluent weirs. A sludge scraper mechanism moves the heavy solids across the tank floor and provides for the removal of scum at the surface of the clarification zone.

Distribution headers were provided for both the primary effluent and recycled sludge to achieve uniform flow distribution. The cross-sectional area at the bottom of the inlet chamber was increased to ensure the release of gases. The opening at the bottom of the rubber baffle wall served as a distribution orifice across the width of the tank for the blended flows. At maximum flow the orifice velocity was 3.5 in/sec. (0.09 m/s). The discharge rate through this orifice into the reactor/clarifier was the controlling hydraulic parameter for sizing the sludge recycle pump. A maximum discharge rate of 100 gal/min/ft (0.021 $m^3/s/m$) of tank width was set by CLA based upon its experience. Maximum sludge recycle pump capacity was set at 3000 gal/min (19 m^3/s).

The overflow rate of 960 gal/day/ft^2 (39.4 $m^3/m^2 \cdot d$) for the clarification zone limited the maximum influent flow rate to 0.86 MGD (0.038 m^3/s) with a minimum recycle ratio of 4 to 1. The nominal average flow rate of 0.5 MGD (0.022 m^3/s) corresponds to an overflow rate of 555 gal/day/ft^2 (22.8 $m^3/m^2 \cdot d$).

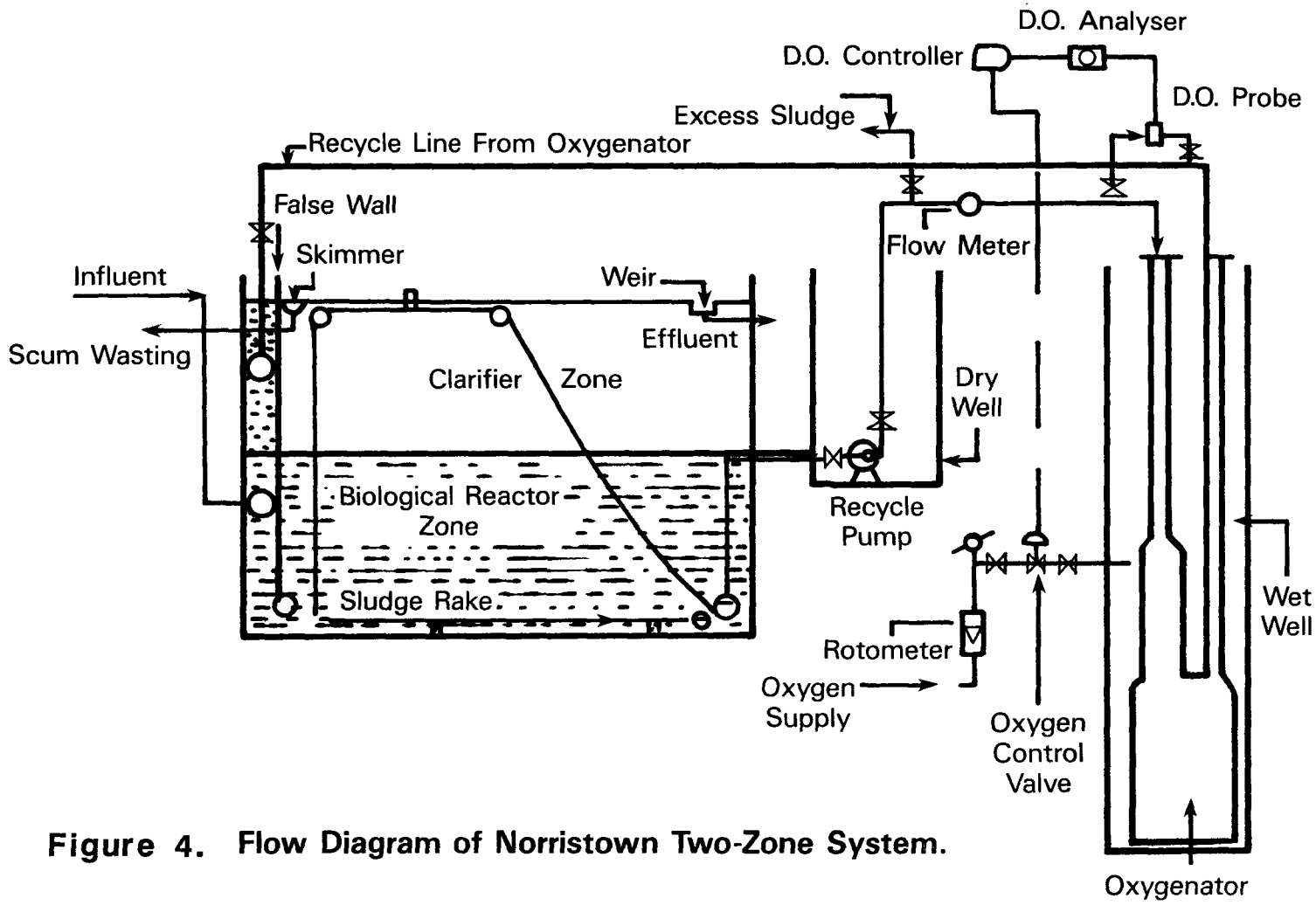
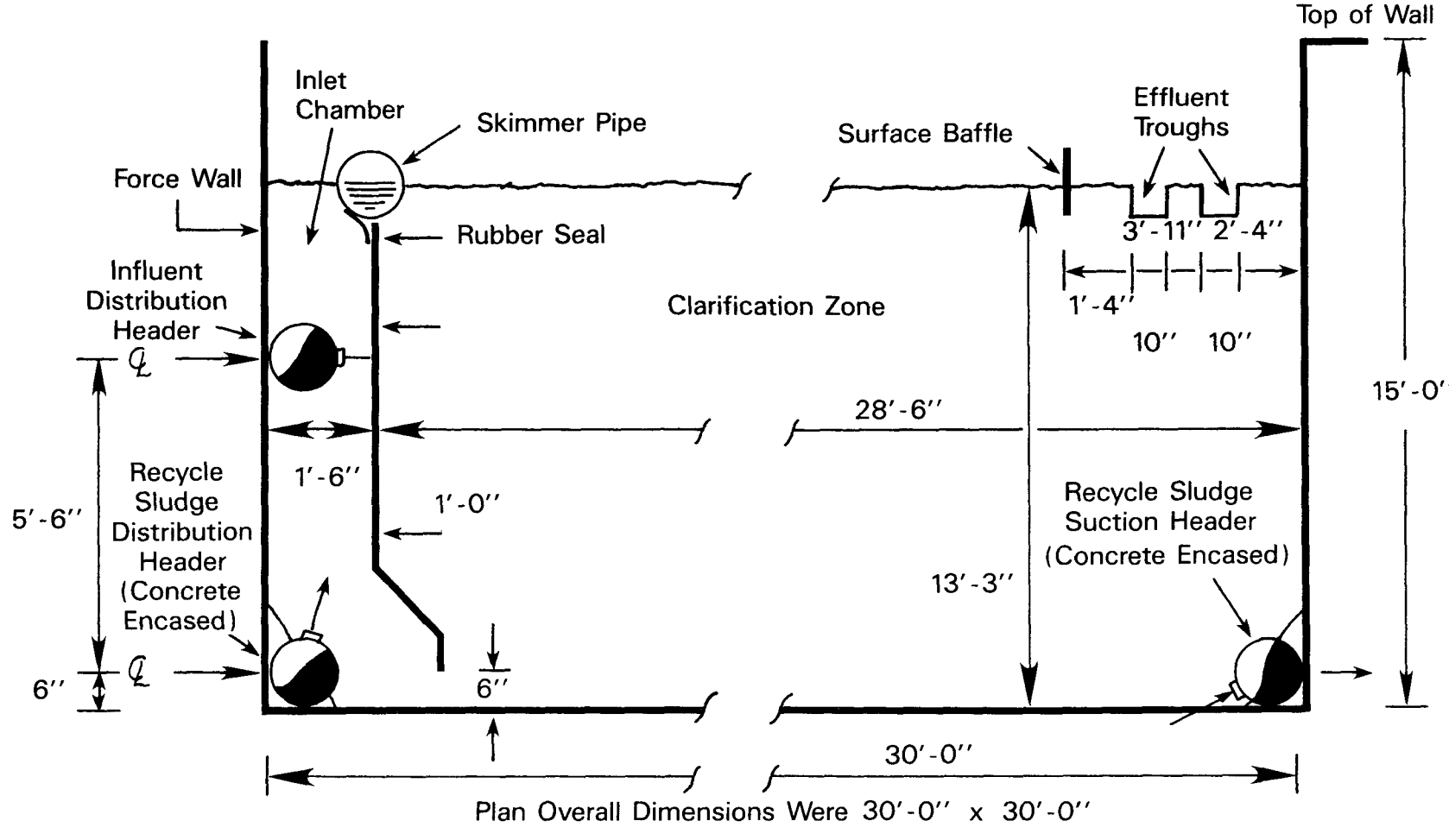


Figure 4. Flow Diagram of Norristown Two-Zone System.



NOTE: 1 FT = 0.305m

Figure 5. Longitudinal Section View of Norristown Two-Zone Reactor/Clarifier

A maximum daily total BOD_5 of 385 mg/L and an assumed oxygen consumption rate of 0.8 lb O_2/lb TBOD applied established the maximum oxygenation rate at 100 lbs/hr (for peak flow conditions). The minimum oxygenation rate of 21 lbs/hr (was based upon a minimum flow rate of 0.3 MGD)(0.013 m^3/s), an influent $TBOD_5$ of 150 mg/L and an oxygen consumption rate of 1.35 lb O_2/lb TBOD applied. The DO concentration of the recycled sludge before reoxygenation was used to control the oxygen feed rate to the recycled sludge.

OXYGEN TRANSFER DEVICE

Liquid oxygen was trucked in and stored on-site to feed pure gaseous oxygen to the sludge recycle stream. The oxygen transfer device used was an oxygenator made by Dorr-Oliver. The main transfer chamber of the oxygenation unit was located in a pit 50 ft (15 m) below the surface level of the Two-Zone reactor where the static pressure aided oxygen transfer. A limited examination of the oxygen transfer characteristics of the oxygenator indicated that an oxygen transfer objective of 90% was feasible.

A characteristic of oxygenators, and other devices that generate supersaturation quantities of oxygen with respect to atmospheric pressure, is that the excess oxygen tends to come out of solution when the pressure is reduced. The release of supersaturated oxygen can be particularly rapid if the stream being oxygenated contains solids which serve as sites for nucleation. Facilities using devices that produce supersaturation should, therefore, be designed to return the oxygenated stream at or below the bottom level of the reactor to maintain maximum pressure.

At Norristown, it was necessary to bring the sludge recirculation line over the end wall of the reactor to avoid cutting through two aeration basin walls. This resulted in release of oxygen from solution and reduced the overall oxygen transfer efficiency, from 85% that was achieved by the oxygenator, to 68% to 80% at a recycle stream oxygen concentration of 20 to 50 mg/L.

A total of six runs were completed under the conditions shown in Table 3.

The performance of the process was excellent with TBOD removals of 82-87% and effluent concentrations of 9-28 mg/L. Soluble BOD removals averaged 86-96% and effluent concentrations of 2-5 mg/L. Suspended solids removals averaged 85-93% with effluent concentrations of 11 to 24 mg/L, with the exception of Run 2 which averaged 34 mg/L and was the Run with the highest hydraulic loading. Effluent DO ranged from 3.2 to 4.9 mg/L.

TABLE 3. TEST CONDITIONS FOR TWO-ZONE EVALUATION

RUN NO.	DAYS	AVERAGE FLOW RATE MGD	AVERAGE RECYCLE RATE MGD	RECYCLE RATIO	TEMPERATURE (°C)
1	22	.51	2.72	5.3	12.9
2	12	.66	2.70	4.1	13.7
3	8	.31	1.89	6.1	21.6
4	26	.36	2.74	7.6	24.5
5	23	.45	3.68	8.2	20.7
6	21	.50	2.99	6.0	18.5

1MGD = 0.043 m^3/s

As shown in Table 4 wastewater detention times varied from 3.3 to 7.0 hours based upon the total reactor/clarifier volume of 12,150 ft^3 (344 m^3) and system influent flow excluding sludge recycle. The relative proportions of detention time in the reactor and clarifier zones varied from run to run and can be estimated by dividing the sludge blanket depth by 13.5 ft (4.1 m) the tank sidewater depth.

The correlation of F/M loading with BOD_5 and COD removals is shown in Figure 6. The F/M loading is the single-pass load in the reactor zone including the soluble BOD_5 in the recycle sludge. This method of calculation increases the loading by about 25% compared to the loading based upon influent BOD_5 only.

The net sludge wastage ranged from 0.95 to 2.01 lbs TSS/lb TBOD removed and averaged 1.33 which is about twice as high as comparable figures for activated sludge systems. These data indicate that the Two-Zone Process is primarily a high rate, bioflocculation process with little utilization of the nonsoluble BOD_5. This is not unexpected given the relatively short solids retention time values for the system of 1.2 to 2.6 days except for Run 4 at an SRT of 3.7 days. There are economic trade-offs between the sludge management costs associated with plant operations with high sludge wastage rates versus the higher oxygen consumption costs associated with the operations to achieve lower sludge production rates. Additional evaluation efforts would have to be conducted to define these trade-offs.

TABLE 4. OPERATING DATA SUMMARY FOR TWO-ZONE EVALUATION

15

PARAMETER	RUN NO. 1	2	3	4	5	6
REACTOR ZONE MLVSS, mg/L	2460	2687	2742	4215	3454	2334
SLUDGE BLANKET DEPTH, ft.	7.6	7.7	2.4	4.7	6.6	5.6
CLARIFIER OVERFLOW RATE, gal/day/ft^2	604	789	369	431	539	594
CLARIFIER SOLIDS LOADING lb/day/ft^2	100	114	78	165	172	108
SLUDGE VOLUME INDEX, mL/g	58	92	57	58	51	54
INITIAL SETTLING VELOCITY, ft/hr	-	-	32	16	17	21
F/M LOADING lb TBOD/day/lb MLVSS	0.64	0.77	0.41	0.33	0.40	0.95
WASTEWATER DETENTION TIME,hrs	4.3	3.3	7.0	6.0	4.8	4.4
VOLUMETRIC ORGANIC LOADING, lbs TBOD/day/1000 ft^3	98	135	102	84	87	133
SRT, DAYS	2.1	1.5	1.5	3.7	2.6	1.2
NET SLUDGE WASTAGE, lbs TSS/lb TBOD REMOVAL	0.95	1.07	2.01	1.27	1.26	1.37

1 ft = 0.305 m
1 gpd/ft^2 = 0.041 m^3/day/m^2
1 lb/day/ft^2 = 4.88 kg/day/m^2
1 lb/day/1000 ft^3 = 0.016 kg/day/m^3

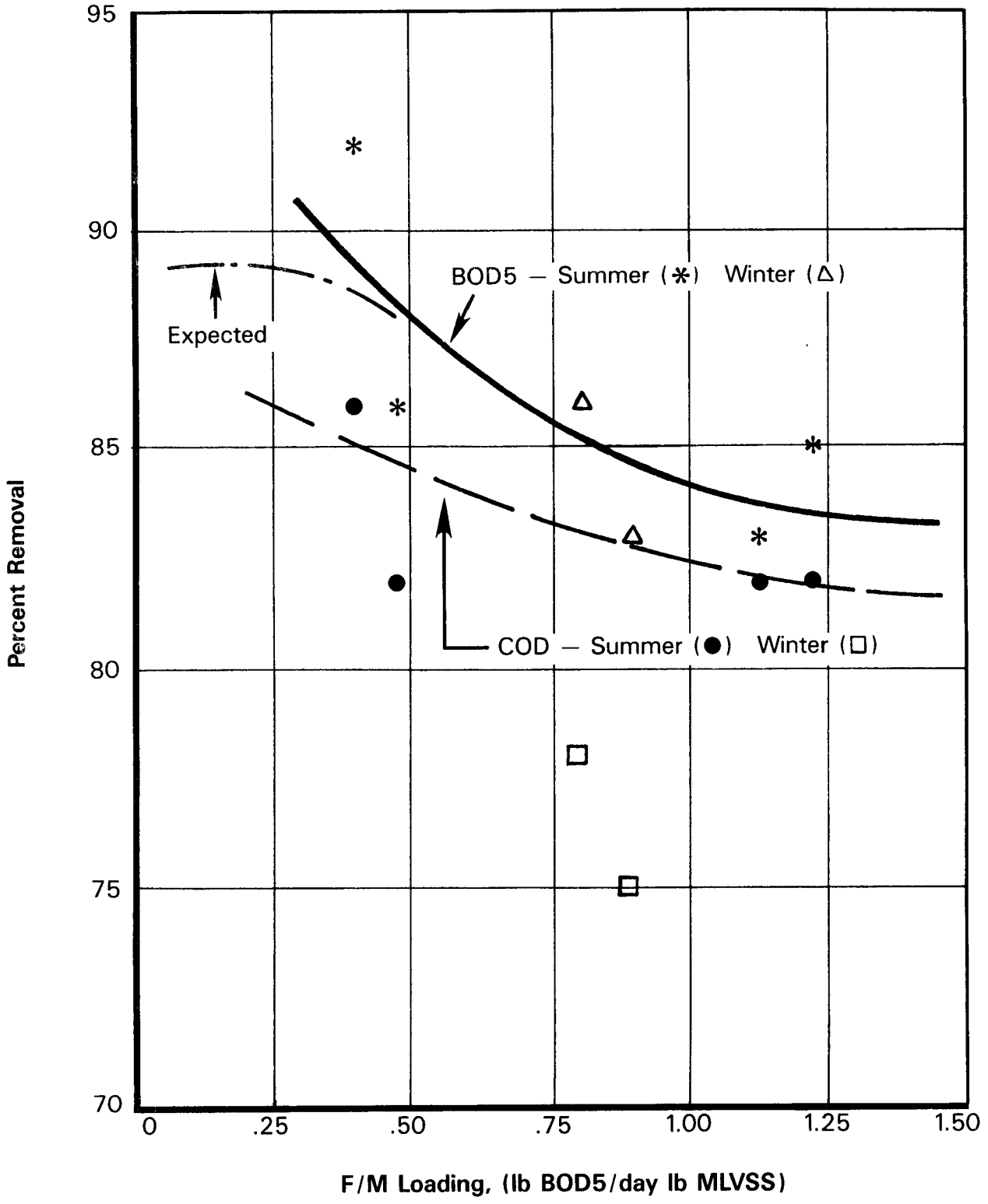


Figure 6. BOD5 and COD Removals as a Function of F/M Loading.

General design application guidelines for the Two-Zone Process include:

- Flow and load equalization should be provided whenever the peak-to-average hydraulic loading ratio and carbonaceous loading ratio, including recycle flow streams, exceeds 2.4 to 2.5 respectively unless the process is being used as a first-stage application.
- Influent flows to the Two-Zone Process should be pretreated with fine screening and primary treatment to avoid fouling of the oxygenator.
- Strong wastewaters with TBOD > 200 mg/L and nitrification within the reactor zone should be avoided due to the high oxygen demand and potential for undesirable flotation.
- Provisions should be made to chlorinate the waste sludges in the event of a Nocardial bloom.
- Provisions to backflush the oxygenator and equalize this return flow are recommended.
- Sludge holding and digestion facilities requiring air should be designed to cope with the higher oxygen demands of the Two-Zone Process sludges.
- A technically skilled operating staff familiar with biological treatment and operation of pure oxygen systems is required.

General sizing criteria for the Two-Zone Process are summarized in Table 5.

Operationally the sludge blanket level was monitored and controlled at a depth of about 7 feet (2.1 m) by wasting solids as needed. There was minimal variation in sludge settling rates and sludge blanket management was influenced primarily by influent flow variations.

SUMMARY

The Two-Zone Process provided excellent $TBOD_5$ removals (83-92%) at average F/M loadings of 0.33 to 0.95 lbs $TBOD_5$/day/lb MLVSS, detention times of 3.3 to 7.0 hr and SRT's of 1.2 to 3.7 days. The process was capable of producing effluent TSS concentrations of less than 20 mg/L at SRT's above 2.5 days. Process oxygen requirements were low averaging 0.5 lb/lb TBOD removed. Due to the low SRT's the process generated a

TABLE 5. GENERAL SIZING CRITERIA

PARAMETER	VALUE	COMMENT
OVERFLOW RATE, gal/day/ft^2		TO DETERMINE MINIMUM SURFACE AREA INCLUDE RECYCLE
AVERAGE	500	
PEAK	1200	
SIDEWATER DEPTH, ft	13.5-15	
FREEBOARD, ft	1.0	
SLUDGE BED DEPTH, ft	53% SWD	WITHOUT FLOW EQUALIZATION
	70%	WITH FLOW EQUALIZATION
TANK GEOMETRY		RECTANGULAR
TANK WIDTH	100 gal/min/ft	FORWARD VELOCITY WITH RECYCLE
	15 ft	STANDARD SCRAPER WIDTH
INLET CHAMBER BOTTOM VELOCITY	0.45 ft/sec	TO CAPTURE FREE GAS BUBBLES
SLUDGE RECYCLE PUMPS		MAXIMUM PUMPING RATE SET BY FLOW LIMIT OF 100 gal/min/ft PROVIDE VARIABLE PUMPING CAPACITY AND 100% RESERVE
OUTLET WEIRS	4,000 gal/day/lineal ft	PROVIDE SCUM BAFFLE

1 gpd/ft^2 = 0.041 m^3/day/m^2
1 ft = 0.305 m
1 gal/mmft = 6.309 X 10^{-5}m^3/s
1 ft/sec = 0.305 m/sec

high level of net sludge and scum production (1.54 lb/lb TBOD) removed. Operator attention was modest with bi-hourly inspection and process adjustments only during the day shift.

VACUUM ASSISTED SLUDGE DEWATERING BED SYSTEMS

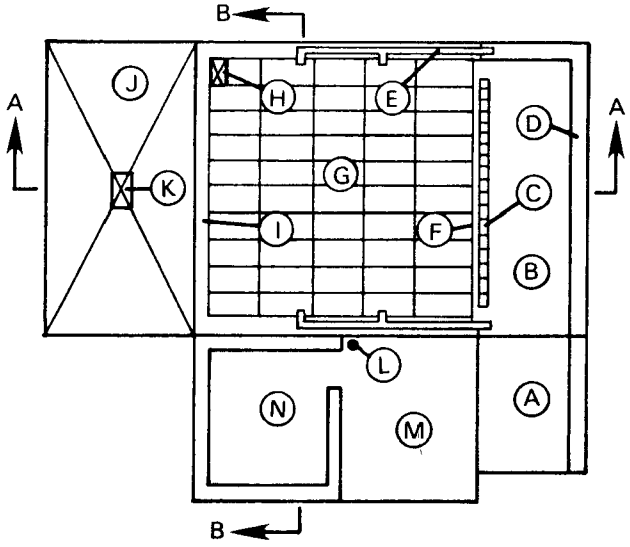
The information on vacuum assisted sludge dewatering bed systems was collected by James M. Montgomery, Consulting Engineers, Inc. and is summarized in reference (5).

The U.S. Environmental Protection Agency evaluated 12 operating Vacuum Assisted Sludge Dewatering Bed Systems (VASDBS) systems that are identified in Table 6. The technology appears to offer a cost effective dewatering technique for plants less than 2 MGD (0.086 m^3/s) in size. The process uses conditioned sludge applied to a rigid but porous support media. A vacuum is applied to the underside of the media to drain the free water that has not drained by gravity in the first 1-3 hours.

Plan and section views of the major components of an uncovered one-bed VASDB system are shown in Figures 7 and 8, respectively. Typically the porous plates are placed upon a concrete support structure. Provision is made for a filtrate collection/drainage system between the media plates and the support slab. Three concrete walls 36 in. (0.92m) in height are fixed; with a bed closure system on the fourth wall to effect closure of the bed during the loading cycle and permit removal of the sludge cake after dewatering. The media plates are sealed to each other as well as the containment walls to prevent migration of solids to the filtrate well. Provisions must be made for feeding and mixing polymer with the incoming sludge as well as flocculating and distributing the mixture onto the support media. Typically float operated filtrate pumps are located in the air-tight sump to convey collected filtrate back to the headworks of the treatment plant. A vacuum system, connected to the filtrate sump and filtrate collection/drainage system, is required to induce a partial vacuum between the underdrainage system and sludge cake on top of the media plates. A high pressure (70 to 120 psig)(480 to 830 kPa) particulate-free wash water source to clean the media surface after each dewatering cycle, and a method to convey wash water to the plant headworks are needed. Typically a control building is provided to house all mechanical and electrical components. Optionally, enclosure of the complete system and possibly heating may be required to maintain day to day operations under wet and cold weather conditions. A front end loader with a rubber covered bucket is usually used to remove the sludge cake from the bed.

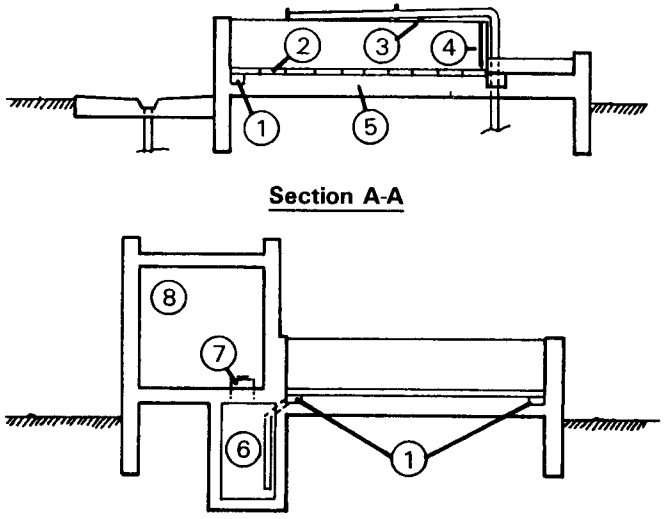
TABLE 6. VACUUM ASSISTED SLUDGE DEWATERING BED SYSTEMS EVALUATED

LOCATION	SLUDGE TYPE	MANUFACTURER
PITTSFIELD, ILL	AEROBICALLY DIGESTED	INFILCO DEGREMONT, INC.
NEVADA CITY, CA	ANAEROBICALLY DIGESTED	"
LUMBERTON, NC	AEROBICALLY DIGESTED	"
TAOS, NM	WASTE ACTIVATED + ALUM	"
MONROE, NC	AEROBICALLY DIGESTED	"
GALENA, IL	AEROBICALLY DIGESTED	SDS COMPANY
SULLIVAN, IL	LIME CONDITIONED PRIMARY + WAS	"
HILLSBORO, IL	IMHOFF TANK	"
LOUISVILLE, CO	EXTENDED AERATION WAS	"
LOCKPORT, IL	AEROBICALLY DIGESTED	U.S. ENVIRONMENTAL PRODUCTS, INC.
SUSANVILLE, CA	OXIDATION DITCH	"
MARIPOSA, CA	OXIDATION DITCH	"



Ⓐ Entrance Ramp
Ⓑ Off-Bed Level Area
Ⓒ Area Drain
Ⓓ Cursing
Ⓔ Sludge Distribution Piping
Ⓕ Bed Closure System
Ⓖ Media Plates
Ⓗ Corner Drain
Ⓘ Bed Containment Wall
Ⓙ Truck Loading Area
Ⓚ Area Drain
Ⓛ Wash Water Supply
Ⓜ Feed Sludge Inventory Tank (Below Grade)
Ⓝ Control Building With
- Sludge Feed Pumps
- Polymer System
- Vacuum Pumps
- Control Panel
- Filtrate Receiver/Pumps (Below Grade)

Plan View of a VASDB System.
Figure 7.



① Filtrate Collection Channel
② Media Plates
③ Sludge Distribution Piping
④ Bed Closure System
⑤ Concrete Support Slab
⑥ Filtrate Receiver/Vacuum Vault
⑦ Filtrate Receiver Hatch
⑧ Control Building

Sections Views of a VASDB System.
Figure 8.

A typical operating cycle is shown in Table 7.

TABLE 7. TYPICAL VASDB OPERATING CYCLE AND MAINTENANCE REQUIREMENTS

CYCLE	TIME	CONDITIONS
READY	0	VACUUM OFF BED CLOSED FILTRATE PUMPS ON AUTO DRAINS SEALED
FILL	0-0.5 HR	SLUDGE FEED ON POLYMER FEED ON FILL TO DEPTH OF 0.5 IN AND THEN OPEN FILTRATE DRAIN
GRAVITY DRAINAGE	0.5 TO 3 HRS. UNTIL 50% VOLUME REDUCTION	FILTRATE DRAIN OPEN OPTIONAL DECANTING OF SUPERNATANT
VACUUM	MEDIAN 20 HR. AVERAGE 14.5 HR. LOW 4.5 HR. HIGH 22 HR.	2-3 IN. Hg. FOR 1 HR. 5-6 IN. Hg. FOR 1 HR. 10-12 IN. Hg. UNTIL CAKE CRACKS
AIR DRYING	VARIABLE	UNTIL CAKE IS LIFTABLE @ 11-13% SOLIDS
CAKE REMOVAL	0.5 TO 1 HR/BED	FRONT END LOADER AND MANUAL REMOVAL
MEDIA CLEANING TOTAL	0.5 TO 0.75 HR/BED	HIGH PRESSURE HOSING
MEDIA DRYING	1 TO 2 DAYS	EVERY WEEK
MEDIA CHEMICAL DRYING	1 DAY	EVERY SIX MONTHS

The most critical steps in the process are proper polymer conditioning, scrupulous cleaning between cycles and periodic drying. Chemical cleaning every six months with alkaline detergents, hypochlorite, enzymes or muriatic acid were used at these plants to keep the media plates clean.

Each manufacturer's system is slightly different in detailed considerations. The media plates are on the order of 2 ft X 2 ft X 3 in (.61 m X .61 m X 7.6 cm). A gravel support base, 1.75 - 2.5 in. (4.5 - 6.4 cm) in depth, with particle diameters of 0.175 in. to 1.0 in. (0.44 to 2.54 cm) holds a surface layer, 0.25 in. (0.64 cm) in depth. The surface layer consists of 1 mm to 3 mm diameter particles which are bonded together with a waterproof, chemically resistent epoxy. The surface material is either sand or Al_2O_3.

Bed sizes vary from 400 to 800 ft^2 (37 - 74 m^2) in area. A minimum of two beds are recommended with five days dewatering operations and two days of drying.

TYPICAL PERFORMANCE AND OPERATIONAL CONSIDERATIONS

Sludge loading rate varied with the type of sludge applied from 0.66 lb/ft^2/cycle (3.2 kg/m^2/cycle) for unthickened oxidation ditch sludge to 7.75 lb/ft^2/cycle (38 kg/m^2/cycle) for the mixture of lime stabilized primary and secondary sludges. A median value of 1.88 lb/ft^2/cycle (9.2 kg/m^2/cycle) was found for typical aerobically digested sludge. This loading corresponds to 24 inches (0.6 m) of sludge with a solids concentration of 1.5%. High loadings can be achieved by using multiple fill/drainage cycles and/or supernatant decanting before applying the final vacuum cycle.

Polymer dose also varied with sludge type and was dramatically reduced with one hour of dilute polymer solution aging prior to mixing with the sludge.

Median operator time requirements were 3.5 hr/ton (3.9 hr/10^3 kg) or 3.5 hr/1000 sq. ft. Operating labor represented about 40% of the system operating costs which averaged about $80/ton ($88/kg) at a loading of 4.0 lbs/ft^2/cycle (19.5 kg/m^2/cycle). Polymer requirements, at $26/ton ($28.7/kg) of dry sludge solids represented about 33% of total operating costs. The remaining operating costs included electricity, front end loader maintenance, plate cleaning and plate replacements. Media replacement costs were estimated to be $27/$ft^2$ ($291/$m^2$) with 3% of the plate area replaced annually.

Capital costs have been variable but a generalized estimate for a facility generating 365 tons/yr (331 X 10^3kg/yr) dry sludge solids would be from $123/$ft^2$ ($11.4/$m^2$) for an uncovered VASDB to $156/$ft^2$ ($14.5/$m^2$) for an enclosed and heated facility. Estimated total VASDB system costs as a function of solids loading are presented in Table 8.

TABLE 8. ESTIMATED SYSTEM TOTAL COSTS[4]

SOLIDS LOADING RATE ($lb/ft^2/cycle$)	TOTAL COST ($/TON) UNCOVERED	ROOFED	ENCLOSED
2	160	166	173
4	105	108	112
6	87	89	92
8	78	80	82

1 lb/ft^2 = 4.88 kg/m^2
1 \$/ton = \$1.1/10^3 kg

SUMMARY

Evaluations of 12 operational VASDB systems indicated that this technology is an acceptable and cost-effective alternative to more conventional sludge dewatering processes, such as sludge drying beds, used at small- to medium-sized wastewater treatment facilities. System modifications can be made by designers and operators to improve performance of both existing and new facility installations.

RIM-NUT PROCESS

A 0.07 MGD (100 m^3/hr) pilot plant study was conducted in South Lyon, Michigan to evaluate the Rim-Nut Process* for a period of six months using primary effluent and a first-stage RBC effluent as feed. Feed water characteristics are shown in Table 9. Total ionic concentrations were high, particularly the concentrations of sulfate, calcium, sodium, bicarbonate and chloride. Ammonium and phosphorus concentrations were typical of the area. However, the phosphorus levels were low relative

*Rim-Nut is a patented process of the Water Research Institute of the National Research Council (IRSA-CMR) in Italy.

TABLE 9. AVERAGE RIM-NUT INFLUENT WATER COMPOSITION [6]

SPECIES	FIRST STAGE RBC EFFLUENT RUNS 3, 9, 10 CONC. (mg/L)	PRIMARY EFFLUENT RUNS 4-8 CONC. (mg/L)
Na^+	267	265
K^+	18	18
NH_4^+(asN)	7.3	19.3
Ca^{2+}	129	142
Mg^{2+}	32	38
Fe^{2+}	0.4	1.3
Cl^-	436	419
NO_3^-(asN)	6.7	0.76
NO_2^-(asN)	0.48	0.05*
SO_4^{2-}	87	101
ALKALINITY (as$CaCO_3$)	326	377
TOTAL PHOSPHORUS (asP)	1.9	2.7
PO_4^{3-}(asP)	1.3	1.8
pH	7.3	7.6
TOTAL CONCENTRATION	0.0215N	0.0226N

*LESS THAN

to national averages. Primary effluent BOD_5 and TSS levels were approximately 60 mg/L and 50 mg/L, respectively. First-stage RBC effluent BOD_5 and TSS concentrations were about 44 mg/L and 58 mg/L, respectively.

The Rim-Nut Process is a physical-chemical process which utilizes selective ion-exchange resin to remove both ammonium and phosphate ions from wastewater and recover ammonium magnesium phosphate (NH_4MgPO_4), a slow release fertilizer according to the following equations.

$$\text{CATION EXCHANGE} \quad RNa + NH_4^+ \underset{\longleftarrow}{\longrightarrow} RNH_4 + Na^+$$

$$\text{ANION EXCHANGE} \quad 2\,RCl + HPO_4^{=} \underset{\longleftarrow}{\longrightarrow} R_2\,HPO_4 + 2\,Cl^-$$

The resins are regenerated with a neutral 0.6M $NaCl$ solution. Magnesium salt is then added to a mixture of the regeneration elutriates to precipitate the ammonium magnesium phosphate as follows:

$$NH_4^+ + Mg^{++} + PO_4^{=} + 6\,H_2O \longrightarrow NH_4MgPO_4 + 6\,H_2O$$

After precipitation the elutriates are recycled.

The pilot plant schematic is shown in Figure 9. The cation exchange columns (C1, C2) contained 0.45 m^3 each of a natural zeolite (Clinoptilolite 1010 A/O-2/AQ, Anaconda Company, Denver, Colorado). For Runs 1 to 3 the anionic exchange columns contained 0.41 m^3 of a strongly basic anion exchange resin (KASTEL A 501D Montedison Company, Milano, Italy), for which preferentially removed sulfate over phosphate. For Runs 4 to 10 the anion resin used was Amberlite IRA 458 from Rohm and Haas Company, Philadelphia, Pennsylvania. The particle sizes of all resins ranged from 20 to 50 mesh. The exchange columns were approximately 0.65 m in diameter and 2.0 m in height and constructed of epoxy painted steel. The sodium chloride regenerant solution was stored in reservoirs S1 and S2 which were 1.0 m in diameter and 1.5 m in height. The regeneration solutions were alternatively directed to the chemical precipitators S3 and S4 where appropriate chemicals were added to precipate the NH_4MgPO_4. The thickened fertilizer was then filtered in unit F and bagged.

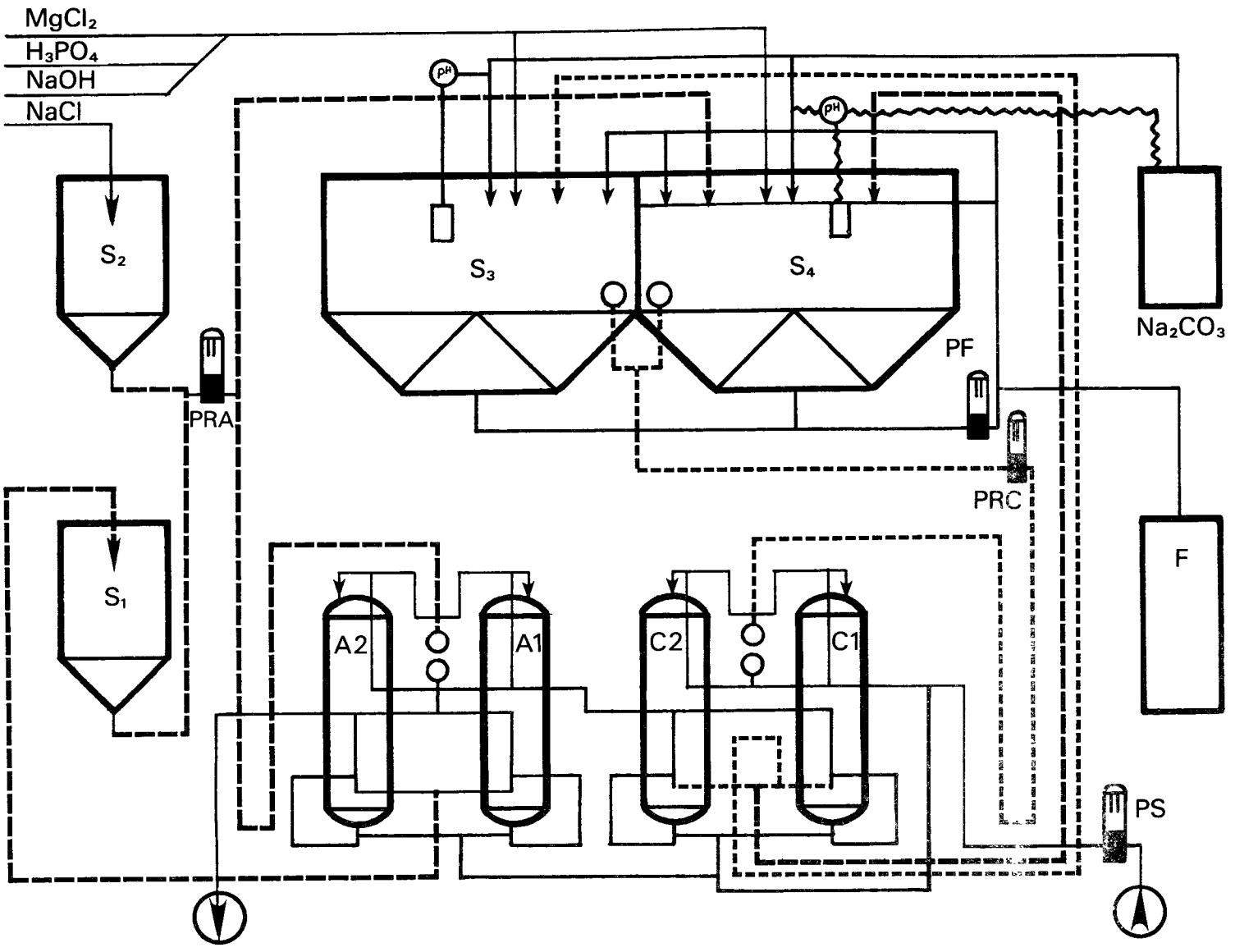


Figure 9. Rim-Nut Pilot Plant Schematic

A summary of results is shown in Table 10. The first two runs indicated that the anion exchange resin had a low operating exchange capacity for phosphorus of around 3 moles phosphorus per cubic meter of resin (P/m^3R). Phosphorus breakthrough occurred at about 60 bed volumes (BV) and could be regenerated with 2 BV. Run 3 utilized a complex mode of operation required independent regeneration of two anion exchange columns in an effort to overcome the effects of sulfate selectivity. Runs 4 through 10 utilized the A458 anion exchange resin which offered improved selectivity for orthophosphate and a greater overall exchange capacity of up to 11.5 moles P/m^3R. The phosphorus breakthrough in Run 4 was extended to 200 bed volumes. With higher concentrations of ammonium-N in the primary effluent feed, ammonium-N breakthrough occurred for the first time in the cation exchange column at about 100 BV.

This anion exchange resin was also regenerated with 2 BV of regenerant. The cation exchange resin required circulating 6 BV of regenerant through the column and precipitator 4 times.

The cationic resin regeneration cycle consisted of passing 6 BV of 0.6 M NaCl from S4 downward through the exhausted resin and collecting it in S3 where Na_2CO_3 was added until pH 9 to 9.5. After ammonium analysis, $MgCl_2 \cdot 6 H_2O$, H_3PO_4 (75%) and NaOH (50%) were added stoichiometrically to precipitate $NH_4MgPO_4 \cdot 6H_2O$. After settling the precipitate, the supernatant solution was recycled. The cationic regeneration procedure efficiency for Runs 4 through 10 averaged 94% in terms of the moles of NH_4 removed divided by the moles of NH_4 exchanged.

The anionic resin regeneration cycle involved two fractions, each 2 BV of 0.6 M NaCl, from Reservoirs S1 and S2 and separately collected. After orthophosphate analysis, the head-fraction, which usually contained 90% of the exchanged orthophosphates, was precipitated in S3 (or S4) together with the last exhausted cationic regenerant fraction. The tail-fraction was collected in S1 to be used as head fraction during the next regeneration cycle. Fresh regenerant solution would be added to S2. Anionic regeneration efficiency for Runs 4 to 10 averaged 100% as shown in Table 11.

The anionic resin was completely regenerated after Run 6 in preparation for Run 7. During Run 7 orthophosphate never broke through with 150 BV processed. The average effluent phosphorus concentration was 0.16 mg P/L. The cationic column exhaustion cycle for Run 7 is shown in Figure 10. No sharp breakthrough of ammonium-N occurred. A gradual

TABLE 10. SUMMARY OF RUNS DATA (5)

RUN	INFLUENT SOURCE	COLUMNS USED	ANIONIC RESIN	RUN-TIME (HRS)	AMMONIUM		ORTHOPHOSPHATE		CATIONIC RESIN O.E.C.* (moles N/ m^3R)	ANIONIC RESIN O.E.C.* (moles P/ m^3 R)
					INF. (mg N/L)	EFF. (mg N/L)	INF. (mg/P/L)	EFF. (mg P/L)		
1	RBC	C_1 A_1	K501	4.5	6.1	2.1	1.1	0.48	79	2.3
2	RBC	C_1 A_1	K501	8			0.99	0.52		3
3	RBC	C_1 A_1 A_2	D501	8.5	10.1	3.7	1.9	0.30	86	2.1 8.9
4	PRIMARY	C_2 A_2	A 458	8.5	15.6	3.5	1.8	0.08	156	11.5
5	PRIMARY	C_2 A_2	A 458	7	21.0	6.5	1.8	0.35	154	8
6	PRIMARY	C_2 A_2	A 458	6	24.0	5.5	1.6	0.30	169	6.1
7	PRIMARY	C_2 A_2	A 458	6	20.0	3.6	1.2	0.16	150	4.9
8	PRIMARY	C_1 A_2	A 458	6	16.0	1.8	2.5	0.13	135	11.2
9	RBC	C_2 A_1	A 458	6.5	6.3	2.2	1.0	0.06	41	4.9
10	RBC	C_2 A_1	A 458	6	5.6	2.6	1.4	0.14	27	6

*O.E.C. (OPERATING EXCHANGE CAPACITY)

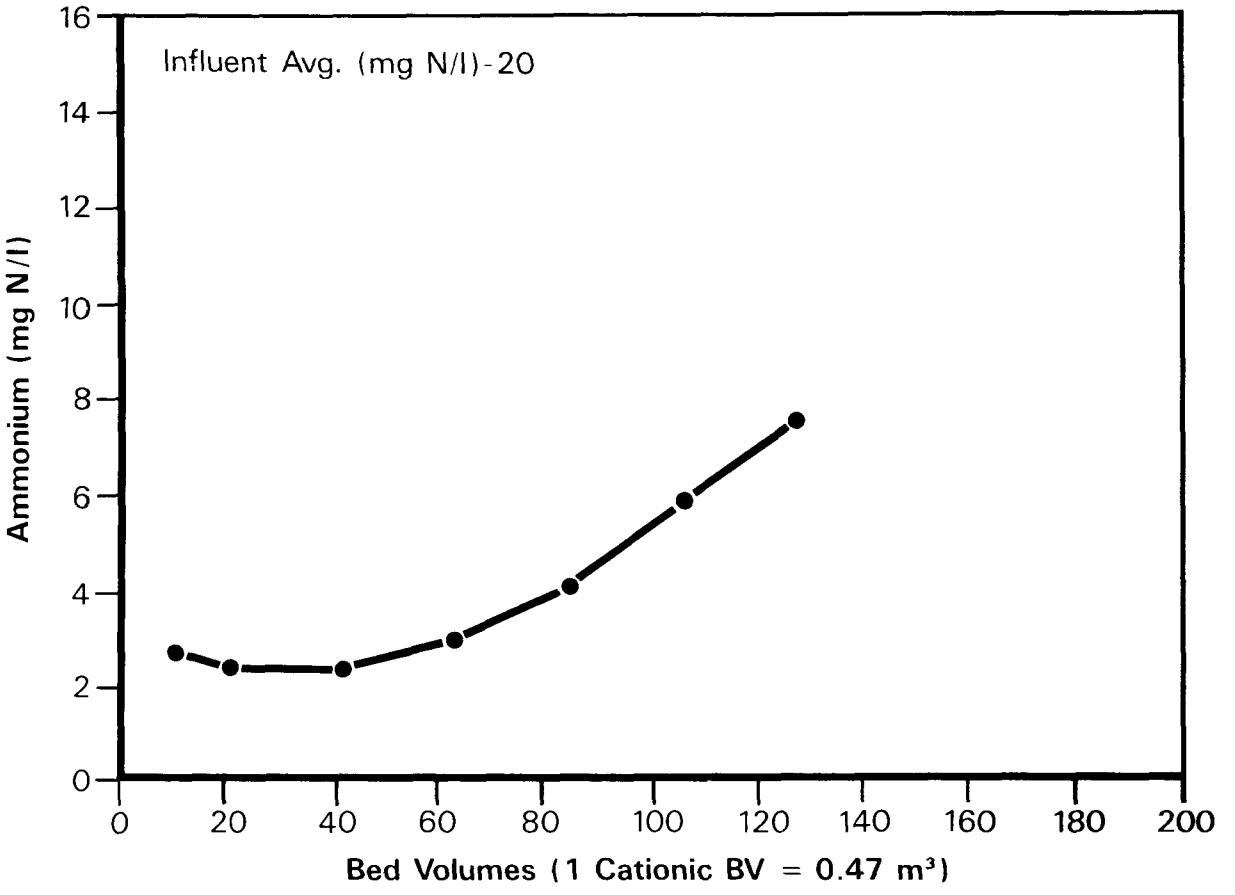


Figure 10. Cationic Exhaustion — Effluent Run 7

TABLE 11. ANIONIC REGENERATION PROCEDURE EFFICIENCY (5)

RUN	NUMBER OF FRACTIONS OF REGENERANT SOLUTION*	MOLES OF P REMOVED / MOLES OF P EXCHANGED X 100
4	2 (1.5_F + 2_F) BV	94
5	2 (2_R + 2_F) BV	88
6	2 (2_R + 2_F) BV	116
7	2 (1.5_F + 2_F) BV	94
8	2 (2_F + 2_F) BV	NOT DETERMINED
9	2 (2_F + 2_F) BV	117
10	1 (2_F) BV	96
	AVERAGE VALUE	100%

*F = FRESH REGENERANT SOLUTION
R = RECYCLED REGENERANT SOLUTION

increase from an initial leakage concentration of 2.7 mg N/L occurred. The average ammonium-N concentration for Run 7 was 3.6 mg N/L. After complete regeneration of the cationic resin following Run 7 the initial ammonium leakage was reduced to 0.7 mg N/L for 70 BV during Run 8.

The Rim-Nut Process achieved average removals of BOD_5, COD, TSS, VSS and Fecal Coliform of 67%, 31%, 53%, 53% and 44%, respectively. With primary effluent as the feed source for Runs 4-8, the Rim-Nut Process produced final effluent BOD_5 values of 6, 18, 26, 22, and 30 mg/L and effluent TSS values of 22, 13, 32, 25 and 19 mg/L, respectively.

A comparision of the calculated and experimentally determined composition of the fertilizer produced during Run 7 is shown in Table 12. The calculated composition compares most favorably with the analyses performed by the Tennesee Valley Authority.

ECONOMIC ANALYSIS

Based upon an influent flow of 1 MGD (0.043 m^3/s), a nitrogen concentration of 8 mg/L as N, and a phosphorus concentration of 2 mg/L, the South Lyon, Michigan plant would produce 215 tons/yr (195 X 10^3 kg/yr) of $NH_4MgPO_4 \cdot 6H_2O$.

Capital and operating costs for such a facility are shown in Table 13.

TABLE 12. CALCULATED AND EXPERIMENTAL COMPOSITION OF FERTILIZER PRODUCED DURING RUN 7 (5)

FRACTION	MOLES PRECIPITATED N	P	Mg	Ca
I	14.6	22.4	35.9	12.7
II	9.3	15.4	14.2	1.6
III	13.3	10.9	13.3	1.6
IV	9.6	4.3	4.0	2.7
TOTAL MOLES	47	53	68	19

M.P.C.P.*	M.W.	MOLES PRECIPITATED	GRAMS PRECIPITATED	WEIGHT FRACTION %
$MgNH_4PO_4$ $6H_2O$	245	47	11,515	74.6
$Mg_3(PO_4)_2$ 4 H_2O	334	3	1,002	6.5
Mg CO_3	84	12	1,008	6.5
$CaCO_3$	100	19	1,900	12.3
$Ca_3(PO_4)_2$	310	0	0	0

CALCULATED FERTILIZER TITLE: N (4.3) = P_2O_5 (24.4) = Mg (10.6)

*MOST PROBABLE CHEMICALS PRECIPITATED

SOLID FERTILIZER ANALYSIS:
TENNESSEE VALLEY AUTHORITY FERTILIZER: N (3.0) = P_2O_5 (24.9) = Mg (8.6)

W. R. GRACE: N (1.5) = P_2O_5 (34) = Mg (17)

TABLE 13. COST ESTIMATE FOR 1 MGD RIM-NUT PLANT [5]

CAPITAL COST	
ION EXCHANGE VESSELS, PIPING VALVES AND INSTRUMENTATION	$640,000
MICROPROCESSOR CONTROL AND PROGRAMS	100,000
EXCHANGE MEDIA	700,000
REGENERATION TANKS/FILTERS	200,000
BUILDING	600,000
TOTAL CAPITAL COSTS	$2,240,000
ANNUAL OPERATING & MAINTENANCE COSTS	
EXCHANGE MEDIA REPLACEMENT	$ 25,000
CHEMICALS	185,000
LABOR (4 OPERATORS)	160,000
POWER AND MISCELLANEOUS	30,000
TOTAL O&M	$400,000

With an amortization schedule of 20 years at 8.25%, the total cost per ton of product to break even is ($224,000 + 400,000)/215 = $2,900/ton ($3.20/kg) or $1.45/lb. The retail price for commercial grade ammonium magnesium phosphate in the U.S.A. was $1.00/lb ($2.20/kg) in 1986.

RUBBER TIDE GATE

One of the more unusual technologies to be evaluted recently by the Water Engineering Research Laboratory is the rubber tide gate (RTG). A cooperative research project was funded with the City of New York to install and evaluate the reliability of a 54 in. (1.4m) neoprene covered, vulcanized rubber tide gate at the E. 89th Street regulator station. New York City spends approximately $1M annually to maintain some 554 tide gates and regulators. A complete inventory and assessment of the status of the New York City system was completed in April, 1985[7]. This study identified an average inflow from tide gates of 25 MGD (1.1 m^3/s) and a peak inflow of 67 MGD (2.9 m^3/s). A program of repair and replacement of tide gates has been recommended.

Table 14 illustrates the potential payback period for replacing the smaller (<72 [1.8m] inch diameter) conventional flap-gates with rubber tide gates based upon projected annual cost savings in both inspection and maintenance costs and excessive inflow treatment costs[8].

During the 18 month period of evaluation, the RTG required almost no maintenance and indicated negligible inflow with good sealing and self-cleaning characteristics. New York City is currently having two 72 inch (1.8 m) RTG's fabricated for installation in two troublesome locations; Port Richmond in Staten Island which has excessive debris problems, and Newtown Creek in Brooklyn which is a difficult gate to access. A number of other cities including Boston, MA; Corvallis, OR; Allentown, PA; Bridgeport, CT; and Hempstead, NY have installed RTG's from 12 to 42 inches (.3 to 1.1m) in diameter in the last several years.

TABLE 14. PAYBACK PERIOD FOR RUBBER TIDE GATE REPLACEMENT PROGRAM

NUMBER OF TIDE GATES (<72 in.)(1.8m)	347
RTG REPLACEMENT COSTS	$13M
RECONDITIONING COST OF CONVENTIONAL GATES	$10M
RTG ANNUAL SAVINGS IN ROUTINE INSPECTION AND MAINTENANCE	$175,000
RTG COST SAVINGS DUE TO REDUCED INFLOW	$250,000
PAYBACK PERIOD	7 YRS
NET PRESENT VALUE (20 YRS @ 7%)	$1.5M

The RTG was developed by Red Valve Company, Inc. of Carnegie, PA based upon a check valve concept. The RTG consists of a flexible tube which tapers to a flattened section with two sealing lobes. A forward hydraulic head of six inches (0.15 m) opens the lobes, releasing flow. Figure 11 shows the discharge flow characteristics of the RTG as a function of headloss and degree of downstream submergence. Reverse hydraulic head due to a rising tide collapses the sealing lobes together, thereby preventing reverse (leakage) flow. The RTG starts to release flow at a lower hydraulic head differential than conventional flap gates for all conditions of submergence. The flap gate discharges a greater flow at a head differential of about 1.5 ft (0.46 m) or greater. The lower maximum flow capability of the RTG requires careful estimation of peak storm flows to size the RTG. The 54 in. (1.4 m) RTG evaluated had a maximum flow capacity of 120 c.f.s. (3.4 m^3/s).

The RTG at E. 89th Street weighed 800 lbs and was affixed to a round stainless steel adaptor plate with a circular clamping ring to conform to the existing tide gate structure opening. The only maintenance the RTG required in the three years since its installation in August, 1984 was due to slippage which occurred during a storm event. It took seven hours to reset the gate. An improved mounting design using bolts through the gate throat and adapter ring together with a ceiling anchor to support the cantilevered end of the lobes are two potential ways to avoid future slippage problems.

During 18 months of frequent inspections, which started at a frequency of once per week and decreased to once per month after eight months, no incidences of leakage flow were reported even though reverse hydraulic head conditions were frequently present. There were also no incidences of trapped debris. Placement of a 4 in. by 4 in. (.1 m X .1 m) length of timber in the RTG to simulate trapped debris caused a 50 gpm (3.15×10^{-3},$^3/s$) leakage flow at a reverse hydraulic head of 2 ft. (0.61 m). The next occurrence of forward hydraulic head flushed the timber out of the RTG. The flexibility of the RTG permits the unit to conform closely to the shape of any entrapped debris thereby minimizing the leakage flow under reverse hydraulic head conditions.

In summary, the RTG provides a potential low maintenance and cost effective alternative to conventional flap gates.

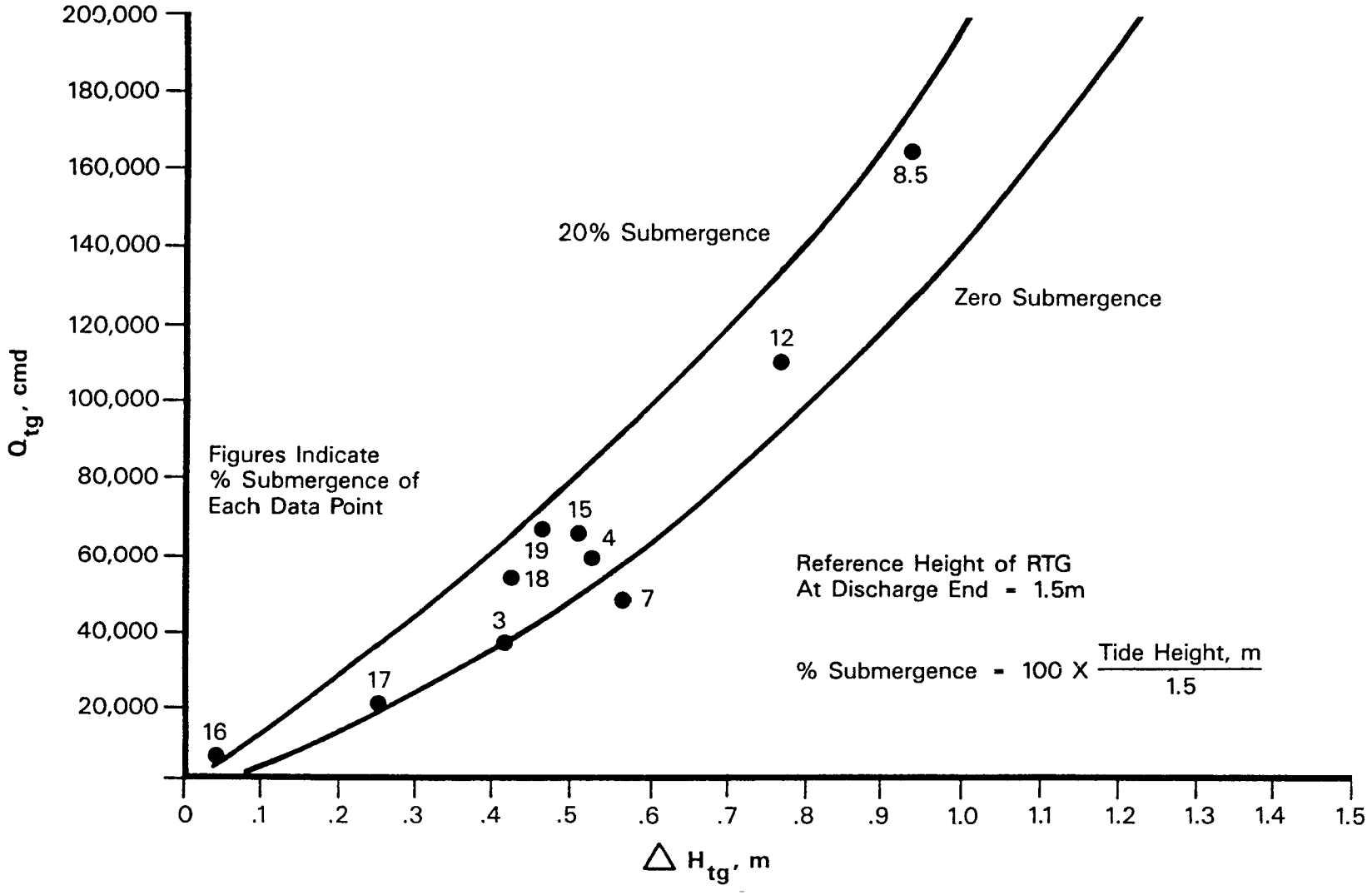


Figure 11. Estimated RTG Flow Characteristics

REFERENCES

1. 1986 Needs Survey Report to Congress; U.S. Environmental Protection Agency, U.S. EPA 430/9-87-001; February 1987.

2. Brown and Caldwell, "Trickling Filter/Solids Contact Process: Full-Scale Studies"; EPA-600/52-86-046; U.S. Environmental Protection Agency, Cincinnati, Ohio (1986).

3. Matasci, R.M., Kaempfer, C. and Heidman, J. A., "Full-Scale Studies of the Trickling Filter/Solids Contact Process", J. Water Pollution Control Federation, 58, No. 11, 1043-1049.

4. Weech, S.R., Stack, V.T., and Orton, G., "Evaluation of the Two-Zone Wastewater Treatment Process at Norristown, PA, EPA/600/2-87/074, 1987.

5. EPA Design Information Report "Design, Operational and Cost Considerations for Vacuum Assisted Sludge Dewatering Bed Systems", J. Water Pollution Control Federation, 59, No. 4, 228-234.

6. City of South Lyon, Michigan - Rim-Nut Demonstration Report; U.S. Environmental Protection Agency Grant No. R-005858-01, April 1987 (Draft).

7. Summary Report - City-Wide Regulator Improvement Program Inventory and Assessment, New York City Department of Environmental Protection, WP-112, Contract No. 1-REG-10A to Hazen and Sawyer, April 1985.

8. Development and Evaluation of a Rubber "Duck Bill" Tide Gate, U.S. Environmental Protection Agency Project Draft Report, CR-807822.

☆U.S. GOVERNMENT PRINTING OFFICE:1988-548158/67111