January 1977

Proceedings of the Utah Water Pollution Control Association Annual Meeting 1977

James H. Reynolds

Donna H. Falkenborg

PROCEEDINGS OF THE

UTAH WATER POLLUTION CONTROL ASSOCIATION

ANNUAL MEETING

April 21-22, 1977
Salt Lake City, Utah

Compiled by
James H. Reynolds and Donna H. Falkenborg

WATER POLLUTION CONTROL FEDERATION · 1928 ·

Utah Water Research Laboratory
College of Engineering
Utah State University
Logan, Utah 84322

April 1977

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Preface

This volume is the first published proceeding of the Utah Water Pollution Associations Annual Meeting. The Technical Program of the Annual Meeting was divided into five separate sessions. The program was developed to attract individuals concerned with management, design and operation of wastewater treatment facilities. Special emphasis was given to the design and operation of wastewater filtration devices.

Unfortunately, four papers are not included in the proceedings because the authors failed to meet the submission deadline. These papers are (1) "Utah Discharge Requirements" by Calvin Sudweeks, (2) "Chlorine, Coliforms, 1977 Standards and You" by Robert A. Sperling, (3) "Panel Discussion" by Ken Watson, and (4) "Panel Discussion" by Michael Miner. One additional paper entitled "Problems of Mounting a Major Water Pollution Control Program" by Martin Lang is not included in the proceedings due to personal committment as National Vice President of the Water Pollution Control Federation.

The Technical Program Committee greatly appreciates the time and effort expended by the authors and advertisers who made these proceedings possible.

James H. Reynolds
Program Chairman

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History of Wastewater Treatment in Utah

*Stephen E. Moehlmann**

Salt Lake City and Ogden constructed wastewater collection systems in 1890, promoted by a need for improved sanitation.

In the 1930s it was clear that sewer systems alone were not the ultimate solution to the waste disposal problems in Utah. But, in addition, construction of adequate treatment facilities was necessary to protect public health, conserve water resources, and protect the State's waters from degradation. At that early time the State Division of Health determined that all wastes should receive at least secondary treatment.

The early 1940s saw the construction of the first modern treatment facilities at military bases such as Hill Field, Ogden Arsenal and Kearns, Dragerton, Horse Canyon, and Geneva Steel. The first modern municipal plant was constructed in 1949 at Nephi.

Greater population growth, the State Law in 1953, and the initiation of the construction grants program under the United States Public Health Service in 1956 further stimulated the construction of municipal facilities. A decade after the opening of the first municipal plant 27 treatment facilities had been constructed.

More rapid population growth and the development of the environmental movement with the formation of the Environmental Protection Agency in 1970 and the passage of the 1972 Clean Water Amendments, demanded both the construction of more treatment facilities, and the enlargement of existing facilities.

Presently in 1977, less than 30 years after construction of the first municipal facility, we have 85 municipal facilities with over 200 operators.

The advent of so many sophisticated secondary treatment facilities in such a short time presented several difficulties. Voters, elected officials, city and district officials, and operators had to be made aware of the necessity and importance of these facilities, and they had to be taught how to properly and efficiently operate these facilities.

The League of Cities and Towns laid the ground work for education of city and district officials with the Waterworks and Sanitation Conference which started in 1944. Since then, the Conference has been a regular part of the League's Annual Meeting in the fall and the regional schools for municipal officials in the spring.

Operator training started in the Spring of 1950 with inception of the Municipal Water and Sewage Works School—sponsored by the University of Utah, Division of Health, League of Cities and later joined by the Utah Water Pollution Control Association and Intermountain Section of the American Water Works Association. This school was held annually from 1950 through 1972 with a total of 23 sessions.

Additional operator training programs were provided by the "In Plant Training Program" offered in 1956, 1957, and 1964 at the Salt Lake City Suburban Sanitary District Treatment Plant, the 1959 "In Service Training" programs and 1964 Short School for Sewage Works Operators offered at the University of Utah.

The need for a professional organization for all of those concerned with wastewater collection, treatment and disposal culminated in the formation of the Utah Water Pollution Control Association in 1957, initially known as the Utah Sewage and Industrial Wastes Association.

Association membership meetings were highlighted by talks and by tours of wastewater treatment facilities or wastewater related industries.

In 1964, Utah State University offered the first Management Institute for Water and Sewer Districts and Municipalities. These Institutes were designed to provide the city and district managers with training and information to improve their operations.

By 1965, the Utah Board for Voluntary Certification of Water and Wastewater Works Operators had adopted a certification plan. This Plan had been approved by the Division of Health, the League of Cities and Towns, the Utah Water Pollution Control Association and the Intermountain Section of the American Water Works Association. The Board offered its first exams in 1966 and each year since.

Certification and growing numbers of untrained operators stimulated the formation of the Utah Water and Wastewater Operators Basic Training School.

This school offered basic and intermediate level courses for water distribution, water treatment and wastewater treatment in 1966, 1967, and 1968. The basic school led to formation of the Utah Water and Wastewater Training Committee, known as the Joint Committee. The Joint Committee's bylaws were adopted by the Parent Associations (the Utah Water Pollution Control Association and the Intermountain Section of the American Water Works Association) in 1969 and their first school was conducted in 1970.

*Stephen E. Moehlmann is a Public Health Engineer for the Bureau of Water Quality, Utah State Division of Health.

In 1971, 1972, and 1974, the State Division of Health sponsored a 44 week operator training course called Utah's Wastewater Treatment Plant Operators On-the-job Training Program.

In 1971, Utah State University offered a short course in Lagoon and Package Plant Operation for the U.S. Forest Service operators.

The Utah Board of Voluntary Certification became a member of the Associated Boards of Certification in 1973.

EPA has sponsored seminars in 1973, 1974, and 1976 for consulting engineers. The seminars discussed upgrading the design and operation of wastewater facilities.

Under an EPA grant, Utah State University offered a Resource Utilization and Environmental Management Institute in 1974 and 1975.

The State Division of Health held five regional Waste Stabilization Pond Seminars for lagoon operators in 1974.

The adoption of new regulations concerning overtime pay by the U.S. Department of Labor prevented the Joint Committee from holding the Joint School in 1975. To fill the gap, the Division of Health offered a nine week Short Course for Wastewater Operators.

To meet increasing needs for qualified wastewater instructors, the Division of Health requested and sponsored the EPA Instructor Basic Techniques course in 1975. This course taught operators how to be instructors.

To comply with requests to reduce time commitment of operators in training, the Division of Health's 44 week course was split into two 27 week courses: Introduction to Wastewater Treatment Plant Operation and Advanced - Wastewater Treatment Plant Operation. Also to meet the needs of more distant facilities, the Introduction training course was held in two locations, Salt Lake City and Orem. The advanced course was only offered in Salt Lake City.

The State Division of Health has sponsored seminars for consulting engineers in 1974, 1975, and 1976. These seminars discussed various aspects and requirements of the EPA Construction Grants program and regulations.

The Joint School was offered again in 1976 but under a new framework. To comply with the U.S. Department Labor regulations, the Joint School was sponsored by Utah State University.

In addition, the Joint School added two new courses, Basic Wastewater Collection Systems and Advanced Water and Wastewater.

Additional instructor training was offered by the EPA Instructor Development Workshop in 1976. The instructors trained in the Techniques Course received more detailed information on class and training material preparation.

To meet the needs of operators unable to attend training courses offered in Salt Lake City, the Division of Health started a Self-Paced Operator Training course in 1976. This course was designed for those operators who couldn't, for various reasons, attend operator training courses offered along the Wasatch Front.

1976 also saw the formation of the Utah Environmental Systems Operations Training Program (UESOTP). The UESOTP was organized to coordinate the numerous training activities for Water, Wastewater, Solid Waste, and Air Pollution personnel now available in the State and offer them under a unified format.

The first course offered under UESOTP in 1976 was the Utah State University's Wastewater Safety Supervisors Course. Next was the NPDES Self-Monitoring Basic Laboratory Skills Workshop sponsored by the Division of Health. This course was designed to give operators the skills necessary to perform the NPDES laboratory tests. Also, a NPDES Permits Seminar was sponsored by UESOTP in 1976. The seminar was to help the permittee more accurately fill in his NPDES Permit Report form.

Programs offered under UESOTP in 1977 include the Joint School, the Division of Health's Wastewater Mathematics Course and the NPDES Self-Monitoring Basic Parameters for Municipal Effluent Workshop.

The 1977 Joint School added an Intermediate Wastewater Collection Systems course.

The Division of Health's Wastewater Mathematics course gave the operator practice with the mathematics geared specifically to plant operation and certification.

The NPDES Basic Parameters Workshop was also sponsored by the Division of Health and was the second half of wastewater laboratory training provided under an EPA grant. It provided actual experience in running laboratory tests required by the permit for operators who have successfully completed the previous NPDES Skills Workshop.

Training efforts in the State of Utah since 1944 have involved nearly everybody concerned with wastewater-elected officials, engineers, managers, and operators.

The League's Waterworks and Sanitation Conference and Utah State University's Management Institute have provided elected officials and managers with training.

Utah Water Pollution Control Association, State Division of Health, University of Utah, and the League have offered the operator a wide variety of training from basic theory through instructor development. This group has also provided the operator with the means to become certified.

With the advent of the Utah Environmental Systems Operations Training Program, the process of operator training will be additionally refined to further coordinate efforts between the Associations, League of Cities, and Towns, the Division of Health, the State's educational institutions and the Board of Certification.

UTAH LEAGUE OF CITIES AND TOWNS REGIONAL SCHOOLS MUNICIPAL WATERWORKS AND SANITATION CONFERENCE

The League's Municipal Waterworks and Sanitation Conference was organized to assist city and district officials and to keep these officials abreast of new technology, operations, and regulations.

The conference has been held each year during the regular League meetings since 1944.

In 1975, the League format was changed and the "conference" changed to the Water and Wastewater Works Section of the Public Works Session.

UNIVERSITY OF UTAH

Municipal Water and Sewage Works School

The Municipal Water and Sewage Works School held annual meetings in the spring of the year from 1950 through 1972. The school lasted for 2 or 3 days with the time split equally between waterworks and wastewater works operator training. The school was organized to provide personnel involved with water and wastewater works operations the basic information to properly operate and maintain their facilities, plan for future expansion and comply with local, State and Federal regulations. In 1965, the name was changed to the Municipal Water and Wastewater Works School. At the end of the 1966 session and all subsequent schools, certification examinations were provided.

The school was sponsored by the University of Utah, Utah State Division of Health, Utah League of Cities and Towns, Utah Water Pollution Control Association and the Intermountain Section of the American Water Works Association. Attendance averaged between 150 and 200.

UTAH WATER POLLUTION CONTROL ASSOCIATION

The Utah Water Pollution Control Association was formally organized in 1957 as the Utah Sewage and Industrial Waste Association. They changed their name to the Utah Water Pollution Control Association in 1967.

The Association has held regular meetings throughout each year in addition to the annual conference. These meetings were often highlighted by plant tours and discussions of pertinent topics.

The Association has been instrumental in stimulating operator training and providing the necessary expertise to run the training programs.

UTAH STATE UNIVERSITY MANAGEMENT INSTITUTE FOR WATER AND SEWER DISTRICTS AND MUNICIPALITIES

In 1964, Utah State University sponsored the first Management Institute for Water and Sewer Districts and Municipalities. It meets for 2 days each spring and averages about 50 participants.

"The Institute aims to provide training and information to water and sewer districts management, district board members and trustees, city mayors, managers, commissioners and councilmen, and others involved in the management and operation of water and sewer facilities."

U.S. FOREST SERVICE LAGOON AND PACKAGE PLANT OPERATOR SHORT COURSE

This course was provided by a grant from the U.S. Forest Service to Utah State University in 1972 to provide training for operators of U.S. Forest Service package plants and lagoons with training.

RESOURCE UTILIZATION AND ENVIRONMENTAL MANAGEMENT INSTITUTE

This Institute was founded by an EPA grant in 1974 and 1975. It was to provide teachers with information regarding the environment and the production of training modules. The course met for 3 weeks and had 15 graduates.

UTAH BOARD FOR VOLUNTARY CERTIFICATION OF WATER AND WASTEWATER OPERATORS

Operator certification efforts officially started with the formation of the Utah Board for Voluntary Certification of Water and Wastewater Works Operators in 1965.

The first exams were offered in 1966 and each year since. The Plan set up separate exams for Water Treatment and Wastewater Treatment with four grade levels.

In 1976, exams for Water Distribution and Wastewater Collection were added to the program.

A special classification of Grade V was set aside for those who did not pass the grade IV exams.

In 1973, the Board joined the "Associated Boards of Certification." The Board is presently making changes necessary to fully comply with the ABC program.

By 1976, there were 156 certified wastewater operators in Utah.

UTAH WATER AND WASTEWATER JOINT TRAINING COMMITTEE

The Utah Water and Wastewater Joint Training Committee, known as the Joint Committee, was officially formed in 1969. The Joint Committee was formed by the adoption of the bylaws by the Parent Association—the Utah Water Pollution Control Association and the Intermountain Section of the American Water Works Association.

The purpose of the Joint Committee is to improve the qualifications of operators by providing them with formal training.

The objectives are outlined as follows:

1. To develop a curriculum for and administer operator training programs.

2. To advance the fundamental and practical knowledge required to effectively and efficiently operate water and wastewater works.

3. To aid operators in progressing with certification.

The Joint Committee sponsored their first training school in 1970 and every year since, except for 1975. The Committee membership is appointed by the Parent Associations. The Committee originally established by adopted bylaws was composed of 14 members—7 from each association. It was expanded in 1976 to 18 members with 9 members from each association.

The history of the Joint Committee goes back to the In Plant Training Programs, In Service Program, the Short School for Sewage Works Operators and the Basic Schools of the 1950s and early 1960s.

The In Plant Training Program was offered by Salt Lake City Suburban Sanitary District #1 in 1956, 1957, and 1964. It met one evening a week for 2 hours for 10 weeks and had guest speakers at each session.

The In Service Program in 1959 and the Short School for Sewage Works Operators in 1964 were cooperative efforts of the Utah Water Pollution Control Association, the Division of Health, the League of Cities and Towns, and the University of Utah. These programs held classes once a week for 3 hours for 8 weeks and had guest speakers at each session.

These training programs led to the formation of the Utah Water and Wastewater Operator Basic Training School in 1966. The Basic School was held in 1966, 1967, and 1968. The Basic Training School established the training framework to be used by the Joint Committee. This framework consisted of basic and intermediate level courses in wastewater treatment, water distribution and water treatment. The Basic Training School held evening classes in the winter. These classes met once a week for 2 to 3 hours for 8 to 10 weeks and had guest lectures for instructors.

The Joint Committee has used the same framework and timing for their courses.

In 1976, the Joint Committee added two new courses, Basic Wastewater Collection Systems and Advanced Water and Wastewater. 1977, the Joint School added Intermediate Wastewater Collection Systems. The Joint School has approximately 50 wastewater graduates each year.

The Committee did not present a school in 1975 because of an unfavorable decision by the U.S. Department of Labor concerning overtime pay for municipal employees. To adapt to the U.S. Labor regulations the 1976 and 1977 sessions of the Joint School have been sponsored by Utah State University.

The Schools have used the New York Manual of instruction for Sewage Treatment Plant Operators, the Texas A & M University Engineering Extension Service Manuals, the Sacramento State College Operation of Wastewater Treatment Plant manual, and the Sacramento State College Operation and Maintenance of Wastewater Collection Systems manual.

STATE DIVISION OF HEALTH

1. Wastewater Treatment Plant Operator On-the-Job Training Program

The Division of Health has sponsored Wastewater Treatment Plant Operator On-the-Job Training Programs in 1971, 1972, and 1974.

The 1971 class was the first course of its kind offered in the United States. The 1971 and 1972 courses were cooperative efforts by the Division of Health and EPA, and were jointly funded. It 1974, the Division of Health solely sponsored and funded the training.

These courses used the New York Manual of Instruction for Sewage Treatment Plant Operators, the Sacramento State College Operation of Wastewater Treatment Plants, WPCF MOP 1, WPCF MOP 6, WPCF MOP 11, WPCF MOP 16, WPCF MOP 18, and WPCF MOP 20.

Objectives of the training program were to transfer those necessary operational skills to operators of wastewater treatment plants to achieve the best effluent possible. More specifically, these objectives included a basic orientation in water supply and wastewater control; development in educational skills—math, communications, and science; coordinating education skills with unit process operation skills; knowledge of local treatment plant design and operation; preparation for unit process operator on-the-job training; and motivation to seek further education for career development in the wastewater field.

The classes averaged 20 graduates. They met twice a week for 4 hours for 44 weeks with approximately 350 hours of classroom instruction and 70 hours of the On-the-Job training.

The 1971 and 1972 courses were held during the evening, and the 1974 course was split into morning and afternoon sessions.

A point of interest, Governor Calvin L. Rampton awarded certificates to the first graduating class in 1971.

2. Waste Stabilization Pond Seminars

In 1974, the Division of Health sponsored five one-day Waste Stabilization Pond Seminars throughout the State. Seminars were held at Salt Lake City, Corinne, Washington, Duchesne, and Ephriam with a total attendance of 49 operators.

The seminars were held to acquaint operators with terminology, operation and problems associated with waste stabilization ponds. A 32 page outline concerning waste stabilization ponds served as the basis of instruction given at the seminars.

3. Construction Grant Seminar

In 1974 and 1975, the State Division of Health sponsored a one-day seminar on the EPA construction grant program for consulting engineers. The seminar reviewed the regulations and requirements for facility plans.

4. Wastewater Treatment Plant Operator Short Course

To fill the gap left by the absence of the Joint School in 1975, the Division of Health offered a nine week Wastewater Treatment Plant Operator Short Course.

The course met once a week for 3 hours. Half of the session was a general review of treatment principles and the other half was mathematics.

Nineteen operators received certificates.

5. O & M Manual Preparation Seminar

The Division of Health sponsored a half day seminar in 1975 called O & M Manual Preparation Seminar. The purpose of the seminar was to assist the consultants or prospective O & M manual writers in organizing and preparing O & M manuals. Division of Health and EPA regional people responsible for review conducted the program.

6. Instructor Techniques Course

Upon request of the Division of Health, EPA held their Instructor Techniques Course - B in Salt Lake City in 1975. This course met for one week—40 hours—with 14 graduates.

Objective of the course was to prepare operators to serve as instructors by giving them the necessary techniques to organize and conduct classes.

7. Introduction to Wastewater Treatment Plant Operation

To reduce the time and personnel commitments of the 44 week OJT Training Program, the Division of Health in 1975 split the course into two 27 week courses, Introduction to Wastewater Treatment Plant Operation and Advanced - Wastewater Treatment Plant Operation.

Introduction to Wastewater Treatment Plant Operation was designed for entry level operators having at least 3 months of experience and interested in taking the grade 3 or 4 certification exam.

This course was held in Salt Lake City and Orem and had 16 graduates. Texts used in the course were New York Manual of Instruction for Sewage Treatment Plant Operators, Sacramento State College Operation of Wastewater Treatment Plant Manual, WPCF MOP 1, WPCF MOP 6, WPCF MOP 11, and WPCF MOP 18. The course met once a week for 3 hours for 27 weeks.

8. Advanced - Wastewater Treatment Plant Operation

Advanced - Wastewater Treatment Plant Operation was the second half of the 44 week course split held in 1975.

It was designed for lead operators, foremen, and superintendents desiring more information on the details of plant operation and design, and who were interested in taking the Grades 1 or 2 certification exam.

This course was held in Salt Lake City and had 9 graduates. WPCF MOP 1, WPCF MOP 16, and WPCF MOP 20 were used for texts. This course was conducted weekly for 3 hours for 27 weeks.

9. Instructor Development Workshop

In the spring of 1976, follow-up week of training to the EPA Instructor Technique Course-B was the Instructor Development Workshop.

The workshop was designed to provide the necessary knowledge and skills to develop, validate and implement an effective instruction package for water and wastewater treatment programs.

10. Self-Paced Wastewater Treatment Plant Operator Course

To meet the needs of those operators who couldn't attend the Wasatch Front Operator Training Courses offered by the Division of Health, Joint School, USU, and others, the Division of Health developed a Self-Paced Wastewater Treatment Plant Operator Course and offered it in 1976.

The course requires students to submit preassignments and post assignments. After preassignments were corrected, the instructor would visit the plant and conduct the class. As a follow-up to the class, the student submitted a post assignment.

The whole course is geared to the operators' own plant. Preassignments usually describe the plant unit to be discussed and the post assignment lists the necessary operation, maintenance and troubleshooting procedures related to the unit. The course involved 19 operators.

The course was divided into 3 parts with a total of 20 lessons.

The New York Manual of Instruction for Sewage Treatment Plant Operators, the Sacramento State College Operation of Wastewater Treatment Plants, WPCF MOP 1, WPCF MOP 6, WPCF MOP 11, WPCF MOP 16, and WPCF MOP 18 were used for textbooks.

11. Appendix C1, C2 Engineering and Construction Contracts

In 1976, the State Division of Health held a seminar discussing the new regulations for EPA's construction grant program in Appendix C1, C2 Engineering and Construction Contracts for consulting engineers. The seminar discussed the new regulations and their effect on the consultants.

ENVIRONMENTAL PROTECTION AGENCY

In 1973, EPA sponsored a 3 day technology transfer seminar for "Upgrading Trickling Filter Plants, Physical-Chemical Treatment, and Upgrading Lagoons" for design engineers.

In 1974, EPA sponsored a 3 day technology transfer seminar at Utah State University for "Upgrading Wastewater Stabilization Ponds to Meet New Discharge Standards" for design engineers.

In 1976, EPA held a one day conference called "Operability, Flexibility, Maintainability of Wastewater Treatment Facilities." Its objective was to consider and discuss techniques to improve the operability, flexibility, and maintainability in the design of wastewater treatment facilities. Improved designs will help assure that those facilities can be operated at their designed level of efficiency and will meet all legal discharge requirements.

UTAH ENVIRONMENTAL SYSTEMS OPERATIONS TRAINING PROGRAM PLAN

The Utah Environmental Systems Operations Training Program (UESOTP) was developed in 1976 to coordinate and promote effective water, wastewater, solid waste and air pollution training programs offered in the State of Utah.

UESOTP was formed by the Utah State Division of Health, Utah Water Pollution Control Association, the Intermountain Section of the American Water Works Association, the Utah Refuse Collection and Disposal Association, and the Utah League of Cities and Towns, and is sponsored by Utah State University.

To this point, the UESOTP has coordinated the following wastewater training programs: USU Wastewater Safety Supervisors Course, the Division of Health's NPDES Self-Monitoring-Basic Laboratory Skills Workshop, the NPDES Permit Report Seminar in 1976 and the Division of Health's Wastewater Mathematics Course, the Joint School, and the NPDES Self-Monitoring-Basic Parameters for Municipal Effluents Workshop in 1977.

1. Utah State University's Wastewater Safety Supervisor Course

Utah State University conducted a Wastewater Safety Supervisor Course in 1976. Its purpose was to aquaint the safety supervisors with the UOSHA regulations and to help them organize and conduct their own safety programs.

The course had 12 sessions which met every other week for two hours and had 12 graduates. The course used the UOSHA regulations and related WPCF materials for the texts.

2. Division of Health's NPDES Self-Monitoring-Basic Laboratory Skills Workshop

The NPDES SELF-Monitoring-Basic Laboratory Skills Course was provided by the Division of Health under a special EPA grant to Red Rocks Community College in Denver, Colorado in 1976.

The course met for one week for 40 hours and had eight graduates. Purpose of the course was to provide the operators with the necessary laboratory skills to perform the laboratory testing required by the permit.

3. NPDES Permit Report Seminar

The NPDES Permit Report Seminar was held for one day in 1976. Personnel from the EPA Region VIII Permits Office conducted the one day seminar on "How to Properly Fill Out the NPDES Permit Form" in 1976. It was designed to help the permittee properly complete his self-monitoring report form. Forty-four people attended the seminar.

4. Division of Health's Wastewater Mathematics Course

In 1977, the Division of Health offered a Wastewater Mathematics Course. The course met 3 hours once a week for 16 weeks and had 12 graduates. The course provided the operators with instruction in mathematics necessary for operation, reports, and certification.

5. Division of Health's NPDES Self-Monitoring—Basic Parameter for Municipal Effluents Workshop

The second half of the laboratory training provided by the Division of Health under the EPA grant to Red Rocks Community College was conducted in 1977. The NPDES Self-Monitoring—Basic Parameter for Municipal Effluents Workshop met one week for 40 hours and had nine graduates.

This workshop required the operator to successfully perform tests for BOD, SS, pH, chlorine residual, and coliforms in order to graduate.

TIME LINE

1980	Salt Lake City - Collection System Ogden - Collection System
1938	Division of Health: Standards for home septic tanks prepared
1941-43	Military Installations - Hill AFB, Ogden Arsenal, Kearns, Wendover Air Base, Industrial Installations (Domestic sewage) - Dragerton, Horse Canyon, Geneva Steel
1944	League: Municipal Waterworks and Sanitation Conf.
1945	League: Municipal Waterworks and Sanitation Conf.
1946	League: Municipal Waterworks and Sanitation Conf.
1947	League: Municipal Waterworks and Sanitation Conf.
1948	League: Municipal Waterworks and Sanitation Conf.
1949	Nephi - First Modern Municipal Plant League: Municipal Waterworks and Sanitation Conf.
1950	U of U: Municipal Water and Sewage Works School League: Municipal Waterworks and Sanitation Conf.

1951 U of U: Municipal Water and Sewage Works School
League: Municipal Waterworks and Sanitation Conf.
"Tentative Standards for Sewage Works" adopted by State Department of Health. Water Pollution Control Legislation referred to Citizens Study Committee

1952 U of U: Municipal Water and Sewage Works School
League: Municipal Waterworks and Sanitation Conf.

1953 U of U: Municipal Water and Sewage Works School
Utah Water Pollution Control Act became law
League: Municipal Waterworks and Sanitation Conf.
"Standards for Sewage Works - Jan. 1954" - adopted by Utah Water Pollution Control Board - Dec. 18, 1953

1954 U of U: Municipal Water and Sewage Works School
"Water Classifications and Standards" - adopted by Utah Water Pollution Control Board - Feb. 4, 1954
League: Municipal Waterworks and Sanitation Conf.

1955 U of U: Municipal Water and Sewage Works School
League: Municipal Waterworks and Sanitation Conf.

1956 U of U: Municipal Water and Sewage Works School
United States Public Health Service - Construction Grants Program
League: Municipal Waterworks and Sanitation Conf.
Salt Lake City Suburban Sanitary District #1: In Plant Training Program

1957 U of U: Municipal Water and Sewage Works School
Utah Sewage and Industrial Wastes Association formed
League: Municipal Waterworks and Sanitation Conf.
Salt Lake City Suburban Sanitary District #1: In Plant Training Program

1958 U of U: Municipal Water and Sewage Works School
League: Municipal Waterworks and Sanitation Conf.

1959 U of U: Municipal Water and Sewage Works School
UWPCA: In Service Training Program
League: Municipal Waterworks and Sanitation Conf.

1960 U of U: Municipal Water and Sewage Works School
League: Municipal Waterworks and Sanitation Conf.
"Utah State Inter-Departmental Committee on Water Pollution" - established by Governor George D. Clyde

1961 U of U: Municipal Water and Sewage Works School
League: Municipal Waterworks and Sanitation Conf.

1962 U of U: Municipal Water and Sewage Works School
League: Municipal Waterworks and Sanitation Conf.

1963 U of U: Municipal Water and Sewage Works School
League: Municipal Waterworks and Sanitation Conf.

1964 U of U: Municipal Water and Sewage Works Operators
USU: Management Institute
UWPCA: Short School for Sewage Works Operators
League: Municipal Waterworks and Sanitation Conf.
Salt Lake City Suburban Sanitary Dist. #1: In Plant Training Program

1965 U of U: Municipal Water and Wastewater Works School
USU: Management Institute
Utah Code of Waste Disposal Regulations (Parts I, - V) adopted by Utah State Board of Health and Water Pollution Control Board (first official authorization for lagoons in Utah)
Voluntary Certification Plan adopted
League: Municipal Waterworks and Sanitation Conf.

1966 U of U: Municipal Water and Wastewater Works School
Certification Exam
Utah Water and Wastewater Operators Basic Training School
USU: Management Institute
League: Municipal Waterworks and Sanitation Conf.

1967 U of U: Municipal Water and Wastewater Works School
Certification Exams
Utah Water and Wastewater Operators Basic Training School
USU: Management Institute
League: Municipal Waterworks and Sanitation Conf.

1968 U of U: Municipal Water and Wastewater Works School
Certification Exams
USU: Management Institute
League: Municipal Water and Sewage Conference

1069 U of U: Municipal Water and Wastewater Works School
Certification Exams
USU: Management Institute
Utah Water and Wastewater Training Committee formed
League: Municipal Water and Sewage Conference

1970 U of U: Municipal Water and Wastewater Works School
Utah Water and Wastewater Joint Training School
Certification Exams
USU: Management Institute
E.P.A. formed

1971 U of U: Municipal Water and Wastewater Works School
Utah Water and Wastewater Joint Training School
Certification Exams
USU: Management Institute
Division of Health: 44 week Wastewater Treatment Plant Operators' On-the-Job Training Program
League: Municipal Water and Sewage Conference

1972 U of U: Municipal Water and Wastewater Works School
Utah Water and Wastewater Joint Training School
Certification Exams
USU: Management Institute
USU: U.S. Forest Service Lagoon and Package Plant Operations Short Course
Division of Health: Wastewater Treatment Plant Operators' On-the-Job Training Program
League: Municipal Water and Sewage Conference
Federal Clean Water Act Amendments

1973 Utah Water and Wastewater Joint Training School
Certification Exams
USU: Management Institute
Board of Certification joins ABC
League: Municipal Water and Sewage Conference
EPA: Upgrading Trickling Filter Plants, Physical and Chemical Treatment, and Upgrading Lagoons

1974 Utah Water and Wastewater Joint Training School
Certification Exams
USU: Management Institute
USU: Resource Utilization and Environmental Management Institute
Division of Health: Waste Stabilization Pond Seminars
EPA: Upgrading Wastewater Stabilization Ponds to Meet New Discharge Standards
Division of Health: Wastewater Treatment Plant Operators' On-the-Job Training Program

1975 Division of Health: Wastewater Treatment Plant Operator Short Course
Certification Exam
USU: Management Institute
Division of Health: Operation and Maintenance Manual Preparation Seminar
Division of Health: Instructor Techniques Course
USU: Resource Utilization and Environmental Management Institute
Division of Health: EPA Construction Grants Seminar
Division of Health: Introduction to Wastewater Treatment Plant Operation
Division of Health: Advanced-Wastewater Treatment Plant Operation
League: Water and Wastewater Section of Public Works Session

1976 Utah Water and Wastewater Joint Training School
Certification Exam
USU: Management Institute
Division of Health: Instructor Development Workshop

Division of Health: Self-Paced Wastewater Treatment Plant Operator Course

Division of Health: Appendix C-1, C-2 Engineering and Construction Contracts Seminar

Utah Environmental System Operation Training Program (UESOTP) formed

EPA: Operability, Flexibility, Maintainability of Wastewater Treatment Facilities

UESOTP: USU: Wastewater Safety Supervisors Course

League: Water and Wastewater Section of Public Works Session

UESOTP: Division of Health: NPDES Self-Monitoring Procedures - Course I Basic Laboratory skills

UESOTP: NPDES Report Seminar

1977 UESOTP: Utah Water and Wastewater Joint Training School

UESOTP: Division of Health: Wastewater Mathematics Course

UESOTP: Division of Health: NPDES Self-Monitoring Procedures - Course II Basic Parameters for Municipal Effluents

The Utah Environmental Systems Operations Training Program— A First Progress Report

*Norman B. Jones**

INTRODUCTION

In recent years the emergence of comprehensive federal and state legislation related to protection of the environment has created substantial demands and obligations for local governments and industry to upgrade and expand their facilities concerned with the areas of water pollution control, water supply, air quality control, and solid waste management. These legislative requirements have placed a heavy responsibility for compliance on the personnel who operate and manage such systems.

Operators of environmental control systems in Utah find themselves being required to achieve job performance levels significantly beyond the level expected of them just a few years ago. Most operators now recognize the need for additional training to achieve these higher performance levels, and indeed, are actively seeking such training opportunities. Additionally, elected officials, as they begin to understand their own responsibilities and obligations under the various laws, are finally sensing the need to encourage and support their personnel in acquiring additional training.

These developments, along with expectations of mandatory certification requirements in the near future, have created a real driving force for expanding and improving operator and management training opportunities.

HISTORY OF TRAINING IN UTAH

In the previous paper, Mr. Moehlmann has presented the historical development of water and wastewater operator training in Utah. Essentially, each training activity that he described emerged as a result of the needs of the time. Today, we stand on the threshold of another new era with respect to training needs in Utah. The task of responding, by developing these expanded training opportunities, presents an awesome challenge for those professionals concerned with providing this leadership. Most certainly, it will require a renewed and innovative approach, building on previous training efforts where possible, but demanding a much greater intensity of participation and cooperation by those organizations and individuals responsible for previous training efforts, such as our own Utah Water Pollution Control Association.

THE UTAH ENVIRONMENTAL SYSTEMS OPERATORS TRAINING PROGRAM

In late 1975, a group of individuals who had been working in various aspects of operator training recognized this need to upgrade and expand the training approach and began discussions related to establishment of a mechanism by which it would be possible to integrate, coordinate, upgrade and extend the impact of training in Utah.

In early 1976, the Utah Environmental Systems Operators Training Program (UESOTP) was conceived. A formal organizational framework and basic guidelines for implementing the program were developed. Letters of support were solicited from participating organizations in order to establish some official recognition and justification for the concept. The goals, objectives, guidelines, and organization (GOGO) of UESOTP as originally adopted are included in Appendix A.

OBJECTIVES AND CURRENT STATUS

The basic objective of UESOTP is to improve the competency and qualifications of water, wastewater and solid waste operations personnel in the operation, maintenance, and management of their facilities and systems through the promotion, development, coordination, and scheduling of appropriate training activities. A related purpose is to promote, support, and complement the objectives of the Utah Voluntary Certification Program for Water and Wastewater Works operators.

Participation in UESOTP is on a voluntary basis and the program and its Coordinating Training Committee (CTC) have no direct powers or controls over training activity in Utah. The basic premise of UESOTP is that by providing leadership in the development and coordination of meaningful training programs, all related training activities proposed will voluntarily utilize the UESOTP framework, its resources, and standards of training quality control as measured by the Continuing Education Unit (CEU).

DEVELOPMENTS SUPPORTIVE OF UESOTP

Two developments subsequent to the creation of UESOTP have added considerable credibility and reinforcement to the validity of its concept.

The first reinforcement for the UESOTP concept came with a report presented at the Water Pollution Control Federation Annual Conference in Minneapolis on October 3, 1976 by the Association of Boards Certification (ABC) entitled "Roles and Responsibilities for Developing a Comprehensive State Water and Wastewater Operator

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Training Program" (cover page and summary in Appendix B). The report findings and recommendations are essentially those adopted in the creation of UESOTP, which was developed independently and prior to the release of this important document.

A second supportive development occurred in November, 1976 when the Environmental Protection Agency, Region 8, Office of Technology Transfer and Manpower development awarded Utah State University a grant in the amount of $18,954 to assist in implementing the Utah Environmental Systems Operations Training Plan. These funds will be used for the acquisition of visual aid training equipment, training materials, travel support to assist in carrying training activity throughout the state, and miscellaneous operating expenses.

SUMMARY

UESOTP represents a well conceived and feasible approach to developing comprehensive training activity for environmental systems personnel in Utah. It can effectively meet the challenge of developing expanded and innovative training that is necessary for the future. Its organizational framework is flexible and adaptive for accommodating new and changing training needs. It has no official authority or power that can be construed or suggested as an attempt to "corner" Utah's training efforts and activity. It is a voluntary organization comprised of dedicated and competent persons, contributing their time and energy for one single purpose—improved performance of environmental control systems through effective training programs.

There is much to be done to achieve this objective. The success for UESOTP looks promising.

In my opinion, it is worthy of your support.

APPENDIX A

THE UTAH ENVIRONMENTAL SYSTEMS OPERATIONS TRAINING PLAN

1.0 Objective:

The basic purpose and objective of this training plan is to improve the competency and qualifications of water, wastewater and solid waste operations personnel in the operation, maintenance, and management of their facilities and systems through the promotion, development, coordination, and scheduling of appropriate training activities. A related purpose is to promote, support, and complement the objectives of the Utah Voluntary Certification Program for Water and Wastewater Works operators.

2.0 Sponsoring and Participating Organizations:

2.1 Utah State University, through its Cooperative Extension Service, will provide basic sponsorship, coordination, and liason for the training activity.

2.2 The following organizations, representing the essential elements and interests necessary to effectively carry out and achieve the stated objectives of the plan, are formally committed to active support of the program concept and training activities.*

2.21 American Water Works Association, Intermountain Section.

2.22 Utah Water Pollution Control Association.

2.23 Utah Refuse Collection and Disposal Association.

2.24 Utah State Division of Health.

2.25 Utah League of Cities and Towns.

2.3 Other organizations and institutions of higher learning will be utilized and involved where specialized training needs or liason is required or deemed appropriate in carrying out the training plan objectives.

3.0 Coordinating Training Committee (CTC)

3.1 A Coordinating Training Committee shall be created for the purpose of:

3.11 Implementing the plan by developing policy and guidelines for its operation.

3.12 Providing liason and feedback to the parent participating organization.

3.13 Providing leadership for the stimulation, development and coordination of training activity.

3.14 Assisting and supporting the Utah Voluntary Certificate program.

3.2 The Committee shall consist of members as follows:

3.21 Two members representing the American Water Works Association, Intermountain Section.

3.22 Two members representing the Utah Water Pollution Control Association.

3.23 One member representing the Utah Refuse Collection and Disposal Association.

3.24 Three members representing the Utah State Division of Health.

3.25 One member representing the Utah League Cities and Towns.

3.26 One member representing the Utah State University Cooperative Extension Service.

*See Appendix A for support letters of intent from participating organizations.

3.3 The committee members shall be appointed by the parent organizations that they represent for a time period deemed appropriate by the parent organization.

4.0 Specific Objectives of the Coordinating Training Committee

4.1 Identification, Coordination, and Scheduling of Training

4.11 Develop a physical inventory of environmental systems operations and personnel for entire state.

4.12 Identify needs and priorities for specific training offerings as a basis for scheduling of training courses.

4.13 Identify and establish regional training centers suitable and equitable for servicing the training requirements of the entire state.

4.14 Develop an operator's Newsletter as a basic mechanism for communications, for improvement of operator morale and professional image, and for general public relation purposes.

4.2 Training Courses, Materials, Resources

4.21 Identify, outline and describe the types of training courses and activities necessary to carry out program objectives.

4.22 Identify, acquire and make available existing relevant training materials and resources.

4.23 Develop new training materials and resources where needed, including correspondence courses, special workshops, etc.

4.24 Actively seek sources of funding where necessary for improvement in the quality and scope of training activities.

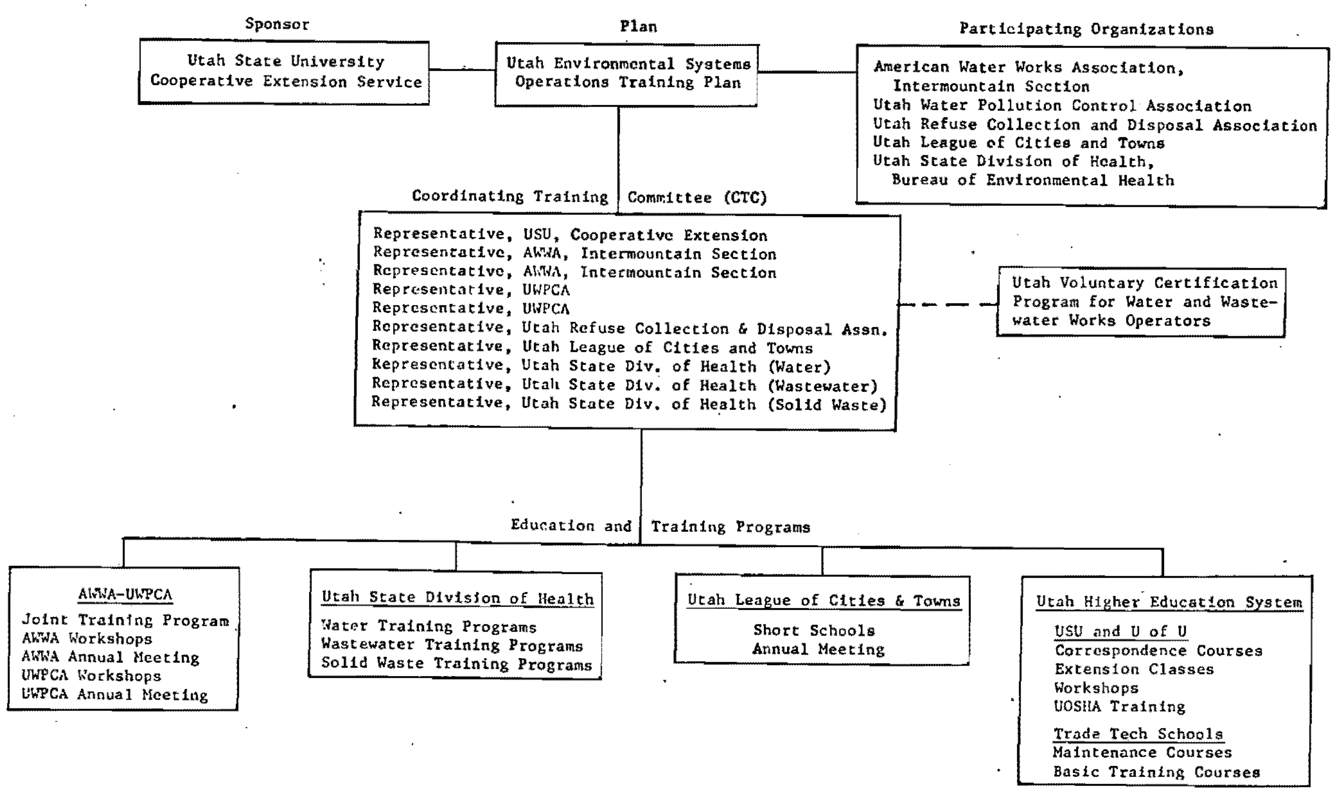
4.3 Relationship to Utah Voluntary Certification Board

4.31 Enhance operator progression toward certification by coordinating training programs with standards and policies of the board.

4.32 Assist Certification Board in recording and documenting professional advancement by implementing the Continuing Education Unit (CEU) concept in association with all training activity.

4.33 Improve communications between operators and the Certification Board by providing Newsletter liason between training and certification activity.

4.34 Continually review certification exam results as one criterion for evaluating training effectiveness.



APPENDIX B

ROLES AND RESPONSIBILITIES FOR DEVELOPING A COMPREHENSIVE STATE WATER AND WASTEWATER OPERATOR TRAINING PROGRAM

Project Report
by the
Association of Boards of Certification
for Operating Personnel
in Water and Wastewater Utilities
ABC Administrative Office
Municipal Building
Ames, Iowa 50010
Robert L. Wubbena, Project Director

for the
Office of Water Program Operations
U.S. Environmental Protection Agency
Grant No. T900661010
July, 1976

TO THE READER:

Numerous studies have shown that many water and wastewater facilities are not meeting design criteria due to poor operation. State Boards for Certification have shown through their examinations and evaluation procedures that many operators may lack the skills and ability to provide proper operation.

This report discusses what is involved in the development of an adequate level of training and education for these operators; it also identifies the roles and responsibilities for the participants in the program development and is intended for use by the following organizations:

AWWA (and Local Sections)
EPA
State Regulatory Agencies
State Boards of Certification
Nongovernmental Training Organizations
Utility Management
WPCF (and Local Associations)
EPS (Environment Canada)
Provincial Regulatory Agencies
Provincial Boards of Certification
Federal Agencies
Education and Vocational Training Institutions
Professional Societies and Organizations

The project was initiated by ABC on behalf of the state and provincial boards of certification. It was financed by a grant from EPA. The study and preparation of the report was directed by a joint committee for AWWA, WPCF, four states, and a representative of the Canadian Government.

The Committee apreciates that the recommendations will not solve all operator training problems. Their implementation will, however, establish much needed coordination and provide a training system that will permit nation-wide pooling and sharing of resources. We urge you full cooperation in achieving these goals.

PROJECT STEERING COMMITTEE
R.L. Wubbena, Project Director
Samuel S. Baxter
E.H. Braatelien, Jr.
William R. Hill
Peter Mack
Alex B. Redekopp
Sam L. Warrington

SUMMARY

Managers and operators of water supply and wastewater utilities need to be highly qualified to achieve effective and economic operation of their facilities in accordance with current day practices and standards.

The independent effort of most states and Canadian provinces to provide essential training has been inadequate. Their experiences clearly indicate the need for leadership to (1) coordinate independent efforts and direct the sharing of their resources, (2) provide guidance for the numerous participants in the development of new training material, and (3) develop training to meet the needs being identified by certification programs. Establishment of this leadership and the provision of some beginning guidelines is the purpose of this study and report.

Recognizing their capabilities for providing this leadership, the American Water Works Association (AWWA), the Water Pollution Control Federation (WPCF), and the Association of Boards of Certification (ABC), appointed representatives to participate in a joint study funded by an Environmental Protection Agency (EPA) grant. They were assisted by four persons knowledgeable of specific state or provincial training activities.

The study is the first nationwide comprehensive analysis of the availability of training and problems associated with the development of effective state programs.

Based on the analysis of 4 state programs and a survey of the other states and 10 provinces, the report identifies the responsibilities and methods for utility management, educational institutions, regulatory agencies, consulting engineers, professional organizations and others in meeting the need.

Included in the report are specific recommendations for action that include the following:

1. That better working relationships be established between state agencies, local units of AWWA and WPCF, colleges and vocational training institutions, industry, consulting engineers, and ongoing training programs in related fields to improve training opportunities;

2. That legal mandates and authorizations be secured to assist in implementation of training;

3. That the basic state training programs be funded by program budget funds and tuition and not by federal grants;

4. That all training material development by AWWA, WPCF and the federal and state governments be based upon the "need to know" criteria developed by ABC and validated by AWWA, WPCF, and other recognized experts.

5. That all new training material be developed in modular form and be assigned a CEU value;

6. That a "means to coordinate training" be established in each state to promote the use of all available training; and

7. That a national committee that includes representation from AWWA, WPCF, ABC, and others be established to provide national leadership in the development of a comprehensive operator training program.

Plant Start-up of the Salina, Utah, Sewage Treatment Plant

*Douglas D. Drury, William J. Spear, and Robert M. McEown**

INTRODUCTION

The design and construction of unique or different processes for small towns often results in operational problems and in poor quality effluent for the sewage plant. The design engineer can insure that these treatment processes will operate as designed by conducting plant start-up and training the operator in the proper control of the treatment process. An aerated equalization basin was designed and constructed for the Salina Sewage Treatment Plant. Plant start-up and operator training began after the plant did not produce the removal efficiencies which were anticipated. Plant start-up identified and corrected many operational problems and design deficiencies. This should enable the treatment plant to meet its discharge permit as originally designed.

Salina City is a town approximately 2,000 people located in south central Utah on the western edge of coal country. Over the last year Salina City has experienced a tremendous growth and development. There were 64 new connections in 1976 alone. This represented a 15 percent increase in total connections in 1976 alone.

HISTORY

The Salina City Sewage Treatment Plant was constructed in 1960 for a design flow of 0.3 MGD. At that time the plant consisted of bar screens, grit removal chambers, a primary clarifier, a standard rate trickling filter, a secondary clarifier, and an anaerobic digester.

The major contributor of flow and BOD to the plant was the local turkey processing plant. The plant operated approximately 10 hours per day from June through November. The turkey plant produced flows in excess of 0.5 MGD and averaged over 250,000 gallons per day. The sewage from the turkey plant would then flow through the sewage treatment plant over a 12-hour period. The BOD's of the sewage from the turkey plant averaged about 400 mg/l with peaks in excess of 1300 mg/l being observed. At that time the city did not employ a full time operator. The operation consisted of the City Maintenance Foreman visiting the plant twice a day to make sure the pumps were running. Needless to say, the plant experienced serious operating problems. By the early 1970s the BOD's and TSS in the effluent of the sewage treatment plant averaged over 50 mg/l with peaks in excess of 150 mg/l being noted. In addition, total coliform concentrations in excess of 230,000 #/100 ml were observed

*Douglas D. Drury is presently the Sanitary Engineer and Operation and Maintenance Specialist for Valley Engineering, Inc. of Logan, Utah. William J. Spear is President of the Project Services, Inc., of Salt Lake City, Utah. Robert McEown in the Plant Operator of the Salina, Utah, Sewage Treatment Plant.

DESIGN

In 1972 Salina City hired Canyon Lands Engineering to design modifications to the plant. By 1973 the design was finished. It was decided that the new construction to the sewage treatment plant should be staged. The initial step would consist of an aerated equalization basin, a recirculation pump station for the trickling filter and repair of the anaerobic digester. The flow diagram for the plant is included in Figure 1. The aerated equalization basin would be used to dampen out the high flows and BOD's produced by the turkey plant during the day. At the same time the turkey plant planned to implement a water conservation program and install flotation tank for the removal of grease and feathers. The plant would then be reevaluated after construction had been completed and after changes had been made within the turkey plant. It was felt that with these changes the plant could meet its N.P.D.E.S. Permit requirements of 25 mg/l for BOD and TSS and 2000 and 200 #ml for total fecal coliforms respectively. The construction at the plant was completed by the spring of 1976. During construction a full time operator was hired from the local community. This man had no previous experience in the operation of sewage treatment plants. The plant went on line in June 1976.

PLANT START-UP

Initially, the plant operated as expected. A characterization of the sewage flows entering the plant during this period appears in Table 1. The flows averaged over 0.3 MGD during the summer. The effluent BOD and TSS for the month of July averaged 19 mg/l and 22 mg/l respectively, well within the permit limits of 25 mg/l. However, there were indications of problems. The dissolved oxygen concentrations in the aerated equalization basin were at 1.0 mg/l and were dropping. By August the D.O. concentration in the aerated equalization basin dropped to only a trace. This adversely affected the effluent quality of the plant. The average BOD and TSS for August were respectively 32 mg/l and 42 mg/l.

At this time, Salina City sought assistance in the operation of the new facility and in training their operator. Plant start-up at the Salina Plant began September 1, 1976, by Valley Engineering, Inc. As a result in the plant start-up the following operational changes were made:

1. More frequent cleaning of grit chambers
2. Minimize recycling of flow back to the head of the plant
3. Optimized the recirculation of flow around the trickling filter
4. Operation of aeration basin as an equalization basin
5. Optimization of wet all pumping

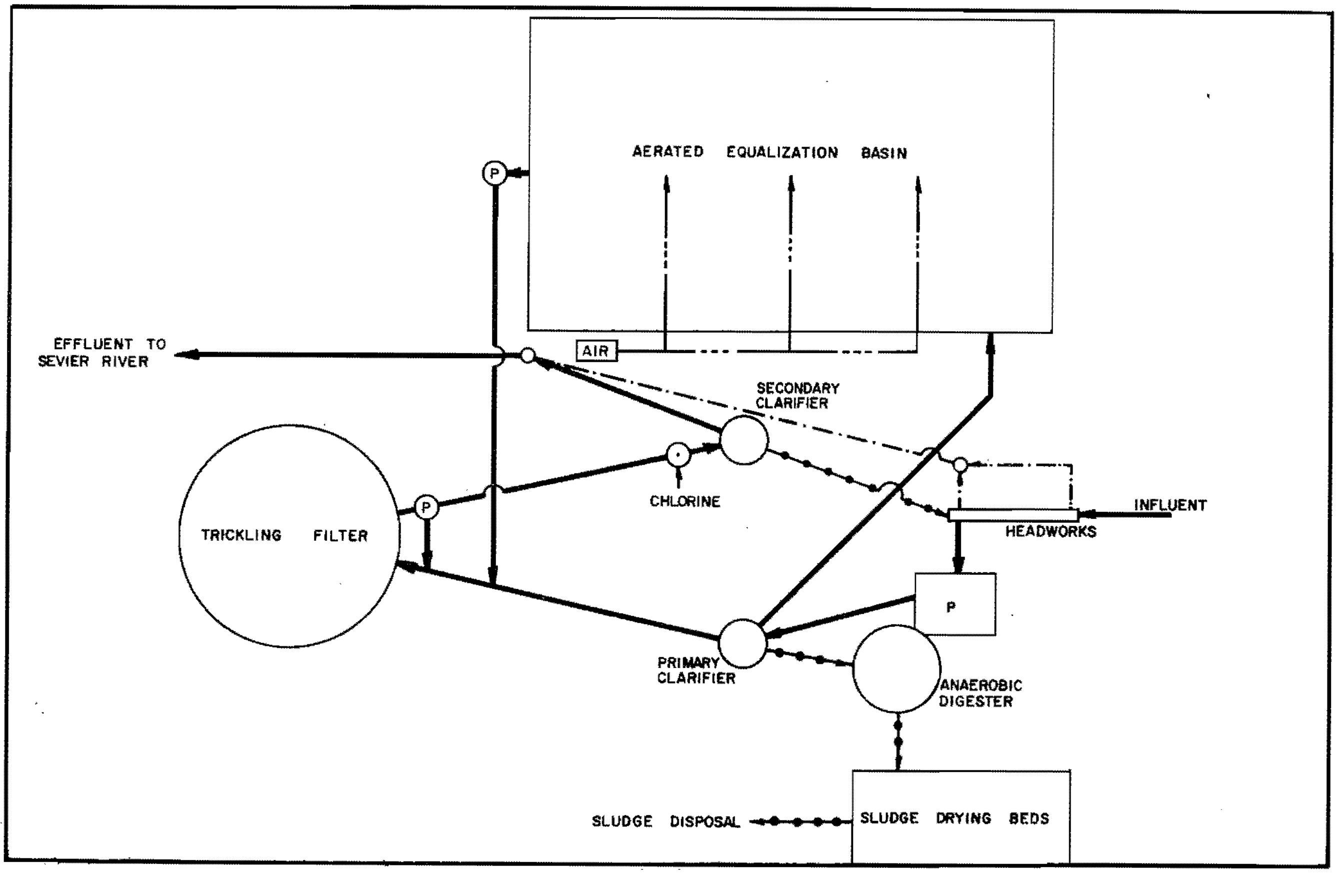


Figure 1. Flow diagram, Salina Sewage Treatment Plant.

Table 1. Characterization of the sewage flows entering the Salina Sewage Treatment Plant in 1976 during the plant start-up.

Influent BOD with turkey plant	= 200-400 mg/l
Influent TSS " " "	= 200-400 mg/l
Influent BOD without turkey plant	= 150-200 mg/l
Influent TSS " " "	= 150-200 mg/l
Maximum Daily Flow with turkey plant	= 0.55 MGD
Average Daily Flow " " "	> 0.3 MGD
Minimum Daily Flow " " "	= 0.05 MGD
Maximum Daily Flow without turkey plant	= 0.27 MGD
Average Daily Flow " " "	= 0.1 MGD
Minimum Daily Flow " " "	= 0.05 MGD

In addition to the operating changes, the plant start-up identified several piping and process changes which were necessary. The following modifications are presently being made to the plant by means of a change order to their existing EPA grant:

1. Install new influent flow recorder and totalizer
2. Modify Plant by-pass to flow to the aerated equalization basin
3. Modify existing wet wall pumps
4. Install new aeration system with increased capacity in the equalization basin
5. Place riprap in the equalization basin down to the bottom
6. Modify blower buildings
7. Pipe supernatant return to the wet well
8. Install sight glass in secondary sludge line
9. Construct new wash water system
10. Modify chlorination system to include post clarification chlorination
11. Obtain laboratory equipment

Because of the difficulty in separating the operational changes from the physical changes needed for a better operation, the major items will be discussed together in an order which will reflect the flow through the plant.

The sewage enters the plant and immediately flows through the Parshall Flume. Two problems were noted with the flume and the flow recorder. The flow entering the flume was not being evenly distributed across the throat of the flume. This problem was caused by the influent pipe not being in alignment with the throat of the flume, thereby causing the water "to bank" on one side of the flume. This problem was drastically reduced by taking a sludge hammer and knocking out one side of the pipe. This produced a more uniform distribution of flow across the flume. The second problem with the meter was with the recorder-totalizer. It needed frequent calibration, and the totalizer was always giving erroneous readings. Parts couldn't be obtained for repair because the meter was obsolete. The only way to solve this problem was to totally replace the recorder-totalizer. The correction of these problems is paramount to the proper operation of the plant. Because only by knowing the flow entering the plant will the operator be able to obtain total equalization of flow. Because of the problems with the flow meter the flows reported in this paper are at best only good estimates of the actual flow.

The sewage then flows out of the Parshall Flume and into the grit channels. Because of the turkey plant operation, the sewage contains large amounts of grit. It was found that the operator was not cleaning the channels frequently enough, thereby allowing grit to enter the wet well. As a result of plant start-up the grit channels are now cleaned two to three times per week during the turkey season.

Two problems were noted with the operation of the wet well. The first problem was the result of an inoperative float valve on the secondary sludge return line. The float had been removed and the operator was returning a constant amount of sludge to the wet well. The recirculation was excessive and when combined with high flows from the turkey plant during the day, it produced flows estimated to be in excess of 600 GPM. The sewage is pumped out of the wet well using two float activated 450 GPM pumps. However, the discharge capacity of one of the pumps had been reduced to about 300-350 GPM, because of wear on the impellar from pumping excessive amounts of grit. When high flows occurred both pumps would kick on and they would pump about 750 to 800 GPM to the primary clarifier. In order to correct this problem, it was necessary to minimize the recirculated flow during high flow periods and obtain better control of the wet well pumps. The float valve was repaired and was set to recirculate flows only at low flow periods which occurred during the night when the turkey plant was not operating. The secondary sludge was then returned to the wet well several times each day by manually opening the float valve. Once the recirculation flows were reduced, one of the wet well pumps could handle the entire flow. It was decided that the best operation for pumping from the wet well would be to produce a uniform flow to the primary clarifier. In order to do this it would be necessary to reduce the flow of the wet well pump so that it would run continuously and not kick off and on. This was done by partially closing a valve on the discharge side of the pump and making the discharge from the wet well pump equal the average flow produced by the turkey plant. After several weeks of trial and error, the desired operation was achieved. One pump ran constantly and variations in flow resulted in fluctuations in the water level of the wet well. Only on rare occasions would the water level raise high enough to turn on the second pump.

Before the changes were made in the operation of the wet well, the excessive flows being pumped to the primary clarifier resulted in operational problems in the primary clarifier. During high flow conditions it was estimated that the surface settling rate was 2400 GPD/ft^2 and the detention time was less than 30 minutes. In addition, during this time there was extensive short circuiting. This problem was probably best characterized by the black grit which would collect in the weir troughs of the primary clarifier. Needless to say, the overall removal efficiencies for BOD and TSS were dramatically reduced. The increased BOD loading to the aerated equalization contributed to its operation problems. Once constant flows were pumped to the primary clarifier, the operation returned to normal conditions. Settleable solids removal are now greater than 90 percent and TSS removals are approximately 50 percent.

The previously mentioned problems of anaerobic conditions in the aerated equalization basin were the direct result of an engineering design oversight. However, it should be mentioned that the inefficient operation of the primary clarifier did not help this problem. Review of the design calculations showed that the aeration system was inadequately designed and that adequate amounts of oxygen could not be supplied by the existing aerator. In addition, the aeration system selected would not allow equalization. The State Health Department's requirements of 3 feet for freeboard resulted in a minimum depth of 5 feet. The minimum suggested operation depth by the manufacturer was 5 feet. This resulted in the maximum and minimum depth being the same. Therefore, equalization could not occur without a loss of aeration efficiency. While a second compressor was available for increased air flow, it could not be used with the system because of excessive pressures which developed. During anaerobic conditions, excessive amounts of sludge from the bottom would rise to the surface of the equalization basin. The equalization then pumped it to the trickling filter, where it would clog the orifices in the rotary distributor. At times it was necessary to clean the distributor arm twice a day. In addition, it was apparent that the anaerobic conditions in the equalization basin significantly reduce the BOD removal efficiencies of the trickling filter process. In order to solve these problems, it will be necessary to install a new aerator with increased capacity and which will lay on the bottom of the basin. The existing aeration system will be used but will be modified so that excessive pressures will not develop when the second compressor is running. The end result of these modifications to the equalization basin will be: 1) to allow the depth to fluctuate, thereby obtaining equalization and improving the performance of the trickling filter and the secondary clarifier; 2) to provide increased aeration capacity, thereby eliminating anaerobic conditions; 3) allow both blowers to run simultaneously, if necessary.

The trickling filter is a 105 foot diameter standard rate trickling filter. It was initially designed very conservatively. It will take flows up to 0.9 MGD and still be classified as a standard rate filter. It was for this reason that a recirculation pump station around the trickling filter was designed and constructed. Soon after construction, problems surfaced with the automatic control and operation of the two pumps. It was determined that the float switches had been wired as if the pumps were to act as wet well pumps. As a result the recirculation pumps would turn on as the flows increased. This is exactly opposite of the proper operation. The system is now operated manually with one pump running continuously. The system will be rewired, so that the second pump will turn on if low flow conditions occur. It should be mentioned that besides providing for increased BOD removal efficiencies, recirculation also helps keep the distributor arm moving during low flow periods when the turkey plant is not operating.

Another problem which was identified during plant start-up was the erratic concentrations of total and fecal coliform in the plant's effluent. The variation was attributed to the pre-clarification chlorination process which was being used at the plant. At the same time the problem wasn't considered critical. The 1.0 mg/l chlorine dosage and the total and fecal coliforms were for the most part kept under the permit limitation of 5000 and 500 #/100 ml respectively. However, the permit conditions were going to change on June 30, 1977. The chlorine residual requirement would then be lowered to 0.5 mg/l and the total and fecal coliforms concentrations would be lowered to 2000 and 200 #/100 ml respectively. It was felt that at this time the permit conditions would be extremely hard to meet with the existing preclarification chlorination process. Since the chlorination process is much more efficient, once the solids have been removed, the permit limitation after June 30, 1977 could be met by post-clarification chlorination. Fortunately for Salina, they have a 1450 foot 15 inch sewer outfall line to the Sevier River. Flowing full, this line can provide over 1 hours detention time and could be utilized as a chlorine contact chamber. In order to use the outfall line, it will be necessary to put a riser on the end of the pipe, thus backing up the water in the pipe and allowing it to flow full. The chlorine feed line will then be extended to the secondary clarifier effluent box. The modified chlorination system will then allow for pre-and/or post-clarification chlorination.

Proper operation of a sewage treatment plant requires the laboratory analysis of various chemical parameters and the Salina sewage treatment plant is no different. During plant start-up the laboratory was found to be deficient. As a result much laboratory equipment was purchased and is now being used. Some of the more notable pieces of laboratory equipment were the dissolved oxygen meter, the pH meter, the turbidimeter and a DPD (Diethyl-p-penylene Diamine) free and total chlorine test kit. The dissolved oxygen meter became invaluable in trouble shooting the problems with the aerated equalization basin. It was used daily during the time the aerated equalization basin was in service. The pH meter was obtained because a daily pH measurement is required by the discharge permit and a more accurate measurement could be obtained at the plant than could be obtained by shipping a sample to a commercial laboratory in Salt Lake City. The turbidimeter will be used to evaluate the performance of the secondary clarifier. In the past the operator has used the settleable solids test for this purpose. But the test becomes useless when the concentration drops below 0.1 ml/l. The turbidimeter will allow the operator to monitor the operation of the secondary clarifier when the suspended solids are very low. The DPD chlorine test kit was obtained because the orthotolidine test kit the plant had did not meet the requirements of its discharge permit.

In addition to the changes already mentioned, many piping changes were made. These changes resulted in the virtual elimination of plant by-passes and easier operation and maintenance of the facilities.

RESULTS

It is very difficult to state what all the exact results of the plant start-up will be. The operator now understands the various processes and is aware of factors controlling their operation. He is now capable of evaluating each process and obtaining optimum operating conditions. Work is still progressing and even after completion, additional time will be required for full evaluation. Significant improvements have been made in the performance of the plant. It is anticipated that the plant will meet its June 30, 1977, discharge requirements for BOD, TSS, chlorine residual and total and fecal coliforms. This could not have been accomplished without the plant start-up and operator training.

CONCLUSIONS

1. When working with smaller towns, the engineer must follow up on unique or nonstandard designs to insure that they will be operated as designed.

2. Often the operator will not have previous operating experience and will require training.

3. Plant start-up and operator training at the Salina sewage treatment plant resulted in the proper operation in the equalization process as well as the other processes in the plant.

4. The improper operation of wet well pumping and excessive recirculation resulted in operational problems for the primary clarifier, which aggrevated the already existing operational problems within the equalization basin, which in turn created operational problems for the trickling filter and created excessive BOD concentrations in the effluent of the plant.

Panel Discussion 208 and Regionalization in Mountainland

*Stephen E. Sowby, P.E.**

ROLE OF MOUNTAINLAND ASSOCIATION OF GOVERNMENTS IN WATER POLLUTION CONTROL

In 1974, elected officials in the MAG area determined that they should be involved in water quality management because of the interest and desire of their member cities and counties. EPA provided funding in 1975 for a 208 areawide water quality management study. MAG is made up of Summit, Utah, and Wasatch Counties and the following communities in those counties.

Coalville	Midway	Springville
Oakley	Lehi	Mapleton
Kamas	American Fork	Spanish Fork
Francis	Pleasant Grove	Salem
Park City	Alpine	Payson
Snyderville	Orem	Spring Lake
Charleston	Provo	Santaquin
Henefer	Heber	Lindon
Salem Hills		

At this stage, it may be well to state that MAG is not another layer of government nor is it a regional government. The association is exactly that—an association—made up of and working for local cities and counties. MAG is working for local government in trying to reduce expenditures while achieving cost-effectiveness and good operating conditions. The association staff does only those things that are requested by the member agencies, and all staff action and plans, including regionalization of wastewater facilities, must be approved by the advisory and executive committees made up of elected officials.

REGIONALIZATION AND 208

At the outset of this presentation, may I state that 208 does not necessarily mean regionalization. Somehow these two words seem to be equated. This simply is not so. The goal of Section 208 of P.L. 92-500 is not to regionalize wastewater treatment plants and do away with the sovereignty of individual cities and towns. Rather, it is an areawide water quality management program to intensively study the overall water pollution control and management problems. Section 303 is a basin approach and Section 201 provides for individual facility construction funds. So let's not automatically equate 208 with regionalization although it has taken 208 studies and staffs to achieve regional cooperation in most cases.

NON-POINT SOURCE POLLUTION

Perhaps the greatest contribution of 208 is in the evaluation, identification, and monitoring of non-point sources of pollution. Very little was said prior to 208 about non-point pollution—not to even mention management of these fugitive sources. Now, however, we recognize the magnitude of these problems and the need to manage and control them. However, our purpose today is not to discuss these non-point sources. So let's turn our attention to what the 208 Study recommends for point source management.

COST EFFECTIVENESS

One main requirement of both 208 and 201 is to develop cost-effective plans for wastewater treatment facilities. This involves studying alternatives to combine municipal plants. Both the 303, the 201's, and now the 208 basically confirm the findings of each other. Regionalization is cost-effective in most cases, especially with new higher levels of treatment required by the State of Utah. With more sophisticated processes, higher O&M costs, expanding population, accelerated construction costs, and ensuing management problems, regionalization in the three-county area of Mountainland is cost-effective.

RESULTS OF THE 208 STUDY

Now for a few facts and figures: These will be general in nature and more specific answers can be given in the Q & A period. Suffice it to say that not everything is set in concrete and many decisions are yet to be made by the local elected officials in the affected cities and counties. The information presented below represents the best recommendations of the MAG 208 Study.

NUMBER AND LOCATION OF FACILITIES

There are now 26 cities and towns in the MAG area, 17 of which had sewer collection systems and treatment facilities last year. It is anticipated that by 1995 there will be 11 or 12 wastewater treatment facilities serving 25 communities. Seven of these 11 plants will incorporate some form of regionalization while four will remain a facility for an individual city only. In other words, regionalization is beneficial to more than half of the communities of Summit, Utah, and Wasatch Counties. Where possible, MAG is merely adopting the completed 201 plans and regionalization schemes as part of the 208 areawide plan.

TREATMENT PROCESSES AND CONSTRUCTION DATES

Construction of these facilities will take place between now and 1985 depending on federal funding. Treatment processes will vary, including single stage trickling filters, two-stage trickling filters, activated sludge, extended aeration oxidation ditches, phosphorus removal, and granular media final filters. Individual schedules and processes will be answered at your requests. Infiltration and storm water are also handled differently in each case, and expansion of collection and treatment facilities will proceed as the need warrants.

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SERVICE CHARGES, BILLINGS, AND MANAGEMENT

Service charges will range from between $4 to $15 per month for residential users with connection fees ranging from $200 - $1,500. Billings in most cases will be done by the existing municipalities, using existing personnel. Management of the collection facilities varies widely with the locality. Included will be existing municipalities, cities, county service areas and sewer districts, special service districts, or a combination of the above. Some management boards are appointed, some elected, with local mayors and city councilmen on the board in most cases.

OPERATIONS

There are now 35 operators at the 17 present wastewater treatment facilities and it is anticipated that there will be a minimum of 60 operators required for the 11 or 12 facilities by 1995. This does not include collection system personnel or other water treatment or distribution system operators. Some supervisors, managers and lab technicians have been included. This is a low figure and could go as high as 80 under full capacity, 24-hour operation, and expanded monitoring requirements. This represents roughly a doubling in the need for trained, qualified personnel.

TRAINING

It can be easily seen that there will be no loss of operator jobs. There is, indeed, a need for more and better-trained operators for water distribution and wastewater facilities. Operators need to be educated and trained in management, new treatment processes, repair and maintenance, electrical systems, and communication skills. Newer equipment and better working conditions will result and personnel need to be flexible and adaptable to changing conditions. Training opportunities are expanding under programs of Utah State University and Utah Technical College. Certification is becoming more necessary and continuing education is a must. Repairs will always be necessary as will maintenance of existing equipment.

CONCLUSION

Specific information on regionalization on a plant-by-plant basis can be obtained from the MAG staff or individual city concerned; but regionalization is becoming a fact of life. There will continue to be a need for good quality operators and training is essential. Sewer service costs will continue to rise and population will continue to grow. You, the operator, are an essential part of this entire process.

Wastewater Filtration, Design Considerations

*E. Robert Baumann**

INTRODUCTION

Wastewater filtration is but one of the design engineer's alternatives which can be considered in wastewater treatment flow schemes to meet specified effluent quality objectives. He should consider it along with other alternatives, finally reaching a decision as to which of the several alternatives is cost effective. This paper presents the questions which must be asked in wastewater filtration, the alternatives available in answering the questions, and the design procedures involved in those alternatives.

The paper presumes that the reader is familiar with granular media filtration from potable water experience or from study of textbook sources (1), and therefore stresses the special aspects related to wastewater filtration. Typical wastewater filtration flow schemes are shown in Figure 1.

The first and most important question the designer must ask is whether filtration can meet the specified effluent quality goals. If the goal is to upgrade the effluent of an existing secondary treatment works, one must first evaluate the present performance and the reasons for that performance. For example, what portions of the present effluent BOD are of soluble and suspended origin? The filter can only remove a portion of the suspended BOD. If the effluent contains high soluble BOD, the only solution may be to upgrade the secondary treatment. If the effluent contains primarily suspended BOD, effluent filtration or upgrading the secondary settling will be possible alternative solutions.

The expected performance of the granular filters can be estimated from the performance at similar plants elsewhere, or by pilot studies at the plant in question. Similar compilations with more data from U.S. activated sludge plants are available in recent EPA Design Manuals (2,3). The mean range of the performance data from these two sources is summarized in Table 1.

The data in Table 1 and the sources from which it was derived indicate that a marginal secondary effluent could easily be upgraded to a 30-30 standard, and probably to a 20-20 standard, by tertiary filtration, i.e., without chemical treatment. A good secondary effluent which already meets the 30-30 standard may approach a 10-10 goal by tertiary filtration. If the effluent quality goal is less than 10-10, some form of chemical treatment will be needed in the secondary or in a tertiary stage prior to filtration.

After considering the effluent quality goals and the ability of granular filtration to achieve those goals, if

Table 1. Median range of performance of wastewater filters, combined data from Appendix A and Reference 2. The data below give the range of mean values and the median of the means including all filtration rates from 2 to 6 gpm/sq ft (inclusive) and media sizes 1 mm effective size.

Filter Influent Type	Suspended Solids (mg/l) Influent Range	Median	Suspended Solids (mg/l) Effluent Range	Median	BOD_5 (mg/l) Influent Range	Median	BOD_5 (mg/l) Effluent Range	Median
Tertiary Filtration of Trickling Filter Plant Effluent	20-51	29	5-13	9	23-35	30	10-14	12
n = (number of observations	n = 31		n = 31		n = 6		n = 6	
Tertiary Filtration of Activated Sludge Plant Effluent	7-55	16	2-10	6	No Data			
	n = 23		n = 23					
Filtration After Chemical Treatment of Secondary Effluent	6-16	10	1-8	1.5	No Data			
	n = 7		n = 6					

filtration is still one viable alternative, the following design questions must be considered in arriving at a successful installation.

1. What are the appropriate flow schemes?
2. What minimum filter run length is acceptable?
3. What filter configurations are appropriate for wastewater?
4. Is pilot scale testing needed, and if so, how should it be conducted?
5. What filtration rate and terminal headloss should be provided?
6. What filter media size and depth should be provided?
7. Should gravity or pressure filters be provided?
8. What system of flow control should be used?
9. What backwash provisions are needed for each filter media alternative being considered to ensure effective backwashing in the long term?
10. What underdrain system is appropriate for the media and backwash regime intended?

FILTER DESIGN—GENERAL CONSIDERATIONS

Minimum Acceptable Filter Run Length

Since the capital cost of a filter is chiefly a function of the area of filter provided, the use of a high filtration rate is usually preferred. In general, the filter design should seek to maximize the net water production per square foot of filter consistent with filter operating feasibility. Useful relationships between net water production and run lengths obtained at different filtration rates are shown in Figures 2 and 3. Two alternatives exist. The first case shown in Figure 2, occurs when filtered water is used for backwashing as in all potable water filtration and most

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wastewater filtration plants. The second case, shown in Figure 3, occurs when unfiltered water is used for backwashing.

The latter method is used for some wastewater filters. However, it is not generally recommended because of potential clogging of underdrain strainer or orifice openings.

The data for both figures was calculated assuming 30 minutes total down time per backwash to allow for draindown time, auxiliary scour time, actual backwash time and start-up time to reach normal rate. Also, the 100 gal/ft^2 total wash water per backwash is typical of volumes adequate for most filtration situations. In the case of recovered wash water, it is assumed that dirty wash water would pass through a holding tank to permit flow equalization of the recirculated water. An example calculation for Figure 2 is shown below:

Backwashes per day (6-hour cycles) = 4
Downtime per backwash = 30 min
Actual filtration time (1,440 - 4 x 30) = 1320 min
Plant production = 4 gal/min/ft^2 x 1440 min/day = 5760 gal/day/ft^2
Backwash water used = 100 x 4 = 400 gal
Needed filtration rate during actual operating time = (5760 gal + 400 gal)/1320 = 4.67 gal/min/ft^2
Backwash water as percent of production = (400/5760) x 100 = 6.9 percent

Figure 2 which is appropriate for most wastewater filtration, would indicate little loss of production if the number of backwash cycles per day per filter is limited to four or less, i.e., 6 hour filter cycles or longer. Thus, under

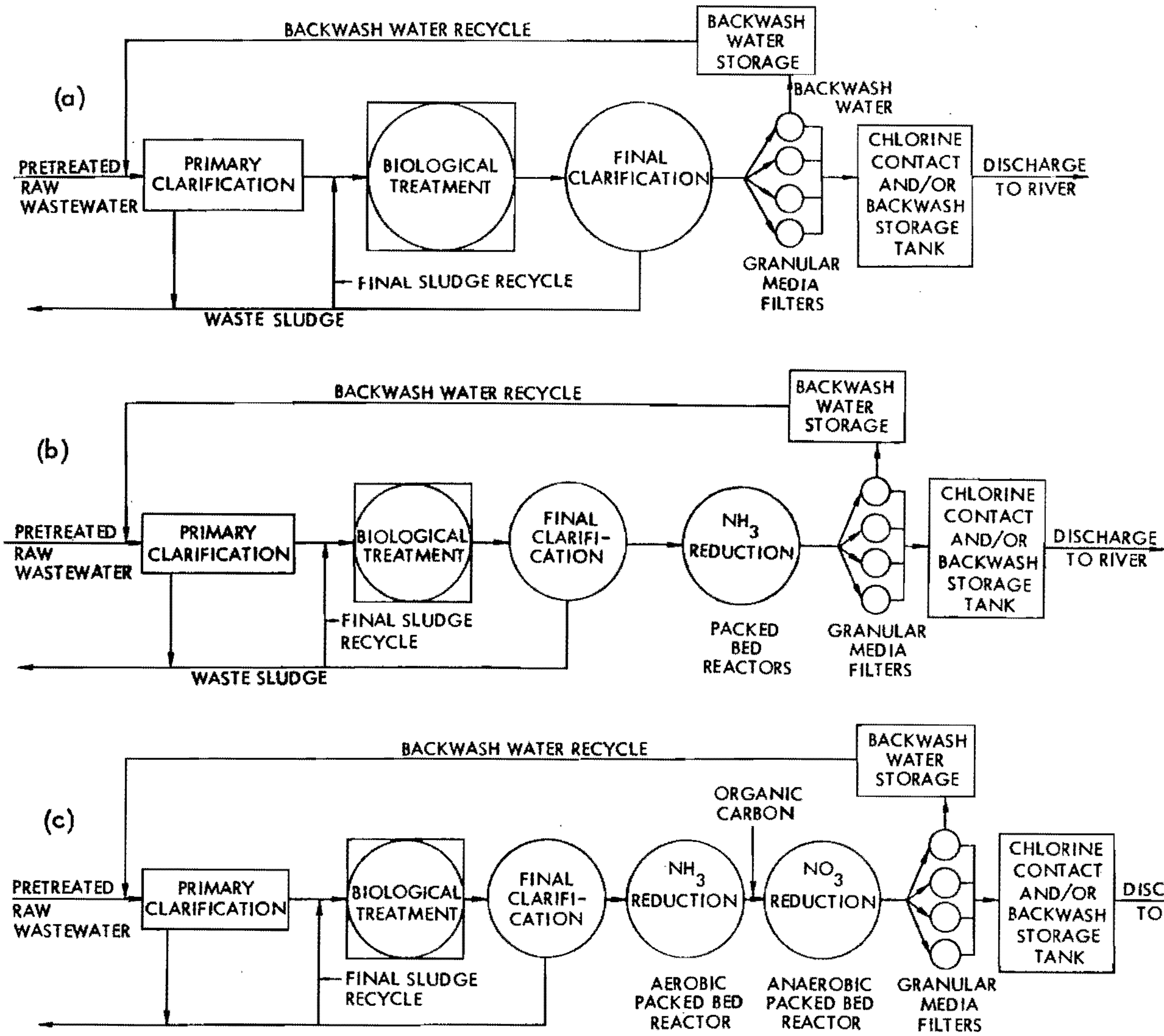

Figure 1. Granular media filters for tertiary wastewater treatment: (a) following biological secondary treatment for carbonaceous BOD removal; (b) following biological secondary and biological tertiary (packed-bed reactors) treatment for carbonaceous BOD and ammonia reduction; (c) following biological secondary and biological tertiary (packed-bed reactors, both aerobic and anaerobic) for carbonaceous BOD, ammonia, and nitrate reduction. (Phosphorus levels may also be reduced by adding ferric or aluminum salts and a polymer feed to solids contact units located ahead of the granular-media filters.)

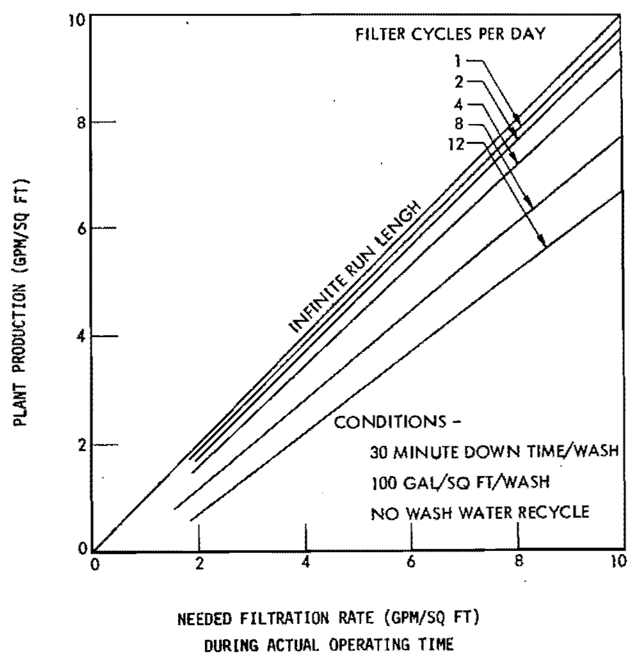

Figure 2. Effect of number of filter cycles per day on filtrate production with filtered water used for backwashing. Plant Production is the average plant output over the full 24 hours of the day.

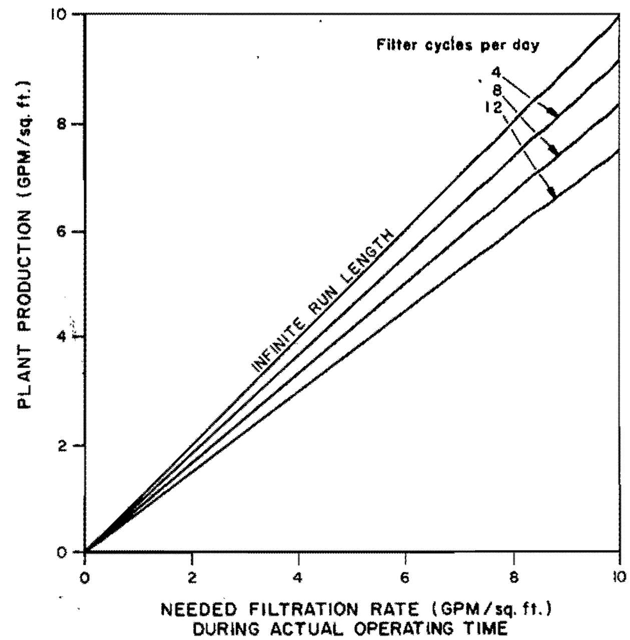

Figure 3. Effect of number of filter cycles per day on filtrate production when using unfiltered water for backwashing. Plant Production is the average plant output over the full 24 hour day.

the peak flow and suspended solids load conditions predicted for the design year, the cycles should be at least 6 hours. Considering typical flow and solids load variations, this should result in 24 hour cycles under average design year loads.

One must keep in mind the conditions selected to construct Figures 2 and 3. Some filters require more than 30 minutes to complete a backwash cycle, especially if complete gravity draindown is essential or desired. Some require more than 100 gal/sq ft/wash. If the downtime and water use for a particular type of filter are expected to deviate significantly from those used above, then the figures should be reconstructed and the cycle length decision reconsidered. Different backwashing routines are discussed in more detail in a later section of this paper.

Filter Configurations

A filter configuration must be selected for a wastewater filter which is appropriate for the higher influent solids anticipated as discussed in the previous sections. A granular media filter is intended to filter in depth, i.e., it is intended that solids removal take place within the filter, and not primarily at the entering surface.

A number of alternate filter configurations have developed to accommodate the higher solids loads described above and to encourage filtration in depth. These are illustrated in Figure 4.

FILTER DESIGN—DETAILED CONSIDERATIONS

The objective of filtration is to produce the desired quality and quantity of filtrate at least cost per unit of filtrate produced. The designer must choose between the various pretreatment alternatives and various performance variables discussed below in reaching a final design. The various alternatives must be tested against the basic objective.

The variables which affect performance fall in two categories: (1) the influent suspended solids variable, such as the type, amount, and filtrability of the solids, and (2) the physical filtration variables such as the rate of filtration, terminal head loss provided; and the size, depth, and type of filter media.

Pilot Scale Testing

When new types of waters are to be filtered containing solids of unfamiliar filtrability, pilot testing may be necessary to arrive at the proper design. Pilot testing on various wastewaters has become increasingly common as such filters are needed in process flow schemes.

The pilot filtration apparatus should have three or more filters which can be run in parallel. This is necessary because the influent solids may change from day to day (even hour to hour) so that various design or operating variables must be compared in parallel rather than sequentially. The three pilot filters can be operated in a series of experiments to evaluate the effect of media size, media depth, media type (single, dual or multi media) and filtration rate on filtrate quality and head loss generation. The filters should be equipped with pressure taps at intervals through the depth so that the extent of depth filtration can be ascertained. The influent and effluent would be monitored for suspended solids, turbidity and other parameters of interest so that the ability to achieve filtrate quality goals can be determined, and the relation of solids load to head loss development can be approximated. The pilot experiments should cover the full range of the variables that may be used in the plant design, e.g., filtration rates of 2 to 8 gal/min/ft^2 and terminal head losses to say 30 feet. A good deal about expected performance can be learned by studying the results of other investigators who have filtered similar influent solids. Substantial data of this type is presented for wastewater filtration in other sources (2, 3). Table 2 contains typical pilot plant data.

Optimization Considerations

The various feasible design alternatives which will meet filtrate quality goals can be compared on a capital and operating cost basis.

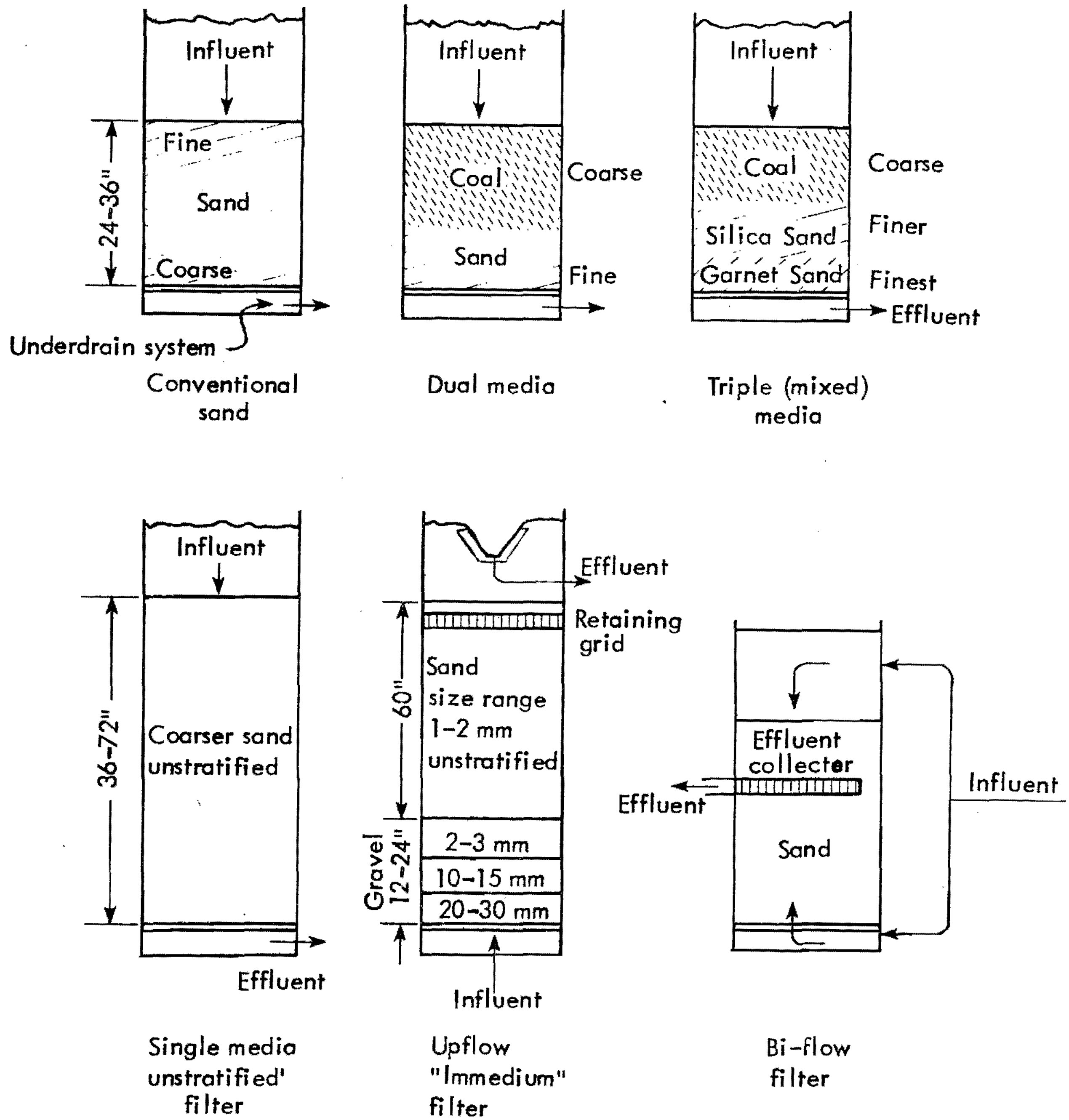

Figure 4. Schematic diagrams of filter configurations for granular media filtration.

Table 2. Solids capture per foot of head loss increase in direct filtration of secondary effluents.

Secondary Effluent Type[1]	Filtration Rate gal/min/ft^2	Mode of Operation[2]	Media Size mm	Solids Capture lb/ft^2/ft Headloss Increase	Reference	Location
TF (Full Scale Sand)	2.5 - 4	C	0.85-1.7*	.04-.05	9 & 10	Luton, Eng.
TF (Pilot Dual Media)	2 - 6	C	1.84 ES	0.07	7	Ames, Iowa
TF (Pilot Sand)	2 - 6	C	0.55 ES	.06-.07	11	Ames, Iowa
TF (Full Scale Sand)	2.3	-	0.55 ES	.03	12	Pretoria
TF (Pilot Dual Media)	2.1	C	1.03 ES	.08	8	Ames, Ia Parallel Operation
TF (Pilot Sand)	2.1	C	2.0-3.6*	.16		
TF (Pilot Dual Media)	3.2	C	1.03 ES	.04	8	Ames, Ia Parallel Operation
TF (Pilot Sand)	3.2	C	2.0-3.6*	.14		
TF (Pilot Sand)	3.2 - 3.8	-	1-2.06*	0.29	13	Finham, Eng.
TF (Pilot Sand)	2	C	2.31 ES*	0.23	14	Ames, Iowa Parallel Operation
TF (Pilot Sand)	2	C	1.82 ES*	0.19		
TF (Pilot Sand)	2	C	1.49 ES*	0.11		
TF (Pilot Sand)	2	C	0.97 ES*	0.06		
TF (Pilot Sand)	4	C	2.31 ES*	0.26	14	Ames, Iowa Parallel Operation
TF (Pilot Sand)	4	C	1.82 ES*	0.21		
TF (Pilot Sand)	4	C	1.49 ES*	0.12		
TF (Pilot Sand)	4	C	0.97 ES*	0.07		
TF (Pilot Sand)	8	C	2.31 ES*	0.31	14	Ames, Iowa Parallel Operation
TF (Pilot Sand)	8	C	1.82 ES*	0.26		
TF (Pilot Sand)	8	C	1.49 ES*	0.15		
TF (Pilot Sand)	8	C	0.97 ES*	0.10		
AS (Pilot Dual Media)	16	C	1.78 ES	.35	15	Cleveland, Oh Parallel Operation
AS (Pilot Dual Media)	24	C	1.78 ES	.093		
AS (Pilot Dual Media)	32	C	1.78 ES	.093		
AS (Pilot Dual Media)	16	D	1.78 ES	.23	15	Cleveland, Oh Parallel Operation
AS (Pilot Dual Media)	22.2	D	1.78 ES	.21		
AS (Pilot Dual Media)	27.6	D	1.78 ES	.12		
AS (Pilot Upflow)	2-5	C	1-2*	.26	16	W. Hertfordshire, Eng.
AS (Pilot Dual Media)	5.1	C	1.08 ES	.24	17	
AS (Pilot Dual Media)	5.1	C	1.45 ES	.34	17	
TF (Pilot Dual Media)	4.24	D	1.28 ES	0.07-0.10	36	Nevada, Iowa
AS (Pilot Dual Media)	4.24	D	1.28 ES	0.01-0.04	36	Marshalltown, Iowa

[1]TF = trickling-filter-plant final effluent; AS = activated-sludge-plant final effluent.
[2]C = constant rate; D = declining rate. *Media size range, unstratified due to backwash provisions.
**ES = effective-size of media, in dual media, only the top coal layer ES is presented.
ES* means unstratified due to backwash provisions.

A particular combination of the physical variables may result in the filter effluent quality reaching its upper limit of acceptability at the same time that the total head loss reaches a selected limit. Such a combination constitutes an optimum (18), or more precisely has been described as an operational optimum (19). A number of operational optimums are possible with a given influent water and filtrate quality goal, but only one would yield water at least cost, i.e., at the economic optimum. In recognition of these concepts, attempts are being made to optimize filter design (19, 20, 21).

Selection of Filtration Rate and Terminal Headloss

Wastewater solids may generate rapid headloss development due to the high solids concentrations in the filter influent and the strong surface removal tendency of the solids. This is especially true in the tertiary filtration of secondary effluents where filtrate quality is not appreciably deteriorated by filtration rates as high as 5 or 6 gal/min/ft^2 using media with effective sizes up to about 2 mm with media depths appropriate to the size. Nevertheless, average rates of 2-3 gal/min/ft^2 and peak rates of 5 gal/min/ft^2 are common to achieve run length objectives (2). Thus, in wastewater filtration, the rate of filtration is dictated more by run length considerations than by filtrate quality considerations.

Modeling of the filtration process has not yet progressed to the point where it is possible to determine precisely what economic filtration rate and terminal headloss should be provided for a granular-media filter. Huang and Baumann (20) found that the most economic terminal headloss for filtration of iron on unisized-sand filters ranged between 8 and 11 feet at all filtration rates from 2 to 6 gal/min/ft^2. Normal American water treatment practice would use a terminal headloss of 8 to 10 feet when using gravity filters. The filtration rate and terminal head should not be so high so as to result in failure of the filtration process by solids breakthrough. However, solids breakthrough does not generally occur in the filtration of secondary effluents. A fraction of the solids pass through the filter during the entire run, but further deterioration does not usually occur as the run progresses.

Studies indicate that pressure drops of as much as 30 feet of water could be used in filtration of trickling filter effluents (7, 23) and in activated sludge effluents (15, 36) through dual-media filters without solids breakthrough. Economic considerations, however, may dictate pressure filters if such terminal headlosses are to be provided.

The selection of the filtration rate and terminal headloss to be provided in design involves consideration of a number of interrelated questions.

What are the desired minimum and maximum filter run lengths? As discussed earlier, run length should be at least 6-8 hours to avoid excessive backwash water use, but less than about 36-48 hours to reduce anaerobic decomposition within the filter and possible detriment to the effluent BOD. The desired run length can be achieved by selecting either the terminal headloss or the filtration rate or both.

Will the backwash operation be automated to avoid manpower costs if short filter runs occur? Automatic backwash is commonly provided in wastewater filtration plants.

Is pressurized discharge desired to a subsequent treatment unit or to an effluent force main? Pressurized discharge would tend to favor the use of pressure filters. In such cases, higher rates and/or higher terminal headlosses may be economically feasible where they would not be with gravity filters.

Is the hydraulic profile of the existing secondary plant such that tertiary filters could be added without repumping by limiting the terminal headloss?

What is the size of the plant, the capital available, and the space available for tertiary filters? A large plant with adequate capital resources may prefer multiple gravity filters, at lower filtration rates and lower terminal headloss, using a more-or-less conventional water plant design. A smaller plant, or one with limited capital or space, may prefer pressure filters operated at higher filtration rates.

Are there any regulatory agency policies which require gravity filters or prohibit pressure filters, or does the client insist upon gravity filters for easier maintenance?

What variations in influent flow rate and suspended solids concentration are expected, and how will they be handled? If influent flow equalization is provided, this concern is partially eliminated. If 24-hour-minimum filter runs are the goal, the hourly variations in load will balance out over the day and become of less concern. On the other hand, if 6-hour-minimum cycles are selected, peak 6-hour loads would be of concern.

To answer these questions rationally, some method of predicting run length as a function of filtration rate, terminal loss, media size, and influent suspended solids is needed. As discussed earlier, pilot plant studies at the plant in question yield the most reliable prediction. In their absence, the designs can be based on a conservative value of solids capture per unit head loss.

If pilot plant data are collected for different filter media and different terminal head losses, the data can be used to select several alternative design combinations of media, filtration rate, and terminal head loss. These can be compared on the basis of capital and operating costs. Furthermore, if the flow and solids load variations are predicted, the operational consequences of those variations can be analyzed. One must be sure to limit the design alternatives to those that have been shown to produce acceptable filtrate quality.

To illustrate the use of pilot plant data, assume that the minimum desired run length is selected to be 8 hours, the maximum 8 hour influent solids concentration is estimated to be 40 mg/l, and the terminal head loss is limited to 10 ft by one of the factors discussed above. From the pilot study, the peak 8 hour filtration rate must then be limited to 6 gal/min/ft^2. If the average annual flow rate is one third the peak, and the average influent solids is predicted to be 20 mg/l; then from the figure, the average run length could be 42 hours. It would be desirable under such loads to wash on a maximum 24 hour override to prevent anaerobic conditions in the filter.

If the design must be based on an assumed solids capture per unit head loss, then alternative designs can be selected as illustrated below.

Assume that a value of 0.07 lb/sq ft/ft head loss has been estimated for a trickling filter effluent and a media size of 1.2 mm ES from Table 2. This value can be used to estimate the terminal head loss that must be provided to achieve a desired filter run length using an estimated secondary effluent suspended solids concentration. For example, find the needed terminal head loss to achieve 24-hour average filter runs under the following conditions:

Average filtration rate = 3 gal/min/ft^2, with range of 2-4.5 during the day
Average secondary effluent suspended solids = 30 mg/l
Average effluent suspended solids = 5 mg/l
Average suspended solids capture = 25 mg/l
Top media size = 1.2 mm

Calculate solids capture per square foot per run:
25 mg/l removed x 3 gal/min/ft^2 x 1,440 min/filter

$$25 \text{ mg/l removed} \times 3 \text{ gal/min/ft}^2 \times 1{,}440 \text{ min/filter run} \times \frac{8.33}{10^6} = 0.90 \text{ lb/ft}^2\text{/run}$$

$$\text{Terminal head loss increase} = \frac{0.90 \text{ lb/ft}^2\text{/run}}{0.07 \text{ lb/ft}^2\text{/ft head loss}} = 13 \text{ ft/run}$$

Thus, a terminal head loss increase of about 13 feet would be required to meet the 24-hour filter run. The initial head loss must be added to this figure to obtain the total terminal head loss. This total is above the normal head loss provided for gravity filters and suggests either that pressure filters be considered, or that the average filtration rate be reduced to 2 gal/min/ft^2.

The filter runs could become substantially shorter during periods of poorer secondary treatment plant performance. For example, if the secondary effluent suspended solids climbed to 50 mg/l, the run length would drop to 13.3 hours, other conditions being unchanged. Peak flows could prevail for such a run length, further accentuating the solids load and reducing the run to 8.9 hours. When filter cycles get this short, the backwash water being returned through the plant becomes substantial and further increases the load on the filters shortening the filter runs.

Selection of Filter Media

The selection of the size and depth of filter media and the appropriate filtration rate are interrelated. In general, filtrate quality is improved by the use of finer media, greater media depth, or lower filtration rates. Similarly, head loss generation rate is increased by finer media, greater media depth and higher filtration rates. With some influent suspensions, these generalizations are not demonstrated significantly. For example, in filtration of secondary effluents, filtration rate has little effect upon filtrate quality over the usual range of rates employed, 2-5 gal/min/ft^2, and increased media depth may not compensate for coarser media in achieving filtrate quality. As evidence, Tables 3 and 4 show that a dual media and a triple media filter provided slightly better filtrate quality than an unstratified coarse sand filter of 46 in. depth. Further, Table 5 shows that changing the depth of the unstratified coarse sand filter had little effect on performance at the filtration rate of 3 gal/min/ft^2. However, greater depth is of benefit in maintaining filtrate quality at higher filtration rates (14).

Granular filter media commonly used in water and wastewater filtration include silica sand, garnet sand, and anthracite coal. These media can be purchased in a broad range of effective sizes and uniformity coefficients. (The term "effective size" indicates the size of grain (in millimeters) such that 10 percent, by weight, of the particles are smaller and 90 percent larger than itself. "Uniformity coefficient" designates the ratio of the size of grain which has 60 percent of the sample finer than itself to the effective size which has 10 percent finer than itself.) The media have specific gravities approximately as follows:

Anthracite coal, 1.35-1.75, Most U.S. anthracite 1.6-1.75, U.K. anthracite 1.35-1.45
Silica sand, 2.65
Garnet sand, 4-4.2

Table 3. Performance of a dual media, triple media and unstratified coarse sand filter when filtering secondary effluent from the trickling filter plant at Ames, Iowa (8). Results are the mean values from periodic composite samples collected during 8 weeks of operation in 1974 at 2.1 gal/min/ft^2.

	Influent	Filter Effluent: Dual Media[a]	Triple Media[b]	Unstratified Media[c]
Suspended Solids (mg/l)	37.49	6.84	6.31	7.92
n = 14[d]	σ=12.03[e]	σ=3.23	σ=3.87	σ=5.80
Turbidity (FTU)	16.38	2.38	2.20	2.89
n = 16	σ= 4.31	σ=0.97	σ=0.56	σ=1.10
BOD_5 (mg/l)	14.61	3.73	4.11	4.73
n = 13	σ= 6.00	σ=1.72	σ=2.03	σ=2.56
Soluble BOD_5 (mg/l)	3.88	1.97	2.20	2.34
n = 10	σ= 1.79	σ=0.96	σ=0.99	σ=1.11

[a] Dual Media: 15 in. of 1.03 mm ES coal, 1.57 U.C.
9 in. of 0.49 mm ES sand, 1.41 U.C.

[b] Triple Media: Same as above plus 3 in. garnet with 0.27 E.S. and 1.55 U.C.

[c] Unstratified Sand: 46 in. of 2.0 mm E.S. sand (2-3.6 mm size range, 1.52 U.C.)

[d] n = number of composites averaged, each representing one filter run

[e] σ = standard deviation

The detrimental effects of the strong surface removal tendancy previously discussed for wastewater filtration must be counteracted by selecting a media size where the flow enters the media which will ensure that the bulk of the suspended solids removal does not occur at the entering surface. Pilot testing of different media is desired if time and budgets permit. If it is not feasible, the following information will assist in selecting the media size or sizes.

Table 4. Performance of a dual media, triple media and unstratified coarse sand filter when filtering secondary effluent from the trickling filter plant at Ames, Iowa (8). Results are the mean values from periodic composite samples collected during 9 weeks of operation in 1974 at 3.2 gal/min/ft^2.

		Filter Effluent[a]		
	Influent	Dual Media	Triple Media	Unstratified Media
Suspended Solids (mg/l)	34.08	7.05	6.82	9.46
n = 14[b]	σ=16.87[c]	σ=4.27	σ=3.10	σ=4.53
Turbidity (FTU)	17.60	4.80	6.78	4.66
n = 15	σ= 6.18	σ=2.28	σ=3.01	σ=2.12
BOD_5 (mg/l)	30.38	12.68	12.99	14.46
n = 15	σ=14.52	σ=6.88	σ=6.82	σ=6.56
Soluble BOD_5 (mg/l)	9.67	7.21	7.27	7.78
n = 15	σ= 3.76	σ=3.72	σ=3.61	σ=3.57

[a] Filter media same as in Table IV-2a except coal depth in dual and mixed media increased to 17 in.

[b] n = number of composites averaged, each representing one filter run.

[c] σ = standard deviation

Table 5. Performance of three unstratified coarse sand filters of different depth when filtering secondary effluent from the trickling filter plant at Ames, Iowa (8). Results are the mean values from periodic composite samples collected during 5 weeks of operation in 1975. Sand size was 2.5 to 3.7 mm size range.

		Filter Effluent[a]		
	Filter Influent	24 in. Depth[b]	47 in. Depth	60 in. Depth
Suspended Solids (mg/l)	31.3	5.9	6.4	5.7
σ(n = 11)[c]	9.7	2.1	2.3	1.8
Turbidity (FTU)	12.6	3.30	3.38	3.14
σ(n = 11)	3.14	1.21	1.14	1.14
BOD_5 (mg/l)	15.6	6.5	7.1	6.6
σ(n = 11)	4.7	2.8	2.5	2.5
Soluble BOD_5 (mg/l)	5.3	3.9	4.0	3.8
σ(n = 11)	1.7	1.3	1.3	1.3

[a] Filtration rate, 3.0 gal/min/ft^2

[b] Filtrate from 24 inches of sand and 12 inches of supporting gravel

[c] σ= standard deviation,
n= number of composites averaged, each representing one filter run

For the tertiary filtration of secondary effluents, media size of at least 1.2 mm E.S. is required, and coarser media is preferred if appropriate backwash is provided. Benefits to filter run length accrue at least up to 2.3 mm E.S. as shown in the prior solids capture data in Table 2.

For the filtration of chemically treated secondary effluents, a media size of not less than 1.0 mm has been suggested (2). However, benefits of coarser media should also occur here, and the sparsity of data makes pilot testing even more important.

Once the size of the media at the entering surface has been selected, the rest of the media specification is dependent thereon. For example, the uniformity coefficients, the size of the sand in dual media, and the depth of each media must be selected.

Low uniformity coefficients (U.C.) are desired to achieve easier backwashing. This is especially true where fluidization of the media is required during backwashing as with dual and triple media filters. This is true for dual and triple media because the entire media should be fluidized to achieve restratification; therefore, the greater the U.C. (i.e. less uniform size range), the larger the backwash rate required to fluidize the coarser grains thus provided. A U.C. of less than 1.3 is not generally practical because of the sieving capabilities of commericial suppliers. A U.C. of less than 1.5 can be obtained at a cost premium and is recommended.

A U.C. of less than 1.5 has the advantage that it will ensure that the coarser grain size in the media (such as the 90 percent finer size, d_{90}) is not excessively large, requiring a large backwash rate. Sieve analyses of filter media will usually plot linearly on either log-probability or arithmetic-probability paper. The ratio of d_{90}/d_{10} for media with a U.C. of 1.5 is 2.0 for the log probability distribution and 1.83 for the arithmetic probability distribution. These ratios are useful in estimating the d_{90} grain size which can then be used to determine the needed backwash rate.

An alternate method of specifying filter media which is used in the U.K. is to specify the range of size within which the media must fall. For example, a 1.4-2.4 mm size range would fall between a U.S. standard 14 mesh and 8 mesh sieve. Some tolerance must be allowed at either end to allow for the sieving capabilities of the suppliers. A 10 percent tolerance at each end is suggested, i.e., 10 percent by weight could be smaller than 1.4 mm and 10 percent coarser than 2.4 mm. This system of specification has the advantages that the effective size could be no smaller than the lower end of the range, and the coarser media is more precisely limited which is of importance is selecting the needed backwash rate.

Dual media

For dual media filters, the sizes of the sand layer must be selected to be compatible with the coal which has been selected. The bottom sand (e.g., the 90 percent finer size) should have approximately the same or a somewhat lower flow rate required for fluidization than the bottom coal to ensure that the entire bed fluidizes at the selected backwash rate.

To assist in the selection of the required backwash rate, and to assess the compatibility question above, empirical data on the minimum fluidization velocity of coal, sand and garnet sand at 25°C are presented in Table 6, as well as empirical correction factors to be applied for other water temperatures. The temperature correction factors agree substantially with data presented by Camp (24).

The effective size of the sand for a dual media filter should be selected to achieve the goal of coarse-to-fine filtration without causing excessive media intermixing. If the coal density is in the typical range of 1.65 to 1.75 g/cm^3, a ratio of the 90 percent finer coal size to the 10 percent finer sand size equal to about 3 will result in a few inches of media intermixing at the interface (24). A ratio of these sizes of 4 will result in substantial media intermixing, whereas a ratio of 2 to 2.5 will cause a sharp interface. Choosing media sizes to achieve a sharp interface will mean that the benefits of coarse-to-fine filtration will be partly lost. Therefore, a size ratio of about 3 is recommended.

Table 6. Minimum fluidization velocities for various uniform sized media to achieve 10 percent expansion at 25°C, observed empirically (8).

Between U.S. Std. Sieves Passing (mm)		Retained	Mean Size mm	Flow rate to achieve 10% expansion at 25°C, gpm/ft^2 Coal	Sand	Garnet
7	2.83	8	2.59	37		
8	2.38	10	2.18	30		
10	2.00	12	1.84	24	41	
12	1.68	14	1.54	20	33	
14	1.41	16	1.30	15.7	27	49
16	1.19	18	1.09	12.5	21	40
18	1.00	20	0.92	9.9	16.4	32
20	0.841	25	0.78	8.4	12.6	27
25	0.707	30	0.65	7.0	9.0	22
30	0.595	35	0.55		6.3	18.0
35	0.500	40	0.46		5.4	13.7
40	0.420	45	0.38		4.0	11.3
50	0.297	60 (0.25mm)	0.27			6.3
Specific Gravity				1.7	2.65	4.1

Temperature correction - The following are approximate correction factors to be applied for temperatures other than 25°C.

Temperature C°	Multiply 25° value by
30	1.09
25	1.00
20	0.91
15	0.83
10	0.75
5	0.68

The use of Table 6 and the foregoing recommendation can be illustrated with an example. Assume a coal of 1.2 mm ES has been selected with a U.C. less than 1.5 (size range of 1.2-2.2 mm, 8 to 16 mesh range). The sand should have an effective size about 0.7 mm to be one third of the coarse coal size. A sand size range of 0.7 to 1.4 mm could be specified (14 to 25 mesh range), or one with an ES of 0.7 mm. The backwash rate for the coarse end of the coal (2.38 mm) is 30 gal/min/ft^2 at 25°C and the coarse sand (1.4 mm) is 27 gal/min/ft^2. Thus, they are compatible. If the peak expected operating temperature is 15°C—the required backwash rate would be 30 x 0.83 = 25 gal/min/ft^2.

It should be noted that no harm would be done if the coarser sand grains were smaller than 1.4 mm. They would merely reach fluidization before the coarser coal grains. There is no danger of inversion of the coal and sand layers during backwashing or complete intermixing as there is with sand and garnet sand. The intermixing behavior of coal and sand, and sand and garnet sand has been experimentally demonstrated (25).

In addition to specifying the gradation of filter media used, the depth of media must be established. At present, there is no reasonable method—other than pilot-plant operation—that can be used to determine the optimum depth of filter media. Huang (7, 23) established that, for filtration of trickling filter plant effluent, a depth of at least 15 inches of 1.84 mm ES coal was desirable. Theoretical considerations would indicate that media depths should increase with media size. For practical designs based on a minimum of available information, the following minimum media depths are recommended for dual media filters:

Anthracite coal, 15 inches minimum to 20 inches
Silica sand, 12 inches minimum to 15 inches

It should be emphasized that the media design illustrated by the foregoing example is one appropriate design for tertiary filtration but it is not the only possibility. A coarser coal would yield longer filter runs but required higher backwash rates. Nor is the example media design necessarily best for chemically pretreated wastewaters, or where polyelectrolytes are to be used as filter aids. In the latter case, a coarser top size may be desired (1.2-1.5 mm).

In dual or triple media filters, after each media layer is installed in the filter, it should be backwashed and skimmed to remove unwanted fines before installing the next layer. This step can be important, for example, because the sand may collect a low density coating after a number of filter cycles. In one case, using alum coagulation of secondary effluent, these coatings caused the fine sand to migrate to the coal surface where it formed a blinding surface layer (26).

Unstratified single media

Single media filter beds comprised of unstratified coarse sand are also being used for wastewater filtration. Sand depths of 4 to 5 ft and size ranges of 1.5-2.5 mm, 2-3 mm, and 2-4 mm are being used.

These filters offer the advantage of using a coarser media size and thus achieve greater solids capture per unit headloss as shown previously in Table 2. However the provision of adequate backwashing is essential.

Because of the coarse sand sizes, backwash by fluidization and bed expansion in the usual U.S. fashion would require excessive wash rates and is not feasible. Therefore, these filters are backwashed with air and water simultaneously at rates just sufficient to cause a pulsing and a slow circulation of the sand in the bed. This is followed by a short water wash to a rate below fluidization to expell some air from the bed.

The overflow level during backwashing must be high enough above the sand surface to prevent excessive loss of sand during the simultaneous air-water backwash. Even though the bed is not fluidized, grains of sand are thrown above the fixed bed surface by the violence of the combined air-water action. A vertical distance to overflow of 24 inches is recommended for the sand sizes mentioned above based on laboratory, pilot and plant scale observations (8). The common wash routines for these sand sizes are presented in Table 7.

Methods of Filter Flow Control

Variable declining rate filtration

Variable declining rate operation is similar to influent flow splitting, and is another desirable method of operation for gravity filters. Variable declining rate operation achieves all the influent flow splitting advantages and some additional ones, without any of the disadvantages. Despite the merits of this method, however, it has not received enough explanation or attention (27).

Figure 5a illustrates the desirable arrangement for new plants designed for variable declining rate operation. Great similarity exists between Figure 5a, the principal differences being the location and type of influent arrangement and the provision of less available headloss.

Table 7. Unstratified sand filter designs for wastewater with appropriate backwash routines.

Media		Simultaneous Wash			Water Wash		Source*
Size range	Depth	Air Rate	Water Rate	Dur.	Rate	Dur.	
(mm)	(ft)	(cfm/ft^2)	(gal/min/ft^2)	(min)	(gal/min/ft^2)	(min)	
1.5-2.5	3	2.7	5.5	10	11	5	1
2-3	4-6	6-8	6-8	15	8	5	2
2.5-3.7	4&5	7	15	10	15	3	3

* 1. Successfully operating full scale plant in tertiary filtration at activated sludge plant in England observed by authors.

2. Manufacturers suggested media and wash routine in the U.S. for 2-3 mm and 2-4 mm sand. Provided acceptable wash of 2-3.6 mm sand in tertiary filtration study at Ames, Iowa (8).

3. Successful wash routine in pilot scale study at Ames, Iowa, in tertiary filtration of trickling filter plant effluent (8).

Figure 5b illustrates the typical water level variation and head loss variation observed with this mode of operation. The filter influent enters below the wash trough level of filters. When the water level in the filters is below the level of the wash trough, the installation operates as an influent flow splitting constant rate plant. When the water level is above the level of the wash trough, the installation operates as a variable declining rate plant. In general, the only time the filter water level will be below the wash trough level will be when all filters are backwashed in rapid sequence or after the total plant has been shut down, with no influent, so that the water level drops below the wash trough. In most cases, the clean filter head loss through the piping, media, and underdrains will range from 3 to 4 feet and keep the actual low water level above the wash trough. The water level is essentially the same in all operating filters at all times; this is achieved by providing a relatively large influent header (pipe or channel) to serve all the filters, and a relatively large influent valve or gate to each individual filter. Thus, head losses along the header or through the influent valve are small and do not restrict the flow to each filter. The header and influent valve will be able to deliver whatever flow each individual filter is capable of taking at the moment. A flow restricting orifice or valve is recommended in the effluent pipe to prevent excessively high filtration rates when the filter is clean and to indicate the approximate clean bed filtration rate.

Each filter will accept at any time that proportion of the total flow that the common water level above all filters will permit it to handle. As filtration continues, the flow through the dirtiest filter tends to decrease the most rapidly, causing the flow to redistribute itself automatically so that the cleaner filters pick up the capacity lost by the dirtier filters. The water level rises slightly in the redistribution of flow to provide the additional head needed by the cleaner filters to pick up the decreased flow of the dirtier filters. The cleanest filter accepts the greatest flow increase in this redistribution. As the water level rises, it partly offsets the decreased flow through the dirtier filters; as a result, the flow rate does not decrease as much or as rapidly as expected.

This method of operation causes a gradually declining rate toward the end of a filter run. Filter effluent quality is affected adversely by abrupt increases in the rate of flow—here, the rate increases occur in the cleaner filters where they have the least effect on filter effluent quality (29). Rate changes throughout the day due to changes in total plant flow, both upward and downward (in all of the filters, dirty or clean), occur gradually and smoothly without any automatic control equipment.

The advantages of declining rate operation over constant rate operation are as follows (27, 28):

For waters that show effluent degradation toward the end of the run, the method provides significantly better filter effluent quality than that obtained with constant rate (or constant water level) filter operation.

Less available head loss is needed compared with that required for constant rate operation because the flow rate through the filter decreases toward the end of the filter run. The head loss in the underdrain and effluent piping system therefore decreases (with the square of the flow rate) and becomes available to sustain the run for a longer period than would be possible under constant rate operation with the same available head. Similarly, the head dissipated through the clogged portions of the filter media decreases linearly with decreasing flow rate.

For the foregoing reasons, declining rate filters are considered to be the most desirable type of gravity filter operation, unless the design terminal head loss is quite high (e.g., greater than 10 feet). Then constant level control or pressure filters may be a more economical choice. A bank of pressure filters can also operate using variable declining rate filtration; however, any rate changes imposed on the plant cause sudden changes in filtration rates with pressure filters.

Some of the concerns and questions raised about variable declining rate filtration are as follows: (1) It appears to be an uncontrolled system with little available operator manipulation. This is, in fact, an attribute which prevents operational abuse of the delicate filtration mechanisms. (2) If the rate limiting device is sized for design year peak loads, it will permit higher than necessary filtration rates in the early plant life. This is true unless one limits the head loss utilized during the early plant life, i.e., backwashes at lower water levels. (3) What is the total available head loss to be provided? This is a difficult question but no more difficult than it has been in the past for constant rate filtration plants. It is best guided by past experience at the plant in question, or by piloting testing. In the absence of these, one must resort to an

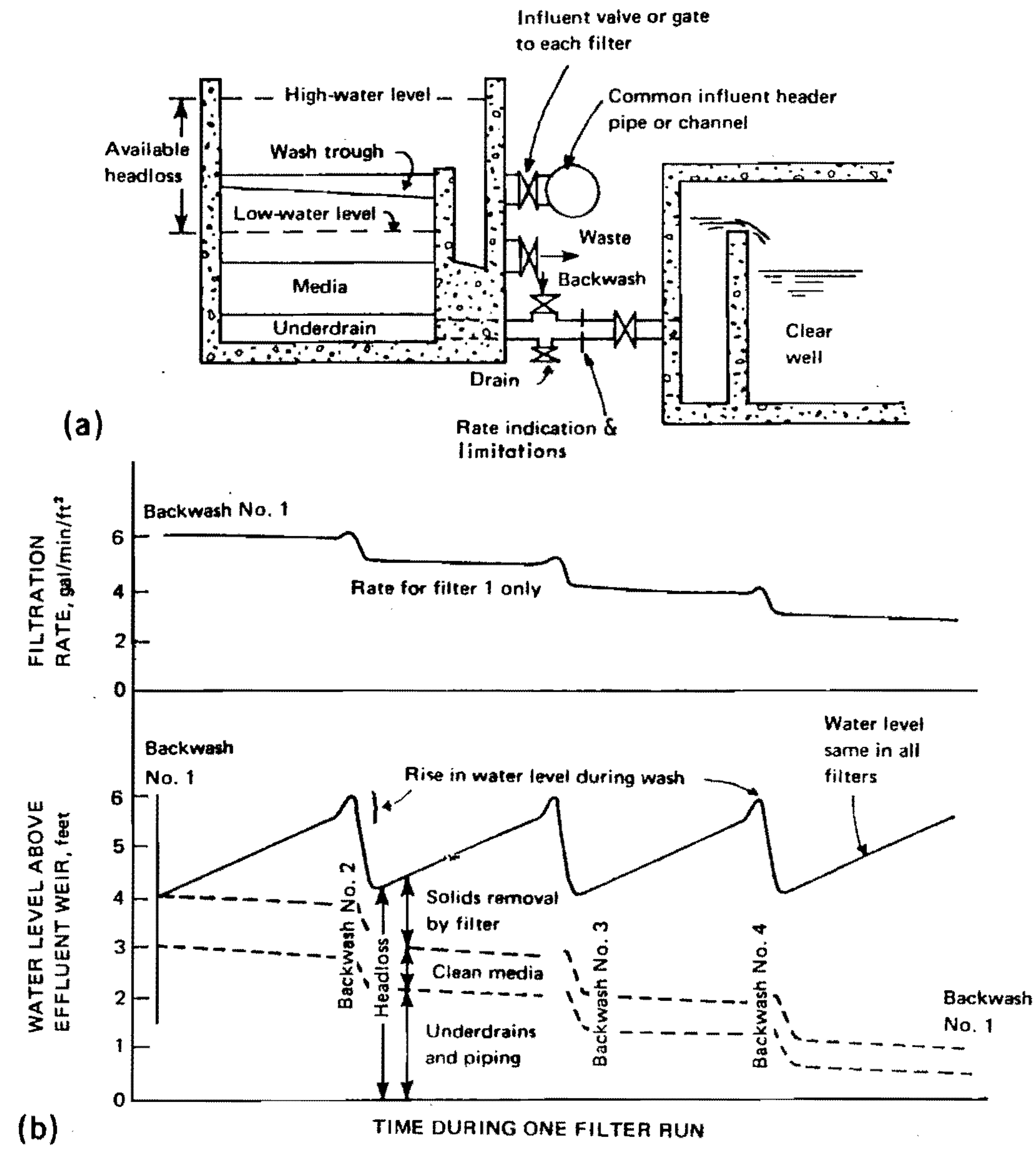

Figure 5. Variable declining-rate filtration: (a) typical filter and clear well arrangement; (b) filtration rate, headloss, and water level during one filter run in a plant having four filters.

assumed solids capture per unit head loss design as discussed previously to select terminal head loss, and make adjustments downward for the head loss recovery discussed above.

Surprisingly, the water level fluctuation in plants operating on this system is not as great as anticipated. Typical variations of 1.5 to 2 ft (0.5-0.7 m) have been reported in potable water plants (31, 32).

BACKWASHING OF WASTEWATER FILTERS

The principal problems in filter operation are associated with maintaining the filter bed in good condition. Inadequate cleaning leaves a thin layer of compressible dirt or floc around each grain of the media. As pressure drop across the filter media increases during the subsequent filter run, the grains are squeezed together and cracks may form in the surface of the media, usually along the walls first.

The heavier deposits of solids near the surface of the media break into pieces during the backwash. These pieces, called mudballs, may not disintegrate during the backwash. If small enough and of low density, they float on the surface of the fluidized media. If larger or heavier, they sink into the filter, to the bottom, or to the sand-coal interface in dual media filters. Ultimately, they must be broken up or removed from the filter or they reduce filtration effectiveness, or cause shorter filter runs by dissipating available head loss.

In wastewater filters, slimes can reduce the average density of the filter grains and can cause more loss of filter media during backwashing, or migration of fine sands in dual media higher into the coal layer. Filimentous growths can cause blinding of the surface layers which shorten filter runs.

Dirty filter media may be chemically cleaned in place as a temporary expedient short of rebuilding the filter bed. Various chemicals have been used, including chlorine, copper sulfate, acids and alkalies. Chlorine may be used where the material to be removed includes living and dead organisms or their metabolites. Copper sulfate is effective in killing algae growing on the walls or medium. Alkalis can be effective on greasy deposits on the filter grains.

However, rather than attempting to correct dirty filter problems after they occur, the backwashing system should be designed to prevent them from the onset.

Potable water filter backwashing practice in the U.S. has used the high velocity wash with substantial bed expansion (20-50 percent). This method does not solve all problems with dirty filters, and it has created problems with shifting of the finer supporting gravel layers. The provision of a surface wash system which introduces high velocity water jets before and during the backwash has largely solved the problem of dirty filter media for potable water filters, but has not solved the problem of shifting gravel. The growing use of wastewater filtration has further demonstrated the weakness of water fluidization backwash. Backwashing is substantially more difficult and problems of agglomerates and filter cracks are prominent.

The problem of shifting gravel and the more difficult backwashing of a wastewater filter has stimulated renewed interest in the air scour method of auxiliary agitation, which has continued in use in European practice. There is also interest in the use of underdrain systems with fine strainers that do not require gravel, a system which was abandoned in the U.S. in the early twentieth century due to clogging and corrosion problems.

Backwashing Recommendations

In view of the difficulty of backwashing wastewater filters, and the various filter media and backwash routines available, a research study was conducted to compare the various alternatives as applied to wastewater filtration. Various granular media filters were studied including single, dual, and triple media. Various methods of backwashing were compared including (1) water fluidization only, (2) air scour followed by water fluidization, (3) surface wash and subsurface wash before and during water fluidization backwash, and (4) simultaneous air scour and subfluidization water backwash.

Some of the conclusions of that study are important to design of wastewater filters and are, therefore, quoted below (8).

"The cleaning of granular media filters by water backwash alone to fluidize the filter bed is inherently a weak cleaning method because particle collisions do not occur in a fluidized bed and thus abrasion between the filter grains is negligible.

"The weakness of water fluidization backwash alone was clearly demonstrated during wastewater filtration studies where a dual media filter which was washed by water fluidization alone developed serious dirty filter problems such as floating mud balls, agglomerates at the walls and surface cracks. These problems were observed when filtering either secondary effluent or secondary effluent which had been treated with alum for phosphorous reduction.

"The heavy mud ball and agglomerate accumulations caused higher initial headlosses and shorter filter cycles. They may also cause poorer filtrate quality in some cases, although such detriment was not demonstrated in this research.

"Simultaneous air scour and subfluidization backwash of unstratified coarse sand filters proved to be the most effective method of backwash. However, this method should not be used for finer filter media such as the coals and sands of the typical sizes used in dual and triple media filters because loss of media will occur during backwash overflow. The choice of simultaneous air and water flow rates must be appropriate for the sand being used and should result in some circulation of the sand for effective backwashing.

"The other two methods of improving backwashing, namely air scour followed by water fluidization backwash, and surface (and subsurface) wash before and during water fluidization backwash, proved to be comparable methods of backwash which can be applied to single, dual and triple media filters. These two methods did not completely eliminate all dirty filter problems, but both auxiliaries reduced the problems to acceptable levels so that filter performance was not impaired.

"The use of some form of air scour auxiliary or some form of surface wash auxiliary is essential to the satisfactory functioning of wastewater filters comprised of deep beds (2-5 ft) of granular material which are backwashed after several feet of head loss development. The auxiliary and the backwash routine must be appropriate to the filter media. For example, subfluidization wash is limited to single media filters because stratification is not essential (or even desired) for such filters. Fluidization capability is essential for dual or triple media filters to permit restratification of the layers in their desired positions at the end of the backwash. Air scour and water backwash simultaneously during overflow is primarily useful on coarse sand filters because finer media will be lost due to the violence of the combined air and water action. However, the simultaneous use of air and water can be useful on dual and triple media filters prior to the onset of backwash overflow. The above conclusion is not intended to apply to all types of wastewater filters such as the various proprietary filters with their special backwashing provisions. Such filters and provisions were not studied.

"The use of graded gravel to support the filter media is not recommended where the simultaneous flow of air scour and backwash water can pass through the gravel by intention, or by accident, due to the danger of moving the gravel and thus upsetting the desired size stratification of the gravel.

"Media retaining underdrain strainers with openings of less than 1 mm are not recommended for wastewater filters due to the danger of progressive clogging.

"The filter influent feedwater (e.g., secondary effluent) is not recommended as a backwash water source because of the danger of progressive clogging of underdrain strainers and/or gravel. The advantages of using feedwater do not justify the risks that result therefrom.

"Air scour is compatible with dual or triple media filters from the standpoint of minimal abrasive loss of the anthracite coal media. However, the backwash routine must be concluded with a period of fluidization and bed expansion to restratify the media layers after the air scour."

The authors urge you to use the foregoing conclusions as design guides. In addition, the following design suggestions concerning the backwashing provisions should also be considered.

First, consider the use of air scour as applied to dual or triple media filters backwashed with fluidization capability. In this case:

Provide operational flexibility in the period of air scour between, let us say, 2 and 10 minutes so the operator could select the period he deems most appropriate.

If supporting gravel is not used, provide the capability for simultaneous air and water backwash. This technique requires provisions to allow for rapid draining of the filter to near the filter media surface, followed by the brief simultaneous air and water backwash until the water reaches within 6 to 8 inches of the wash troughs. The simultaneous wash is then stopped, and either air alone or water alone may be continued. The water rate during the simultaneous air water wash should be below fluidization velocity to extend the time duration of the action to the maximum.

Provide a backwash volume of at least 100 gal/ft^2 of filter per wash. This is based on the observation that when backwashing at rates above the fluidization velocity for the media, the total wash water required for effective cleaning is about the same regardless of the backwash rate—about 75 to 100 gal/ft^2 of filter. This observation is for typical U.S. wash trough spacing with the trough edges about 3 feet above the surface of the filter media. Larger spacing between troughs, or greater height of trough above the media, would increase the wash water requirements. No economy of total wash water use is achieved by adopting lower backwash rates (above fluidization), because the length of required backwash must be increased proportionately.

Second, consider the use of air and water backwash simultaneously without fluidization capability. In this case:

Provide a backwash water volume of about 150-200 gal/ft^2 of filter per wash. This is larger than the prior case because less experience is available.

Because of the effective solids transport capability of air and water used simultaneously, wash troughs can be eliminated in favor of a single overflow trough along the length of the filter if the transport distance is limited to 12 feet.

Third, consider the use of surface wash auxiliary in dual or triple media filters backwashed with fluidization capability.

Provide a subsurface washer (as well as the surface washer) to attack the mud balls that sink to the interface between the coal and the sand. The subsurface jets should be located at the expected depth of the expanded interface. The writers have no information on the ability of full scale rotary subsurface washers to remain operational in the long term due to the greater drag they encounter, and the hostile environment. The pilot rotary subsurface washer used in the foregoing research was not a good model of a full scale unit, and considerable difficulty was encountered in keeping it operational.

Two additional backwashing problems are of importance in wastewater filter plant design.

Where do we get the water for backwashing?
What do we do with the dirty backwash water?

The best source of water for backwashing will be the effluent from the filters. If disinfection of the plant effluent is practiced, the chlorine or ozone contact tank should provide sufficient capacity to permit drawing backwash water from this tank. If disinfection is not provided, then a special backwash storage tank should be provided, through which all filter effluent should be directed before final discharge. The backwash water storage tank should normally have sufficient capacity to store all the water needed to backwash at least three filters in succession with the volumes suggested above.

The dirty backwash water must be returned to the plant influent for further treatment. Because of the nonuniform scheduling of filter backwashing, the backwash water presents a significant slug load on the primary or secondary treatment facilities if returned to them at the rate of backwashing. For that reason, dirty backwash water should be sent to a dirty backwash storage tank and delivered from there at a nearly constant rate to the plant influent or secondary influent. If flow equalization is not being practiced at the plant, it would be desirable to return the backwash wastewater during the low flow period of the day. This would entail a larger wastewater storage tank and return pumping capability, but it would assist in flow balancing.

SUMMARY

The key questions involved in the proper design of granular filters for wastewater filtration have been discussed in the foregoing sections, and design recommendations have been presented. These recommendations are summarized as follows:

The variable hydraulic and suspended solids load in secondary effluents must be considered in the design to avoid short filter runs and excessive backwash water requirements.

A filter that allows penetration of suspended solids is essential to obtain reasonable filter run lengths. The filter media on the influent side should be at least 1.2 mm for tertiary filtration, and preferably larger if appropriate backwash is provided.

The filtration rate and terminal head loss should be selected to achieve a minimum filter run length of 6 to 8 hours if flow equalization is not provided. Estimates of head loss development and filtrate quality preferably should be based on pilot scale observations at the particular installation. If such studies are not feasible, head loss development should be based on past experience on the suspended solids capture per foot of head loss increase from other similar installations.

The effect of recycling of used backwash water through the plant on the filtration rate and filter operation must be considered in predicting peak loads on the filters and resulting run lengths.

High filtration rates (3 gal/min/ft^2 or higher at average load) and/or high influent suspended solids to the filters (30 mg/l or higher at average load) will cause high terminal head losses and may favor the use of pressure filters over gravity filters, especially for smaller plants with limited capital resources.

Lower filtration rates or lower influent suspended solids may permit the economical use of gravity filters, especially in larger plants where multiple filters will be needed. At least two, and preferably four, filters should be provided. If only two filters are provided, each should be capable of handling peak design flows to allow for one filter to be out of service for backwashing or repair. If four or more gravity filters are provided, the variable declining rate method of operation is strongly recommended.

The success of the wastewater filtration plant depends upon the provision of an effective backwash system which is appropriate for the media selected. Details of backwashing requirements for dual and triple media filters and for unstratified coarse sand filters are presented.

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APPENDIX

Performance Data for Wastewater Filtration from the Literature

REPORTED EFFICIENCIES FOR
DIRECT FILTRATION OF TRICKLING FILTER PLANT EFFLUENTS‡

Location	Ref.	Media Type	Media Size* (mm)	Media Depth (in.)	Filter Rate gpm/sq ft (U.S.)	N**	Suspended Solids (mg/ℓ) Influent Range	Influent Avg	Effluent Range	Effluent Avg	BOD5 (mg/ℓ) Influent Range	Influent Avg	Effluent Range	Effluent Avg	% Removal
Luton, England (1945)	8														
Lab Study		sand	.85-2.0	21	2.0	5	34-77	53	1-20	6					
		coal	1.0-2.0	21	2.0	7	41-67	51	4-13	7					
		sand	0.5-0.85	21	2.0	2	40-59	50	1-2	2					
		sand	0.5-1.0	21	2.0	2	49	49	0	0					
Pilot Study		sand	0.85-1.7	24	2.33	3m	20-37	28	2-5	3	24-40	30	8-15	11	
		sand	0.85-1.7	24	2.66	1m	20	20	3	3	31	31	15	15	
		sand	0.85-1.7	24	2.83	5m	15-25	19	2-5	3	17-28	21	6-9	7	
		coal	0.85-2	18	1.7	2m	20-37	29	3-5	4	27-40	34	10-14	12	
			2-5	10	2.8	7m	15-28	21	2-5	3	17-28	23	6-13	8	
Bingham, England Pilot Study (1949)	8	sand	1.0-2.0	24	1.6	m	32		5		26		10		
		sand	1.0-2.0	24	1.9-2.4	m	40		6		31		10		
		sand	1.0-2.0	24	2.9-3.2	m	35-36		7-8		22-25		10-13		
		sand	1.0-2.0	24	3.2-3.8	m	34-35		7-9		27-38		11-14		
		coal	1.0-2.0	24	1.4-1.9	m	43		10		32		14		
			2-5	3	2.0-2.3	m	38		5		27		10		
					2.4-2.8	m	32		11		35		14		
					2.9-3.2	m	34		8		28		13		
					3.2-3.7	m	34-35		7-8		30-37		14		
Luton, England Full Scale (1957)	2	sand			2.5	12m	7-18	13	1-7		6-15	9	2-8	4	

Prepared by Gary A. Rice and John L. Cleasby, Iowa State University, March, 1974.

Location	Ref.	Media Type	Media Size* (mm)	Media Depth (in.)	Filter Rate gpm/sq ft (U.S.)	N**	Suspended Solids (mg/ℓ) Influent Range	Influent Avg	Effluent Range	Effluent Avg	BOD5 (mg/ℓ) Influent Range	Influent Avg	Effluent Range	Effluent Avg	% Removal
Luton (Cont'd)															
Full Scale (1967)	7	sand	0.85-1.7	36	3.4	3m	28-35		9-10		9-10		3-4		
		sand	0.85-1.7	36	5	3m	13		8		5		3		
Pilot Scale Upflow		sand	0.85-1.7	60	3.4	3m	29-35		5		9-10		3		
		sand	0.85-1.7	60	4.8	3m	13		6		5		3		
Derby, England Pilot Study (1970)	4	sand	1.2-2.3	24	2.0	4	25-35	29	7-14	10					
					3.0	2	29-31	30	10-13	12					34-45
					4.0	5	27-31	29	12-14	13					11-39
					8.0	2	24-29	27	16-18	17					
		sand	1.2-1.7	24	2.0	4	26-35	29	7-15	10					45
					3.0	2	29-31	30	9-13	11					15-47
					4.0	5	24-32	28	11-14	13					
					8.0	2	23-29	26	16-18	17					
Triple Media "a"		coal	1.4-2.3	8	2.0	2	25-35	29	6-12	8					
		sand	1.2-1.4	8	4.0	4	27-28	27	11-13	12					33
		garnet	0.7-0.85	8	8.0	3	23-29	26	15	15					
Upflow Sand "a"		sand	0.7-2.3	24	2.0	2	28-37	33	9-15	12					
					4.0	3	27-30	27	13-14	13					
					8.0	2	23-29	26	16-17	17					
Triple Media "b"		coal	1.4-2.3	8	3.0	2	29-31	30	7-11	9					38
		sand	0.85-1.0	8	4.0	2	29-32	31	10	10					38
		garnet	0.7-0.85	8											
Upflow Sand "b"		sand	0.85-2.3	24	3.0	2	29-31	30	10-14	12					
					4.0	2	28-32	30	11-14	13					

Location	Ref.	Media Type	Media Size* (mm)	Media Depth (in.)	Filter Rate gpm/sq ft (U.S.)	N**	Suspended Solids (mg/ℓ) Influent Range	Influent Avg	Effluent Range	Effluent Avg	BOD_5 (mg/ℓ) Influent Range	Influent Avg	Effluent Range	Effluent Avg	% Removal
Derby, England Pilot Study After Trickling Filter Improvement	4	sand	1.2-2.3	24	3.0	2	22-25	24	9-10	10					31
					4.0	5	20-24	22	7-10	9					50-64
					6.0	2	19-26	23	10-11	11					35
		sand	1.2-1.7	24	3.0	2	22-25	24	9-10	10					54
					4.0	4	21-24	23	8-10	9					53-65
Triple Media "b"		triple (see above)			3.0	1	22	22	8	8					
					4.0	5	20-24	22	6-9	8					
					6.0	2	19-26	23	9	9					
Upflow Sand "b"		sand (see above)			3.0	2	22-25	24	8-10	9					69
					4.0	4	21-24	23	8-11	10					59-71
Upflow Sand		sand	1.2-2.3	36	6.0	4	19-26	23	9-11	10					43-67
Ames, Iowa Pilot Study (1965)	6	sand	0.55 ES 2.36 UC	24	2.0	15	11-49	20	1-15	6	38-115	56	13-49	24	
					4.0	15	10-58	19	1-24	6	29-130	53	15-65	23	
					6.0	15	8-60	18	2-27	6	25-132	50	13-74	24	
Pilot Study (1973) Unpublished		dual media													
		coal	0.9 ES	12	1.7	12	21-75	33	1-8	4	39-85	57	6-19	13	
		sand	0.4 ES	12											

‡ Blank spaces in table due to data missing or not presented in manner needed for table, e.g., for averaging. All mg/ℓ values rounded to nearest 1 mg/ℓ.
* Range in size given, British practice, or ES (effective size) and UC (uniformity coefficient), U.S. practice.
**N = number of values reported in the range and average presented.
N generally represents individual filter runs unless followed by the letter m which indicates the number of average monthly values presented.
m without numeral means average of several months data (unspecified duration).

REPORTED EFFICIENCIES FOR
DIRECT FILTRATION OF ACTIVATED SLUDGE PLANT EFFLUENTS ‡

Location	Ref.	Media Type	Media Size* (mm)	Media Depth (in.)	Filter Rate gpm/sq ft (U.S.)	N**	Suspended Solids (mg/ℓ) Influent Range	Influent Avg	Effluent Range	Effluent Avg	BOD_5 (mg/ℓ) Influent Range	Influent Avg	Effluent Range	Effluent Avg	% Removal
West Hertfordshire England (1968) Pilot Scale "Boby" Upflow Filter	10	gravel	40-50	6	2.16		10-89	44	1-2	2		58		3.9	
		gravel	8-12	10	4.0		9-70	37	2-7	3.7		53		4.6	
		gravel	2-3	10	5.0		12-128	55	1-17	7.1		42		5.6	
		sand	1-2	60	6.0		5-97	37	2-22	9.9		35		4.7	
Letchworth, England (1968) Pilot Scale "Boby" Upflow Filter	9	gravel	20-30	4	3-4	81	10-26	22	1-12	5.5					
		gravel	10-15	4	4-6	66	10-28	16	2-15	6.7					
		gravel	2-3	4	6-8	65	7-24	14	6-14	8.8					
		sand	1-2	60											
Los Angeles, Calif. (1961) Preliminary Tests	5	sand	0.95 ES 1.6 UC	11	2	5	19-34	27	7-21	15	6-15	10	2-8	4	
Philowith, Ore. (1967) (Extended Aeration AS)	1	mixed media	-	30	5		30-2180	59	1-20	4.6	17-36	26	1-4	2.5	
Peoria, Ill. (1964) (High rate AS)	3	sand			1.1			35		8		45		17	
Cleveland, Ohio	11	dual media													
		coal	1.78 ES 1.63 UC	60	8	1	20		5		19		14		
					16	1	27		8		9		5		
		sand	0.95 ES 1.41 UC	24	24	2	22-23		9-11		9-10		4-6		
					32	1	29		14		9		7		
Declining Rate Filters at Cleveland. (Avg filter rate presented)		dual media													
		coal	4.0 ES 1.5 UC	60	8	1	13		4		7		5		
					16	1	13		4		7		5		
		sand	2.0 ES 1.32 UC	24	24	1	13		6		7		5		

Intermittent Sand Filter Operation

*J.H. Reynolds, S.E. Harris, D.W. Hill, D.S. Filip and E.J. Middlebrooks**

INTRODUCTION

Intermittent sand filtration has been proposed for upgrading waste stabilization lagoon effluent. Considerable research has been conducted to determine the ability of intermittent sand filters to improve lagoon effluent, however, studies have not been specifically conducted to determine various operational problems associated with intermittent sand filters.

This paper presents a brief overview of the operations of intermittent sand filters.

LITERATURE REVIEW

Slow Sand Filters

Intermittent sand filtration is actually a modification of slow sand filtration. Therefore, the experience gained in the operation of slow sand filters has application to operation of intermittent sand filters.

Initially, slow sand filters were operated by applying a continuous volume of raw or chemically treated water to the filter until a predetermined headloss (usually 3 to 4 feet; 0.194 to 1.22 m) was reached. At this point the filter was taken out of service and usually allowed to dry. Men and machines would then go onto the filter to recondition the surface by one of several methods.

One method of reconditioning involved scraping the top two inches (5.1 cm) of sand, transporting the scraped sand by hydraulic ejectors to a sand washer, washing the sand, storing the sand or transporting it back to the filter, and restarting the flow to the slow sand filter at a slow rate until the filter became "ripened" (a "schmutzdecke" or filtering skin buildup) at which time normal hydraulic loading rates were used (Fuller, 1908a; Gaub, 1915).

Another filter reconditioning method involved intensely raking the surface of the slow sand filter to breakup the surface mat. Story (1909) reported that raking followed by a drying period, provided an economical method of restoring the filter to its original filtering ability. Saville (1924), at the Hartford, Connecticut plant, found that four rakings between scrapings provided an economical method of maintenance. Saville (1924) reported that five men could rake a bed in two hours, while it took eleven men sixteen hours to scrape and wash the same bed.

A simple method of filter cleaning called the Brooklyn method was reported by de Varona (1909), Fuller (1908b), and Gaub (1915). The Brooklyn method consisted of lowering the water depth over the filter to just a few inches. Boards were driven in the sand surface to separate the sand filter into sections. After this, unfiltered water was run in streams over each section of the sand filter, while men with rakes and shovels agitated the sand to suspend the dirt and organic matter. Gaub (1915) considered this method costly because it required considerable manpower. A foreman and fourteen men were required to clean a 0.75 acre (0.384 hectare) bed in one eight hour working day (Gaub, 1915).

Hydraulic ejectors, reported by Gaub (1915) and Karalekas (1952), utilized water under pressure entering from the bottom while sand was shoveled into the top. This formed a suspension between the sand and water which could be transported by lengths of hoses to sand washers, storage bins, or slow sand filters. Gaub (1915) reported that the ejectors had a tendency to stratify the sand and recommended specifying a low uniformity coefficient sand for use in slow sand filters.

The mechanical sand washers utilized a method of agitating the dirty sand by clean wash water so that the organic material, fines, and debris were suspended and withdrawn to waste. The sand being heavier dropped to the bottom of the machine and was transported by ejectors to filters or storage. Gaub (1915) reported that the Nichlas washer was the predominant sand washer in use during the early 1900s, while Karalekas (1952) reported the use of Allan Hazen sand washers at Springfield, Massachusetts, in 1952.

Fuller (1908a, 1908b) reported in 1908 that the use of present cleaning methods (scraping and raking by manpower) were seriously retarding the use of slow sand filters. Fuller (1908a) reported the use of a mechanical sand washer which washed the sand as it lay in place on the filter while the slow sand filter was in full operation. Other authors in the literature have also reported the use of the Blaisdell machine and how higher loading rates were possible from its use (Anonymous, 1918; Bailey, 1937; de Varona, 1909; Gaub, 1915).

Smith (1945) referred to a machine used by the McMillan Slow Sand Plant which operated on a dry bed. A screw conveyer attached to the front pushed sand into a receiving box. This receiving box was attached to an ejector which transported the sand to a sand separator located at the top of the machine. The sand was washed and deposited on the filter behind the machine as it traveled across the filter. This machine could scrape and clean 7 cubic yards (5.35m^3) of sand an hour. Smith (1945) also noted that a mechanical raking machine was used to breakup the "schmutzdecke" (filtering skin) between filter cleanings at this plant.

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Operational problems have been cited by several authors Flu, 1922; Madiley, 1921, 1927; Story, 1909). Flu (1922) reported that insects, crabs, and fish created a nuisance at the Weltercreden, Dutch East Indies slow sand filter plant because they bored through the filtering layer. Madiley (1921) agreed with Flu's assessment that fish, crabs, and insects caused a deterioration in the effluent quality because of the breaking and floating of the filtering skin ("schmutzdecke"). Madiley (1921) noted that the sunlight caused excessive algal growth in the filtering skin. Madiley (1921) suggested putting screens over the influent pipes to solve the fish and crab problem and increase the depth of water over the filters to solve the algal growth problem.

On the same filters at Madras City, Madiley (1927) cited a failure of slow sand filters. Ferrous sulfide presence in the quartz filter sand produced hydrogen sulfide gas in the hot, humid climate. The gas collected in pockets within the sand bed and eventually burst through the filtering layer. No amount of cleaning of the filters or pretreatment of the water seemed to help. It is emphasized that the problems cited by Flu (1922) and Madiley (1921, 1927) took place in tropical climates. Madiley (1921) stated that slow sand filters located in the tropics worked quite differently than slow sand filters located in more moderate climates such as England.

Story (1909) reported on the operation of filters at Ludlow Reservoir at Springfield, Massachusetts. This was a temporary solution for Springfield until a new source of potable water could be found. In June 1907, when the slow sand filters were placed into operation for the year, the raw water had large numbers of *Uroglena sp.* and *Asterionella sp.* (diatom). These organisms formed a cement like layer on the filter causing rapid clogging. Story (1909) found that intense raking followed by a period of sunlight and drying worked almost as well as scraping in renewing the filtering ability of the sand. *Anabaena sp.* (a blue-green alga) appeared in the water supply in late June, and when these organisms died they created numerous problems for the filtering plant. Lengths of filter runs were short and taste and odor problems plagued the plant. However, the filters continued to give a good quality effluent though this difficult period. Story (1909) found that subsurface clogging had taken place during the summer of 1907, because new sand had been laid over old unscraped sand during the spring. The clogged sand was removed and length of filter runs improved slightly. Story (1909) reported that intermittent sand filtration was also tried at the Ludlow plant, but this produced about the same results as slow sand filtration.

INTERMITTENT SAND FILTERS

The operation of intermittent sand filters for treating sewage was much like the operation of slow sand filters for treating water except for the intermittent operation. Daniels (1945) has stated that unless they are carefully and intelligently operated, intermittent sand filtration can be a nuisance and even suffer total failure. Daniels (1945) noted that the term "intermittent" was often overlooked. Many intermittent sand filters were continuously operated, and this had a serious effect upon the bed. The sand filter needs a rest period between applications to keep the bed aerobic and functioning properly. This resting period is needed because the filtered substances must be mineralized or oxidized within the top layer of sand or the pores will rapidly clog (Fair et al., 1968). Steel (1960) stated that complete resting of the bed is needed if septic conditions are present in parts of the bed. The resting period should be at least one week and two to four weeks if the condition is serious.

An example of a well operated filter was one at the Lawrence Experiment Station (Massachusetts Board of Health, 1912). Sand has not been removed from the surface of this filter in 23 years of operation. It had a surface area of 1/200 acre (0.002 hectare). This anonymous (1912) article stated that within this time 2,395,532 gallons (9,068.1m^3) of sewage containing about 6,000 lbs (2,727.3 kg) of organic matter had been applied to the intermittent sand filter. This example should demonstrate the potential of intermittent sand filtration for upgrading wastewater treatment plants.

Even when intermittent sand filters were operated properly eventually the filters became plugged and cleaning was necessary. Plugging occurred when the daily dosage of sewage failed to percolate through the filter bed in a 24-hour period. For multiple loadings, a cleaning was necessary when the preceding dosage still covered the surface at the time of the next loading (Furman et al., 1955).

When cleaning was necessary, the bed was taken out of service and allowed to dry. The surface mat of strained solids would crack and curl up. This mat, composed of organics and sand, was then scraped off and wasted or washed to remove the organic portion. An economical number of rakings between scrapings was used to increase filter runs. The amount of sand surface removed depended upon the condition of the influent sewage as well as other external conditions. Usually only a 1/4 - 1/2 inch (0.6 - 1.2 cm) thickness of sand surface needed to be removed, but this was extended to 2 inches (5.1 cm) at times (Babbitt and Baumann, 1958). Cleaning and removing of the sand continued until the minimum depth of filter sand was reached. At this point the intermittent sand filter was thoroughly scraped and clean sand was added.

The winter operation of intermittent sand filters presented special maintenance and operational problems as the sand surface of the filters could not be allowed to freeze. Daniels (1945) discusses three methods of managing intermittent sand filters during the winter. The first method, called the Brockton method, involves furrowing and ridging the beds at the start of winter. When the ice sheets are formed, they would come to rest upon the ridges and eventually would break up. At the start of a cold spell the beds are loaded heavier to provide extra protection against the freezing of the sand surface.

The second method, called the Worchester method, is similar to the Brockton method except that during the last scraping of the filters in the fall the scrapings are heaped into piles. These piles serve as a support for the ice layer and also require much less cleaning and rearrangement when spring comes.

The third method of managing the sand bed is identical to the regular summertime operation. However, much care has to be taken to prevent the ice layer from settling upon the flat sand surface and solidly freezing the surface. If the incoming influent dosage is unable to thaw out the settled ice layer, the filter will be unusable until the spring thaw arrives (Metcalf and Eddy, 1935). Although more expensive, intermittent sand filters could be covered by wooded planks or plastic covers during the winter to prevent freezing.

COLD WEATHER OPERATION

The operation of intermittent sand filters during harsh winter conditions in states like Utah has been a serious concern to many design engineers. A study was conducted at Utah State University (Harris et al., 1975) to evaluate several difficult operational modes. The study was divided into a warm weather experimental period and a winter experimental period.

During the winter experimental period a hydraulic loading rate of 3700 m^3/ha·d (0.4 mgad) was applied to four of the six filters (one remained at 1900 m^3/ha·d (0.2 mgad) loading, one was out of service) employed in the study. It was anticipated that cold weather and freezing would create serious winter operational problems. Therefore, four separate operational modes were studied. The first mode used a furrow technique. That is, the surface of the filter was plowed into furrows (small hills and valleys). The second operational model involved placing 0.3 m (1 ft) wooden stakes at 1.22 m (4 ft) centers across the filter surface to break up any ice sheets which formed. The third method involved maintaining at least 0.3 m of water standing on the filter at all times (flooding). The fourth operational mode was the control and involved making no changes in the filter operation or configuration.

The winter experimental period was conducted under fairly harsh climatic conditions. Ambient air temperature dropped to below -23°C (-10°F) for several consecutive days on several occasions and nighttime temperatures were constantly below freezing. Thus, ice sheets formed on the top of each filter. However, in general all operational modes studied performed satisfactorily.

All of the data collected during the winter experimental period have been averaged and these average values are reported in Table 1. The number of individual data points averaged depends on the length of the particular filter run. However, in no case were fewer than three data points used to obtain an average value. The biochemical oxygen demand removal and suspended solids removal performance for each filter is shown graphically in Figures 1 and 2.

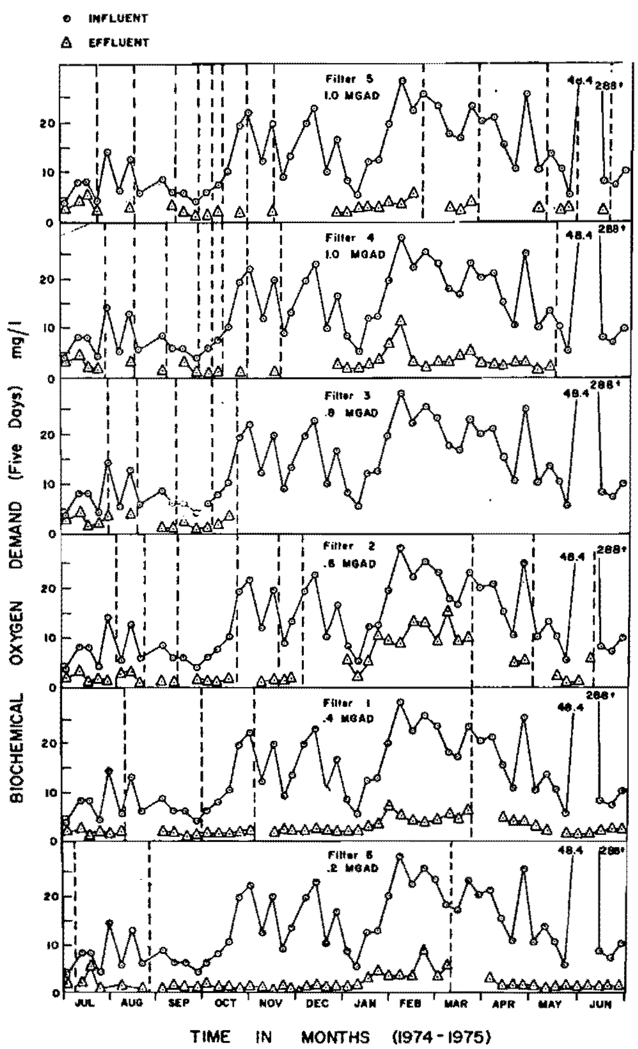

Figure 1. Filter biochemical oxygen demand performance (mgad = 9360 m^3/ha·d).

Table 1. Average of all samples during winter (1974-1975) operation.

Filter No.	Treatment	BOD_5 (mg/l)	COD (mg/l)	SS (mg/l)	VSS (mg/l)	Total Phosphorus (mg/l)	O-PO_4-P (mg/l)	NH_3-N (mg/l)	NO_2-N (mg/l)	NO_3-N (mg/l)	pH	Temp. (°C)	DO (mg/l)
Influent		18.0	64.3	28.3	25.6	3.462	2.866	4.961	0.017	0.084	8.6	3.3	9.9
1	3 700 m^3/ha·d furrowed	4.1	17.9	3.5	3.2	3.072	2.909	1.149	0.029	4.335	7.5	3.0	8.0
2	3 700 m^3/ha·d head maintained	9.4	33.2	9.6	7.6	3.105	2.840	4.609	0.037	1.031	7.9	2.8	7.8
4	3 700 m^3/ha·d staked	4.0	19.2	5.1	3.9	3.209	2.975	1.777	0.126	5.065	7.7	2.7	8.6
5	3 700 m^3/ha·d no modification	2.6	18.0	3.3	2.7	3.247	3.078	1.983	0.093	3.208	7.7	2.7	8.6
6	1 900 m^3/ha·d no modification	3.7	16.7	3.4	2.8	3.106	2.888	2.347	0.022	2.527	7.7	2.2	8.3

The averages for filter number 2 show a marked difference from the other four filters in operation. Anaerobic conditions caused by a constant head on the filter greatly reduced its removal capacities. As can be seen, BOD_5, COD, SS, VSS, and NH_3-N are twice as high in the effluent of filter number 2 as in the others. The NO_3-N concentration is half the value of the four other filters. The DO concentration is almost as high, but probably because of agitation as the water flowed from the sample ports. A very objectionable odor accompanied the effluent of filter number 2.

Filters 6, 1, 4, and 5 (control, furrowed, staked, and raked respectively) all performed satisfactorily. There were minor variations among them, but each produced quality effluents. However, the overall quality dropped below that of the warm weather period.

The length of filter run during the winter experimental period is shown in Table 2. The length of run varied from 58 days for the filter which was raked to 130 days for the furrowed filter. Each of the filters had a hydraulic loading rate of 3700 m^3/ha·d (0.4 mgad). Filter number 6 had a filter run length of 188 days; however, the hydraulic loading rate was only 1900 m^3/ha·d (0.2 mgad).

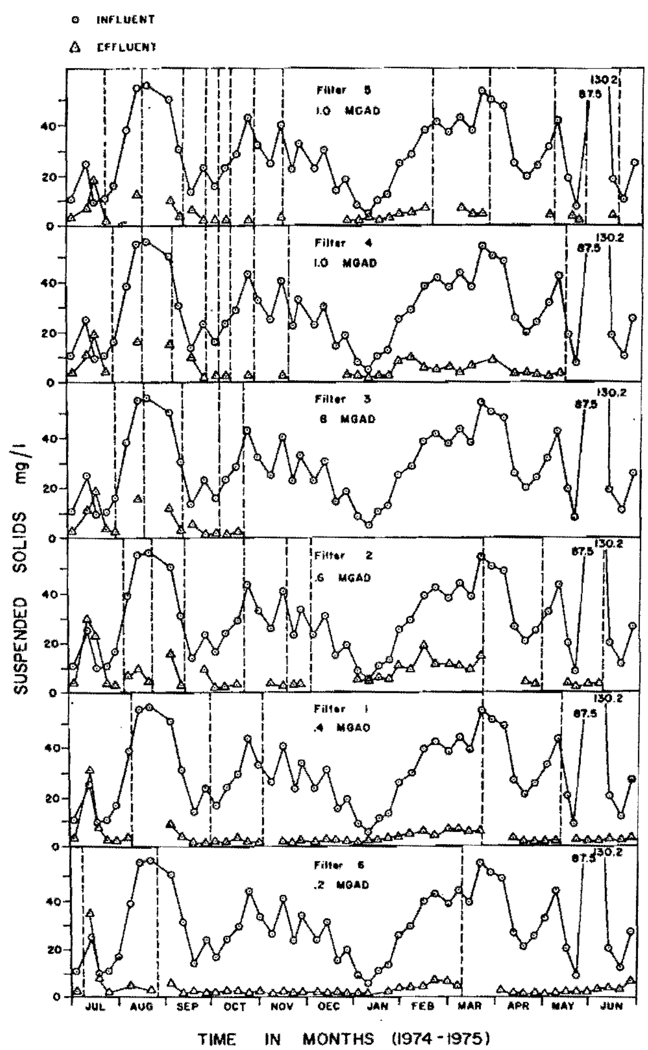

Figure 2. Filter suspended solids performance.

Table 2. Length of filter run for winter experimental period.

Mode of Operation	Filter No.	Hydraulic Loading Rate (m^3/ha·d)	Length of Filter Run (days)
Control	6	1 900	188
Furrowed	1	3 700	130
Flooded	2	3 700	73
Staked	4	3 700	92
Control (raked)	5	3 700	58

CLEANING

Intermittent sand filters are generally loaded hydraulically once a day during a four to six hour period. That is, the amount of water to be applied to the filter is placed on the filter in a period of four to six hours and then allowed to percolate through the filter bed for the remainder of the twenty-four hour period. A filter is considered to be plugged and to require cleaning when the applied dose of water will not percolate through the filter in a single twenty-four hour period.

When the filter is plugged, it is taken out of service and the top two or three inches of filter sand is removed. For small systems (less than 2 MGD), this filter sand could be removed with a small tractor with a blade and front end loader. For larger systems (greater than 2 MGD) more sophisticated cleaning equipment may be justified (see Literature Review).

The spent filter sand may be washed in a conventional sand washer and then replaced on the filter. It may be disposed in a landfill or employed as a soil conditioner. Because the sand is rich in organic material, when mixed with a clay soil, it produces a nutrient rich fertile soil. Studies conducted at Utah State University (Elliott et al., 1976) indicate that spent filter sand has an excellent affect on soil productivity.

SUMMARY

The operation of intermittent sand filters is relatively simple and economical. Experience in the operation of slow sand filters and sewage intermittent sand filters can be applied to the operation of intermittent sand filters to upgrade lagoon effluent. Winter operation has not presented any serious operational problems to date. Cleaning of the intermittent sand filters appears to be the greatest and most costly operational problem associated with intermittent sand filters.

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Management Alternatives for Training Wastewater Treatment Plant Operators

*Robert A. Gearheart**

INTRODUCTION

A need exists in the complex training area of wastewater treatment plant operations to effectively determine training needs. As the need for operating personnel increases with new and upgraded plants, and the process state of the art increases, the continuing education and entry level training problems become more complex. The skill levels in wastewater treatment plant operation is quite varied and in many cases is based upon specified attainment of formal degrees or certificates of completion. A training management system has been developed to assist in identifying the training needs for any type of treatment training. The training management system identifies those training modules needed to perform a given task for specific treatment process, orders the training modules in sequence of prerequisities, determines the time necessary for training, and the cost of implementing the training. The system was designed specifically for training personnel associated with municipalities, wastewater authorities, industry, and state regulatory authorities. Transfer of knowledge in a training sense is nothing more than a simulation of the working world. A careful survey of the tasks associated with operation of waste treatment process was the initial step in developing the training management matrix. The various tasks must then be arbitrarily placed under job descriptions. For any given plant these tasks might be reordered under different job descriptions. As an example, under one-man plant operations all of the tasks will be under one job description.

Under the present mode of wastewater treatment plant operator certification there is a minimum of relationship between what it takes to perform a job and those skills necessary to successfully pass certification examinations. This training matrix does not address certification examination not based upon operational skills. Hopefully, in the near future this discrepancy will not exist and training will satisfy both requirements.

Task analyses have been performed on wastewater treatment plant operators for various types of treatment processes. A task analysis is comprised on an action performed by a worker on a subject or object with some result or output.

Table 1 shows the various elements of wastewater treatment operation subfunctions. Each of these operations can then be further divided into waste treatment plant processes.

THE SYSTEM

The purpose of this system is to provide a tool by which a manager may study the training needs of his personnel. The current usage is oriented toward sewage treatment plant personnel, but the program could be used for any situation in which the jobs and training requirements can be sufficiently defined.

The definition process consists of several steps and results in a set of data that is stored on disc pack file (auxiliary storage on the computer). This file will be called the database. Describing the database definition process will be a good base for understanding the other functions of the system. The first step in the process would be to compile a set of tasks that can be combined to describe all the jobs in the particular situation under examination. The second step is to develop definitions of training modules that will cover the educational needs of all of the tasks. The module definition must include: a descriptive name, the

Table 1. System subfunction.

I. Separation of coarse-suspended and floating matter—bar racks, comminuters, screens.

II. Separation of grease and oil—scum collectors, skimming tank.

III. Separation of finely suspended matter and various types of settleable solids—grit chambers, primary clarifiers, chemical precipitation tanks.

IV. Separation or stabilization of organic matter in suspension, the colloidal state, or solutions; biological treatment; soil mantle treatment (land treatment, irrigation), trickling filter, activated sludge, oxidation ponds.

V. Reduction and/or disposal of mineral organic solids separated in preceding operations (sludge treatment), anaerobic digestion, filtration, elutrication, incineration, concentrating (pressing, centrifuging).

VI. Tertiary treatment—removal of suspended and dissolved organic matter removed above—removal of nitrogen and phosphorus species. (Filtration, absorption, stripping, chemical precipitation, ion exchange.)

VII. Disinfection of wastewater of fecal contaminant indicator, chlorination, ozonation.

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total number of hours required to complete it, the total number of weeks required to complete it, a dollar cost of the module to the employer, and the modules that would be prerequisites. The third step is to list for each previously defined task, the training modules that would provide the educational background needed to perform the task. The total definition is punched on cards and is edited by a program called TRADATA. If no errors are found in the definition, TRADATA writes it to the database disc pack file. When the database for a given situation has been created, a manager can use the interaction program, TRAMODEL, to define sets of jobs and generate reports that will allow him to study their training requirements.

OBJECTIVE

The matrix depicts the modules of instruction which have been determined to be necessary to perform the given task. The matrix would serve as a curriculum guide for training an individual who must perform a given set of tasks with designated processes. The matrix would serve as planning tools in development of training materials (instructional packages) and visual aid requirements.

FORMAT

All the identifiable tasks (Table 2) are listed on the vertical dimensions of the matrix, somewhat grouped into function components such as: manager, supervisor, operator, laboratory technician, maintenance personnel, and laborers. The task analysis is independent of the functional nomenclature in terms of analysis, though, it serves simply as an arbitrary grouping of tasks. The important idea is that all of the tasks are listed that are performed by someone at the plant.

The instructional modules are listed on the horizontal dimensions of the matrix. These instructional modules are arbitrary groupings of training behaviors commonly used in training of operators. The arrangement of the modules are in somewhat ascending order of complexity of objectives. The only values in this is the pictoral arrangement for management reasons. The intersection of instructional modules and task performed is determinal by backing out the training sequence or produce a given act or task and given types of processes. The linkages between the modules will not be shown due to space limitation, but will be shown for a typical training need. The matrix will be developed such that a computer program will be used to identify modules and the sequence of presentation to meet a given set of tasks.

MANAGER

The manager has overall responsibility for the operation and maintenance of the wastewater treatment plant, the total system. This operational responsibility includes the overall delegational responsibility and scheduling of personnel, maintaining adequate personnel work records, resolving personnel problems, assigning maintenance personnel to replace or repair malfunctioning or cooperative equipment, and establishing priorities for the maintenance of equipment, supervising the installation of new equipment, and the planning and ordering of all supplies to support the day-to-day operation of the

Table 2. Task Analysis.

0100 Unskilled Labor

0101 - Prepare surfaces for protective coatings
0102 - Apply protective coatings to surfaces
0103 - Ground keeping task
0104 - Install and remove equipment and/or parts of plant processes
0105 - Deliver or pick up information or parts necessary for operation of plant
0106 - Perform housekeeping duties
0107 - Deliver process chemical to point of application
0108 - Remove solids to point of ultimate disposal

0200 Maintenance

0201 - Recognize inoperative equipment in the various unit processes
0202 - Read blueprints of plant design including structural, electrical, hydraulic, and mechanical
0203 - Repair mechanical equipment by replacing parts, welding and cutting
0204 - Prepare preventative maintenance procedures
0205 - Prepare normal operation procedures to reduce maintenance requirements
0206 - Prepare parts inventory and orders for replacement parts
0207 - Repair instrumentation used in lift stations and wastewater treatment plants
0208 - Lubricate equipment
0209 - Calibrate equipment
0210 - Repair equipment by a construction replacement parts
0211 - Maintenance of power system for plant
0212 - Read maintenance and repair manuals for various pieces of equipment both by mechanical and electrical in plant

0300 Laboratory Technician

0301 - Obtain samples from plant processes
0302 - Order chemicals and supplies for analysis
0303 - Perform standard wastewater analysis both for process control and/or regulatory reporting
0304 - Log process control and regulatory reporting data
0305 - Perform process control bench scale experiments
0306 - Perform standard industrial wastewater analysis (heavy metals, solvents, refractory organics). Analytical instrumentation
0307 - Analyze plant performance data for purposes of reports and operational strategies
0308 - Perform routine maintenance of analytical equipment used in laboratory
0309 - Perform routine laboratory cleaning duties, work area, glassware, equipment

0400 Operator

0401 - Manipulate valves pertaining to process control
0402 - Perform routine cleaning of unit processes

Table 2. (Continued)

0403 - Collect samples for analysis to control processes, observe unit process and indicate normal or abnormal operations
0404 - Perform routine operations, housekeeping functions on plant premise
0405 - Compute data concerning flow volume, characteristics of influent and effluents, and amount of chemicals used in treatment
0406 - Enter computed data into plant records
0407 - Record accurate readings and occurrences in plant log books
0408 - Monitor control readouts for purposes of plant operations
0409 - Perform routine preventative maintenance to system and subfunctions, unit processes, pumps, motors, controllers, etc.

0500 Supervisor

0501 - Prepare work schedule
0502 - Observe process operation by observing operation, consulting with operator, and examining laboratory control data
0503 - Transfer daily operation log to acceptable form to be received by management
0504 - Review and implement preventative maintenance program
0505 - Establish operational procedure
0506 - Implement plant safety program
0507 - Devise laboratory analysis program for plant operators and regulatory agency
0508 - Identify manpower needs as to tasks needed to be performed
0509 - Translate total system objectives to operators
0510 - Observe and document operating experience of various pieces of equipment for future information in replacement or expansion
0511 - Observe and document effectiveness and efficiency of plant personnel for purposes of promotion, training, and manpower needs

0600 Management

0601 - Prepare annual budgets for plant operation
0602 - Prepare capital expenditure proposal for federal, state, and private financing
0603 - Determine alternatives for purchasing replacement and new equipment
0604 - Present operating, financing, and technological alternatives to public and private groups
0605 - Establish personnel policies
0606 - Initiate and sustain relationship with local, state, and federal regulatory agencies
0607 - Prepare annual report including cost of service, plant efficiency, personnel changes, and abnormal events
0608 - Establish plant safety program
0609 - Implement plant safety program
0610 - Establish plant training program
0611 - Implement plant training program
0612 - Establish plant operation procedures
0613 - Prepare emergency operational procedures

Table 2. (Continued)

0614 - Establish and maintain data bank on plant operations
0615 - Design information storage and retrieval system for plant operation data, payroll data, plant financial data, etc.
0616 - Conduct meetings with state and local officials, engineering and construction officials, and plant personnel

wastewater treatment plant and associated pumping stations. The manager is also responsible for coordinating with consulting engineers, state and federal regulatory agencies, and proprietory equipment representatives.

SUPERVISOR

The supervisor has responsibility for the operation of a wastewater treatment plant. This responsibility includes day-to-day scheduling and keeping of personnel records for each work shift, holidays, and weekends. The supervisor evaluates personnel for promotions, pay increases, education benefits, and termination of employment. The supervisor is responsible for storing and distributing supplies and equipment as needed and for requesting additional supplies, responsible for maintaining safe and clean work conditions. The supervisor must maintain complete and accurate records on samples taken on a scheduled basis, supervisory or actually doing some of the analytical analysis, insuring that laboratory samples are ready for collection, responsible for providing on-the-job training to employees, daily checking the operation of the entire plant and the functioning of all major items of equipment, review of operational procedure for each unit processes to optimize efficiency, insuring that the relieving day shift knows what has transpired on the previous shift, and replacing all personnel who are not available for work on a particular day. The supervisor is the information link between total plant objectives and the personnel who operate the plant.

OPERATOR

The operator has the responsibility during an assigned shift for the actual operation of a wastewater treatment plant. This responsibility includes operating pumps, valves, motors, and related machinery and equipment, regulates and adjusts meters, flow meters, chlorinators, and digester temperature. He performs minor maintenance on motors, pumps, gages, and chlorinators, conducts numerous daily and periodic tests of sewage, and inspection of meters, gages, and indicators. The operator maintains a log of plant operation and performs house-keeping duties in connection with the maintenance of buildings and grounds. The operator is the information link between the total plant objectives and the equipment which does the work.

MECHANIC-ELECTRICIAN

The mechanic-electrician has the responsibility for the preventative and corrective maintenance of all mechanical, electrical, and hydraulic equipment at the wastewater treatment plant and at the pumping station. This maintenance responsibility includes the replacement of defective parts, minor repair work, lubrication and oiling of equipment, checking on the operation of motors and pump, and making adjustments as needed. In addition, the mechanic-electrician is responsibile for the preventative maintenance of electrical equipment such as cleaning contact paints as well as the installation of new and relatively simple electrical equipment.

LABORATORY TECHNICIAN

The laboratory technician has the responsibility of performing and recording the analytical analysis required for state and federal regulatory agency compliance. In addition, the laboratory technician is responsible for performing and recording process control analyses which are required for efficient plant operation. The laboratory technician is responsible for cleaning and maintenance of all laboratory equipment and facilities and for recommending the purchase of supplies, equipment, and chemicals. The laboratory technician is responsible for communicating the results of analysis to the plant operation staff for their use in plant operations. Pertinent operational parameters will be graphed by the laboratory technician to facilitate changes in flow, strength, temperature, etc. on process operations.

UNSKILLED LABOR

The unskilled laborer has the responsibility for the routine and repetitive operating and housekeeping tasks required during each shift. On orders received from the shift operator, the laborer could make adjustments in valving pump settings and other devices as well as maintain and lubricate certain pieces of equipment. The laborer directed by shift operator could be involved in process chemical storage and distribution and final solid handling procedures. Routine housekeeping tasks associated with the exterior buildings and processes, interior of buildings and processes. The grounds and protective coating application are the duties of unskilled labor.

TASK LISTING

This report is a listing of the tasks as they have been defined in the database. It contains the task code number, the descriptive name, and the module code numbers of the training modules needed to provide the educational background to perform the task. This listing is in order on task code numbers. It can be used to edit the entries in the database or as a reference when analyzing a set of jobs. It is written by either TRADATA or TRAMODEL.

JOB TASK DEFINITIONS

CODE	NAME	TRAINING MODULES
101	PREPARE SURFACE	110 120
102	APPLY PROTE COAT	110
201	MAINTENANCE 01	110 120 130
202	MAINTENANCE 02	110 120 130 309 310 320 410 415 420 510
203	MAINTENANCE 03	110 120 130
301	LAB 01	110 120 130 309 310 320 410 420 510 520 530 602
302	LAB 02	110 120 130 309 310 320 410 420 510 520 530 602 604
303	LAB 03	110 120 130 309 310 320 410 420 510 520 530 602
401	OPER 01	110 309 410 510
402	OPER 02	110 120 130 309 310 320 410 415 420
403	OPER 03	110 120 130 309 310 320 410 415 602 604
404	ACTIVATED SLUDGE	110 120 309 310 320 410
501	SUPERVISOR 01	110 120 130 309 310 320 410
502	SUPERVISOR 02	110 120 130 309 310 320 410 415 602 604
503	SUPERVISOR 03	110 120 130 309 310 320 410 415 420 510 520 530 602
601	MANAGEMENT 01	110 120 130 309 310 320 410
602	MGT 02	110 120 130 309 310 320 410 415
603	MGT 03	110 120 130 309 310 410 415
604	MGT 04	110 120 130 309 310 410 415 420 510 520 602
605	MGT 05	110 120 130

SYNTHETIC MANAGEMENT MATRIX

This report shows the relationship between the training modules and the tasks as defined in the database. It is a two-way table with the training module codes across the top and the task codes down the side. The entries in the table are 1 if the module is required for a specific task or 0 if it is not required. It is written in command by TRAMODEL.

SYNTHETIC MANAGEMENT MATRIX

SEQ NUM	TASK CODE	TASK NAME	TRAINING MODULES 110	120	130	309	310	320	410	415	420	5[illegible]
01	101	PREPARE SURFACE	1	1	0	[illegible]	0	0	[illegible]	0		
02	102	APPLY PROTE COAT	1	0	0	0	0	0		[illegible]		
03	201	MAINTENANCE 01	1	1	1	0	0	0	[illegible]		[illegible]	
04	202	MAINTENANCE 02	1	1	1	1	1	1	[illegible]	[illegible]	1	[illegible]
05	203	MAINTENANCE 03	1	1	1	0	0	0		0	0	
06	301	LAB 01	1	1	1	1	1		1		1	
07	302	LAB 02	1	1	1	1	1	1	1		1	
08	303	LAB 03	1	1	1	1	1	[illegible]	1		1	1
09	401	OPER 01	1	0	0	1	0	0	1		[illegible]	
10	402	OPER 02	1	1	1	1	1	1	[illegible]	1	1	
11	403	OPER 03	1	1	1	1	1	1	1	1		
12	404	ACTIVATED SLUDGE	1	1	0	1	1	1	1			
13	501	SUPERVISOR 01	1	1	1	1	1	1	1	[illegible]		
14	502	SUPERVISOR 02	1	1	1	1	1	1	1	1	[illegible]	
15	503	SUPERVISOR 03	1	1	1	1	1	1	1	1	1	
16	601	MANAGEMENT 01	1	1	1	1	1	1	1	0		
17	602	MGT 02	1	1	1	1	1	1	1	1		
18	603	MGT 03	1	1	1	1			1	1		
19	604	MGT 04	1	1	1	1	1	0	1	1	[illegible]	[illegible]
20	605	MGT 05	1	1	1	0	[illegible]		0	[illegible]		

TRAINING MODULES NEEDED

This report is basically the same format as the Training Module Listing except that it contains only the modules required for the current job set and it also lists the number of people that would need each module. It is written during the analysis process in TRAMODEL.

TRAINING MODULES NEEDED FOR THIS JOB SET

CODE	NAME	PEOPLE	HRS	WKS	COST	PREREQUISITES
110	COMMUNICATION	12	2.00	1.00	30.00	
120	SENTENCE STRUCT	12	5.00	2.00	75.00	110
130	RESOURCE MATER	10	4.00	2.00	60.00	120 410 510
309	INTRO SLIDE RULE	10	1.50	1.50	22.50	
310	ALGEBRAIC EXPRES	10	12.00	4.50	160.00	309
320	LINEAR FUNCTIONS	10	9.00	3.00	120.00	310
410	SOLIDS, LIQUIDS	10	2.50	1.00	27.50	309
415	HEAT, TEMP, CALO	10	1.50	1.00	22.50	410
420	FUSION, VAPOR	10	6.00	2.50	90.00	415
510	CHEMISTRY 1	10	9.00	3.50	120.00	310 415
520	CHEMISTRY 2	06	4.50	2.00	67.50	510
530	CHEMISTRY 3	06	10.50	4.50	157.50	520
602	MICROBIOLOGY 2	06	6.00	3.00	90.00	320 415 530
604	MICROBIOLOGY 4	06	6.00	3.00	90.00	602

TRAINING MODULE PRECEDENCE MATRIX

This report shows the required modules for the current job set and the prerequisites. This is the basis for

TRAINING MODULE PRECEDENCE MATRIX

ORIGIN CODE	DESTINATION CODE 001	110	120	130	309	310	320	410	415	420	510	520	530	602	604	999
001	0	1	0	0	1	0	0	0	0	0	0	0	0	0		
110	0	0	1	0	0	0	0	0	0	0	0	0	0	[illegible]		
120	0	0	0	1	0	0	0	0	0	0	0	0	0	0	[illegible]	
130	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	[illegible]
309	0	0	0	0	0	1	0	1	0	0	0	0	0	0	[illegible]	[illegible]
310	0	0	0	0	0	0	1	0	0	0	1	0	[illegible]	0	0	[illegible]
320	0	0	0	0	0	0	0	0	0	0	0	0	0	1		
410	0	0	0	1	0	0	0	0	1	0	0	0	0			
415	0	0	0	0	0	0	0	0	0	1	1	0	0	1		
420	0	0	0	0	0	0	0	0	0	0	0	0	0	[illegible]		
510	0	0	0	1	0	0	0	0	0	0	0	1	0		[illegible]	
520	0	0	0	0	0	0	0	0	0	0	0	0	1			
530	0	0	0	0	0	0	0	0	0	0	0	0	0	[illegible]		[illegible]
602	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
604	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	[illegible]
999	0	0	0	0	0	0	0	0	0	0	0	0	0	0	[illegible]	

the algorithm that figures the module schedule. It is a two-way table with the prerequisite modules across the top and the acquired modules down the side. The entries in the table are a 1 of the top module and it is a prerequisite for the side module and 0 is not. It is written during the analysis process in TRAMODEL.

SCHEDULE TIMES

This report shows each training module that is required for the current job set in chronological order on their starting times. It contains the code number of the module, its starting time in terms of weeks (EVENT), (BEVENT), and the difference between the two (SLACK). Time 0.00 is the beginning of the first week. Time 1.00 is the beginning of the second weeks, etc. It is written during the analysis process of TRAMODEL.

SCHEDULED TIMES

SEQ	CODE	EVENT	BEVENT	SLACK
1	001	0.00	0.00	0.00
2	110	0.00	0.00	0.00
3	120	1.00	0.00	- 1.00
4	130	9.50	0.00	- 9.50
5	309	0.00	0.00	0.00
6	310	1.50	0.00	- 1.50
7	320	6.00	0.00	- 6.00
8	410	1.50	0.00	- 1.50
9	415	2.50	0.00	- 2.50
10	420	3.50	0.00	- 3.50
11	510	6.00	0.00	- 6.00
12	520	9.50	0.00	- 9.50
13	530	11.50	0.00	- 11.50
14	602	16.00	0.00	- 16.00
15	604	19.00	0.00	- 19.00
16	999	22.00	22.00	0.00

CHRONOLOGICAL SYNTHETIC MANAGEMENT MATRIX

This report shows each job in the current job set, with the tasks that have been specified as making up the job, and all the training module codes for each task. The report is written during the analysis process of TRAMODEL.

CHRONOLOGICAL SYNTHETIC MANAGEMENT MATRIX

SEQ NUM	TASK CODE	TASK NAME	001	110	309	120	310	410	415	42[illegible]	320	510	1[illegible]
	100	LABORER											
01	101	PREPARE SURFACE	1	1	0	1	[illegible]	0	0	0	0	0	[illegible]
02	102	APPLY PROTE COAT	1	1	0	0	[illegible]	0	0	0	0	0	0
	200	MAINTENANCE											
03	201	MAINTENANCE 01	1	1	0	1	0	0	0	0	0	0	1
04	202	MAINTENANCE 02	1	1	1	1	1	1	1	1	1	1	1
05	203	MAINTENANCE 03	1	1	0	1	0	0	0	0	1	0	1
	300	LABORATORY TECH											
06	301	LAB 01	1	1	1	1	1	1	0	1	1	1	1
07	302	LAB 02	1	1	1	1	1	1	0	1	1	1	1
09	303	LAB 03	1	1	1	1	1	1	0	1	1	1	1
	400	OPERATOR											
09	401	OPER 01	1	1	1	0	0	1	0	0	0	[illegible]	
10	402	OPER 02	1	1	1	1	1	1	1	1	1		1
11	403	OPER 03	1	1	1	1	1	1	1	0	1		1
12	404	ACTIVATED SLUDGE	1	1	1	1	1	1	0	0	1		
	500	SUPERVISOR											
13	501	SUPERVISOR 01	1	1	1	1	1	1	0	0	1	[illegible]	1
14	502	SUPERVISOR 02	1	1	1	1	1	1	1	0	1	[illegible]	1
15	503	SUPERVISOR 03	1	1	1	1	1	1	1	1	1	1	1
	600	MANAGEMENT											
16	601	MANAGEMENT 01	1	1	1	1	1	1	0	0	1		[illegible]
17	602	MGT 02	1	1	1	1	1	1	1		1	[illegible]	[illegible]
18	603	MGT 03	1	1	1	1	1	1	1	[illegible]			1
19	604	MGT 04	1	1	1	1	1	1	1	1	0	1	1
20	605	MGT 05	1	1	0	1	0	0	[illegible]	[illegible]	0	[illegible]	1

SEQ NUM	TRAINING MODULES CONT. 520	530	602	604	999
	100 LABORER				
01	0	0	0	0	0
02	0	0	0	0	0
	200 MAINTENANCE				
03	0	0	0	0	0
04	0	0	0	0	0
05	0	0	0	0	0
	300 LABORATORY TECH				
06	1	1	1	0	0
07	1	1	1	1	0
08	1	1	1	0	0
	400 OPERATOR				
09	0	0	0	0	0
10	0	0	0	0	0
11	0	0	1	1	0
12	0	0	0	0	0
	500 SUPERVISOR				
13	0	0	0	0	0
14	0	0	1	1	0
15	1	1	1	0	0
	600 MANAGEMENT				
16	0	0	0	0	0
17	0	0	0	0	0
18	0	0	0	0	0
19	1	0	1	0	0
20	0	0	0	0	0

CHRONOLOGICAL LISTING OF REQUIRED TRAINING MODULES

This report is basically the same as the Training Modules Time report. The difference is that the modules are listed chronologically by starting time and totals already figured for the time required and costs. The total of course weeks is a straight total and does not take into account that they may overlap. The total professional time is the total hours involved if one job required all the training modules, not the total time actually involved for each person whose job was entered in the current job set to take only the modules required for his job. This report is written in the analysis process in TRAMODEL.

CHRONOLOGICAL LISTING OF REQUIRED TRAINING MODULES

COURSE NUMBER	COURSE NAME	COURSE WKS	SCHEDULE TIME	SLACK TIME	PROFESSIONAL TIME (HRS)	NUMBER ENROLLED	COST
110	COMMUNICATION	1.00	0.00	0.00	2.00	12	$ 30.00
309	INTRO SLIDE RULE	1.50	0.00	0.00	1.50	10	$ 22.50
120	SENTENCE STRUCT	2.00	1.00	0.00	5.00	12	$ 75.00
310	ALGEBRAIC EXPRES	4.50	1.50	0.00	12.00	10	$180.00
410	SOLIDS, LIQUIDS	1.00	1.50	0.00	2.50	10	$ 37.50
415	HEAT, TEMP, CALO	1.00	2.50	0.00	1.50	10	$ 22.50
420	FUSION, VAPOR	2.50	3.50	0.00	6.00	10	$ 90.00
320	LINEAR FUNCTIONS	3.00	6.00	0.00	8.00	10	$120.00
510	CHEMISTRY 1	3.50	6.00	0.00	8.00	10	$120.00
130	RESOURCE MATER	2.00	9.50	0.00	4.00	10	$ 60.00
520	CHEMISTRY 2	2.00	9.50	0.00	4.50	6	$ 67.50
530	CHEMISTRY 3	4.50	11.50	0.00	10.50	6	$157.50
602	MICROBIOLOGY 2	3.00	16.00	0.00	6.00	6	$ 90.00
604	MICROBIOLOGY 4	3.00	19.00	0.00	6.00	6	$ 90.00
	TOTAL	34.50			77.50		$ 1162.50

SCHEDULE OF INSTRUCTIONAL TRAINING MODULES

This report is a graph showing how the required modules could be scheduled into the shortest time period, taking into account all prerequisites. The report is written in the analysis process of TRAMODEL. This report can be utilized by the training management director to insure proper timing of instructors and insures that trained manpower will be available for startup.

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SCHEDULE OF INSTRUCTIONAL TRAINING MODULES

                                              WEEKS
COURSE    COURSE                       1    1    2    2    3    3    4    4
NUMBER     NAME             1    5    0    5    0    5    0    5    0    5

 110   COMMUNICATION        X    .    .    .    .    .    .    .    .    .
 309   INTRO SLIDE RULE     X-   .    .    .    .    .    .    .    .    .
 120   SENTENCE STRUCT      .XX  .    .    .    .    .    .    .    .    .
 310   ALGEBRAIC EXPRES     .-XXXX    .    .    .    .    .    .    .    .
 410   SOLIDS, LIQUIDS      .--  .    .    .    .    .    .    .    .    .
 415   HEAT, TEMP, CALO     . --.     .    .    .    .    .    .    .    .
 420   FUSION, VAPOR        .   -XX   .    .    .    .    .    .    .    .
 320   LINEAR FUNCTIONS     .    . XXX.    .    .    .    .    .    .    .
 510   CHEMISTRY 1          .    . XXX-    .    .    .    .    .    .    .
 130   RESOURCE MATER       .    .    -X-  .    .    .    .    .    .    .
 520   CHEMISTRY 2          .    .    -X-  .    .    .    .    .    .    .
 530   CHEMISTRY 3          .    .    . -XX-X  .    .    .    .    .    .
 602   MICROBIOLOGY 2       .    .    .    . XXX.    .    .    .    .    .
 604   MICROBIOLOGY 4       .    .    .    .    XXX  .    .    .    .    .

  TRAINING PERIOD X = 1 WEEK   SLACK PERIOD - = 1 WEEK
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An information retrieval system has been designed which allows several modes of output for the some 4000 pieces of information stored in the training modules, task analysis, and waste treatment matrix. An interactive computer program named TRAMOD (Training Module) has been written to access information for various educational management decisions. The program was designed so a user with no knowledge of computer programming could input raw data, ask pertinent educational management questions, and to receive usable output data. Hopefully this system will be utilized by the various agencies who are involved in wastewater educational programs.

Sewer Use Charges

*Richard A. Johnston, P.E.**

The problem of disposal of our liquid wastes has been with us since civilization began. As a utility, sewers have been around far longer than other major utilities such as gas, electric, or telephone but the service of providing collection and treatment of wastewater has historically been the simplest and most taken for granted of the utilities. No problems of building dams, reservoirs and water purification plants, or problems of power outages and continuous maintenance of power poles and lines, or problems of constantly upgrading fuel and gas supplies occur with sewer utilities. Until recently sewage service has amounted to laying pipe and directing wastes to a simple primary or secondary plant and almost letting the system run itself. Operation and maintenance costs for sewer utilities have always been relatively minor when compared with major utilities. Historically many localities have been able to fund their sewer system costs from general government funds or other sources without the need for a special sewer rate as other utilities have had to do.

But as increasing population demands and more stringent water quality requirements have forced the need for more expensive sewage collection and treatment facilities, a corresponding need arose for far more funding; funding not only for capital improvement costs but also to provide operation and maintenance of these larger more complicated facilities.

The result of all these problems and increasing costs is that sewer systems are now entering the spotlight with other utilities. Sewage collection and treatment is today given much more attention than in the past by both citizens and politicians and the importance of establishing efficient and well run sewage systems becomes paramount. It now becomes more logical and important than ever that sewer utilities establish and maintain efficient and sound collection and treatment facilities, and they they provide mechanisms to pay for these costs through the use of sewer use rate changes.

The hard fact must be recognized by all, including the citizens and the politicians that as in the other major utilities, the costs to operate and maintain our sewer systems is increasing at an alarming rate. Such operational costs as power for pumping, chemicals, and labor costs have increased drastically in the last few years and have in some cases more than doubled. As we increase sewer rates to meet these higher costs, sewer utilities become more and more under close scrutiny from all rate payers demanding that the rates be justifiable, fair, and equitable.

Not only does a good sewer use charge system need to be developed to satisfy our needs as I have mentioned, but an "approved" system must be developed before EPA will approve funding of major treatment facility construction projects. Section 204 of the Federal Water Pollution Control Act Amendment of 1972, PL-500 set forth guidelines to be followed in the establishment of user charge systems. Since we all anticipate spending EPA funds in the upcoming years, I would like to discuss these guidelines and how they can be followed in establishment or upgrading of user charge systems.

The basic intent of this Act is to evenly distribute the costs of operating and maintenance of treatment works to the pollutant source and to promote self sufficiency with respect to operation and maintenance. There are two basic user charge systems suggested by EPA. The first is the establishment of a system where each user pays his share of operation and maintenance costs. Idealistically this would mean a separate charge being figured for each connection into the sewer system based on the amount of sewage flow which actually enters to that system in proportion to the entire sewage flow to the treatment facilities. The establishment of this type of system would be dependent on establishing actual flow rates for each individual user, either by the use of sewage flow meters or through providing estimates of the sewage rate to water consumption rates and charging according to water consumption meter rates. The second basic recommendation for a user charge system is to establish classes of users based on similar flow and wastewater characteristics (BOD, suspended solids) and assign that class its share of waste treatment costs. An example of this would be to determine the proportion of total waste flow which comes from all residential users in the system and base the residential charge on what that proportional share of the overall costs are.

Again the intent of these two recommendations and EPA's basic requirement is that "Fair and proportionate share of costs be apportioned to each user based on his contribution to the costs of operation and maintenance of the facilities." It is of course unrealistic to accomplish this task completely as a sewage flow meter would have to be placed at every sewer connection in the system, and this of course is totally unfeasible. So the main problem here is determining a method by which the above basic requirement can be realistically met without undue administrative and regulatory hassles. EPA has developed outlines for development of user charge systems. These guidelines are shown on "Federal Guidelines—User Charges for Operation and Maintenance of Publically Owned Treatment Works" which follows this paper.

Model No. 1: If a treatment works flow dependent or if most users have similar wastewater strength characteristics the user charges can be developed on a volume basis by determining a proportion of each user's volume to the total volume of all users.

*Richard A. Johnston is Assistant City Engineer, Salt Lake City Corporation.

This requires an estimate of sewage flow rate which for most users would have to be based on the water consumption rates. Certain heavy industries which have varying percentages of water consumption to actual sewage flow rate would probably have to install their own sewage flow meters.

Model No. 2 is based on proportionate shares of BOD, SS or other pollutant surcharge. This requires extensive sampling of users to determine BOD, and SS strengths.

Model No. 3 is a combination of Models 1 and 2 and is called the quantity/quality formula. This model is recommended for treatment facilities which handle both domestic and industrial flows.

As you can see, the effective use of these models is dependent on the sewer utility having a great deal of information as to the flow rate and wastewater strength characteristics of each user or user class. Lacking this information or a reasonable estimate of this information, a utility may have a difficult time in convincing EPA to approve their user charge system.

There are different methods which are used by varying localities and sewage systems to establish these equitable sewer use charges. The first method is the use of flat rate charge. This method is used in areas where water meters are not available or where it has been determined that flat rate charge is simpler and more accurate. A flat rate charge based on each type of user paying a specified rate each month is used. The advantage of a flat rate charge is that it establishes a fixed consistent rate and is easy to budget for. Disadvantages of this rate are that it can be inconsistent and unfair, and can also be very hard to keep track of and cumbersome for commericial and industrial type users. An example of how cumbersome this type of rate can be is the previous Salt Lake City sewer charge system which was updated in 1973.

In this system there was a charge for hotels and motels of $1.00 for each unit of occupancy per month, less an automatic allowance of 10 percent for vacancies. One can imagine the administrative nightmare involved in determining a charge for each hotel and motel based on monthly figures for vacancy rates. Another charge was for restaurants, cafes, dining rooms, lodges, and private clubs of $10.00 per month. One type of charge would be totally inconsistent and unfair in that the same charge would be levied for a small cafe as would be for a very large restaurant. Many cities have gotten around the inconsistency in such a charge by setting a charge based on the number of tables or the area of each restaurant. Again such a charge would be cumbersome and result in an administrative problem.

The second basic method of establishing a sewer use charge, and the one which appears to be the most feasible by EPA guidelines is to base that charge on a proportion of the water consumption rate. Usually this will give a more equitable figure as to actual sewage flow contributed by each user. The problem in this type of charge is again that it can be inconsistent and unfair. The percentage of sewage flow to water consumption varies for not only each class of user but for varying users in the same class. For example a small restaurant specializing in take-out foods may contribute very little of its water consumption to the sewage flow whereas a large restaurant which completely prepares the food may contribute a good deal more sewage percentage. Another problem in respect to the residential users is providing for the times of the year in which lawn sprinkling takes place. The best way to get around this problem has been found to determine the charge based on the water consumption rate during the winter months and thereby eliminating the charging of sewer fees for water which winds up on lawns and gardens. However, providing a winter rate consumption charge can become an administrative problem depending on the type of billing and accounting procedures which are set up for the water utility.

Salt Lake City in adopting its latest sewer use charge system has gone to a combination of both flat rate charges and charges based on water consumption. Because of the administrative problems which the City would have in using a winter based water consumption charge we have decided to go to a flat rate charge for residential units. This charge was established by determining the total amount of residential sewage flow to the treatment plant and determining a share of the operation and maintenance costs for that flow. This share was then apportioned evenly over the 44,000 residential units (single-triplex) in Salt Lake City. This is an example of using a user class estimate for determining a sewer use charge to meet EPA requirements. All other users both commercial and industrial were charged based on a water consumption rate charge. Additionally, Salt Lake City will provide a surcharge for any users whose sewage strength exceeds the following: 200 parts per million BOD and 250 parts per million suspended solids. Because we have never collected on a surcharge basis before, and therefore have no experience in how this will be administered, we cannot relate how we will completely set up our sampling programs and surcharge systems. We do anticipate however that some administrative problems will have to be overcome.

I have attempted to point out the importance of establishing a fair and equitable sewer use charge and insuring that this charge adequately provides the needed compensation for efficient operation and maintenance of sewage treatment facilities. And as pointed out in the report it is not an easy task to determine an effective and equitable means of establishing a sewer use charge. Each locality should carefully study their own peculiarities and characteristics before determining which type of user charge system is most adaptable for their needs. Again I think it is important in determining these sewer charges to always keep in mind the basic federal guidelines put out by EPA "The charges will assure that each recipient of waste treatment services will pay its proportionate share of the cost of operation and maintenance."

FEDERAL GUIDELINES

User Charges for Operation and Maintenance of Publicly Owned Treatment Works

(a) Purpose—To set forth advisory information concerning user charges pursuant to Section 204 of the Federal Water Pollution Control Act Amendments of 1972, PL 92-500, hereinafter referred to as the Act. Applicable requirements are set forth in Subpart E (40 CFR Part 35).

(b) Authority—The Authority for establishment of the user charge guidelines is contained in section 204(b) (2) of the Act.

(c) Background—Section 204(b) (1) of the Act provides that after March 1, 1973, Federal grant applicants shall be awarded grants only after the Regional Administrator has determined that the applicant has adopted or will adopt a system of charges to assure that each recipient of waste treatment services will pay its proportionate share of the costs of operation and maintenance, including replacement. The intent of the Act with respect to user charges is to distribute the cost of operation and maintenance of publicly owned treatment works to the pollutant source and to promote self-sufficiency of treatment works with respect to operation and maintenance costs.

(d) Definitions—(1) Replacement.—Expenditures for obtaining and installing equipment, accessories, or appurtenances which are necessary to maintain the capacity and performance during the service life of the treatment works for which such works were designed and constructed. The term "operation and maintenance" includes replacement. (2) User charge.—A charge levied on users of treatment works for the cost of operation and maintenance of such works.

(e) Classes of users—At least two basic types of user charge systems are common. The first is to charge each user a share of the treatment works operation and maintenance costs based on his estimate of measured proportional contribution to the total treatment works loading. The second system establishes classes for users having similar flows and wastewater characteristics; i.e., levels of biochemical oxygen demand, suspended solids, etc. Each class is then assigned its share of the waste treatment works operation and maintenance costs based on the proportional contribution of the class to the total treatment works loading. Either system is in compliance with these guidelines.

(f) Criteria against which to determine the adequacy of user charges —The user charge system shall be approved by the Regional Administrator and shall be maintained by the grantee in accordance with the following requirements:

(1) The user charge system must result in the distribution of the cost of operation and maintenance of treatment works within the grantee's jurisdiction to each user (or user class) in proportion to such user's contribution to the total wastewater loading of the treatment works. Factors such as strength, volume, and delivery flow rate characteristics shall be considered and included as the basis for the user's contribution to ensure a proportional distribution of operation and maintenance costs to each user (or user class).

(2) For the first year of operation, operation and maintenance costs shall be based upon past experience for existing treatment works or some other rational method that can be demonstrated to be applicable.

(3) The grantee shall review user charges annually and revise them periodically to reflect actual treatment works operation and maintenance costs.

(4) The user charge system must generate sufficient revenue to offset the cost of all treatment works operation and maintenance provided by the grantee.

(5) The user charge system must be incorporated in one or more municipal legislative enactments or other appropriate authority. If the project is a regional treatment works accepting wastewaters from treatment works owned by others, then the subscribers receiving waste treatment services from the grantee shall have adopted user charge systems in accordance with this guideline. Such user charge systems shall also be incorporated in the appropriate municipal legislative enactments or other appropriate authority.

(g) Model user charge systems —The user charge system adopted by the applicant must result in the distribution of treatment works operation and maintenance costs to each user (or user class) in approximate proportion to his contribution to the total wastewater loading of the treatment works. The following user charge models can be used for this purpose; however, the applicant is not limited to their use. The symbols used in the models are as defined below:

Ct = Total operation and maintenance (O & M) costs per unit of time
Cu = A user's charge for O & M per unit of time
Cs = A surcharge for wastewaters of excessive strength
Vc = O & M cost for transportation and treatment of a unit of wastewater volume
Vu = Volume contribution from a user per unit of time
Vt = Total volume contribution from all users per unit of time
Bc = O & M cost for treatment of a unit of biochemical oxygen demand (BOD)
Bu = Total BOD contribution from a user per unit of time
Bt = Total BOD contribution from all users per unit of time
B = Contribution of BOD from a user above a base level
Sc = O & M cost for treatment of a unit of suspended solids
Su = Total suspended solids contribution from a user per unit of time
S = Concentration of SS from a user above a base level
Pc = O & M cost for treatment of a unit of any pollutant
Pu = Total contribution of any pollutant from a user per unit of time
Pt = Total contribution of any pollutant from all users per unit of time
P = Concentration of any pollutant from a user above a base level

(1) Model No. 1.—If the treatment works is primarily flow dependent or if the BOD, suspended solids, and other pollutant concentrations discharged by all users are approximately equal, then user charges can be developed on a volume basis in accordance with the model below:

$$Cu = \frac{Ct}{Vt}(Vu)$$

(2) Model No. 2.—When BOD, suspended solids, or other pollutant concentrations from a user exceed the range of concentration of these pollutants in normal domestic sewage, a surcharge added to a base charge, calculated by means of Model No. 1, can be levied. The surcharge can be computed by the model below:

$$Cs=(Bc(B)+Sc(S)+Pc(P))Vu$$

(3) Model No. 3.—This model is commonly called the quantity/quality formula:

$$Cu=VcVu+BcBu+ScSu+PcPu$$

(h) Other considerations.—(1) Quantity discounts to large volume users will not be acceptable. Savings resulting from economies of scale should be apportioned to all users or user classes.

(2) User charges may be established based on a percentage of the charge for water usage only in cases where the water charge is based on a constant cost per unit of consumption.

Pretreatment of Industrial Wastewater at Hill Air Force Base

*Colonel Harry C. Russell**

Almost all industrial processes produce wastes whose uncontrolled disposal can result in serious environmental pollution. The Air Force has long recognized the potential for environmental problems arising from the operation of its highly industrialized installations. Since the early 1950s, the Air Force has emphasized environmental pollution abatement to minimize the effect of its operations.

In Utah, Hill Air Force Base, one of the largest industrial complexes in the state, has been one of the leaders in pollution abatement. Since 1957, pretreatment of wastewater generated by various industrial operations at Hill has been standard practice. This practice has materially reduced the impact of Air Force operations on the Wasatch Front environment during the past 20 years.

In the mid-fifties, when pretreatment of industrial wastewater was initiated at Hill Air Force Base, the major sources of wastewater requiring treatment were segregated from both sanitary and storm sewers to maximize the effectiveness of the treatment plant at that time. The major sources of industrial wastewater included: aircraft washrack, aircraft maintenance shops, engine test cells, chemical laboratories and some metal plating and finishing operations. The wastewater generated by these sources can be best described as a mixed liquior whose constituents were heavy metals; chromium and cadmium; oils and greases; and suspended solids. Concentrated wastes of chromium, cadmium, cyanides, acids and alkalis were also being produced in significant quantities by the metal finishing and plating industry.

To minimize and control the potential degradation of the environment from the uncontrolled discharge of these pollutants, the Air Force designed and constructed a pretreatment facility at Hill Air Force Base. In 1957, this facility represented perhaps the ultimate in advanced state-of-the-art in industrial and wastewater treatment. The plant constructed for only $191,000 employed a chemical-physical treatment scheme to neutralize and remove the major pollutants from the wastewater. The plant was designed to handle a rather small hydraulic load of only 0.33 MGD.

Figure 1 illustrates the 1957 facility constructed to treat industrial wastewater generated at Hill Air Force Base. The principal unit processes of the plant consisted of:

1. A bar rack and screen for removal of rags, sticks and large objects.
2. Primary sedimentation for suspended solids removal.
3. One 150,000 gallon flow equalization tank to minimize fluctuations in flow and waste concentrations.
4. Chemical addition for hexavalent chromium reduction.
5. Dissolved air flotation for removal of oil, grease, emulsions and fine suspended solids.
6. Neutralization for pH control.

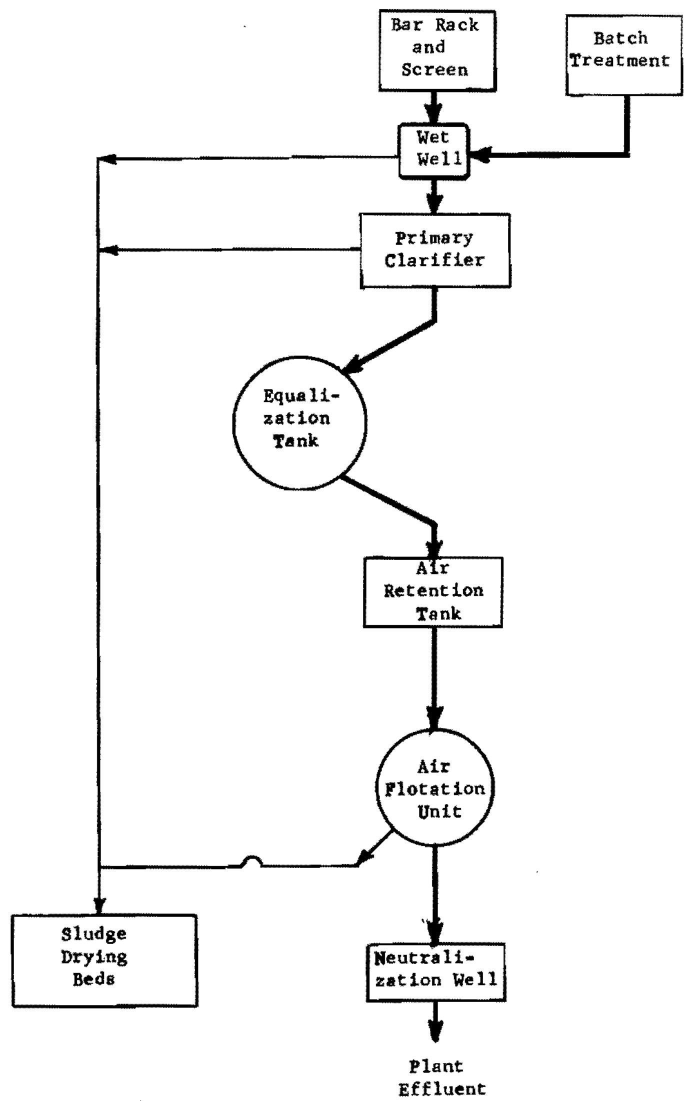

Figure 1. Flow diagram—original treatment plant, 0.33 MGD capacity.

*Colonel Harry C. Russell is Chief, Bioenvironmental Engineering Division, Hill Air Force Base, Utah.

Treatment of concentrated cyanide, chromate, acid and alkali batch wastes was achieved by chemical addition in batch holding/mixing bays, followed by delivery to the plant influent.

Wastewater entering the plant through the bar rack was pumped to the primary clarifier from a wet well for suspended solids and primary fuel oil and grease removal. From the primary clarifier, the wastewater flows by gravity to the equalization tank where fluctuations in flow and constituents were normalized. After equalization, ferrous sulfate was added to the wastewater to reduce hexavalent chromium to the trivalent state. Chemically, this reaction can be expressed as follows:

$$2Cr^{+6} + 3\ FeSO_4 \rightarrow 2Cr^{+3} + 3(SO_4)^{-} + 3Fe^{+3}$$

While this reaction proceeds more rapidly at a pH of 2, the presence of cyanide in the influent prevented the use of this procedure. To complete the removal of chromium, the wastewater was maintained at or near a pH of 8, which promotes the following reaction:

$$Cr^{+3} + 3OH \rightarrow CrOH_3$$

Mixing is achieved by pumping the wastewater to the dissolved air flotation unit through the air retention tank. In the air retention tank, air is dissolved into the wastewater under pressure. Optimum pressures for flotation of heavy metal sludges and oils and greases range from 40 to 50 psi.

From the air retention unit, the wastewater was pumped to the dissolved air flotation unit. Upon release of this pressurized water to the atmosphere, small bubbles are formed in the DAF as the water releases the dissolved air. These bubbles capture oils, greases, and fine suspended solids forcing them to the surface where they are removed by skimming.

The clarified effluent was then discharged to a wet well where the pH is controlled to the limits of the discharge standards. The effluent of the plant was then discharged into the North Davis County Sanitary Sewer for additional treatment. Ultimately, the North Davis plant discharged the treated wastewater into a 1500 yard long stream, which empties into the Great Salt Lake.

Concentrated wastes of chromium, cyanides, acids, and alkalis from spent plating baths were delivered to the treatment plant in carboys for treatment. Originally, we were treating only those wastes generated by Hill Air Force Base, but today we receive these types of wastes from as far away as North Dakota for treatment.

Treatment of cyanide wastes is accomplished by chlorination under basic conditions to neutralize the cyanide. Chemically, this neutralization process can be expressed as follows:

(1) $NaCN + Cl_2 \rightarrow CNCl + NaCl$
(2) $CNCl + 2NaOH \rightarrow NaCNO + NaCl + H_2O$
(3) $2NaCNO + 3Cl_2 + 4NaOH \rightarrow 2CO_2 + N_2 + 6NaCl + 2H_2O$

The first reaction occurs at any pH and is almost instantaneous. However, the formation of CNCl (cyanogen chloride) represents a hazard unless the reaction is impaired by a pH of 10 or greater. This hydrolysis reaction is virtually complete in approximately 20 minutes. After neutralization, the wastewater is introduced into the head works of the plant.

The neutralization of concentrated chromium wastes is accomplished by procedures that essentially follow those described for the flow through portion of the plant. Other wastes are neutralized by standard acid/alkali additions to meet pH standards for the plant. Here again, the neutralized wastes are pumped to the head of the plant for final treatment.

Over the years, industrial growth at Hill Air Force Base has been significant. This growth has required two major modifications to upgrade the hydraulic capacity of the plant. In 1960, the hydraulic capacity was increased to 0.5 MGD with the addition of a second dissolved air flotation unit and associated pumping capacity, and piping to handle this increased loading. Again in 1971, the hydraulic capacity of the plant was increased. This time to 1.0 MDG. This modification included a third dissolved air flotation unit and an additional 150,000 gallon equalization tank (See Figure 2).

Following the 1971 modification, the operation of this plant was evaluated and found to be operating satisfactorily, however, a lot has happened since 1971. New standards for treatment and discharge of industrial wastes have been promulgated and upgraded, not only by the State, but the EPA as well. By 1974 it was apparent that the present plant was no longer capable of producing a satisfying effluent to meet these new requirements. Additionally, new pretreatment standards required plants to be operated more efficiently and required closer scrutiny of daily operations.

To satisfy these new requirements, the Air Force is upgrading the existing plant to meet these future standards. Congress approved this project at $2.7 million of our original estimate with construction scheduled to begin in late 1977.

This new plant was designed to meet the EPA's pretreatment guidelines, which under Sections 304 and 307 of the 1972 Federal Water Pollution Control Act (PL 92-500), were to take effect in 1977. In Dcember 1976, however, EPA withdrew these guidelines in the light of new data documenting certain inaccuracies and inequities in these standards. EPA is currently developing new strategies and standards for pretreatment of industrial wastewater to eliminate these inequities. We believe that our new facility will meet all but the most severe standards, a zero discharge limitation.

As previously mentioned, the effluent of the Hill Industrial Waste Treatment Plant is discharged to the sanitary sewer system of the North Davis County Sewer District, which operates a 19 MGD two stage trickling filter plant with intermediate classification. Currently, the plant is treating appoximately 11 MGD of which approximately 2.0 MGD is attributed to Hill Air Force

Base. The plant is efficiently run as designed to remove domestic sewage pollutants, e.g., BOD_5, suspended solids, etc. However, any industrial wastes such as cyanides, and heavy metal would pass through the plant without significant removal.

At the present time, the National Pollution Discharge Elimination System (NPDES) permit for the North Davis County Sewer District does not include restrictions on industrial pollutants, although the effluent must be sampled and analyzed for these types of pollutants monthly. Their current NPDES permit expires 1 January 1978 and a more stringent replacement is anticipated. The new permit will probably incorporate the Utah Class "C" Water Standards and may provide impetus to the Sewer District to revise the current 1956 sewer code discharge limitations.

Table 1 lists the current discharge limitations imposed by the North Davis County Sewer District Code. It is easily seen that with the exception of two or three

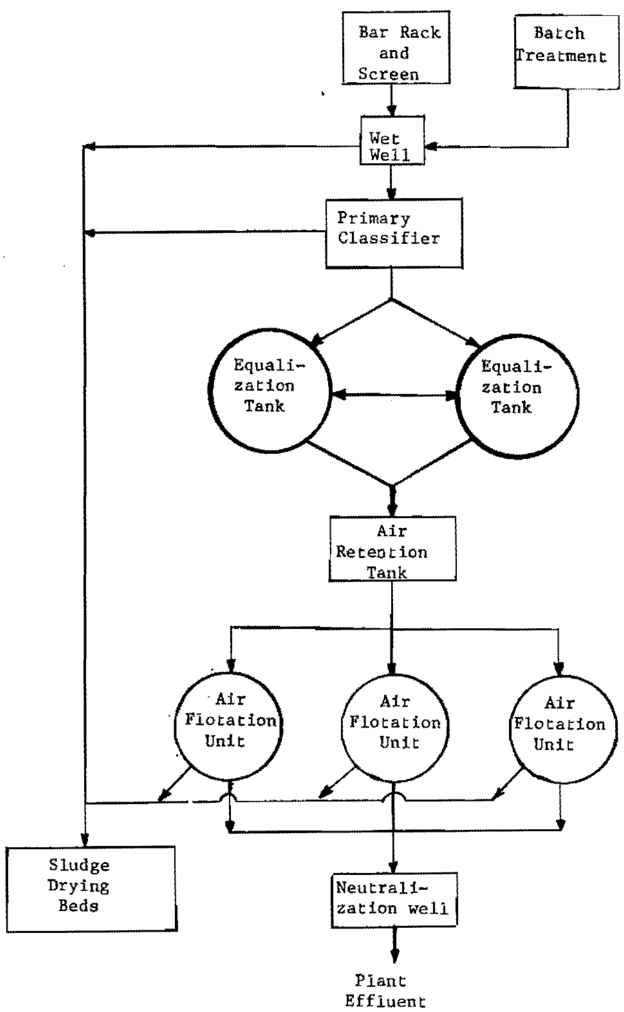

Figure 2. Flow diagram—original treatment plant, 0.33 MGD capacity. (1960 modification—0.5 MGD capacity; 1971 modification—1.0 MGD capacity.)

Table 1. North Davis Sewer District Discharge.

Total Solids	1500
Sodium Compounds	750
Fluoride	50
Hexavalent Chromium	3
Phenols	50
Oils	20
Cyanide	2
Arsenic	0.01
Copper	1.0
ABS	0.5
pH	6.5-8.7

standards, this code incorporates acceptable discharge limits. The limitation of 20 mg/l for oils and greases appears to be quite low when compared to a concentration of 50 mg/l suggested in most literature as typical of a weak domestic sewage, like that discharged by Hill Air Force Base. Another example of the code which may require revision is the 3 mg/l total chromium limitation. This concentration is excessively high, even considering dilution, if the Utah Class "C" Water Standards are imposed stringently on the so called "stream" into which North Davis discharges.

The water quality standards established both by EPA and the state to limit discharges into streams were designed to protect the water quality of the stream and its ultimate receiving body of water from pollution. In so doing, these waters may be used as drinking water for down stream users. In this particular case, the application of these standards to this stream seems unrealistic. In light of the natural "sink" attributes of the Great Salt Lake, the ultimate receiving body of water, the uninhabited area through which the stream flows, and the length of the "stream," it would appear more logical to develop different criteria for this case. Presently, the Great Salt Lake is not being used as a source of drinking water, nor to our knowledge, are plans being made to develop this water source for drinking purposes. It would appear, therefore, that these discharge standards are being improperly applied. Unfortunately, state law may not allow development of different criteria to solve this problem.

We expect more stringent standards to be imposed on the North Davis County Sewer District by the State in 1978. In order for North Davis to comply with these impending new restrictions, two options are foreseeable. North Davis could install tertiary treatment facilities to remove industrial pollutants in the wastewater effluent, or, they might decide to revise the current code to limit discharges from various users. Of these, the later appears the least costly in terms of investment capital and most likely represents the action selected.

There are three possible approaches the North Davis Sewer District might take to limit industrial discharges so they can meet these new standards. One approach would be to limit industrial pollutants from each discharge point to the limit of their new NPDES permit limit. This might incorporate the class "C" water discharge standards. A second approach might be to allow individual users to discharge industrial waste such that the dilution of their industrial wastes with their non-industrial wastes would meet the permit limitation. A third approach might be to allow all sources of industrial waste to be diluted with all other non-industrial wastewater entering the plant. This approach would require a total mass balance of the system to insure that permit limitations are not exceeded.

Applying each of these approaches to Hill Air Force Base, one can calculate numerical values for limiting discharges into North Davis Sewer System. Assuming a hypothetical NPDES permit based on Utah Class "C" Standards, Table 2 presents a comparison of these options. Level I concentrations, Option 1, allows no dilution and essentially represents a zero discharge. This approach represents a drastic philosophical change in wastewater treatment. Neither EPA nor the State to our knowledge has adopted this approach, and we believe such an approach is unrealistic. It would require substantial capital investment and provide very little obvious benefit to the environment. The cost of upgrading our new pretreatment facility to meet such a hypothetical limitation would soar to an astronomical 5 to 6 million dollars. At the least, the adoption of this approach would certainly require the Air Force to assess the economics of total recycling of wastewater. We believe such an approach cannot be justified either economically or environmentally.

Level II concentrations, Option 2, depict concentrations of a hypothetical discharge code calculated as the aggregate concentration of both the industrial waste and non-industrial wastes produced by Hill Air Force Base. In adopting this approach, the North Davis Plant could comply with the NPDES limits, but would not require total recycling of wastewater by the Air Force.

The concentrations calculated under Level III assume no industrial wastes are being discharged to North Davis except those of Hill Air Force Base. These concentrations are the least restrictive of the options presented. As shown, Hill Air Force Base could meet Level III concentrations with the present plant effluent in all parameters except that for iron. These concentrations are so lenient that it is unlikely that this approach would be taken by North Davis.

With our present plant effluent, Level II concentrations would be consistently exceeded for cadmium, chromium, and possibly cyanides, mercury and nickel;

Table 2. Hypothetical industrial pollutant discharge limitations for Hill AFB EHL/K Survey, Hill AFB UT, 14-30 June 1976.

Parameter	Present NDCSD NPDES Permit Limitations (mg/l)	Utah Class "C" Water Standards (mg/l)	EPA Quality Criteria for Water (mg/l)	Level I Hypothetical Revised NDCSD NPDES Permit Limitations (mg/l)	Level II Hypothetical Revised NDCSD Sewer Code (mg/l)	Level III Hypothetical Revised NDCSD Sewer Code (mg/l)	Hill AFB IWTP Effluent
pH (Units)	6.5-9.0	6.5-8.5	5-9	6.5-8.5	6.5-8.5	6.5-8.5	7.3
Phenol	*	0.001	0.001	0.001**	**	**	3.70
Cadmium	*	0.1	0.01	0.01	0.03	0.20	0.08
Chromium (+6)	*	0.05	0.05	0.05	0.17	0.98	0.01
Chromium (Total)	*		0.05	0.05	0.17	0.98	0.85
Copper		1.	1.0	1.0	3.3	19.6	0.21
Cyanides	*	0.01	0.01	0.01	0.03	0.20	0.08
Iron		0.3	0.3	0.3	1.0	5.9	9.04
Lead		0.05	0.05	0.05	0.17	0.98	0.05
Manganese			0.05	0.05	0.17	0.98	0.14
Mercury	*		0.00005	0.00005	0.00017	0.0001	0.005
Nickel	*		0.01	0.01	0.03	0.20	0.11
Silver		0.05	0.05	0.05	0.17	0.98	0.01
Zinc		5	5	5	17	98	0.07
Aluminum							0.99
Arsenic		0.01	0.01	0.01	0.03	0.20	0.01
Barium		1.0	1.0	1.0	3.3	19.6	0.1
Beryllium			0.1	0.1	0.3	2.0	0.001
Selenium		0.01	0.01	0.01	0.03	0.20	0.01

*Effluent monitored for this parameter, but no limitations.
**This value should be higher than the Utah Class "C" Standards, since biological treatment/removal of up to 500 mg/l is possible.

however, with our new plant, these levels would be achieved. We recognize that these standards may be considered lenient by North Davis since they assume non-industrial wastes generated on base do not contain industrial pollutants. For this reason, we believe that the revised North Davis Sewer Code will probably restrict discharges to levels somewhere between the hypothetical concentration to Level I and II.

Since the treatment processes of our existing plant cannot possibly meet either Level I or II limitations, we have undertaken an in-house project to maximize the efficiency of our current operation. Here, let's look at some of our present operational problems with the plant. Suspended solids overflow the weir of the primary clarifier almost continuously. They can also be observed to build up on the weir surface. This is due in part to the 1960 and 1971 modifications to upgrade the hydraulic capacity of the plant. In both modifications, which increased the hydraulic capacity of the plant, no attempt was made to upgrade the air retention tank supplying air for the dissolved air flotation units. As a result, this portion of the plant is still sized for a 0.33 MGD operation. Instead of an optimum pressurization of 40 to 50 psi, we are currently operating at a pressure of only 28 psi. The flotation and removal of suspended solids, oils, and greases in the wastewater is less than optimum.

The heavy metal flocs produced by the addition of ferrous sulfate to reduce the hexavalent chromium to the trivalent state tends to settle in the dissolved air flotation units. These flocs must be continuously recycled from the dissolved air flotation units to the primary clarifier for removal. This action not only reduces the efficiency of the dissolved air flotation units, but it overloads the primary clarifier.

Table 3 illustrates typical influent concentrations entering the plant in comparison with effluent concentrations during 1974 and 1976. During 1974, the percent removal of chromium was only 55 percent, which is considered good for a plant not designed for total chrome removal. But as shown in 1976 we were and are achieving a 90 percent removal of total chrome with the present facility. How we did this is novel and represents considerable effort on the part of the plant operators.

The operation of dissolved air flotation units are maximized to achieve the best possible operation. Skimmer arms are observed every 15 minutes to assure they are set at proper elevation to remove floated solids. This is required because of the fluctuations in flow through the plant. Recycling of settled sludge in the dissolved air flotation units is only accomplished when the unit is taken off line for an hour to allow maximum sedimentation. After sludge removal to primary clarifier, the unit is placed back into service. This procedure has allowed us to reduce our average discharge to chrome from 6.4 mg/l in 1974 to 1.3 mg/l in 1976.

Another problem associated with the present operation of the existing facility are sludge drying beds. As part of our in-house project to maximize plant efficiency in 1975, we constructed new drying beds at the plant to eliminate

Table 3. Percent removal of pollutants by IWTP.

PARAMETER	DAILY AVERAGE CONCENTRATION				PERCENT REMOVAL	
	Influent mg/l		Effluent mg/l		(Inf - Eff) X 100 / Inf	
	1974	1976	1974	1976	1974	1976
Suspended Solids	23.0	39.3	97.0	43.2	**	**
Oils/Grease	9.8	27.5	6.6	24.0	33%	13
Phenol	5.9	4.537	5.1	3.698	14	18
Cadmium	0.25	0.13	0.17	0.08	32	38
Chromium (Hexavalent)	4.8	1.88	0.05	0.01	99	99
Chromium (Total)	14.1	8.95	6.4	0.86	55	90
Copper	0.45	0.41	0.45	0.21	**	49
Cyanide	0.061	0.10	0.045	0.08	26	20
Iron	2.5	1.08	11.7	9.04	**	**
Lead	0.5	0.06	0.5	0.05	**	17
Nickel	0.28	0.19	0.25	0.11	10	42
Zinc	0.18	0.17	0.15	0.07	17	59

** Added during treatment

the potential problem of ground water pollution. Previously, these beds had no bottoms. With the new beds, effluent of the sludge dewatering is recycled to the head of the plant. In our new plant we plan to separate the oils and greases from the heavy metal sludges to improve the effectiveness.

Another part of our in-house effort to maximize efficiency of the current plant until the new modification can be completed, has been directed at the individual industrial wastewater sources. We are developing capability to segregate individual wastewaters into maximum treatment efficiencies. We have also improved the head works of the plant by installing a grit chamber and parshall flume, and we have also constructed a 50,000 gallon holding tank at the front of the plant to contain accidental spills so that effective treatment can be taken without further overloading of the plant. All of these in-house improvements will ultimately become part of our new industrial wastewater treatment plant.

A new industrial wastewater treatment facility currently is undergoing final design to meet the new recinded EPA guidelines for treatment of industrial wastewaters. Figure 3 illustrates the treatment scheme that will be employed at this facility. In this scheme, wastewaters generated by various industrial processes will be segregated at the source to maximize contamination with other wastes and maximize plant efficiency. The wastes are then piped separately to the plant for treatment. In employing this treatment scheme, the new facility should produce an effluent of sufficient quality that the normal operations of the North Davis Plant is not upset. The plant will not achieve consistent removal of the industrial pollutants to a zero discharge level, but it maximizes efficiency of each unit.

Oils and grease removal should be significantly improved in this treatment scheme. Since the cyanides wastes will completely segregate from other wastes, the reduction of the pH of the plant influent to a pH of 2 is now possible. This will maximize not only the oxidation-reduction potential for chromates, but will also improve the removal characteristics of the dissolved air flotation units. But perhaps more importantly, the segregation of the oils and greases will produce heavy metal sludges more ammenable to dewatering.

Another improvement in operations will be the use of SO_2 instead of ferrous sulfate for reducing hexavalent chrome to the trivalent state. This should reduce significantly the volume of sludge produced and the concentration of iron in the finished water.

Heavy metal removal will be accomplished by sedimentation, a two step process. Chromium is best removed at a pH of 8, while other metals such as Cadmium and Zinc are best removed at a pH of 10.5. Flocculation

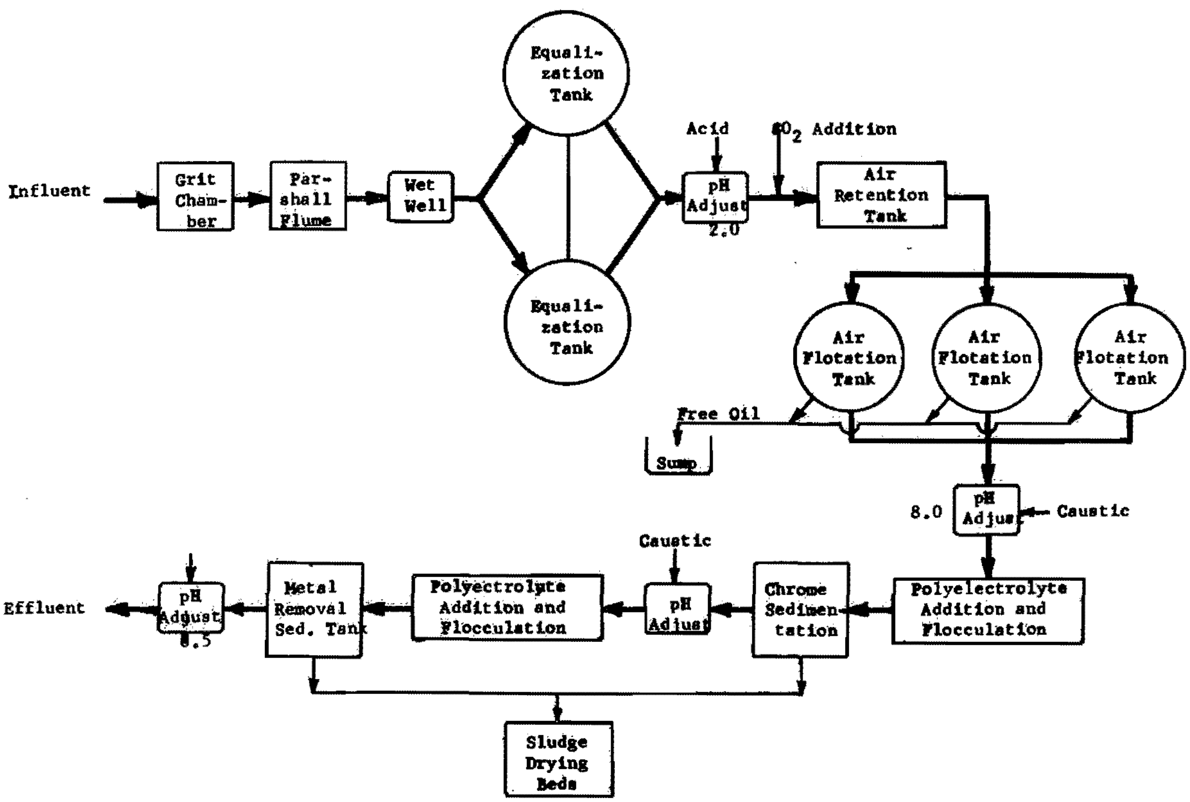

Figure 3. Flow diagram—modified treatment plant.

aids will also be used although the specific aid has not been identified at this time. Following sedimentation, the heavy metal sludges will be subjected to thickening and finally dewatered on the drying beds described previously.

Finally, the improvements planned to allow complete segregation of both continuous and batch wastewater sources entering the plant will materially improve plant capabilities and operations. All of these planned actions should allow our new upgraded facility to produce a polished industrial wastewater effluent that will consistently meet any rational discharge standard.

SUMMARY

In summary, this paper has described the long term interest and investment the Air Force has made in protecting the water quality along the Wasatch Front. Past efforts in treating industrial wastewater generated by Air Force operations at Hill Air Force Base are described in detail.

New Federal and State water quality protection standards have required the Air Force to upgrade its present plant in order to meet these new criteria. A $2.7 million treatment plant to accomplish this is currently being designed. This plant will incorporate advanced state-of-the-art treatment processes to meet all but the most severe zero discharge limitation. Additional treatment would be required to meet this standard and most assuredly would result in complete recycling of wastewater by Hill Air Force Base.

The approach the Air Force is taking in addressing these problems illustrates their intent to invest even more in the water quality emphasis and will continue to support national environmental protection goals.

Utah Power and Light Co. Wastewater Handling System—At Gadsby and Huntington Plants

*K. M. Neuschwander**

I will present an over view of two wastewater handling systems now in operation at the Huntington and Gadsby plants. We have five operating plants and each plant has a different requirement, therefore, a different treatment system. The system at our Huntington plant represents the latest concept in wastewater handling systems and the Gadsby plant represents a retrofit system to one of our older operating plants.

The Huntington Steam Plant the newest operating plant on the UP&L system is located in south central Utah in Huntington Canyon 8 miles northwest of the town of Huntington. This plant, consisting of two 430 MW units, is located near the source of fuel, deep mined coal, and a good water supply for the plant's cooling system. The plant site was originally laid out as a 2000 MW plant consisting of 4 units.

Make up water for the plant is obtained from Huntington creek at the diversion structure. The water is carried through a quarter mile long pipe line to the raw water holding pond. Pumps convey the water from the raw water pond to the condenser cooling water systems making up water lost through evaporation in the cooling tower and blowdown from the system. Each unit consumptively uses between 6000 and 8000 acre ft. of water per year. The condenser cooling system consists of a main steam turbine condenser, and two small boiler feed pump turbine condensers, several hundred feet of 72 inch circulating water line, two circulating pumps and a 10 cell mechanical draft cooling tower. The main equipment in the plant is the steam generator or boiler and the turbine-generator along with the associated auxiliary equipment.

Power produced by the plant generators is delivered to the plant switch yard then fed out over the 345 kV transmission system to the load centers of the Utah Power & Light service system.

In the last 8 to 10 years Environmental Legislation has been responsible for important changes in the handling of wastewater from new electric generating plants and for that matter all other industrial wastewater discharges also. To better describe the wastewater control and treatment at the UP&L Huntington plant, a brief explanation of the water supply system is necessary.

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To meet the water needs for a plant of 2000 MW, and to assure sufficient water for the operation of the plant including those years when precipitation is below normal, two dry years back to back, UP&L purchased water rights from the local Irrigation Companies, developed excess surface run off rights in the upper Huntington Canyon area by the construction of the Electric Lake Reservoir and contracted with the Bureau of Reclamation for uncommitted water in the Joes Valley Reservoir. One stipulation in this water contract with the Bureau of Reclamation was that no wastewater would be discharged from the plant.

This stipulation laid the basic requirement for the wastewater handling system—a zero discharge requirement. The system required several years of development and design, requiring innovation and use of an unproven technically, contrary to normal utility practice. Utilities are basically conservative in the pioneering of new concepts because of the high availability and reliability factor required for steam plant operation.

The wastewater handling system that was accepted included a vapor compression brine concentrator, a wastewater holding and detention structure, a holding pond for reusable boiler and cooling tower water and a large evaporation pond for the disposal of final wastewater.

The wastewater holding structure is used to collect the unusable wastewater which is then pumped to the evaporation pond for final disposal by solar evaporation.

The reusable wastewater pond is used to hold water drained from the boiler and the cooling towers (condenser cooling water system) when the units are shut down for overhaul. This water can be reused in the condenser cooling water system thus cutting down on the amount of wastewater that must be disposed of.

The major source of wastewater is from the blowdown of the condenser cooling water system. Blowdown from this system is required to maintain control of the dissolved solids, particularly sulfates and silica, along with other dissolved solids. These solids are concentrated due to the evaporation of the heated water in the cooling tower to dissipate the heat picked up in the main condenser from the steam cycle.

This blowdown water is still usable and is used as make-up water for the ash sluicing system, and will be used as make-up water to the Unit No. 1 SO_2 scrubber when it goes into service. Water is lost in the ash handling system through evaporation, water retained with the ash in the dewatering bin, and water used to wet the fly ash as it is loaded into trucks and hauled to the ash disposal area. This use consumes about 30 percent of the blowdown water.

The excess blowdown water can be handled in two ways, it can be pumped to the evaporation pond where it is disposed of by solar evaporation or used as feed water to the RCC brine concentrator.

Early in the plant design engineering for the first unit, all logical water reclamation processes were investigated. The systems in the final evaluation demonstrated a high usable water yield with minimum waste stream. The three systems were, reverse osmosis, multi-stage flash evaporation and vapor compression evaporation.

The Resources Conservation Company (RCC) vapor compression evaporator showed great promise and the decision was made to enter a lease-buy agreement for a unit rated at 156 gpm from RCC. The agreement was for RCC to install the unit under a lease and if the unit met all its design specifications during a one year demonstration period UP&L would purchase the unit, if not RCC would remove the unit.

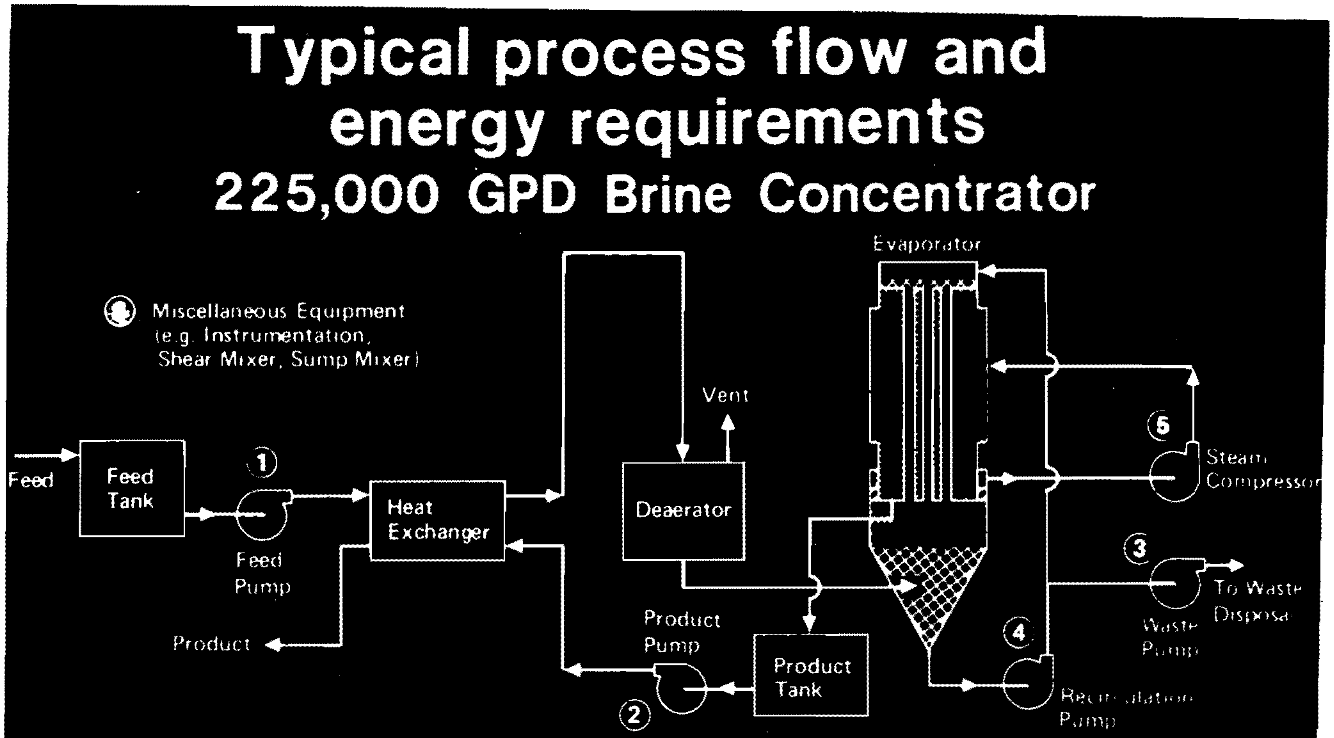
The Resources Conservation Co. evaporator is a falling film vapor compression type brine concentrator designed to recover up to 96 percent of the cooling tower blowdown feed water as nearly pure water, less than 10 ppm TDS. The schematic of the over all process is shown in Figure 1. The main components of the process are:

1. Pre-treatment Section—Here acid is added to the feed water for pH control before heating it to the boiling point. It then goes to the deaerator where the non-condensable gases, oxygen, carbon dioxide, etc. are removed.

2. The Evaporator Sections—Here the feed water is concentrated to waste brine composition and the nearly pure effluent water is recovered. Scale formation in this section is controlled by a seed slurry technique.

3. The Seed Recycle Section—A portion of the precipitated solid slurry is recycled back to evaporator slurry tank to maintain an adequate suspended solid level in the evaporator brine, and the final waste is pumped to the waste disposal sump and then to the evaporation pond.

For a more detailed explanation of the system, sulfuric acid is fed to the cooling tower blowdown water for pH control just before it enters the feed tank. The feed tank is designed to have a residence time of 10 to 50 minutes based on a designed flow of 158 gpm. The acidified water is pumped by the high pressure feed pump through the primary heat exchanger, this is a plate type heat exchange



Equipment	①	②	③	④	⑤	⑥	Total
Power Requirement, KW	45	32	10	70	630	53	840

Total Energy Requirement – 90 KWH/1000 Gal

Figure 1. Typical process flow and energy requirements 225,000 GPD Brine Concentrator.

and the heat source is from the counter flowing distilled effluent water. The water then passes through the trim heat exchanger and finally through the scavenge heat exchanger. At this point in the cycle the feed water temperature has been raised to about 6°F above its atmospheric boiling point. This water enters the deaerator where it is allowed to flash at near atmospheric pressure releasing the dissolved gas. The steam and gases flashed off in the evaporator are pumped to the scavenge heat exchange to return heat to the system. The deaerated water then enters the evaporator sump where it mixes with the concentrated brine. The concentrated brine is recirculated over the heat transfer surfaces within the evaporator where the flow is distributed around the tubes and allowed to flow down the tubes in a thin film keeping the tubes wet at all times. Water is evaporated from the brine and passed through a demister to the suction side of the vapor compressor. The vapor is then pumped by a single-stage centrifugal compressor which raises its saturation temperature 6 to 8°F above the boiling point of the brine. The compressor is driven by an 800 horsepower electric motor which converts electrical energy to thermal energy in the saturated steam. This is the main source of energy input to the system.

The compressed steam is pumped to the inside of the evaporator tubes supplying the heat to vaporize the water from the brine. The condensed condensate or product water is then pumped from the product water tank through the heat exchanger to give up its thermal energy to the incoming feed water.

A bleed stream in taken from the brine blowdown line to maintain the proper brine concentration in the evaporator brine tank.

The process is controlled by an automatic control system that keeps the system within close operating tolerance to produce an effluent of less than 10 ppm total dissolved solids.

The RCC system met all the requirements of the specifications during the one year test period and was purchased by Utah Power & Light.

The main operating problem experienced with the system was fouling of the plate heat exchanger. A filter has been installed in the make-up water line and this should minimize this problem.

The effluent from the brine concentrator is used as make-up water to the mixed bed demineralizer which produces the ultra pure water for boiler make-up needs. The RCC effluent materially extends the demineralizer runs and produces a plus in the over all system economics, plus reducing the wastewater flow to the evaporation pond by about 50 percent.

I will now discuss the retrofit wastewater handling system at our Gadsby plant as contrasted to the new system at Huntington.

The Utah Power & Light Company's Gadsby plant is a three unit steam electric generating plant located west of downtown Salt Lake City on the west bank of the Jordan River. The first unit, rated at 66 MW went into service in 1951 followed by the second, a 75 MW unit, in 1952, and the third a 100 MW unit in 1955.

The water supply system for the plant consists of a diversion dam in the Jordan River, an intake structure with a rotating screen, a circular clarifier, a holding tank and pumps. This system supplies the make-up water to the condenser cooling water systems for the three units. Water is lost from the system through evaporation in the cooling towers and system blowdown the same as at the Huntington plant.

The main wastewater streams from the plant are from the condenser cooling system blowdowns, the clarifier blowdowns, the wash water from the ash handling system, the sodium and hydrogen zeolite water softener back wash and rinse drains, and boiler and evaporator blowdown. There is also a thermo drain from the steam heated pitched tank heaters in the oil tank farm area.

Originally these waste streams discharged into the Jordan River on the east side of the plant or to the industrial canal to the west of the plant. This was the logical and accepted procedure at the time these units were designed and constructed in the early 1950s.

The passage of the Clean Water Act in 1972 and the implementation of the National Pollution Discharge Elimination System regulations ushered in a new set of standards and parameters for wastewater handling and discharge control requirements which we must now meet.

The first permits for the Gadsby plant were received late in 1973 and were for a three year period. The permit limits for TSS was for an average of 30 ppm for one of the five discharge points, the other four discharge points had to meet 30 ppm by January 1, 1974; the pH range was 6.5 to 9.0; temperature 90°F and 95°F; chlorine residual of 1.0 ppm; and oil or grease at 10 ppm.

Renewal permits were received in 1976 and are for a five year period. The total suspended solid limit was reduced to an average of 25 ppm with a maximum of 50 ppm. The average and maximum value for TSS will be further reduced to 10 ppm and 25 ppm respectively in 1980. The chlorine was changed to 1.0 ppm free available chlorine. Temperature remained at 95°F and oil and grease remained the same at 10 ppm.

The issuing of the discharge permits with these low limitations on the quality of wastewater allowed from the plant discharge was the catalyst that set into motion the engineering and design process for controlling the various wastewater streams.

Two studies were initiated to determine the total flow rate of all water discharges, where they originated in the system and the average and maximum parameter of these streams.

As stated before, the Gadsby plant is located on the west side of the Jordan River with the intake structure, the water treating plant, the cooling tower for the No. 1 unit and the pitch tank farm all located east of the plant, therefore all drains from the No. 1 unit discharge into the Jordan River.

When Units 2 and 3 were constructed, the cooling towers for these two units were constructed west of the plant and the blowdowns from these units discharge into the industrial canal.

It was, therefore, necessary in the pre-engineering studies to determine accurately, if possible, the source of all the wastewater streams and the present operating parameters and conditions.

The studies indicated that TSS was the most serious problem faced, and occurred mainly from drains on the east side of the plant. The greatest TSS load was from the air washer drain for the ash handling system and the clarifier blowdown. The boiler and evaporator blowdowns had a component of basically low TSS along with temperature, also a temperature problem occurred with the pitch farm heater condensate drain. The back flush and regeneration drains from the sodium and hydrogen zeolite water softening system also required control to minimize pH swings.

The design of the new system also addressed the problem of reducing the need for city water to meet the make-up water requirements during peak load periods especially during the hot summer months, therefore, a second clarifier was designed into the system.

The new wastewater system contains an ash water sump where the air washer water was collected along with the boiler and evaporation blowdowns. This water was pumped to a thickner where ash and other TSS are settled out. The heavy under flow is pumped to a tank truck and the waste disposed of in a land fill area. The clear effluent is discharged to the new clarifier and supplements raw river water producing additional make-up water for the three condenser cooling water systems. The under flows from the two clarifiers, and the zeolite softener back flush and rinse drains are piped to the thickner for disposal. This reduces the number of discharge points to the Jordan River to one, which handles the No. 1 cooling tower blowdown and the emergency over flows of clean water from the thickner or clarifier.

This system was placed in service in May of 1976 and has undergone several changes in concept and in equipment. Several pump failures were experienced along with failures in other equipment components, mainly in the control systems. One major system change was also required. The draining of the hot boiler and evaporator blowdown water into the ash water pump sump increase the calcium carbonate disposition in the line from the pump sump to the thickner. The blowdown line has been changed to discharge into the No. 1 cooling tower basin in hopes of easing this problem. An agitator was also installed in the ash pump sump to minimize ash build up in the sump and to prevent plugging of the pump suction screens, this seems to be working very well.

The blowdown from the No. 2 and 3 towers are now planned to be diverted to the Jordan River this summer and during the summer of 1979 the No. 1 cooling tower will be diverted to this line. A sand filter will be installed in this line to bring the TSS of wastewater discharges to the Jordan River into compliance with the 10 ppm TSS limitation, by 1980. At the Carbon plant we have a sand filter operating on the cooling tower blowdowns from the two cooling towers at this plant with very good success.

There are emergency conditions that imposed abnormal loads on the wastewater discharge system that were not adequately considered in the initial design phase, to correct this a surge or holding pond is being designed to handle the short term heavy flows from emergency situations without violating the discharge limitations and over loading the wastewater treatment system.

The presently installed system and modifications planned should allow the wastewater discharges from the Gadsby plant to meet all of the requirements in our NPDES permits. So far this has required the expenditure of approximately $675,000 and increased the operating expense at this plant which must be compensated for by increased cost of electrical power for our customers. We have a mandate to keep our expenditures for new equipment and operating expenses at a minimum, therefore any changes in wastewater control requirements by congress or EPA should be evaluated as to the soc-economic impact they will have along with the cost-benefit ratio of the regulation. We must strive to keep our operating costs down and to meet accepted environmental standards.

Biological Treatment of Petrochemical and Refinery Wastes

*Davis L. Ford**

INTRODUCTION

The control of wastewater discharged from a modern petrochemical or refinery complex will depend on an understanding of all processes contributing to the waste, available options for treatment and waste disposal, effluent quality standards and operation of the treatment plant.

The successful treatment of wastewaters discharged from refineries and petrochemical plants is predicated on many factors and while it is recognized that each industry poses its individual problems, there is a logical approach to selecting efficient treatment processes.

The present and future levels of wastewaters and residuals depend on the sources and management of these inputs. Figure 1 depicts the six general sources as process streams, utility wastes, sewage, contaminated storm water, ballast from ships and miscellaneous discharges.

The solution of a specific pollution problem normally involves the completion of a series of individual phases, with the final result being the construction of the necessary treatment process units. The engineering effort needed to obtain design data is divided into phases and further grouped into tasks. Phase I consists of the process inventory, the wastewater survey, localizing major pollutional streams, identifying sources of major pollutants, outlining and implementing an analytical schedule, and misinterpretation of data. Under Phase II, candidate processes are reviewed and selective wastestreams screened for toxic reactions prior to bench-scale and pilot plant studies. Phase III consists of preparing a preliminary engineering report which translates information developed from the treatability study to rational engineering designs. This phase outlines the conceptual design, the sizing of the major unit processes, the estimated cost of the system, and the finalized conclusions and recommendations. The final phase in the USA involves preparing bid documents; namely, the construction drawings and specifications.

Information gained during the wastewater survey forms the basis for all future decisions and it is, therefore, imperative that careful considerations be given to this phase of the water pollution abatement program. In order to accomplish these objectives, a series of investigations should be performed as follows:

1. Review process information and plant records;
2. Establish sampling points based on potential waste reuse, product recovery, and segregation of specific waste streams;
3. Establish a sampling and flow measurement program;
4. Conduct field surveys to locate wastewater collection points; and

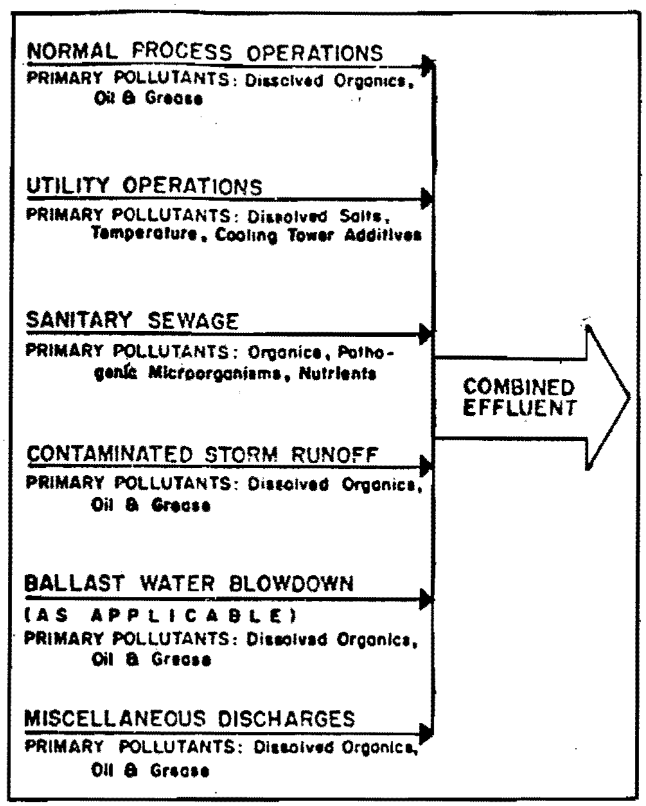


Figure 1. General sources of wastes.

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5. Perform the sampling and analytical program based on field conditions, treatability parameters, and regulatory quality requirements.

The importance of representative samples cannot be overemphasized. In some instances, wastes must be prepared from a variety of sources. When industrial plants exist, grab samples may be acceptable and even necessary in cases of unstable constituents, but composited samples are desirable for the treatability studies.

The trend toward more stringent effluent standards has generated an increased interest in the handling and treatment of storm runoff. Because storm flow is intermittent and unpredictable in nature and little data has been collected to typify its characteristics within industrial installations, the design of handling and treatment facilities presents a challenging engineering problem. Since direct treatment of the high flow volumes experienced during a storm is normally impractical, the contaminated runoff must generally be surged and temporarily stored prior to treatment. The design of appropriate storage facilities must be predicated upon the degree of segregation to be practiced, a reasonable determination of required volumes, peak flow rates anticipated and the development of an economic balance between required storage volumes and the additional treatment capacity needed to handle peak flows due to runoff.

Ballast derived from ships and barges should not be overlooked. Ballast and storm runoff may be considered as batch discharges and usually some storage is required.

Petrochemical and refinery wastewaters are highly variable with regard to both quantity and quality, and as a consequence, characterization is difficult. The organic component of the wastewater is usually estimated in terms of the five-day biochemical demand (BOD_5), chemical oxygen demand (COD), total oxygen demand (TOD), or total organic carbon (TOC). The BOD_5 analyses are sometimes more difficult to undertake; consequently, COD, TOC, and TOD have gained in popularity.

Inorganic characterization is also an important part of the overall treatability program. These analyses should include potential toxic elements such as heavy metals and possible pollutants such as acidity, alkalinity, suspended solids, nitrogen, phosphorus, chlorides, etc.

Wastes from the petroleum industry may include various volatile fractions, lubricants, gas oil, fuel oil, wax, asphalt, petroleum coke, etc. Additionally, chemicals are derived from petroleum derivatives and natural gas.

Traditionally, refinery wastewaters have been categorized as clean water or process wastewater. These general classifications are often further segregated into high and low total dissolved solids (TDS) streams, dirty and clean storm water, high and low organic streams, sanitary wastewaters, and ballast water. Low TDS streams are usually segregated for their reuse potential.

Most hydrocarbon wastes usually occur from leaks, spills, and product dumps. Steam condensate from reflux systems contain significant amounts of both hydrogen sulfide and mercaptans and, depending on the plant processes and temperatures, sulfide concentrations of combined sewers may be between 2 and 5 mg/l. Petrochemical wastewaters are often alkaline since caustics are used to purify various hydrocarbon streams, and these wastes are often toxic, oxygen demanding, and disagreeable to the senses. Phenols and cresols are produced by various cracking processes used in refinery operations and these wastes are among the most troublesome pollutants produced by the petroleum industry. Ammonia arises from two sources: ammonia added directly to process streams for the control of corrosion, and ammonia resulting from the breakdown of nitrogenous compounds. Corrosion inhibitors, particularly heavy metals, may create some waste disposal problems.

CHARACTERIZATION OF INDUSTRIAL WASTEWATERS

The analysis, design, operation, and control of biological treatment systems are all based on the characterization of the liquid waste using selected parameters. The proper analytical techniques as well as correct interpretation of the results are, therefore, of prime importance when considering the biological treatment of industrial wastes.

Parameters used to characterize industrial wastewaters can be categorized into organic and inorganic analyses. The organic content of wastewater is estimated in terms of oxygen demand using biochemical oxygen demand (BOD), chemical oxygen demand (COD) or total oxygen demand (TOD). Additionally, the organic fraction can be expressed in terms of carbon using total organic carbon (TOC). It should be recognized that these parameters do not necessarily measure the same constituents. Specifically, they reflect the following:

1. BOD—biodegradable organics in terms of oxygen demand
2. COD—organics amenable to chemical oxidation as well as certain inorganics, such as sulfides, sulfites, ferrous iron, chlorides, and nitrites.
3. TOD—all organics and some inorganics in terms of oxygen demand
4. TOC—all organic carbon expressed as carbon.

Another organic parameter commonly used in defining industrial wastewaters is the measurement of oil and grease. This analysis is particularly important since oils have both a recovery value and reduce the efficiency of biological treatment systems.

Extraction techniques using various organic solvents such as n-hexane, petroleum ether, chloroform, and trichloro-trifluoro-ethane are used to determine the oil and grease content of wastewaters. The method outlined by the Environmental Protection Agency (EPA) measures hexane extractable matter from wastewaters, but excludes hydrocarbons that volatilize at temperatures below 80°C (3). Additionally, not all emulsified oils are measured using these extraction techniques. However, a modified procedure provides for the release of water soluble oils by saturating an acidified sample with salt (4).

The inorganic characterization schedule for wastewaters to be treated using biological systems should include those tests which provide information concerning:

1. Potential toxicity, such as heavy metals, ammonia, etc.
2. Potential inhibitors, such as total dissolved solids (TDS), chlorides, sulfates, etc.
3. Contaminants requiring specific pretreatment such as pH, alkalinity, acidity, suspended solids, etc.
4. Nutrient availability.

CORRELATION OF ORGANIC PARAMETERS

A comparative analysis of organic parameters alludes to a more interpretive definition of the nature of the wastewater organic component. Moreover, correlation of these parameters can result in the more effective operation and control of existing biological facilities as well as predicting the applicability of these systems in treating the wastewater(s) in question. A discussion of these organic parametric relationships follows.

BOD/COD Relationship

The BOD/COD relationship is generally considered to indicate the fraction of the chemically oxidizable organics which are amenable to biological degradation. For example, if the BOD_{ult}/COD ratio of a wastewater approached unity, a major fraction of the organic materials in the waste would be considered as biodegradable. Conversely, a BOD_{ult}/COD ratio of 0.1 to 0.3 would indicate that a major portion of the organics which are amenable to chemical oxidation are resistant to biochemical oxidation, and a proposed biological treatment system should be considered as questionable on this basis. It is, of course, possible that a large fraction of the observed COD is attributable to the oxidation of reduced inorganic constituents, but this can be determined by performing ancillary chemical analyses.

An evaluation of BOD and COD for selected chemicals categorized into four groups is presented in Table 1. The results are tabulated in terms of the COD and BOD yield as a percent of theoretical oxygen demand (ThOD) (5). As a rule, the higher the percentage of BOD yield, the more applicable one would expect biological treatment to be. Another list which states relative biodegradability of certain organic compounds is tabulated in Table 2 (6).

BOD-COD/TOC Relationships

In attempting to correlate the BOD or COD of an industrial wastewater to TOC, certain factors which might constrain or discredit the correlation should be considered at the outset. These include:

1. A portion of the COD may be attributable to the oxidation of inorganics as previously described while the TOC analysis does not include the oxidation of these compounds.
2. The BOD or COD tests do not include those organic compounds which are partially or totally resistant to chemical or biochemical oxidation. However, all of the organic carbon is recovered in the TOC analysis.

Table 1. Evaluation of COD and BOD with respect to theoretical oxygen demand—test organic materials.

CHEMICAL GROUP	ThOD (mg/mg)	Measured COD (mg/mg)	COD/ThOD (%)	Measured BOD_5 (mg/mg)	BOD_5/ThOD (%)
ALIPHATICS					
Methanol	1.50	1.05	70	1.12	75
Ethanol	2.08	2.11	100	1.58	76
Ethylene glycol	1.26	1.21	96	0.39	29
Isopropanol	2.39	2.12	89	0.16	7
Maleic acid	0.83	0.80	96	0.64	77
Acetone	2.20	2.07	94	0.81	37
Methyl ethyl ketone	2.44	2.20	90	1.81	74
Ethyl acetate	1.82	1.54	85	1.24	68
Oxalic acid	0.18	0.18	100	0.16	89
Group Average			91		56
AROMATICS					
Toluene	3.13	1.41	45	0.86	28
Benzaldehyde	2.42	1.98	80	1.62	67
Benzoic acid	1.96	1.95	100	1.45	74
Hydroquinone	1.89	1.83	100	1.00	53
o-Cresol	2.52	2.38	95	1.76	70
Group Average			84		58
NITROGENOUS ORGANICS					
Monoethanolamine	2.49	1.27	51	0.83	34
Acrylonitrile	3.17	1.39	44	nil	0
Aniline	3.18	2.34	74	1.42	44
Group Average			58		26
REFRACTORY					
Tertiary - butanol	2.59	2.18	84	0	0
Diethylene glycol	1.51	1.06	70	0.15	10
Pyridine	3.13	0.05	2	0.06	2
Group Average			52		4

Table 2. Relative biodegradability of certain organic compounds.

Biodegradable Organic Compounds*	Compounds Generally Resistant to Biological Degradation
Acrylic Acid	Ethers
Aliphatic Acids	Ethylene Chlorohydrin
Aliphatic Alcohols (normal, iso, secondary)	Isoprene
Aliphatic Aldehydes	Methyl Vinyl Ketone
Aliphatic Esters	Morpholine
Alkyl Benzene Sulfonates w/exception of propylene-based Benzaldehyde	Oil
Aromatic Amines	Polymeric Compounds
Dichlorophenols	Polypropylene Benzene Sulfonates
Ethanolamines	Selected Hydrocarbons Aliphatics Aromatics Alkyl-Aryl Groups
Glycols	Tertiary Aliphatic Alcohols
Ketones	Tertiary Benzene
Methacrylic Acid	Trichlorophenols
Methyl Methacrylate	
Monochlorophenols	
Nitriles	
Phenols	
Primary Aliphatic Amines	
Styrene	
Vinyl Acetate	

* Some compounds can be degraded biologically only after extended periods of seed acclimation.

3. The BOD test is susceptible to variables which include seed acclimation, pH, temperature, toxic substances, etc. The COD and TOC tests are independent of these variables.

One would expect the stoichiometric COD/TOC ratio of a wastewater to approximate the molecular ratio of oxygen to carbon (32/12 = 2.67). Theoretically, the ratio limits would range from zero, when the organic material is resistant to dichromate oxidation, to 5.33 for methane. Higher ratio values possibly infer the presence of inorganic-reducing agents. Reported COD and TOC values for several chemical and refinery wastewaters investigated indicate the COD/TOC ratio varying from 2.19 to 6.65 (7). This variability is attributed to the COD yield, and waste streams containing a portion of these substances would be subjected to a fluctuating COD/TOC ratio in the event of relative concentration changes. The greater the variability in the characteristics of an industrial waste stream, the more pronounced will be the change in its COD/TOC ratio. This in itself is a good indicator of the degree of consistency of wastewater constituents and can be a valuable aid in predicting the design organic load applied to a biological treatment facility.

The BOD_5/COD and COD/TOC ratios recently reported for various industrial production facilities are shown in Table 3. (8). Samples were taken directly from the process units and in most cases excluded dilution from cooling tower or boiler blowdown sources.

COD/TOD Relationship

The COD and TOD values have been correlated for several waste streams, although extensive correlation data is not presently available. The TOD concentration usually can be expected to be higher than the corresponding COD values by virtue of the fact that chemical oxidation is less efficient than that obtained in the catalyzed combustion chamber of the TOD analyzer. Preliminary unpublished data indicate that the COD yield of refinery wastewaters ranges from 70 to 80 percent of the total oxygen demand. Unusually high COD/TOD ratios favor the chemical oxidation of inorganics over their oxidation in a catalytic combustion chamber. If the COD/TOD value was unusually low, then the presence of constituents resistant to chemical oxidation would be inferred, or perhaps a more complete oxidation of inorganics in the combustion tube was observed than that obtained chemically. Reported COD/TOD values for untreated industrial wastewaters are tabulated in Table 4 (9) (10). These data indicate that average COD/TOD values for the raw industrial wastewaters cited approximate unity, with the variations being attributed to the aforementioned factors.

Table 3. Individual wastewater characterization.

WASTEWATER	BOD_5/COD	COD/TOC
Acrylonitrile	0.19	2.0
Ammonia	0.06	-
Ammonia	0.55	4.4
Ammonia + Utilities	0.37	3.4
Butadiene-Styrene	0.05	3.8
Chlorine-Soda	0.03	22.5*
Cumene	0.12	5.6
EDC-Direct	0.49	1.6
EDC-Oxyhydrochlorination	0.64	1.8
Ethylene Oxide	0.35	17.0*
Olefins	0.25	3.4
Polystyrene	0.44	3.3
Polyvinylchloride	0.10	1.9
Propylene Oxide	0.45	5.0
Propylene Glycol	0.48	4.9
Propylene Tetramer	0.34	0.7
Sewage	0.37	3.4
Synthetic Rubber	0.51	3.9
Urea	0.79	0.8
Vinyl Chloride	0.04	0.9

* TDS > 20,000 mg/l

PROCESS DESIGN FORMULATION

Once the industrial wastewater has been characterized, decisions can be made regarding pre- or primary treatment requirements, the type of biological processes to be considered, and the degree of bench or pilot scale treatability studies necessary for adequately developing process design criteria.

Pre- or Primary Treatment

One of the critical features in designing a biological treatment system receiving industrial wastewaters is inclusion of the necessary pre- or primary treatment processes. There are many constituents in industrial wastewaters which adversely affect biological treatment systems. Reported limiting or inhibitory concentrations for some of these constituents are listed in Table 5 (7). The characterization results of the industrial wastewater will indicate which, if any, of the contaminants should be removed prior to being treated biologically. Of those cited in Table 5, the organic load variation generally is the most significant. Most industrial effluents are highly fluctuative both in terms of flow and constituents. Such variations are highly detrimental to the biological process and indicate the need for equalization. The size of the equalization basin and its degree of mixing will determine the effectiveness of dampening these variations. If sufficient flow and quality information is obtained during the wastewater characterization study, a rational basis for designing the equalization

Table 4. COD/TOD ratios for untreated industrial wastewaters.

TYPE OF WASTEWATER	COD/TOD
Refinery Waste	0.99
Pesticide Manufacturing Waste	0.95
Petrochemical Waste	0.98
Petrochemical Waste	1.20
Petrochemical Waste	1.12
Plastics Manufacturing Waste	1.25
Cryogenics Plant Waste	1.04
Refinery Waste	0.71
Combined Refinery-Petrochemical Waste	0.75

Table 5. General tolerance limits for biological treatment.

CONSTITUENT	LIMITING OR INHIBITORY CONCENTRATION	PRETREATMENT
Suspended Solids	>125 mg/l	Lagooning, sedimentation, flotation
Oil or Grease	>50 mg/l	Skimming tank or separator
Heavy Metals	<1-10 mg/l	Precipitation or ion exchange
Acidity	Free mineral acidity	Neutralization
Organic Load Variation (based on 4 hour composite)	>4:1	Equalization
Sulfides	>100 mg/l	Precipitation or stripping
Chlorides	>(8,000 - 15,000 mg/l)	Dilution, deionization
Phenols	>(70 - 160 mg/l)	Stripping, provide complete mixing
Ammonia	>1,600 mg/l	Dilution; pH adjustment and stripping
Dissolved Salts	>20,000 mg/l	Dilution, ion exchange

facility can be developed. One approach is the use of the following equation:

$$x_{(t+\Delta t)} = C_t \; 1 - \exp -\frac{Qt}{v} + x_t \; \exp -\frac{Qt}{v} \quad \ldots\ldots\ldots\ldots (1)$$

where:

- t = time increment chosen for the numerical step-by-step calculation
- C_t = input concentration averaged over Δt
- x_t = basin concentration before addition of the increment of flow at concentration of C_t
- $x_{(t+\Delta t)}$ = basin concentration after addition of increment of flow
- Q = volumetric flow rate
- v = basin volume
- t = time, varies between zero and Δt in the equation. The expression need only be evaluated at $t = \Delta t$.

Using this model, the equalization basin concentration of a critical pollutant or that discharged to the biological process can be calculated at selected time intervals for various equalization volumes. This assumes that the critical pollutant in the industrial discharge was measured at time intervals of sufficient frequency to accurately define the variation. The standard deviation of the equalized concentration will decrease with increasing basin retention time. The relationship then can be used for selecting the retention time which corresponds to the maximum fluctuation that can be tolerated in the biological system.

Oil and grease are of paramount importance when designing biological systems for industrial wastewaters such as those discharged from petroleum refinery and petrochemical installations. Hexane extractables adversely affect a biological system as the concentration in the mixed liquor approaches 50 to 75 mg/l. A recent study conducted for the Environmental Protection Agency indicated that an activated sludge system will perform satisfactorily with a continuous loading of hexane extractables of 0.1 lbs per lb of mixed liquor suspended solids. It was recommended that the influent to the biological system should contain less than 75 mg/l hexane extractables, and preferably less than 50 mg/l. The most significant problem related to oils in biological systems was attributed to lowering floc density to a level where the sludge settling properties were destroyed (11). The removal of free, and to some extent, emulsified oils through gravity separation, air flotation or possible filtration, its therefore required in many instances.

BIOLOGICAL TREATABILITY STUDIES AND DESIGN CRITERIA

The necessary prerequisite in the formulation of design criteria for biological systems, particularly where complex organic wastewaters are involved, is a process simulation study programmed to provide key information relative to the removal of pollutants. The preliminary characterization analyses as previously described may be indicative of the applicability of biological treatment application, but a treatability study is necessary in many cases to describe and relate process removal kinetics to the nature of the wastewater and the obtainable effluent quality.

There are several approaches which can be utilized to evaluate candidate biological systems. The most obvious technique for process evaluation is to simulate alternate systems on a bench or pilot scale and measure biological responses to various organic and environmental conditions. It should be recognized, however, that the accuracy of information developed from process simulation depends on several factors.

These include:

1. **The characteristics of the wastewater used in the treatability tests are representative of those anticipated in field tests.**
2. **The physical nature of the bench or pilot scale process is similar to the proposed full scale unit.**
3. **Independent and dependent operational variables are considered.**
4. **Environmental parameters affecting process efficiency are defined.**

It is apparent from these constraints that process simulation techniques can provide predictor relationships and mathematical expressions for the treatment process and wastewater in question, but does not necessarily define a specific model with general applications. However, a treatability study which is properly programmed and judiciously implemented does afford the basis for the logical development of unit process selection, design, and predictive performance.

Continuous flow and batch biological reactor systems are used in the laboratory to assess the treatment capacity

and process kinetics of a fluidized high rate biological system such as extended aeration or activated sludge. A batch reactor is primarily used in "screening" analyses; namely, determining toxic thresholds by varying the concentration of wastewater or delineating biologically "treatable" wastewaters from those which are not amenable to biological degradation. Such reactors are normally "fill and draw" type units, with the biological solids and wastewater being aerated until the organic constituents are reduced to a specific level. It is not recommended that batch developed bio-kinetics be applied to a continuous system.

Continuous flow systems in the laboratory are designed to provide a steady supply of wastewater through the reactor, permitting a continuous withdrawal of the spent substrate or treated effluent.

There have been many biological pilot facilities constructed, varying in size and design. Basically, the extent of pilot plant operations is a function of:

1. The degree of reliability required:
2. The size of the full scale facility;
3. The nature of the wastewater to be treated; and,
4. The time and budgetary allowances.

This basically includes an evaluation of substrate removal, sludge production, and oxygen requirements.

There is an increasing use of completely mixed biological systems, particularly in the activated sludge treatment of industrial wastes. In this case, the soluble BOD in the effluent is equal to that in the aeration tank. A material balance results in the following relationship:

$$Q\,S_o - Q\,S_e = \frac{dS}{dt}(V) \quad \ldots\ldots (2)$$

where:

S_o = raw waste COD, BOD
S_e = effluent COD, BOD
V = tank volume
t = detention time
Q = flow

Substituting the simplest form of dS/dt in terms of a retardant equation will yield the relationship:

$$(S_o - S_e)/(X_a t) = KS_e^n \quad \ldots\ldots (3)$$

where:

X_a = VSS undergoing aeration
K = substrate removal rate
n = exponent (for a first order approximation, $n = 1$)

It is recognized that the rate of biochemical reaction (K) is temperature dependent, and the most traditional expression for relating K with temperature in the range of 5°C to 35°C is the following equation:

$$K_T = K_{20°C}(\theta)^{T-20} \quad \ldots\ldots (3\text{-A})$$

where:

K_T = substrate removal rate at temperature "T"
$K_{20°C}$ = substrate removal rate at 20°C
T = aeration basin temperature, °C
θ = coefficient

The coefficient "θ" is a function of many variables—namely, the nature of the wastewater and the type of process. Reported values range from 1.02 for domestic wastewaters to over 1.08 for soluble industrial wastewaters (12). Where there is seasonal variation in temperature, winter conditions will control design.

It has been established previously that the total oxygen requirements in a biological system are related to the oxygen consumed to supply energy for synthesis and the oxygen consumed for endogenous respiration. This assumes that oxygen must be supplied to the system in order to:

1. Provide oxygen for biological organic removal ($a'S_rQ$);
2. Provide oxygen for endogenous respiration where cells lyse and release soluable oxidizable organic compounds ($b'X_aV$); and,
3. Provide oxygen required for chemical oxidation as measured by the immediate oxygen demand (K^oQ).

This expression is:

$$R_rV = a'S_rQ + b'X_aV + K^oQ \quad \ldots\ldots (4)$$

where:

R_r = oxygen utilization per day
a' = fraction of substrate (BOD or COD) used for oxidation
S_r = substrate (BOD or COD) removed
b' = fraction per day of VSS oxidized (oxygen basis)
K^o = chemical oxygen demand coefficient (as measured by immediate oxygen demand)

Sludge accumulation in the activated sludge system from the biological oxidation of wastewaters can be computed using a similar approach. The components of a mathematical relationship would include:

1. Increase in sludge attributable to influent SS (QX_i)
2. Increase in sludge due to cellular synthesis (aS_rQ)
3. Decrease in sludge due to cellular oxidation or endogenous respiration (bX_aV)
4. Decrease in sludge due to effluent SS (QX_e)

The expression is:

$$\Delta X = [QX_i + aS_rQ] - [bX_aV + QX_e] \quad . . . . . (5)$$

where:

ΔX = sludge production day
a = fraction of substrate (COD, BOD) converted to new cells
b = fraction per day of VSS oxidized (sludge basis)
X_i = influent SS
X_e = effluent SS

A graphical solution for determining the design coefficients can be obtained by varying organic loadings to the bench or pilot scale units and measuring the parametric responses. The substrate removal rate indicated in Equation 3 can be estimated by plotting the response data in accordance with Figure 2-A. If a non-removable COD or BOD persists as shown in Figure 2-B, then Equation 3 must be modified accordingly:

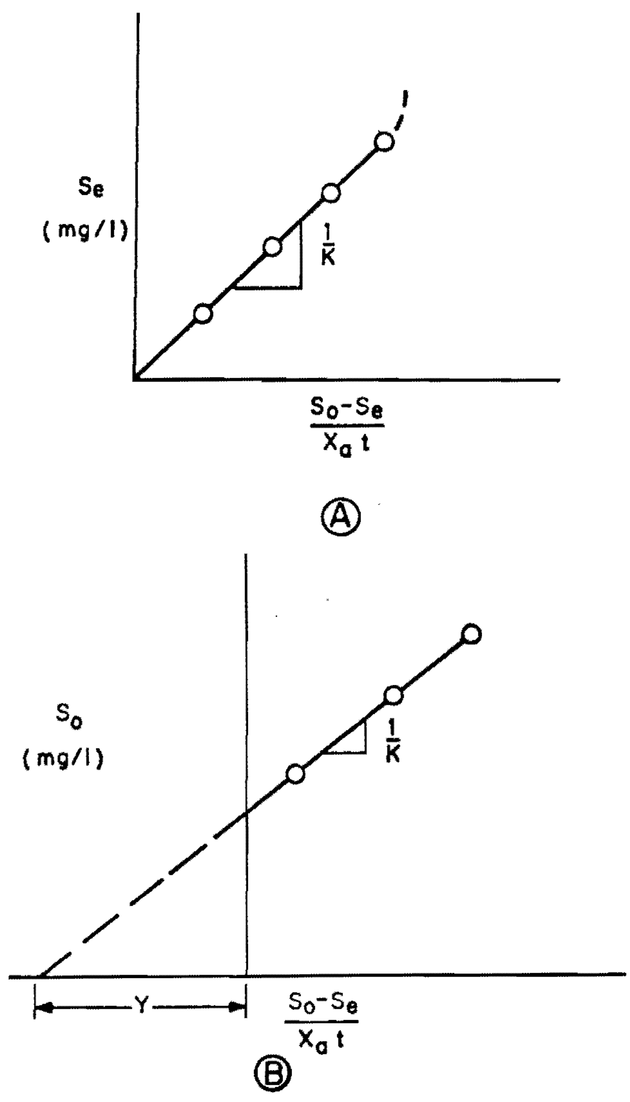

Figure 2. Substrate removal rate.

$$(S_o - S_e)/(X_a t) = KS_e - y \quad . . . . (6)$$

The system oxygen requirements can be estimated by rearranging Equation 4:

$$(R_r)/(X_a) = (a'S_r)/(X_a t) + b' \quad . . . . (7)$$

where:

t = V/Q and K^oQ is neglected assuming this oxygen demand is satisfied prior to testing. The a' coefficient is taken as the slope and b' as the intercept when plotting the data as shown in Figure 3-A.

The synthesis sludge production is predicted by rearranging Equation 5 and neglecting or accounting for the influent and effluent suspended solids.

$$(\Delta X)/(X_a) = (aS_r)/(X_a t) - b \quad . . . . (8)$$

the "a" and "b" coefficients are taken as the slope and intercept values respectively of the plot shown in Figure 3-B.

It is to be emphasized that a key parameter in the analysis of the data is:

$$(S_o - S_e)/(X_a t)$$

This parameter is known as the removal velocity and has the units of pounds substrate removed/pound MLVSS/day.

An equally important parameter is:

$$(S_o)/(X_a t)$$

This parameter is referred to as the organic load and has the units of pounds of substrate applied/pound MLVSS/day. It should be noted that the removal velocity is approximately equal to the organic loading in the lower ranges when the effluent concentration of substrate (S_e) is small.

EFFLUENT POLISHING

Increasingly stringent quality standards which are either being imposed or considered for many industries indicate the requirement for polishing or further upgrading the biologically treated effluent. This suggests the use of carbon or filtration processes following biological treatment. The concepts of biological-carbon systems treating refinery and petrochemical wastewaters were recently reported based on extensive pilot work conducted at various industries in the Eastern and Southwestern regions of the United States (13). This study described the interrelationship of the biological-carbon adsorption system and predicted the effluent quality obtainable by polishing the biologically treated effluent with fixed-bed carbon columns. This effluent quality projection is shown in Table 6. In certain cases, filtration alone is sufficient as an effluent polishing step. This is particularly true when most of the organic materials in the biologically treated effluent are in suspended form. In a recently conducted study for a refinery, 93 percent of the TSS, 78 percent of the BOD_5, and 37 percent of the COD were removed from a biologically treated effluent using an upflow sand filter as the polishing device (14).

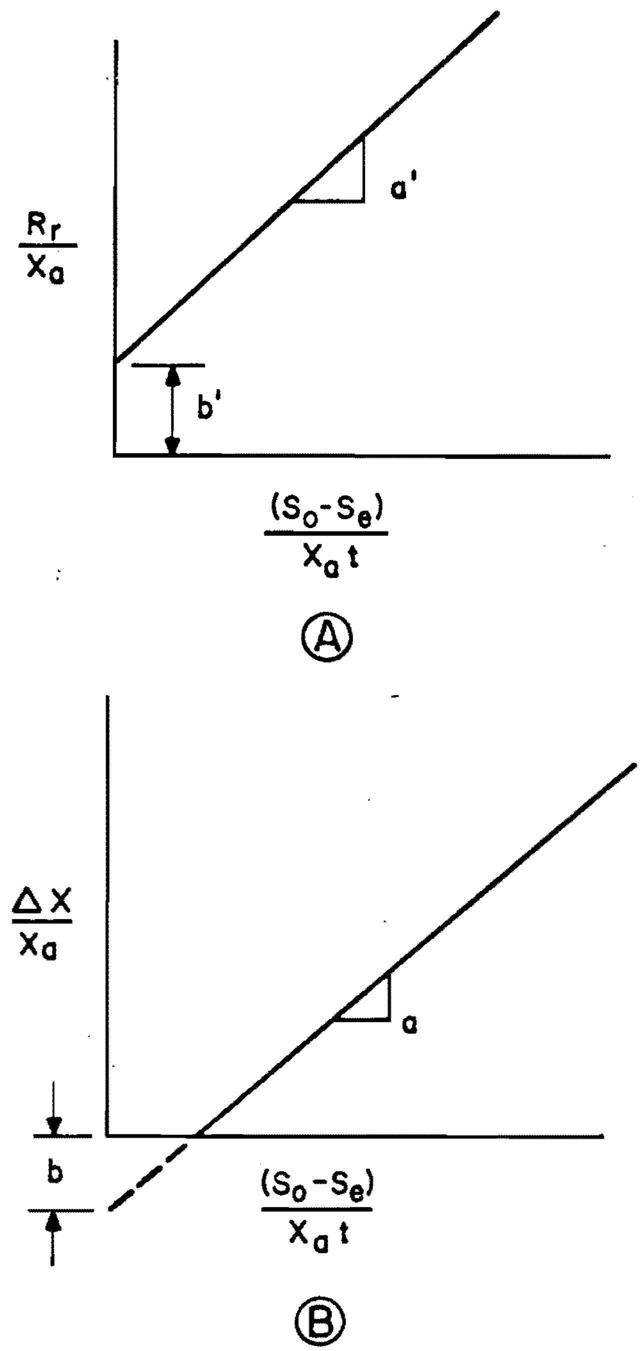

Figure 3. Oxygen requirements and sludge production.

SUMMARY

In summary, biological systems have been and will continue to be an effective process for treating many industrial wastewaters. It is important, however, that the applicability of these systems be proven before the design is finalized. The required pre- or primary treatment should be carefully considered. An adequate wastewater characterization program will indicate the need for equalization, oil removal, or other forms of treatment required as a pretreatment step if the biological system is to perform effectively.

Polishing processes, such as carbon adsorption or filtration, can be applied following biological treatment as required when more stringent effluent quality requirements are established.

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*Table 6. Estimated effluent quality for the activated sludge, carbon, and combined treatment of refinery wastewaters.**

CONSTITUENT	Mean Value Range Primary Effluent	Activated Sludge Effluent	Combined Activated Sludge-Carbon Effluent	REMARKS
COD	500-700 mg/l	100-200 mg/l	30-100 mg/l	Exact COD residuals vary with complexity of refinery & design contact times in the Activated Sludge and Carbon Treatment Plants.
BOD_5	250-350 mg/l	20-50 mg/l	5-30 mg/l	BOD residual depends on BOD/COD ratio which characterizes relative biodegradability of wastewater.
Phenols	10-100 mg/l	<1 mg/l	<1 mg/l	Phenols (ics) are generally amenable to biological and sorption removal.
pH	8.5 - 9.5	7 - 8.5	7 - 8.5	pH drop in Activated Sludge systems attributed to biological production of CO_2 and intermediate acids. pH change in carbon columns depends on preferential adsorption of acidic and basic organics.
SS	50-200 mg/l	20-50 mg/l	<20 mg/l	Primary effluent solids depend on design and operation of oil removal units. Activated Sludge effluent solids depend on effectiveness of secondary clarifier. Low effluent solids characterize carbon column effluent.
TDS	1500-3000 mg/l	1500-3000 mg/l	1500-3000 mg/l	TDS is essentially unchanged through both systems.
NH_3-N	15-150 mg/l	5-100 mg/l	2-100 mg/l	Exact concentration depends on pre-stripping facilities, nitrogen content of crude charge, corrosion additive practice and biological nitrification.
P	1-10 mg/l	<1-7 mg/l	<1-7 mg/l	Only removal attributed to biological synthesis.

* Based on wastewater characterization data and treatability studies conducted by the author at eight refineries and petrochemical installations.

TZ

Cleaning, TV Inspection and Rehabilitation of Sanitary Sewer Lines

*C. Sketchley and James J. King**

SEWER CLEANING

Rodding-Cleaning Machines, Inc. is very appreciative for having this opportunity of speaking to you today. My talk will be based on three subjects—Sewer Cleaning, TV Inspection, and Sanitary Sewer Rehabilitation. The purpose for Cleaning, TV Inspection and Rehabilitation of Sanitary Sewers is the preservation of pipelines and structures to assure a useful life and ability to withstand the effects of corosion, erosion, age and settling, and further, to make corrections of existing structural deficiencies from all causes and finally the reduction or elimination of infiltration and exfiltration. The elimination of stoppages or control of infiltration and or exfiltration must be accomplished in three steps. The first step is a cleaning program. A cleaning program must be set up for preventative maintenance of the system. This is the first step to the control of stoppages, corosion, build up of sedimentation and sulfides. With the present day sewer cleaning equipment, the guess work of what to do and how to do it is a thing of the past.

Let's take a moment and talk about what cleaning equipment is available for the purpose of cleaning and maintaining a sanitary sewer system. Do you realize that it has only been in the last 20 years that equipment has been available for the maintenance of sewers? Up to the early forties, cleaning and maintenance was performed by either hand rods, tires, cones, or winches. If this did not work, other methods would be performed on a trial and error basis. The results would be temporary. But more important, it developed poor relations with industry and the private citizens of the community. To overcome this type of situation the community had to be educated to the fact that you just don't install a sewer system and forget it. This re-education to the public, board members, and budget officials was a difficult task. It was not until an accident occurred, such as a cave-in in the street, and a person was hurt or killed, or a shut down of a plant that replacement must be performed. In the past 20 years great strides have been made in sewer maintenance. This came in part from communities and cities forming a sewer maintenance section where they would get together once a month and discuss their problems and equipment needs. Through these meetings and supplying requests for sewer equipment, the sewer departments were able to meet the needs and perform their jobs.

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The purchasing of modern cleaning equipment is not limited to rodding machines, bucketing equipment, high pressure water cleaning units or hydraulic balling equipment, but also includes portable equipment such as scooters and self-propelled scrappers. With this type of equipment available, all types of maintenance and sewerage problems can be dealt with.

TV INSPECTION

The introduction of TV Inspection in the early sixties was the foundation for sewer rehabilitation. It is through TV inspection that conditions of sewer lines can be seen and evaluated. Problems such as deterioration, shifting of lines, cracks, sags, off-set joints, infiltration and exfiltration can be seen from this, and a plan of attack can be made to eliminate them. Not only from a preventative maintenance is TV Inspection useful, but also when new sewer lines are installed, the cities and or sewer districts can be assured of a proper and acceptable job. To go another step further with TV Inspection, the Environmental Protection Agency insists on Closed Circuit TV Inspection with Audio-Video Tapes to uphold the findings and determinations of Engineering Firms doing sewer system analysis. Through TV Inspection, all peoples concerned will have a complete visual record of the problems in their system and from that, recommendations for correction can be made.

SEWER REHABILITATION

From this point, we enter into the third subject of my presentation—Sanitary Sewer Rehabilitation. As of this date, there are three methods of sewer system rehabilitation—INTERNAL GROUTING, POLYETHELENE SLIP LINING, AND TOTAL REPLACEMENT.

Internal Grouting is the most common and widely used rehabilitation method. This type of rehabilitation consists of locating a sealing packer over a joint and with air pressure testing this joint for tightness and if the joint fails this test, then seal it with chemical grout. This grout not only seals the joint within the connection, but also penetrates outside the joint into the ground and combines itself with this material and forms a lasting seal around the pipe and joint. Depending on the drop of air pressure when applied to the joint, the setting time for the gel can be calculated. If the air pressure drops very slowly, then a fast setting time should be set. If the air pressure drops fast, then a slower gelling time is calculated so that penetration can be made into and around the joint to secure a tight seal. Internal Grouting can be used effectively to correct infiltration and or exfiltration when the following conditions are encountered: (1) Structurally

Sound Pipe with deteriorated joints, off-set joints, open joints or occasional small cracks; (2) House Service Connections; (3) Manhole walls, bases, and inverts. Internal Grouting has its limitations and should not be used as a structural repair for broken, crushed or badly cracked pipes. There may be some of you here today that don't agree with the use of Internal Grouting for the reduction of infiltration and or exfiltration from sewer systems. Let me assure you that this method is proven and reliable. Of the three methods of Sanitary Sewer Rehabilitation, it is the most economical and has been proven on hundreds of projects throughout the country. Let me emphasize at this time that a grouting project is only as good as its applicator.

Polyethelene Slip Lining involves the pulling of a polyethelene pipe into and throughout an existing Sanitary Sewer. This technique was introduced in the early seventies for rehabilitation projects, and many of the projects that couldn't be Internal Grouted successfully can now be Slip Lined at a lower cost and expense than if it had to be totally replaced. Such advantages as minimum excavations, savings in trenching costs and safety, heat fused joints, minimum interruption for traffic and (50 percent) savings time for installation versus Total Replacement enhance the method of rehabilitation by Polyethelene Slip Lining. A major advantage that everyone cares about is cost. With this type of rehabilitation, there is an approximate savings of 30 percent over Total Replacement. Polyethelene Slip Lining can be used effectively when the following conditions exist: (1) Deteriorating pipe having shallow grade, septic conditions, and corosive liquids; (2) Extensively cracked pipe in unstable soil conditions; (3) Pipe with massive and destructive root intrusion problems; (4) Off-set and Open joints; and (5) Deteriorated House Service Connections. Total Replacement involves the removal of existing pipes and or excavations, backfill, and paving. The cost of this method is extremely expensive and much higher than the previous two types.

You are aware of the procedures involved in this method and an extensive discussion would not enlighten you to any degree. The main advantage of total replacement is when sewer lines are so badly deteriorated that extensive excavations would be needed in order to correct the alignment so that a liner could be installed. When this situation arises, there is no alternative but to totally replace the line.

At this time, I would like to add that combination methods for sewer system rehabilitation are needed. In many instances where Sewer System Evaluation Studies have been performed, the corrective methods suggested have been combinations of all three types of rehabilitation. Depending on the conditions encountered, Internal Grouting, Polyethelene Slip Lining, and Total Replacement may be needed. Rehabilitation is what it says. The method or the combination of methods is what makes up that final program that eliminates sewer problems.

To summarize, the three subjects that I have talked about are all interrelated. Our main objective is the preservation of the sewer system. Through Cleaning, TV Inspection, and when needed, a proper rehabilitation program, Sanitary Sewer Systems will be maintained, checked, and corrected; and, thus, problems will be eliminated.

Flow Reduction by Pipe Insertion Renewal at Heber, Utah

*Carl H. Carpenter, P.E.**

INTRODUCTION

Heber, Utah is a small rural community of about 3,500 population situated 40 miles southeast of Salt Lake City. It has a sanitary sewer system which was constructed in 1939 and a single-stage trickling filter plant of 1.5 MGD capacity which was constructed in 1954. The sewer system includes more than 112,000 feet of concrete pipe ranging in size from 6 to 15 inches. (see Figure 1.) The pipe joints were caulked with mortar which has deteriorated over the years and allowed excessive infiltration to take place. The source of infiltration is irrigation water delivered throughout the city in a system of open ditches. Thus, the sewer system has acted as a drain during the irrigation season, and picked up massive amounts of water which is conveyed to the treatment plant. When irrigation water is applied to the city lots in the spring and summer each year the flow at the plant increases from 0.6 MGD to 6.0 MGD in a matter of a few days greatly overloading the plant capacity. (See Figure 2.)

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1969 STUDY

In 1969, the city retained Nielsen, Maxwell & Wangsgard to investigate a means of correcting this massive infiltration problem. After a year of study it was recommended that the ultimate solution would be to better control the irrigation water by installing a pressure irrigation system throughout the city. The city adopted the

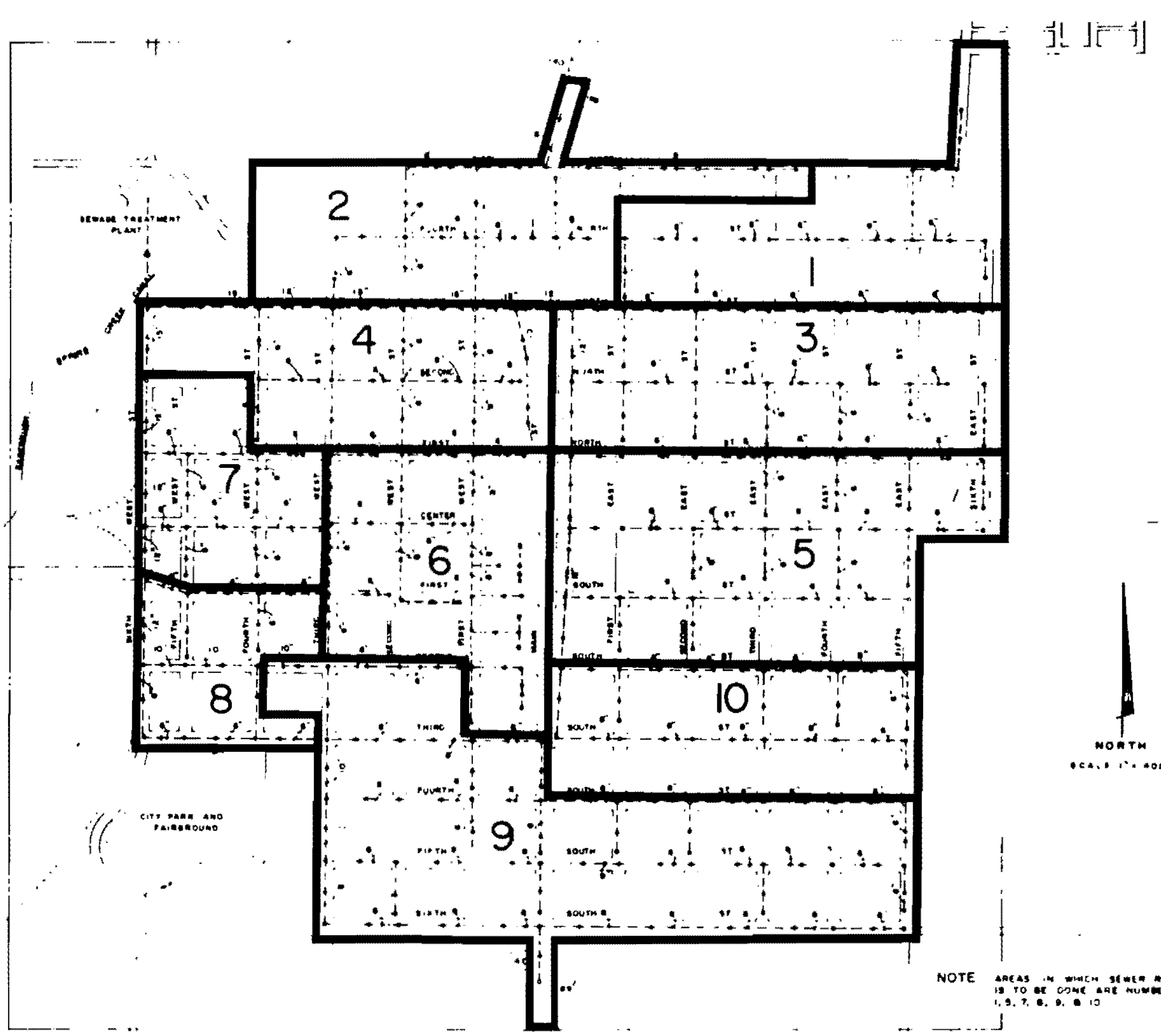

Figure 1.

project but it was defeated in a bond election in 1970. A scaled-down version was also turned down in a 1971 bond election.

In 1972 and 1973 about 8,500 feet of sewer line was rehabilitated by the chemical grouting method. This proved to be somewhat effective; however, in areas where this was done, the sewers no longer picked up irrigation water and the rising water table flooded basements.

1975 I/I STUDY

In 1975, with the aid of an EPA Step 1 Grant, an intensive study was made of the infiltration problem and a cost-effective analysis made to determine the best means of correction. Several alternatives were evaluated which included sewer sealing, subsurface drains, drainage wells, replacing the entire system, enlargement of the existing plant, and construction of a new and larger plant. It was determined that a project which included a combination of sewer rehabilitation and subsurface drains was the most cost-effective. (See Figure 3.) This combination was necessary to separate the sanitary flow from the groundwater and keep the water table under control. The city had limited funds with which to construct the project and did not want to risk another bond election. Therefore, an intensive study of the I/I situation in July 1975 was used to pinpoint those portions of the city where infiltration was most critical and also where the fluctuating water table was a problem. The method of correlating sanitary loading and flows at key manholes with 24 hour sampling of flows at the treatment plant (as reported in 1976 by Luce & Kisana) was to delineate critical areas. This study indicated an average sanitary flow of 0.36 MGD and infiltration amounting to 5.9 MGD.

THE PROJECT

A project was developed to correct the infiltration, and a Step 2 Grant from EPA was used to prepare plans and specifications. The project included installation of 16,000 ft. of subsurface drain in the city ranging in size from 8 to 36 inches. (See Figure 4.) It also included a proposal to rehabilitate 54,000 feet of sanitary sewer ranging in size from 6 to 15 inches by either chemical grounting or polyethylene pipe insertion renewal (sliplining). (See Figure 5.) The project was bid in two contracts: one for the drains and one for the sewer rehabilitation. The bidders on the sewer rehabilitation were given the choice of chemical grouting or sliplining. A bid opening February 1976 provided a low bid of $275,000 for the drains, $130,300 for chemical grouting of sewers, and $365,000 for sliplining. It was the city's desire to do the sewer rehabilitation with sliplining rather than grouting even

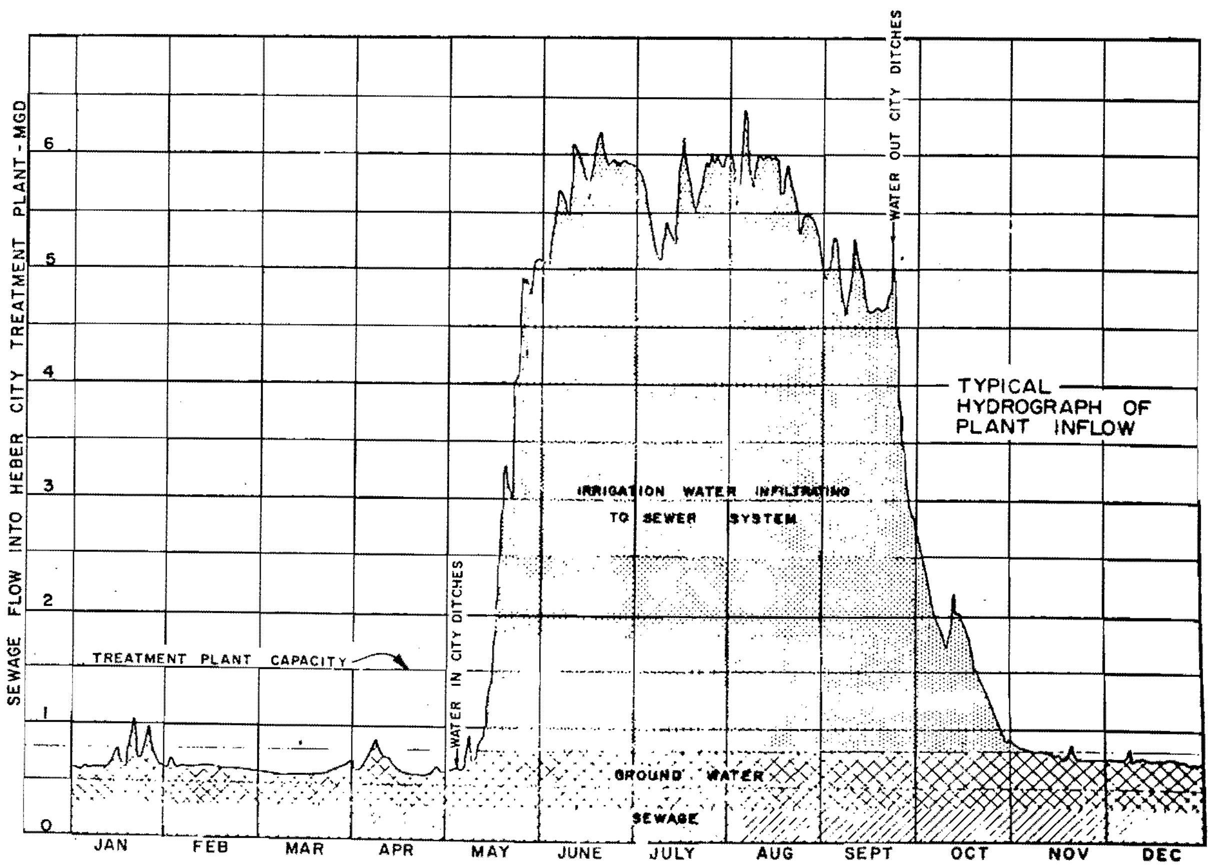

Figure 2.

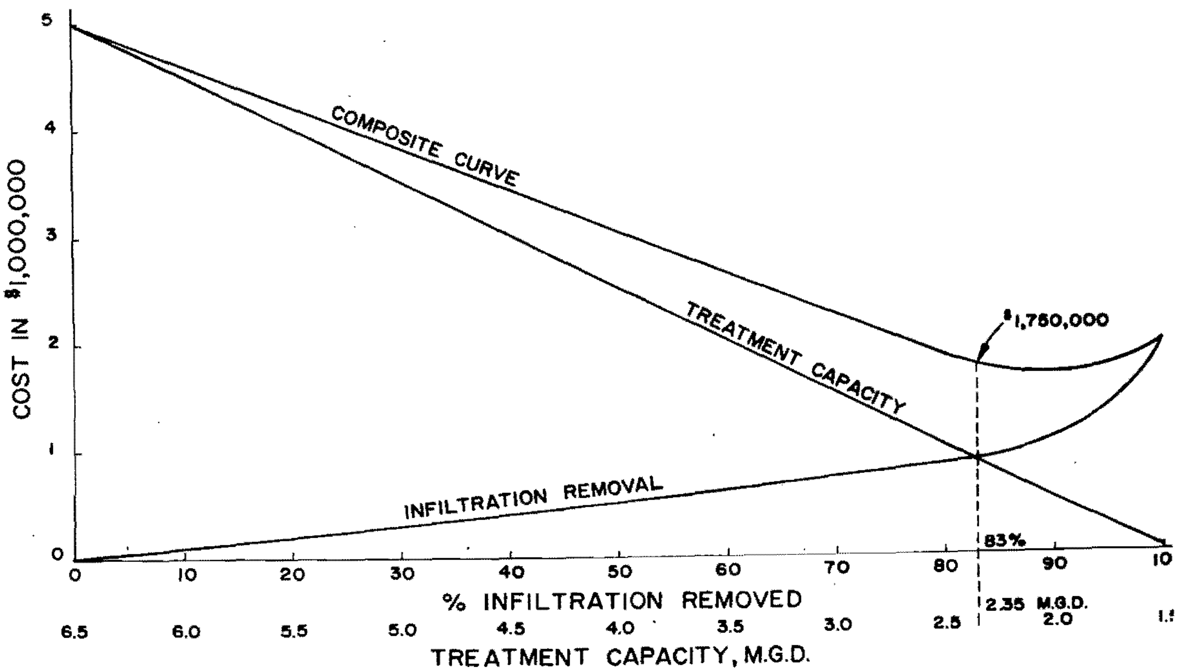

Figure 3.

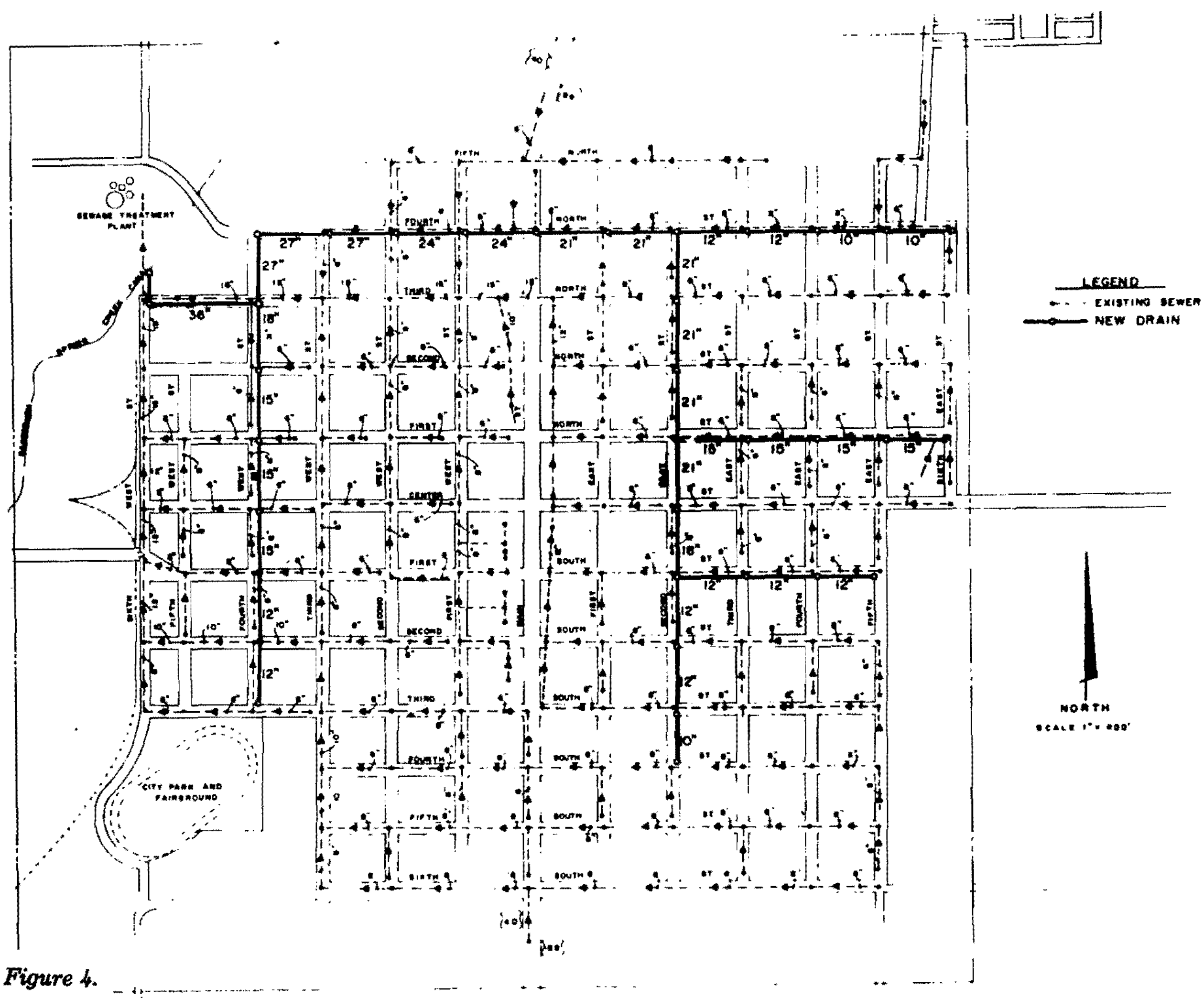

Figure 4.

though it was more expensive, because it provided for a new, tight service connection at each house lateral; whereas, the grouting did not. The I/I study has shown that leaking service laterals were a big contributor to infiltration. EPA agreed to this request and a Step 3 Grant was provided to pay for 75 percent of the two contracts.

Construction on both contracts began April 1, 1976 and they ran concurrently through the summer with completion by November 1. The drain contractor was Knudsen Construction Company of Ogden, and the sewer rehabilitation contractor was Rodding-Cleaning Machines, Inc. of Los Angeles, California.

Drain construction was completed using conventional methods. Because some trenches were more than 14 feet in depth, a box or shield was used throughout the project. Concrete pipe with lugs cast in the bell to provide an open joint was used. A gravel envelope was placed around the pipe to keep sediment out of the drains and allow flow to enter the joints.

The sliplining required cleaning and televising each line before pipe insertion. The exact location of each house lateral was required and this was done with the television camera. One serious problem was differentiating between active and inactive connections. This was resolved by asking homeowners to discharge water to the sewer during televising. Access shafts were required at manholes to allow pulling the slipliner through. A hole was cut where the old sewer connected to the manhole, and the slipliner with a pulling head was inserted. A cable was threaded through the line to the pulling manhole and attached to the pulling head. The exact length and size of slipliner was fused together on the surface and laid out in the city street. It was then attached to the pulling head, and pulled through the sewer line to the pulling manhole. Excavations had to be made to expose each house lateral before pulling and a hole in the old concrete line was cut to allow access to the slipliner. As soon as the slipliner was pulled, crews immediately began to cut holes in the liner at each house lateral to make that connection. A fitting was fused to the liner at each connection and also fused to the house lateral pipe to provide a tight seal. Concrete was poured under each connection and backfill placed. The connection was then sealed at each manhole. One to two blocks were pulled at a single operation. Preparation time, including fusing liners, excavating access shafts, and exposing house

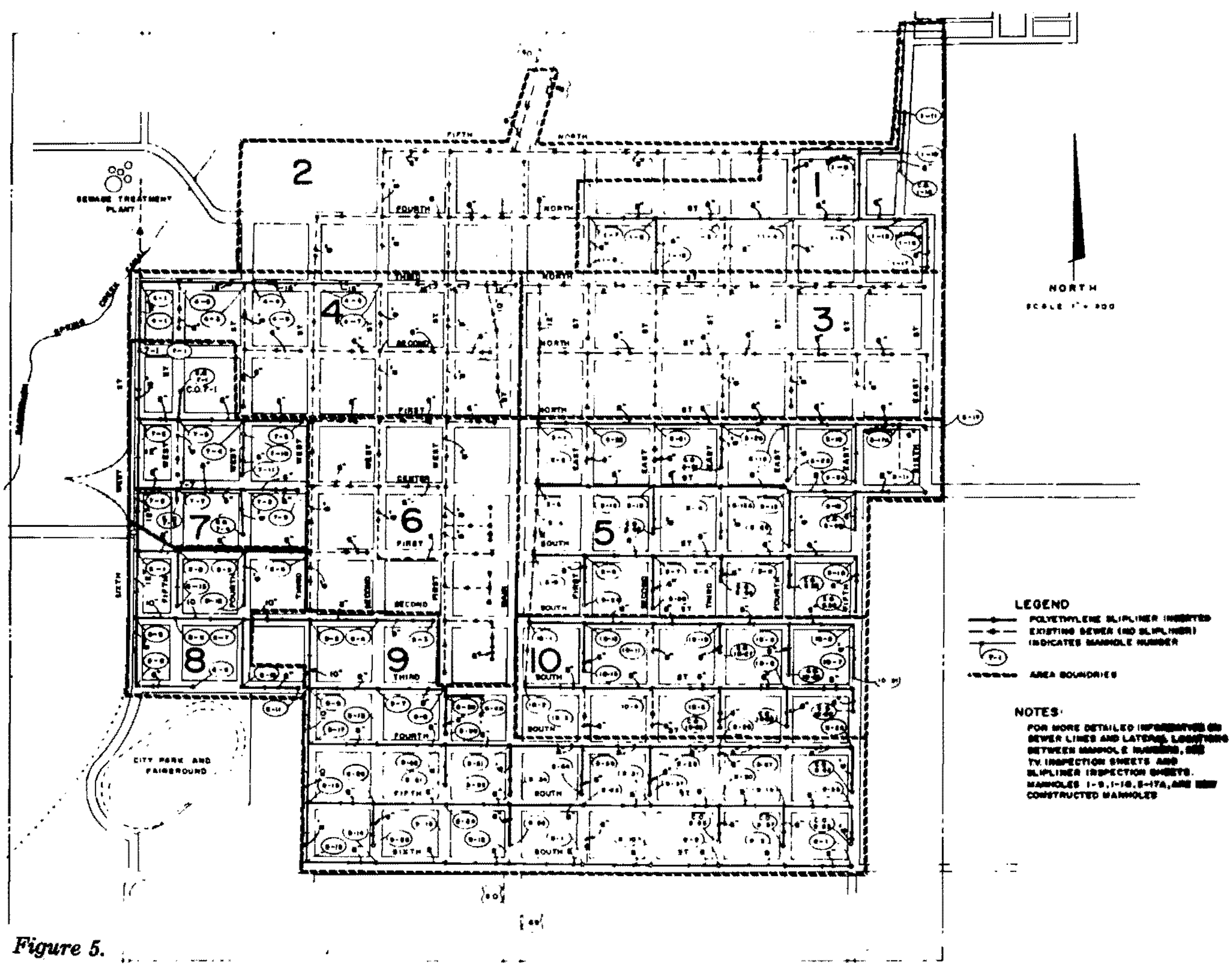

Figure 5.

laterals would take several days; but, pulling the liner averaged about 2 minutes. While the house connection was being made, sewage was allowed to dissipate in the trench excavation. No bypassing was required.

The cost to clean the lines ranged from 30 to 40 cents per foot, to televise averaged 30 cents per foot, sliplining ranged from \$3.25 to \$8.85 per foot, access shafts cost \$200 apiece, and each service connection cost \$300. The polyethylene liner was supplied by Dupont with brand name Alydl "D" and had a SDR ratio of 32 with a wall thickness ranging from 0.223 to 0.418 inches. The material was quite flexible and the fuxed joints were as strong as the original material.

RESULTS

Even though the project was not completed until November, the results of both the drains and lining were quite dramatic. Discharge of the drain outlet was about 12 cfs during the July to September period, and the sampling of the water shows so far that it meets the State's Class "C" standard; thus, indicating that the sewers have been effectively sealed. Flow at the treatment plant did not get higher than 3.6 MGD during the summer months even though the lining job was not completed until fall. This compares with a peak of more than 6.5 MGD for the previous year. (See Figure 6.) The results of lining the areas delineated by the I/I study confirm that the method used to determine the areas of high infiltration was accurate. In conclusion, there was not a single complaint of basement flooding in Heber City in 1976. Total cost per foot to do the sliplining, including all items, was \$6.70 per foot; and, for installation of the drains, \$16.97 per foot. Approximately half of the sewer system was rehabilitated.

When funds become available, the city will probably complete the sewer rehabilitation in the remainder of the system.

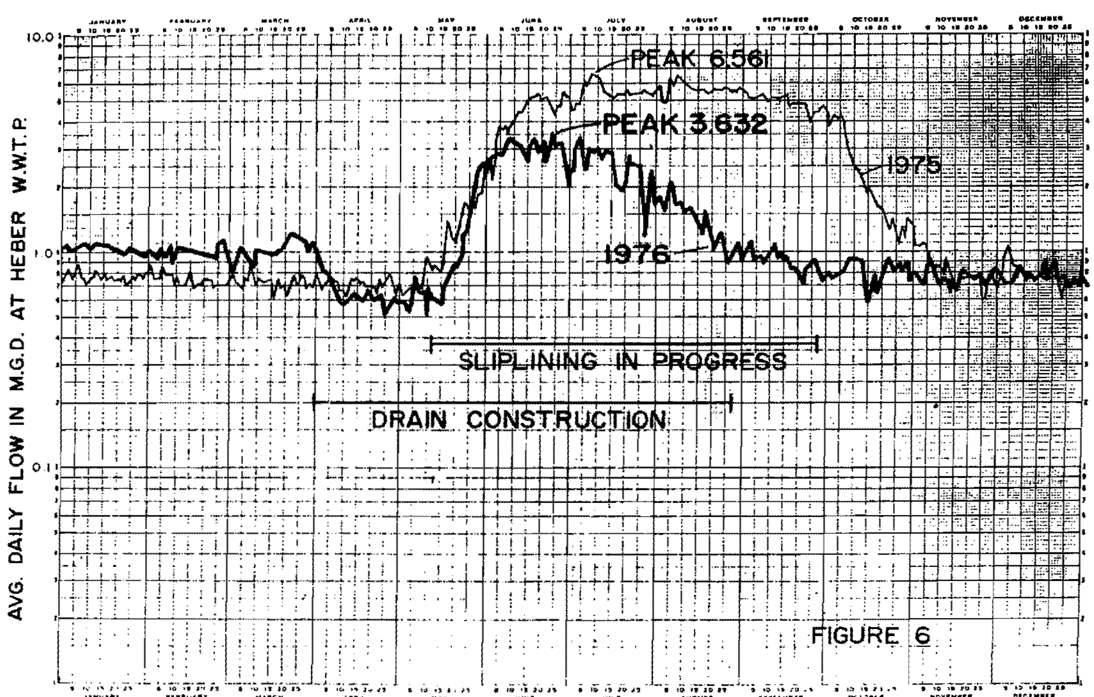

Figure 6.

Experiences in Wastewater Filtration with the Low Head Automatic Backwash Filter (ABW)

*Ronald F. Culp**

INTRODUCTION

The Automatic Backwash Filter, as manufactured by Environmental Elements Corporation, a subsidiary of Koppers Company, Inc., was originally invented in the late 1930s by the Hardinge Company of York, Pennsylvania, now a part of Koppers. Since that time much development and continual updating of design has been accomplished, however, the overall concept of continuous or uninterrupted filtering has remained essentially the same and has been used in municipal and industrial water and wastewater treatment facilities for more than thirty years.

This unique filter design has a media bed divided into a number of compartments or cells and an automatic mechanism is used to backwash and clean individual compartments sequentially, while the remaining cells continue to filter the water or wastewater applied (Refer to Figure 1).

Referring to Figure 1, a channel distributes influent along the length of the filter. The influent enters the filter through evenly spaced ports. The filter bed and underdrain are partitioned. The water is approximately 8"-10" deep. The sand in each compartment is 11" deep, the aluminum oxide support plate developed especially for this unit is 1" thick, and the underdrain channel is approximately 8" deep. Water flowing downward through the bed passes into the effluent channel through a port from each underdrain section.

The automatic backwash mechanism, suspended from the motor driven carriage, draws finished water from the effluent channel (via a separate backwash pump) and discharges it into the underdrain of the compartment simultaneously covered by the hood. Another pump withdraws the washwater from under the length of the hood, discharging into a washwater trough for removal. Normally, the mechanism moves slowly and continuously along the length of the filter, backwashing each compartment until all have been cleaned and the loss of head across the filter has returned to normal.

Because the entire filter remains in operation, except for the compartment being backwashed, there is plenty of clean water in the effluent channel, eliminating the need for separate washwater storage. Automatic compartmental backwashing requires only about 150 gpm. Rarely does the automatic backwash system use more than 2.0 percent of the total throughput to clean the filter bed. The short cleaning cycle, repeated regularly, keeps the sand in a nearly clean condition and limits penetration to the upper 1 to 2 inches. Also, the presence of some material on the bed aids in removing particulate from the flow.

WASTEWATER TREATMENT EXPERIENCES

In 1961, the Los Angeles County Flood Control District ran studies using the ABW Filter and others to determine if wastewater from a secondary treatment plant could be reclaimed suitable for well injection (1). It was of considerable importance to remove the major portion of water-borne solids and precipitate forming constituents which could deposit in soil voids causing a loss of aquifer permeability. Tests with the ABW confirmed that the high quality effluent necessary for well injection could be produced.

Later, in 1968, comparative studies were undertaken by the Metropolitan Sanitary District of Greater Chicago at the Hanover Park, Illinois Treatment Facility to determine tertiary filtration efficiencies of the ABW and microstrainers(2). The filter produced an effluent superior in quality to the Illinois Sanitary Water Board criteria of 5 mg/l suspended solids and 4 mg/l BOD. Percent removal of suspended solids ranged from 76.4 to 77.5 percent. Hydraulic loading was approximately 2 gpm/ft^2. Since this study was undertaken, a design for additions to the treatment facility has been completed and construction began in 1976 for the installation of additional equipment, including four new filter units.

Much data collected by treatment facilities using the ABW filter illustrated suspended solids removal at a conventional rate of 2.0 gpm/ft^2. Since many of these plants are not yet operating at design capacity, or rates higher than 2.0 gpm/ft^2, data on removal of suspended solids at higher hydraulic loadings was not readily obtainable in detail. To demonstrate and test the removal efficiency at higher rates, a pilot ABW unit was constructed and used at several facilities. The pilot plant is an automatic backwash filter having 36 square feet of filter area divided into eighteen separate cells, each measuring 8" x 36", or approximately two (2) square feet. Silica sand, 11" deep, having an effective size of .599 mm and a uniformity coefficient of 1.252 was used as the filtering media. The filter was placed in a trailer equipped with flow meters, headloss gages, and miscellaneous laboratory equipment.

A trickling filter plant operating with an average daily flow of 45 MGD was chosen as one of the first testing sites. The pilot ABW operated continuously for more than 500

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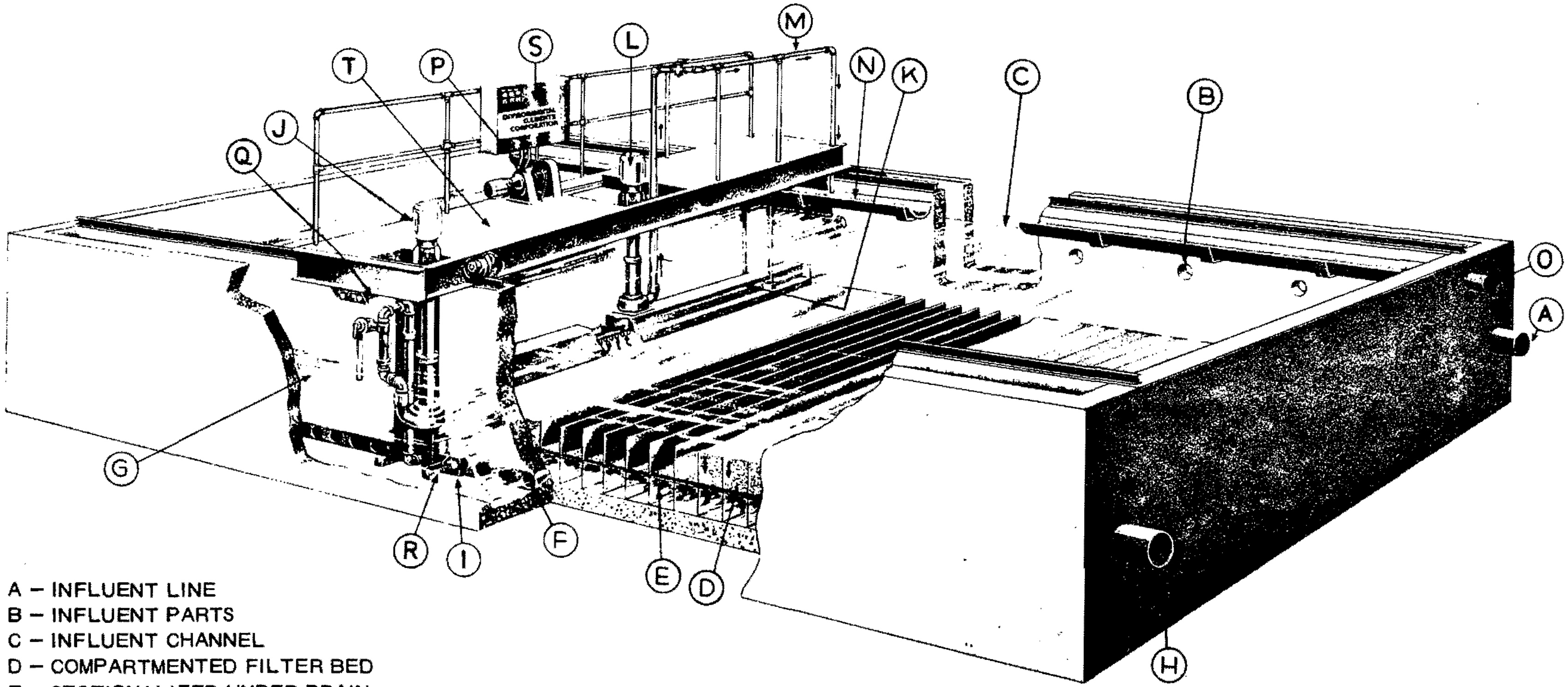
ENVIRONMENTAL ELEMENTS CORPORATION

AUTOMATIC BACKWASH FILTER



A – INFLUENT LINE
B – INFLUENT PARTS
C – INFLUENT CHANNEL
D – COMPARTMENTED FILTER BED
E – SECTIONALIZED UNDER-DRAIN
F – EFFLUENT AND BACKWASH PARTS
G – EFFLUENT CHANNEL
H – EFFLUENT DISCHARGE LINE
I – BACKWASH VALVE
J – BACKWASH PUMP ASSEMBLY
K – WASHWATER HOOD
L – WASHWATER PUMP ASSEMBLY
M – WASHWATER DISCHARGE PIPE
N – WASHWATER TROUGH
O – WASHWATER DISCHARGE
P – MECHANISM DRIVE MOTOR
Q – BACKWASH SUPPORT RETAINING RINGS
R – PRESSURE CONTROL RINGS
S – CONTROL INSTRUMENTATION
T – TRAVELING BACKWASH MECHANISM

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FIGURE I

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hours at rates ranging from 106,000 to 250,000 gpd, the source of influent being secondary clarifier overflow. Grab and composite samples were taken throughout the testing period by plant personnel and analyzed for suspended solids concentrations according to Standard Methods(5). Table 1 lists a summary of results at the various hydraulic loadings. Terminal head loss was low, ranging from 5" to 11".

Following this study, the pilot ABW was placed in operation in an activated sludge plant having an average daily flow of 1.5 MGD. The unit operated for more than 137 hours processing approximately 100,000 gpd. As shown in Table 2, influent suspended solids ranged from 32 to 86 mg/l. Percent removal of suspended solids averaged 87.2 percent. Both grab and composite samples were taken in this case also, and suspended solids concentrations were determined by a State approved laboratory according to Standard Methods. Additional effluent samples were taken following backwashing to determine if solids breakthrough occurred.

SUMMARY

This recent testing further illustrated the capability of this unique single media filter design to process influents at higher rates with higher than normal suspended solids, as has been shown with conventional filter units using a dual or multi media(4). Proper design and coordination of components such the backwash pump discharge valve, carriage speed, underdrain, washwater collection, and method of operation contribute significantly to the efficiency of the ABW unit.

Studies are presently underway to determine the effectiveness of using dual media (anthracite coal and silica sand). These studies are planned to determine if filtering efficiency, better than that reported above, will result.

Table 2. Operating Summary Activated Sludge Wastewater Treatment Plant, December 1976.*

OPERATING SUMMARY*
ACTIVATED SLUDGE WASTEWATER TREATMENT PLANT
DECEMBER, 1976

FLOW RATE (gpm/ft^2)	DATE	TIME	SUSPENDED SOLIDS (mg/l) INFLUENT	SUSPENDED SOLIDS (mg/l) EFFLUENT(1)	% REMOVAL
1.929	12/3	8:00	37.0	3 - 5	86.5 - 91.9
		10:00	41.0	2 - 4(2)	90.2 - 95.1
		12:00	34.0	2 - 3	91.2 - 94.1
		2:00	36.0	1 - 4	88.9 - 97.2
		4:00	32.0	2	93.8
		Composite(4)	38.0	4	89.4
	12/6	8:00	34.0	3 - 4	88.2 - 91.2
		10:00	86.0	3	96.5
		12:00	64.0	4 - 5	92.2 - 93.8
		2:00	71.0	9 - 14(3)	80.3 - 87.3
		4:00	66.0	10 - 12	81.8 - 84.8
		Composite(4)	67.0	7	89.5
	12/7	8:00	38.0	7 - 7.5	80.2 - 81.6
		10:00	58.0	9 - 10	82.7 - 84.5
		12:00	43.0	6 - 7	83.7 - 86.0
		2:00	39.0	6 - 6.5	83.3 - 84.6
		4:00	36.0	8 - 10	72.2 - 77.7
		Composite(4)	42.0	7.5	82.1

NOTE: Average Terminal Headloss was 5"

(1) Two effluent samples were analyzed vs. one influent

(2) Sample taken immediately after backwash (10 min.)

(3) Sample Taken shortly after backwash (20 min.)

(4) Composite consisted of samples taken every two hours from 8:00 a.m. to 4:00 p.m.

* The above results are based on 51 data points.

Table 1. Operating Summary Trickling Filter Wastewater Treatment Plant, November 1976.*

INFLUENT FLOW RATE (gpm/ft^2)	INFLUENT SUSPENDED SOLIDS RANGE (mg/l)	EFFLUENT SUSPENDED SOLIDS RANGE (mg/l)	AVERAGE REMOVAL (%)	TERMINAL HEADLOSS (inches)
2.049	13-20	1.5-4.8	84.4	5
3.05	15-18.7	2.4-4.3	80.3	6
4.059	16-19.3	2.9-5.0	77.6	8
4.823	17.3-31	3.6-6.5	78.9	11

*The Above results are based on 58 data points

The ABW provides a number of advantages other than those already mentioned:

1. Mudballing, a common problem in conventional filters, does not occur.

2. The bed does not pack, therefore, there is no danger of a packed bed cracking and short-circuiting or breakthrough occurring. The filter uses a low driving force.

3. The low operating head permits water to be transferred from the pretreatment section or secondary clarifier with minimum free-fall which avoids floc breakup. This low head also limits the shearing forces to which floc in the bed is subjected. Floc breakup is also avoided since influent pumping is usually not a requirement.

4. Construction of the filter itself is simple, its profile is low.

5. Through the use of channels and the automatic travelling bridge mechanism, the pipe gallery has been eliminated.

6. There is no longer a need for an operator to be in attendance to control backwashing.

7. There is no "surge" of washwater from the filter during backwashing. The continuous flow of washwater eliminates the need for storage to prevent upset conditions at the head of the plant or point of discharge.

All of these result in a lower installed cost, minimum connected horsepower, low operating costs, and minimal maintenance. Thus, the ABW Filter provides the cost effectiveness currently desired by regulatory agencies and consulting engineers. The many plants presently in operation give evidence to its world wide acceptance.

NOTE: Reference of specific testing locations has been intentionally deleted. Supportive information is available upon written request to the author.

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Design Considerations for Filtration of Secondary Effluent

*William R. Kirkpatrick and Nicholas L. Presecan**

In the past few years there has been an increased legislative emphasis on reducing the adverse environmental effects of pollution from urban wastewaters. More stringent standards are being set which reduce not only the allowable mean concentration of pollution constituents but limit the frequency of deviation from the mean allowable discharge concentration. Filtration of secondary effluent provides an excellent method of reducing the variability and concentration of some effluent constituents. More stringent public health aspects of pollution control have resulted in the need to reduce bacterial and virus levels in effluents. Filtration used in proper combination with chemical pretreatment and post filtration disinfection will produce an effluent which significantly reduces bacteria and virus levels in the final effluent. In addition, filtration provides an ideal treatment ahead of more advanced forms of tertiary treatment such as carbon adsorption, electrodialysis, and reverse osmosis by preventing secondary solids carryover which would tend to foul the advanced process operation.

The major applications of secondary effluent filtration are: (1) direct filtration, (2) filtration of chemically pretreated effluents, and (3) filtration of chemically coagulated and clarified effluent. In many cases, direct filtration is all that is required. These are cases for which the effluent is not destined for intimate contact with animal and plant life subject to significant human contact. In addition, operational and testing results indicate that for very low suspended solid and turbidity concentrations, chemical coagulants prior to filtration have little effect. Where, however, this type of contact or potential significant contact produces a reasonable public health concern, regulatory agencies may require additional pretreatment with chemicals.

This paper discusses the various significant design considerations necessary to achieve proper filtration of secondary effluents from the conventional activated sludge process. In the cases discussed, the controlling regulatory agencies have determined that chemicals injected prior to filtration will in fact aid in the production of an effluent low in bacteria and virus levels in addition to reducing suspended solids, BOD, and turbidity. In particular, the paper discusses the design of the Whittier Narrows Water Reclamation Plant effluent filtration facilities, County Sanitation Districts of Los Angeles County, and the Burbank Water Reclamation Plants, City of Burbank, California. Where appropriate, other similar filtration facilities and filtration research are cited.

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CASE HISTORIES

The Whittier Narrows effluent filters are open top, dual media gravity filters. There are six common wall basins 16-feet wide by 32-feet long and 27-feet deep (total), constructed of cast-in-place reinforced concrete. The 27-foot depth includes the underdrain plenum, underdrains, media, and head above the media including extra space required to convert the filter to a deep bed carbon filter. The principal components of the completely automated system are the chemical pretreatment system, dual media filters, low headloss underdrains, air-water backwash system, backwash recovery system, and the filtered effluent chlorination system. The filtered effluent flows into a wet well where it is lifted by several vertical turbine pumps and discharged into the chlorine rapid mixing basin. The chlorinated effluent continues through the chlorine contact tanks, a dechlorination tank, and then is discharged into the groundwater recharge channels of either the Rio Hondo or San Gabriel Rivers. The two river channels are subject to intensive public use for swimming, sun bathing, biking, and horse trails (Reference 1).

The Burbank Water Reclamation Plant effluent filters are enclosed, dual media gravity filters. The three prefabricated steel tanks are 26-feet in diameter and 10-feet deep; each tank has three independent equal area filter compartments (cells). The principal components of the automatic filtration system are the pumps, dual media filters, air-water backwash system and the chemical feed system. The filtered effluent passes through a chlorine contact tank and discharges either to the Los Angeles Flood Control Channel or to the Burbank Steam Power Plant for use as makeup cooling water.

DESIGN CONSIDERATIONS

Many factors are involved in proper filtration design. Current published filtration technology is rather thorough in terms of presenting a list of considerations as well as good design "numbers" and "rules of thumb." This paper will not reiterate these items but will present specific significant design considerations used for two aforementioned pollution control facilities to produce the desired effluent. The design considerations discussed include pre-filtration chemical treatment requirements, filter loadings, media selection for "in-depth" filtration, low headloss underdrains, backwash systems, filter galleries with virtually no valves and piping, and the effects of secondary effluent quality on system functions. Additional design parameters include control techniques, hydraulics and post filtration requirements.

Pre-filtration Chemical Treatment

Perhaps the single most important reason for filtering secondary effluent which discharges to receiving water, subject to reasonably constant human contact, is to significantly aid in the reduction of bacteria and virus concentrations. Investigations performed to date have generally shown that direct filtration without any form of chemical pretreatment results in poor removal of viruses (References 2 & 3). The same investigations show that proper development of chemical floc prior to filtration results in an improved removal efficiency. It would appear that virus removal by sand media, at least, is poor without chemical pretreatment; perhaps this can be attributed to inefficient attachment of these particles to the filter media surfaces.

The purpose of the chemical pretreatment at both the Whittier Narrows and the Burbank effluent filtration facilities is to coagulate the suspended and colloidal solids and flocculate them such that they can be filtered. The chemicals used at the plants are polyelectrolytes at Burbank and polyelectrolytes and alum at Whittier Narrows.

Pilot studies (Reference 4) performed specifically for determining the chemical doses required for proper operation of the Whittier Narrows effluent filters (and others) resulted in doses of 5-7 mg/l alum and 0.01 mg/l anionic polymer. (Cationic polymers were also investigated and found to have comparable results. Availability of the anionic polymer made it the preferred polymer for testing.) Flexibility must be designed into the plant chemical feed systems owing to scale factors, potential shock load situations, and variability of wastewater characterization. This flexibility is not only to allow for wide variability in dosages but to allow for the use of both dry and liquid polymers. Systems with such flexibility are readily able to optimize chemical usage as well as to allow for changes in the type of chemical used should economic or other considerations change during the system design life. The dosage ranges are 2:1 on the alum and a polyelectrolyte range of 50:1. Both the alum and the polymers are mixed at medium mixing intensities (about 7 ft-lb/sec/cu ft) and injected just prior to the filters. The alum is fed first and allowed approximately five minutes for the formation of the floc; the polymer is injected and mixed with the flocculating wastewater just prior to the filters.

FILTER DESIGN

With adequate land area and site conditions, it is generally felt that the more commonly used gravity downflow filter offers the most cost-effective design. It is energy conservative, flexible, easy to operate and allows a simple, non-complex design in terms of mechanical equipment required. This section will discuss filter flow proportioning and loading, media designs used, low headloss underdrains, and galleries with virtually no piping or mechanical equipment required. The media discussion addresses the pilot testing and results which led to the selection of filter media at the Whittier Narrows effluent filters.

Flow Proportioning and Filter Loading

The filters at Whittier Narrows are all equally loaded at an average flow rate of 3.4 gpm/sq. ft. with a peak loading of 5.4 gpm/sq. ft. (The latter loading will occur at peak plant flow with one filter backwashing.) The flow to each of the six filters is the same and is controlled by free-flowing sharp crested weirs which receive their feedwater from a tranquil influent channel. The operating water surface over the media cannot fall to below the top of the packed media as it is controlled by the backwater from the filtered effluent wet well. The water surface during filtration is maintained within three inches of a preset elevation, twelve feet above the media, by a pneumatically operated modulating butterfly valve which gradually opens as the water surface in the filter tends to rise.

Filter run time is regulated by a timer or a high level override and is further discussed in a later section. Timer control is preferable to assure that the filters do not reach a point when either several or all require backwashing at the same time. For filters set to backwash at a "high head loss" level only, it can be shown that the need for several filters to backwash at the same time will in fact occur if there are any differences in the operating characteristics of the filters at all. This is an extremely important consideration if the design relies on flow from other filters for backwash water. (See Backwash Systems for detailed discussion.)

Media Design

Filtering secondary effluents is decidedly different from filtering potable water supplies. In water filtration plants, the floc to be filtered is a chemical precipitate and has a relatively weak structure; hence, it is desirable to use a fine media, thus "straining" the solids from the flow. This results in rapidly increasing headlosses and a diminished allowable loading on the filter. On the other hand, biological floc in a wastewater effluent is stronger and can be driven into a filter media with larger voids thus increasing the length of the filter runs without increasing headlosses beyond a reasonable limit or reaching breakthrough. Dual media filter designs exhibit these characteristics and allow for greater solids loadings, and hence, longer and more economical filter runs. The nomenclature given this type of filtration is "In-Depth" filtration; the filtration occurs throughout most of the depth of the media rather than merely straining at the filter surface.

Pilot studies were performed on four dual media designs for the Whittier Narrows Plant. (Reference 4.) Tests were performed on secondary effluent from the plant itself. With the exception of one, each of the media was a design with full scale operating experience. Five separate runs were made over a several month period and each of the pilot filter runs was operated over a 14 to 22 hour period. The primary criteria for filter evaluation was effluent turbidity and headloss. All of the designs produced comparable turbidities, yielding reductions of 65 to 90 percent with composite influent turbidities ranging from 1.2 to 6.5 FTU. It should be noted that the filtered effluent turbidities remained consistently between 1.0 and 2.0 almost without regard to fluctuations in influent turbidity. The times the filter effluent turbidities substantially deviated from the above values (before "breakthrough") were during periods of extremely high values of turbidity and suspended solids in the secondary effluent. In the headloss category, two of the four media designs exhibited very similar performance and both were five to ten-fold superior when compared to the other two designs. It is interesting to note, however, that when the same pilot testing program was performed at another secondary wastewater treatment plant, one of the filter media designs that performed poorly at both Whittier Narrows and a third treatment plant, showed itself superior to the two media designs that showed best performance at the latter two plants. Thus, suffice it to say that pilot testing for each wastewater is of extreme value to the designing engineer and should be performed if at all possible before finalizing the filtration system characteristics.

Low Headloss Underdrains

Headloss in underdrain systems is only of importance during the backwash as most, if not all, underdrains impose insignificant headloss at standard filtration surface loading rates.

Conventionally, underdrains are designed to impose a controlling loss of head at the interface between the filter media and the underdrainage system. Recent experience (Reference 5) has shown that with proper underdrains design this "imposed" headloss is not required. Such "low headloss" designs are being used in many successful filtration operations and such a design is used at the Whittier Narrows plant.

The design used is a 12-inch wide V-shaped precast reinforced concrete underdrain. Small 1/2-inch flow orifices are located in the underdrain walls. At a backwash rate of 20 gpm/ft^2 the headloss through the underdrains is one foot as compared to a conventional underdrain with loss in excess of three to four feet. This design not only provides a cost effective support for the gravel and media, but assures uniform distribution of backwash water and backwash air. During the backwash cycle, air is introduced by PVC laterals from a central air header; located in the underdrain plenum; the laterals are located under alternate V-blocks. When the combination air-water backwash is operating, the water and air are able to continuously and uniformly discharge from the orifices and thoroughly mix before arriving at the media-support gravel interface.

Installation of the precast V-block underdrains is simple. A sling may be used to place the blocks in position on their support beams. To insure proper fit, a gap of 1-inch is designed into the spacing, the gap is grouted to within 1/4-inch of the orifice invert. To date, the performance of the V-block underdrains in the water treatment plants such as the 150 MGD Robert A. Skinner Filtration Plant is entirely satisfactory. Another similar design, the M-block underdrain used at the Reynosa, Mexico Water Treatment Plant, performs in the same manner as the V-block and its performance is also satisfactory (Reference 6).

Backwash Systems

Given the selection of "In-Depth" filtration as perhaps the most effective form of filtering a biologically treated effluent, special consideration should be given to the backwash system as the imbedded particulate matter must be thoroughly and efficiently removed. In addition, if future conversion to activated carbon media is considered, as it is at the Whittier Narrows facilities, deep penetrating particulate matter or biological growths will require that the backwash and scour systems be capable of being effective throughout the entire filter bed.

The backwash concept consists of water moving counter to the flow direction during filtration to pick up the imbedded solids and "lift" them to the wastewater trough. To effectively accomplish this, two primary conditions are required. First, the filter media must be fluidized or expanded to allow the particulate matter to escape. Fluidizing the bed will allow perched solids to rise up with the upflow current and out of the filter via the washwater troughs. However, to free the media from attached and adsorbed solids, a scouring system must be introduced with enough energy to cause the media to collide and scour and shake the particulate matter loose while not injuring the media. Merely fluidizing the media does very little if any scrubbing as in general, collisions between grains in a fluidized bed are nearly non-existent (Reference 7).

For single media sand filters where foreign particulates are removed in the upper few inches of the media, various types of surface agitation systems efficiently provide the desired scrubbing action. With "In-Depth" filtration, where virtually the entire depth of the upper anthracite layer and the upper few inches of the sand layer are used to remove and hold the solids, effective scrubbing of the media deep in the bed cannot be attained solely with a surface wash system. A subsurface system to create sufficient and uniform turbulence is required.

To provide proper agitation, two primary methods can be used: (1) the introduction of air into the bottom of the filter (air-water backwash) or, (2) by high pressure water scrubbers. Both of these systems are presently in use and each represents advantages in certain applications. The following discussion compares the two systems (Reference 8):

(1) For filtering systems utilizing the "In-Depth" concept, the filter media must be scrubbed throughout its entire depth during backwash to remove solids, slimes, and potential for mud ball formation.

(2) To get adequate total bed cleansing with a water scrubbing system, rotary arms or fixed nozzles should be installed at several levels throughout the depth of the expanded bed.

(3) At Whittier Narrows, the three foot deep dual media filter bed eventually may be replaced by activated carbon, the carbon depth is proposed to be six feet deep. Because of this depth a three foot gravel spacer is required to raise the initial dual media filter to a level for efficient removal of rising particulate during fluidization. With this gravel support bed for the inert filter media and without an air backwash, there might be a tendency for anaerobic conditions and slime growths to develop in the lower portions of the media unless the operation includes a continuous application of chlorine to the filter. With the uniformly applied air-water backwash this would be eliminated. With future conversion to a six-foot deep

activated carbon bed, there will be increased need for in-depth cleansing of the bed. Only air scour can effectively and uniformly clean the deeper portions of the carbon bed.

(4) With an air scour system, there is no requirement to remove piping and fixtures to a fixed subsurface mechanical scour when replacement of the dual media is contemplated.

(5) The attrition loss of activated carbon due to abrasion is expected to be the same with either the use of an air-water backwash or the subsurface water scrubbers. The amount of abrasion anticipated is expected to be generally proportional to the energy used to scrub the bed, regardless of whether the energy is created from air or high pressure water.

(6) With air-water backwash, the energy input to the filter bed is essentially uniform over the entire bed and moves in the direction which supports the transport of solids, whereas, with a water scrubbing system, the energy is highest near the nozzles and decreases as the distance from the nozzles increases. Thus, with a given level of scrubbing energy input to the filter during backwash, there may be a tendency for more abrasion between carbon particles near the nozzles in a water scrubbing system. In addition, the nozzle energy is generally dissipated transverse to the flow and thus primarily mixes without aiding the desired upward materials transport.

In view of the greater problems associated with wastewater filtering, the need for very efficient cleaning of the filters, the potential for future conversion to deep bed activated carbon, and the uniform control and scour intensity of air backwash systems, and the prevention of anaerobic conditions developing in the filter bed due to the inherent biological activity level in secondary effluent, air-backwash system seems to offer the best system in terms of assuring efficient media cleaning. Both Whittier Narrows and the Burbank effluent filters use the air-water backwash system.

Operationally, Burbank and Whittier Narrows differ slightly. Burbank scours with air alone and then follows with the water wash to fluidize the bed and remove the loosened particulate. Degremont (Reference 9) recommends using an air scour with a minimal non-fluidizing backwash flow. Apparently, some concern exists regarding the downward movement of mud balls and other solids in deep bed filters. Air scour alone may create some downward movement which may result in lodging impurities low in the deep bed filters and they might not be removed during fluidization.

At Whittier Narrows, the backwash cycle begins by closing the influent valve, and draining the filter to within a foot or two of the top of the media. The water wash control valves then slowly modulate from the closed position and accelerate the backwash flow-rate to about 4 gpm/sq. ft. at which time the air scour will start. The air scour will operate for a preset time ranging from two to five minutes. Concern is expressed in the literature regarding the loss of filter media during backwash in dual media filters if air and water are used simultaneously; at Whittier Narrows the simultaneous air and low flow water backwash will occur with the water surface below the washwater troughs and thus no media can be lost. When air scour is terminated, the backwash rate will accelerate to between 15 and 20 gpm/sq. ft. to remove the loosened particulate from the fluidized bed. After sufficient backwash, the backwash flow is linearly reduced to 4 gpm/sq. ft. over a preset adjustable time period to allow proper restratification of the anthracite and sand media. At the 4 gpm/sq. ft. flow rate the valve is rapidly shut off.

Backwash water, laden with solids removed from the filters, can be disposed of and treated in several ways. In wastewater treatment, the flow is usually returned to a point in the treatment train. At Burbank, the spent backwash water is returned to the primary clarifiers. At Whittier Narrows, the spent backwash water discharges to a holding tank and from the holding tank it is pumped at a uniform rate of about 900 gpd/sq. ft. to a clarifier. The solids settle and the clarified effluent discharges prior to the chemical pretreatment basin and undergoes filtration once again. The backwash sludge at Whittier Narrows is collected by chain and flight collectors and then discharged to a large trunk sewer for transportation and treatment by a downstream treatment plant, as the Whittier Narrows plant has no final solids treatment and disposal facilities.

NEW DEVELOPMENTS

Complex pipe galleys seem to be automatically associated with filtration facilities, however, such an extravagant design is not required. Lee Streicher[1] an accomplished and innovative engineer in the area of filtration and water quality, developed a filter design which:

(a) eliminates the need for a filter pipe gallery and its associated piping, valves, and controllers;

(b) eliminates the possibility of negative head developing in the filter which causes air-binding and high velocity channeling resulting in a decrease in effluent quality;

(c) requires only two flap gates for operation of each unit; and

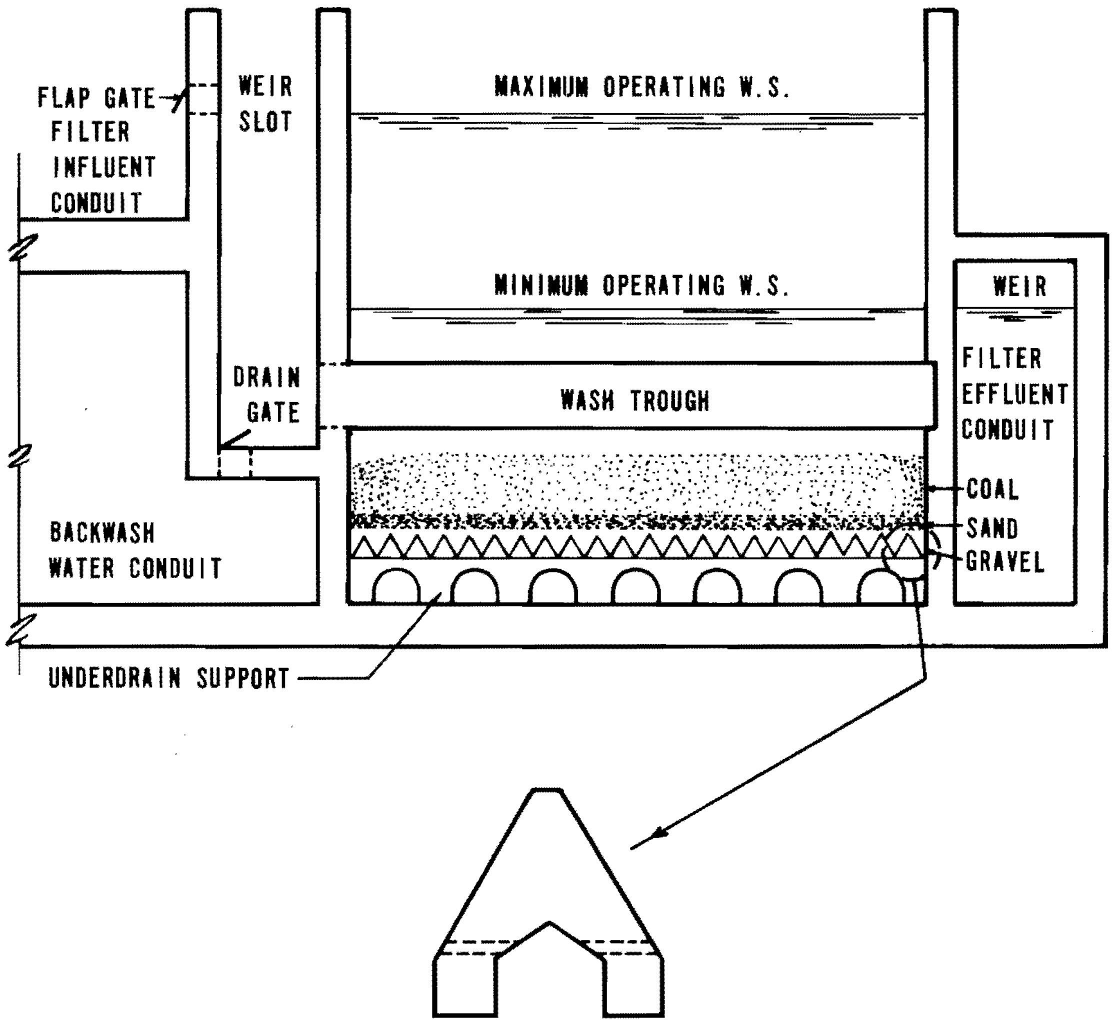
(d) eliminates the need for a washwater storage tank if the plant flow is sufficient to maintain the backwash flow rate of a single unit.

The Streicher concept (See Figure 1) currently being used in the design of a water treatment plant for the City of Oceanside, California (Reference 10). The filters are constant rate type with the feed-rate controlled by influent weirs with an automatic flap gate cover which closes to isolate the filter during backwash. The filtered water flows through the low velocity underdrain plenum and into the filtered effluent conduit. The level in this conduit is controlled by a sharp-crested weir which fixes the minimum water level that can occur within the filter. The

[1]Lee Streicher: Chief Water Treatment Engineer, Engineering-Science, Inc.; formerly with Metropolitan Water District of Southern California (36 years).

FIGURE 1

LONGITUDINAL SECTION THROUGH STREICHER CONSTANT RATE FILTER



SECTION THROUGH PRECAST CONCRETE UNDERDRAINS overflow elevation is adjustable and set above the top of the media.

The backwash water is fed into the low velocity underdrain plenum from the filtered effluent channel. The backwash energy grade line is developed by virtue of the elevation difference between water surface and the filtered effluent channel and water surface in the filter while backwash water is entering the washwater troughs.

Experience at the Robert A. Skinner Water Filtration Plant in Los Angeles County (where the Streicher concept is being successfully used) shows that between 20 and 38 inches of head differential is required to achieve adequate backwash rates with complete media fluidization. Backwash water is supplied directly from the "on-line" filters; hence, no backwash water storage tank is required. (This, however, requires that the plant be large enough to produce enough filtered water to meet the backwash flow rate requirements, i.e. four to five filters minimum in addition to the backwashing filter.)

Upon thorough backwashing, the effluent flap gates close and the backwash flow gradually and smoothly diminishes, owing to the decreasing head differential, allowing the filter media to restratify and settle uniformly. At a preset filter water level the influent flap gate is opened and the filter returns to service. With this operation, the filter gradually accelerates to the design surface loading rate; this minimizes the initial filter loading shock and eliminates the need for a filter-to-waste period.

For the Oceanside, California, plant presently under design, it was found that there was a twenty percent savings in capital cost by using the Streicher filter concept when compared to a conventional "pipes and valves" type filter design. This does not recognize the additional cost savings in operations.

POST FILTRATION DISINFECTION

It is felt that a discussion considering effluent filtration and its effect on virus and bacteria removal must deal with the post filtration advantages obtained in the disinfection process. Here, the discussion is limited to disinfection by chlorination. Filtration prior to chlorination aids the disinfection operation in that it removes particulate matter which not only exerts a chlorine demand but harbors and protects bacteria and virus from the chlorine.

Perhaps the single most important operation in the entire treatment train for reducing bacteria and virus levels is effluent disinfection. Key factors in design of these facilities are simple and should produce good results if properly adhered to. The following design considerations are those used at the Whittier Narrows plant and are the result of an extensive design research and preparation for final design of the disinfection facilities.

In short, the chlorine should be injected and mixed with the filtered effluent as rapidly as possible and with considerable turbulence or mixing intensity (about 25 ft-lb/sec/cu ft) and long detention times. Recent literature suggests that contact times should be on the order of one and one-half hours at peak flow and the system should provide a plug flow reactor condition. This is achieved at Whittier Narrows by a five-pass contact tank with an overall length to width ratio of 74:1; the minimum recommended is 40:1 (Reference 11).

At the end of the chlorine contact tank is a sulfur dioxide dechlorination system consisting of a vertical baffle five-pass system. The sulfur dioxide contact basin should be covered to inhibit the growth of algae.

SUMMARY

There are several ways of improving filtration design to assure a cost-effective design and operation with minimal energy costs. Each of the design considerations discussed herein were applied to the design of filtration facilities which were being added to existing secondary treatment plants. The design criteria were obtained from actual operating experience or pilot testing.

It should be emphasized that pilot scale operations are warranted almost without exception. Areas of particular importance are media selection and operational control. Where it is unfeasible to pilot a particular unit operation, flexibility must be designed into the full scale plant. In effluent filter designs, the most important areas requiring flexibility are in the chemical feed systems and the media backwashing system. Each of these are very important to the success of the filter and the poor performance of one operation will negate the benefits of the other.

At this writing full scale plant operational data is limited on the effluent filters presently in operation. The complete pilot study results (Reference 2) will be published by April, 1977. Some preliminary results can be summarized as follows:

(1) Suspended solids removal through the filter ranges from 80 to 90 percent. For the most part in the activated sludge plant effluents considered suspended solids concentrations in the secondary effluents range from 5 mg/l to 15 mg/l. Filtered effluents suspended solids ranged from 2 to 3 mg/l. In addition, mild secondary upsets were not reflected to any large degree in the filter effluents. For example, secondary effluent SS concentrations of 30-50 mg/l were consistently reduced to less than 5 to 10 mg/l for the wastewaters tested.

(2) Turbidity removals are on the order of 65 to 90 percent. Strong correlations between secondary effluent turbidity and SS were observed. Filtered effluent turbidities ranged between 1 and 2 FTU with frequent ranging to 6 FTU.

(3) COD removals were on the order of 40 percent. All of the pilot testing work was performed on the basis of COD. The results of actual operation at the Burbank effluent filters show little effect of the filters on BOD_5 removal. The secondary effluent BOD_5 at Burbank is, however, very low ranging around 5 to 10 mg/l. In these ranges, most of the BOD_5 is soluble and thus little effect is exerted by the filters.

(4) Both extensive pilot testing and full scale filter operation show that the SS, turbidity, and BOD_5 removal efficiencies are directly proportional to the secondary

effluent concentrations. Additionally, with relatively consistent secondary activated sludge process results (i.e. mild to medium extremes in effluent concentration of the above constituents) the filtered effluent will negate the affect of these upsets thus eliminating the extreme final effluent values.

(5) Post filtration treatment with chlorination at long detention times in plug flow systems with good initial mixing is the single most important process in bacteria and virus removal. The success of this disinfection operation is dependent upon receiving a secondary effluent low in suspended solids which effluent filtration can provide. Pilot test findings (Reference 2) show that in a chemically treated, filtered secondary effluent, considerably more than 50 percent of the virus removal is accomplished by the chlorination facilities. These tests were performed in plug flow reactors, with chlorine doses of 10 mg/l, and detention time in excess of 2-1/2 hours.

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Why Settle for Only Secondary Treatment?

*William L. Berk**

With the continued demand for more rigid effluent requirements, it becomes necessary to investigate different approaches to meet these requirements. The Oxidation Ditch is one approach that should be investigated because of its effective performance capabilities, the simplicity of operation and also its cost effectiveness.

Region VII of the United States Environmental Protection Agency made a multiyear study of 225 operating secondary treatment plants (1) in their region using analytical results from multi-day, 24 hour influent and effluent composite samples. Of all the various processes studied, only the group of Oxidation Ditch plants averaged secondary treatment or better. This study included a rigorous 30 plant special winter operation study to further confirm their original findings.

The Oxidation Ditch process is a modified form of the activated sludge process, and may be classified in the complete mix, long term aeration group. The process is a fresh, unique and economical approach for the treatment of municipal and most industrial wastes. The Oxidation Ditch provides effective secondary treatment with BOD_5 and suspended solids reductions of 90 to 98 percent. In addition to this excellent secondary treatment, additional purification of the contaminated flow is also occurring. Nitrification of ammonia and organic nitrogen is virtually complete under normal operation. Phosphorus removal can be achieved with standard Oxidation Ditch plant design simply by the addition of chemicals upstream of the rotor. A full scale Oxidation Ditch plant study at Port Elgin, Ontario (2) showed 75 percent total phosphorus removal was achieved 80 percent of the time. Some denitrification is also present and with controlled and close operator control, substantial nitrogen removal is possible. The federal EPA demonstration grant for the Oxidation Ditch at Dawson, Minnesota (3) produced 51 percent nitrogen removal and it was felt with better operational control, this could have been increased to 80 percent.

The Institute of Public Health Engineering, TNO, Holland (4) in an effort to develop a highly efficient and cost effective treatment process, developed the Oxidation Ditch plant. The first full scale plant was installed at Voorschoten, Holland in 1954 (5). In 1963, the first Oxidation Ditch in the United States was installed at the Tektronix plant compound, Beaverton, Oregon. At present, there are well over 700 installations of this process in the United States and Canada.

PROCESS FLOW SHEET

There is normally no primary settling tank used in the Oxidation Ditch process, see Figure 1. Raw sewage passes directly through a bar screen to the ditch. The bar screen is necessary for the protection of mechanical equipment such as the rotors and return sludge pumps.

The Oxidation Ditch forms the aeration basin and here the raw sewage is mixed with the active microorganisms. The rotor is the aeration device that entrains the necessary

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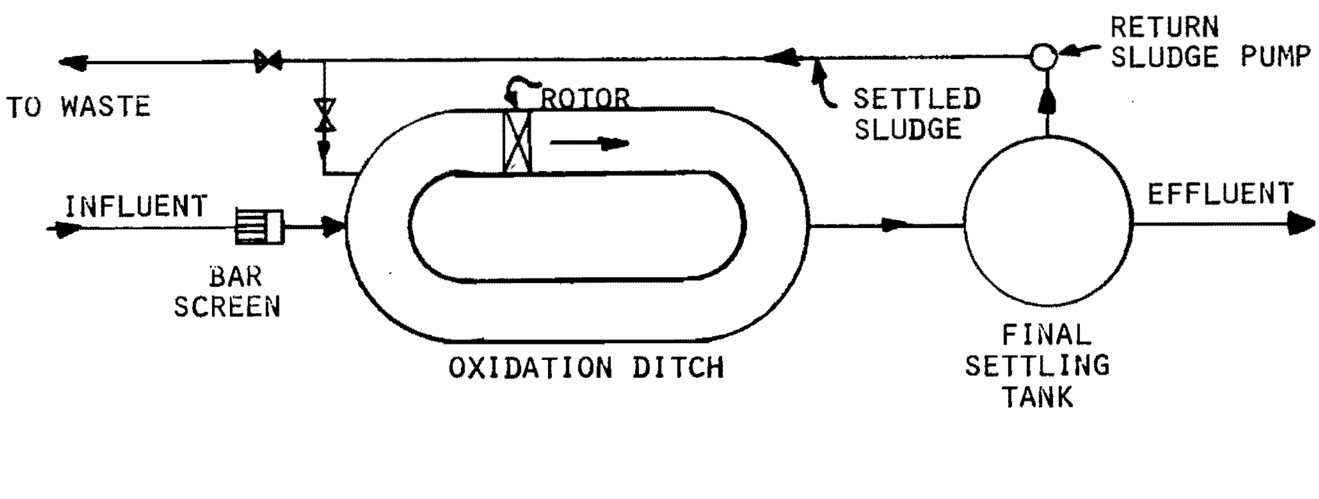

Figure 1. Oxidation Ditch Plant, line diagram.

oxygen into the liquid for microbial life and keeps the contents of the ditch mixed and moving to insure ready contact of all microorganisms with the incoming sewage or food supply. Velocity of the liquid in the ditch must be sufficient to prevent settling of solids. The ends of the ditch must be well rounded or baffled to prevent eddying or dead areas. The mixture of treated sewage and microorganisms, called mixed liquor, formed in the ditch, flows to the final clarifier for separation. Quiescent conditions in the clarifier afford separation of the solids, formed in the ditch, from the liquid. The clarified liquid passes over the effluent weir and may be discharged to the receiving stream or sent for further treatment such as disinfection, tertiary filtration, or post aeration.

The settled sludge is removed from the bottom of the clarifier by an air lift or pump and is returned to the ditch. All sludge formed by the process and settled in the clarifier is returned to the ditch. Scum which floats to the surface, in the race of the clarifier, is removed and also returned to the Oxidation Ditch.

The Oxidation Ditch is operated as a closed system and the net growth of volatile suspended solids will increase until it will be necessary to periodically remove some sludge from the process. Excess sludge formed by the process is stable and requires no further treatment. This excess, odor-free sludge may be applied direct to drying beds, sludge lagoons, hauled away for land disposal or further processed by mechanical dewatering equipment.

PROCESS CHARACTERISTICS

The Oxidation Ditch process, with its long term aeration basin, is designed to carry mixed liquor suspended solids concentrations of 3,000 to 8,000 mg/l. This provides a large active microbial mass within the system. Food to mircoorganism ratio is low, ranging from 0.03 to 0.1 lbs of BOD per day per pound of volatile suspended solids. This low food to microorganism ratio produces a system that can absorb shock loadings without upsetting the total operation. By carrying concentrations of this magnitude, the Oxidation Ditch can provide continual BOD and suspended solids removal of 90 to 98 percent. Virtually complete nitrification of ammonia and organic nitrogen is just an added bonus to this effective secondary treatment operation. The operation of the plant is relatively simple. It does not require continual manipulation to produce high quality effluents. The oxygenation capacity of the rotor is readily adjustable by manipulation of the adjustable weir installed at the ditch effluent which controls the liquid level and rotor immersion. Control of the rate of sludge return is possible by adjustment of a telescoping sludge valve or adjustment in the speed of the recirculation pump. There are no odor problems and there is no foam problem once the concentration of mixed liquor suspended solids reaches the minimum recommended operating level. When excess sludge accumulates in the process, it may be wasted directly for drying or disposal without additional treatment. As recorded in the Region VII report, (1) the Oxidation Ditch plants were least affected by the caliber of operator competency.

There are few components to the Oxidation Ditch plant and therefore it makes the operator's task much more simple. The rotor sits completely above the ditch level with the exception of the tip of the blades immersed in the liquid. The equipment is readily accessible for necessary greasing of bearings which is required weekly and the changing of oil in the reducer required on a semi-annual basis. Similar maintenance is also required on the drives for the final tank and the return sludge pumping equipment. There is no primary tank to maintain, aerobic or anaerobic sludge digestion equipment to service and maintain, or diffusers or spargers that must be pulled from the liquid and cleaned on a regular basis.

Another very important characteristic of the Oxidation Ditch plant is the economic operation and the economic first cost. Rotor Aerators are highly efficient mechanical surface aerators with a very large flexibility in actual operation. Minor adjustments in rotor immersion provide considerable variation in oxygen input with almost no change in the rotor's pumping capabilities. The actual power draw by the rotor is dependent upon the operating immersion of the rotor and not the nameplate rating of the drive assembly. The cost effectiveness of the Oxidation Ditch plant over other treatment processes is because of the earthen lined ditch or channel that is used for the aeration basin instead of a reinforced concrete structure. This economical first cost is one of the major reasons for the rapid growth in the employment of an Oxidation Ditch for waste treatment.

APPLICATION

The Oxidation Ditch may be used for treatment of any waste that is amenable to aerobic degredation. The basic design criteria recorded herein will be directed toward normal domestic waste. Modification of this criteria is required when handling various types of industrial wastes. The plant is normally sized based on the average daily design flow and the average BOD_5 waste strength. With normal operation, basic design will produce BOD and suspended solids removals of 90 to 98 percent and will provide complete nitrification.

The process is based on the theoretical destruction of all organic matter applied using an extended aeration period. Untreated wastewater containing organic matter that is amendable to biological degradation is attacked by the bacteria previously formed in the system. The food to microorganism ratio is low and will range from 0.03 to 0.1 lbs of BOD per day per pound of volatile suspended solids.

Most plants are proceeded by only a hand rake bar screen. The bar screen should have a clear opening of 1" and a drain rack must be provided at the top of the screen. Comminutors or mechanical bar screens can be provided. Normally the flow passes directly from the bar screen to the aeration chamber. Plants that will be handling wastes from combined sewers or wastes which contain large quantities of grit should be preceded by some type of grit chamber. No primary tank need be employed.

OXIDATION DITCH

The Oxidation Ditch forms the reactor or aeration tank. The initial ditch plants were generally an elongated oval, with sloping side walls and center island. This elongated oval may be straight, bent at one end, bent at both ends or circular. The prime criteria is that it does form a complete circuit. It's also possible to employ an elongated channel, either with sloping or vertical side walls, a center dividing wall and flow guide baffles at either end. The liquid depth employed in the ditch can vary between 3' and 5'-6" when you employ a Cage Rotor or a Mini-Magna Rotor and these liquid depths can be increased to 10' to 14' when the Magna Rotor is employed. When the Magna Rotor is used in liquid depths greater than 7', a horizontal baffle downstream of the rotor is required.

For domestic sewage, the volume of the ditch is sized based on a loading of 13.5 lbs of BOD per thousand cubic feet. The ditch volume for weak wastes may be as low as 8 lbs of BOD per thousand cubic feet and for the stronger wastes, the design loading may be increased to 15 lbs of BOD per thousand cubic feet. Regardless of the loading, the minimum detention time in the ditch should not be less than 18 hours. Loadings of up to 40 lbs of BOD per thousand cubic feet of ditch volume have been employed for strong industrial wastes. The actual loadings used for an industrial waste, dependent upon the strength of the waste, type of waste and the waste's amenability to aerobic degradation.

Normally the Oxidation Ditch has sloping side walls. The compacted earthen ditch is preferably lined with 4" to 6" of poured concrete or shot-crete. A variety of other construction materials such as asphalt, wood, preformed materials and clay have been used. The rotor moves the liquid in a horizontal plane and it is preferred to have the rotor sitting upstream of a straight section of a least 40' in length. Where center islands are used, they should be wide enough to provide a smooth flow around the bend. The width of the island should vary depending upon the width of the ditch at the liquid level. Figures normally used should be a 12' center island for ditch widths up to 13', 16' for ditch widths from 13' to 24' and larger islands where the ditch width is 25' greater.

Ditches with vertical side walls can be used and these can be furnished with a center island or a center dividing wall. Where a center dividing wall is used, additional consideration has to be given to flow guide baffles at either end of the ditch. These flow guide baffles should be off center in the direction of flow. Another method would be to form a bulb type of return bend at either end of the ditch.

The Oxidation Ditch aeration channel produces the most effective, controlled, complete mix aeration basin at present available. This controlled mixing basin insures the most efficient use of oxygen and aeration volume. The rotor accelerates the liquid until the entire ditch basin is moving at a near constant velocity around the ditch. The raw waste and return sludge, added just upstream of the rotor, are thoroughly mixed and distributed across the cross section at the rotor. There is a continuous addition of raw flow or new food supply to the mixed liquor concentration as it passes the rotor. There is no short circuiting around the rotor which provides the most effective utilization of rotor oxygenation.

ROTORS

Rotors are mechanical surface aerators which rotate in a plane horizontal with the liquid surface and are placed perpendicular across the aeration channel. Operation of the rotor carries out a twofold function of supplying the necessary propulsion and complete mixing of the ditch contents and inducing the necessary oxygen to support biological activity.

The length of rotor used for a given project should be the maximum length computed by either the velocity criteria or the required oxygenation capacity. In selecting the length of rotor, first calculate the length of rotor required to satisfy the velocity criteria. The ditch velocity is based on the propulsion capability of the rotor and the frictional resistance of the wetted perimeter of the ditch. Sufficient propulsion must be provided to produce a minimum velocity of approximately 1' per second so that all solids are maintained in suspension.

Lakeside manufactures three different rotors. Cage Rotors are 27-1/2" in diameter and are manufactured in 1' intervals from 3' through 12' in a single length. Mini-Magna Rotors are 28" in diameter and are available in 1' increments from 10' through 16' in a single length. Operation of these rotors is from 60 to 90 rpm and from 3" to 10" immersion.

To meet the needs of larger plant designs and also to provide greater O_2 input per foot of rotor for strong industrial wastes, the Magna Rotor was developed. Magna Rotors are 42" in diameter and are manufactured in 1' increments from 6' through 30' in a single length. Operation of the Magna Rotor is from 50 to 72 rpm and from 4" to 15" immersion.

A rotor assembly can have multiple rotor lengths, but these must be supported by intermediate bearings. All rotor lengths should be constructed with bearings supporting both ends of each rotor length. Rotors may be mounted directly on concrete pads, on vertical piers, side wall mounted, suspended from an overhead support structure or mounted on floats. Side wall mounted rotor assemblies are preferred in that this mounting provides a clean and dry area, easily accessible for lubrication, maintenance and repair, if needed, of the drive assembly and outboard bearing. Reduced ditch cross sections at rotor mounts should not be used. This reduced cross sectional area produces increased velocities in the contriction at the rotor and reduces oxygenation capcity of the rotor.

Ditch velocity criteria is based on actual experience accumulated from operating plants. For plant designs with population equivalents less than 600 persons, using the Cage or Mini-Magna Rotor with a lined ditch, the ditch volume should not exceed 13,000 gallons per foot of rotor. For plant designs with population equivalents above 600 persons, using a Cage or Mini-Magna Rotor, with a lined ditch, the ditch volume should not exceed 16,000 gallons per foot of rotor. Where the Magna Rotor is used with a lined ditch, the ditch volume should not exceed 21,000 gallons per foot of rotor. In the few cases where unlined ditches are used, the above figures should be reduced.

OXYGENATION

To suitably apply mechanical surface aerators to various aeration processes and various wastes, it is essential that the oxygenation capacity and power requirements are known. It is impractical to test an aerator in each used water application, and it is therefore necessary to adapt some sort of standard for testing. With suitable formulation it is then possible to properly size the aeration units for a given project. The standard technique is to evaluate the aerator in a properly sized tank of de-oxygenated tap water and determine the rate of reoxygenation of this water by the aerator. Test results are then converted to standard conditions of 20°C., 76 mm mercury pressure and 0 dissolved oxygen. Net power requirements are also recorded during each run.

Several series of tests were conducted under the direction of Dr. E.R. Baumann of Iowa State University, Ames, Iowa. The testing of the Cage Rotor was done at Iowa State University and the procedure adopted duly recorded (6). The work on the Magna Rotor, under the direction of Dr. E.R. Baumann, was done in a full scale Oxidation Ditch plant at Somonauk, Illinois. The end product of this work was the development of the family of Magna Rotor curves for oxygenation, recorded in Figure 2. and power requirements, recorded in Figure 3, at various rpm's and immersions. The curves give the Magna Rotor oxygenation and power requirements in tap water under standard conditions. A rather formidable looking equation, Figure 4, has been developed to convert oxygen input under standard conditions to oxygen input required for various forms of activated sludge and different wastes that are amenable to aerobic degradation.

Suitable figures have been developed for the alpha and beta factors for domestic waste and these can be substituted into equation 2. By solving equation 2 for N_0 in terms of N, you arrive at what can be called a conversion factor. The normal conversion factor, for domestic waste, for Oxidation Ditch application is 2.35. Adjustments of this conversion factor must be considered for industrial wastes. A correction must also be taken into account for water pollution control facilities that are located at elevations greater than 2000 feet. As an example, the corrected conversion factor for a plant site at 4500' elevation would be 2.92 in lieu of the 2.35. Correction for elevation is accomplished by utilization of equation 1.

With the conversion factor, the BOD can be converted to pounds of oxygen required at standard conditions. This is then divided by the total length of rotor arrived at by the velocity criteria to come up with the pounds of O_2 required per hour per foot of rotor. Under normal conditions we would enter the oxygenation curve at 72 rpm and determine the proper immersion required to give us the necessary oxygenation. The flexibility of the oxygenation capacity of the Magna Rotor is noted from reviewing

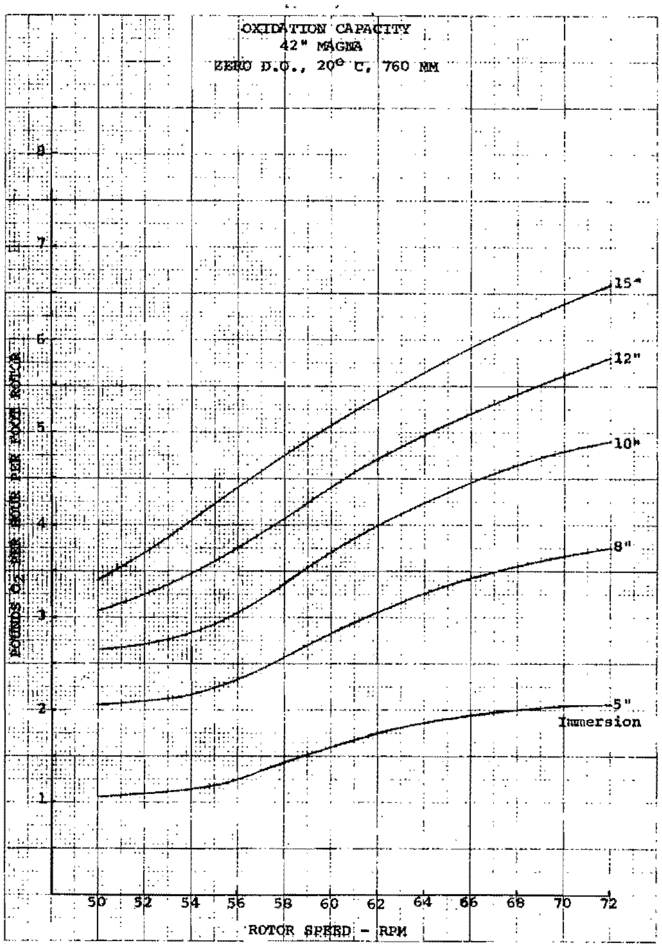

Figure 2.

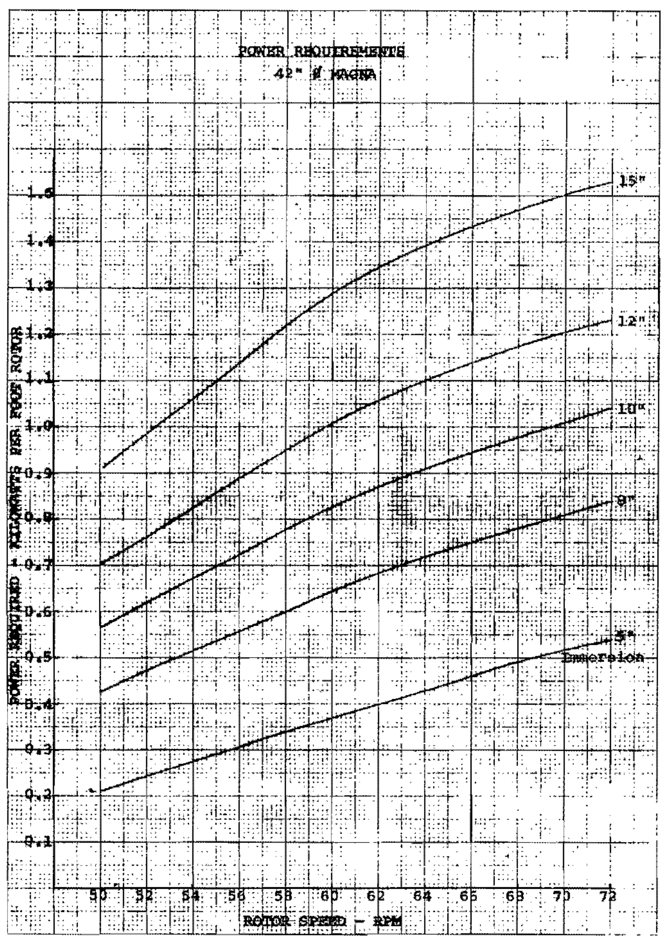

Figure 3.

Correction for Elevation

$$C_{sw} = \frac{A_p \times C_s}{A_o} \quad (1)$$

Where:

C_{sw} = Saturation value of oxygen in waste at operating temperature and plant elevation.

C_s = Saturation value of oxygen in waste at operating temperature.

A_p = Atmospheric pressure at plant elevation.

A_o = Atmospheric pressure at sea level.

Oxygenation Capacity

$$N_o = \frac{1.5 \times N}{\alpha\left(\frac{\beta \times C_{sw} - C_L}{C_{st}}\right) \times \left[1.024^{(T-20)}\right]} \quad (2)$$

Where:

N_o = lbs. O_2/day transferred to water at zero D.O. and 20° C.

α = Oxygen transfer ratio.

β = Ratio saturation of waste to saturation of water.

C_{st} = Saturation value of oxygen used in test operation.

C_L = Operation D.O. level.

T = Temperature of waste degrees Centigrade.

1.5 = Conversion from 5 day BOD to ultimate BOD.

N = lbs. of O_2/day transferred to the waste mixture.

Figure 4.

Figure 2. At 72 rpm and 15" immersion, we have an O_2 input of 6.60 lbs of O_2 per foot of rotor and at 5" immersion, the unit is capable of 2.05 lbs of O_2 per hour per foot of rotor. This gives an increase in O_2 input of 3.22 times merely by operating the rotors through its full range of immersions. It should also be noted that additional variation in O_2 input is also possible by regulating the speed of the rotor. At 50 rpm and 5" immersion, the actual input of the rotor is 1.06 lbs of O_2 per hour per foot of rotor which gives a total difference between this and the maximum point of 6.23 times.

The actual power requirements depends upon the rotor immersion and not the nameplate rating of the driving motor. Once the design conditions have been selected for O_2 input, you can go to Figure 3 and obtain the kilowatts per foot of rotor at the selected rpm and immersion. The required motor size is calculated as follows:

$$HP = \frac{\text{Length of Rotor} \times \text{Kilowatts per Foot} \times 1.34}{\text{Drive and Reducer efficiency}} \quad (4)$$

Efficiency of 0.95 is used, figuring 4 percent loss in the double reduction helical gear reducer and a 1 percent loss in the belt drive. For domestic waste, a 1-1/2" increase in rotor immersion above the design immersion is used in selecting the actual motor size. This is to afford some variation in the liquid level of the aeration basin to handle the fluctuation of flows coming to the plant. A 2" increase in immersion above design is required when you're considering industrial waste.

Good operation of the process depends on the rotor supplying the proper oxygenation to the waste. For the best operation, a D.O. concentration of 0.5 mg/l should be registered just upstream of the rotor. Over-oxygenation wastes power and excessive D.O. levels can form a pinpoint floc which settles poorly in the final tank and allows excessive loss of solids over the effluent weir. Various types of handwheel operated weir assemblies are available to regulate the rotor oxygenation by adjusting the rotor immersion. Preferred design is to employ weir lengths that will provide variations in liquid crests over the weir, between maximum and minimum flows, of something less than 1".

FINAL TANK

The Oxidation Ditch plant operates as a closed system in that all solids formed by the process should be retained within the process. Operation of the process must be such that a suitable size flocculated solid is formed that can be separated from the liquid when it is subjected to the near quiescent volume in the final settling tank.

The final settling tank is sized based on hydraulic loadings using the average daily flow. The tank should have, based on average design flow, a surface settling rate not exceeding 600 gallons per square foot per day and a detention time of at least 3 hours. Special consideration should be given to plants receiving their total daily flow in less than 16 hours by sizing the final tanks based on an hourly rated flow. Where excessively large pumps lift directly to the ditch or final tank or the plant receives extremely high peak flows, consideration should also be given to modifying the final tank design.

The Oxidation Ditch can provide efficient aeration, excellent mixing, sufficient velocity and good flocculation, but if you do not provide an efficient settling basin, excessive solids will be lost over the effluent weir. A small settling tank or a suitably sized tank with an inefficient settling mechanism can materially hamper the overall effectiveness of the process. The peripheral feed Spiraflo Clarifier has proven to be the most effective mechanism for this process. The construction of the peripheral feed clarifier reduces short-circuiting within the basin. All flow must move into the race area, move around the race and down and then pass underneath the bottom of the skirt and then flow up through the sludge blanket. Many of the fine colloidals, normally lost from other types of clarifiers, are retained within the sludge blanket as the flow rises through the sludge blanket.

SLUDGE

All of the activated sludge formed by the process and settled in the clarifier is returned to the ditch. Grease and floating solids retained in the race of the clarifier are also collected and returned to the ditch. The return sludge pump or air lift for domestic waste should be sized to handle a minimum of 100 percent of the average design flow. For strong industrial waste, the rate of sludge returned should be somewhat greater.

Adjusting the rate of sludge returned is a means of regulating plant operation. Continuous return is preferred over intermittent operation. Intermittent operation of a pump taking its suction directly from the bottom of the clarifier is definitely not recommended. The rate of sludge return can be controlled by the use of telescoping sludge valves or pumps fitted with variable speed motors. For medium and large size plants, Screw Pumps are an excellent mechanism for return activated sludge.

The Oxidation Ditch process is operated as a closed system and with efficient operation, there will be a slow growth or increase in suspended solids. In that the process operates at the lower end of the endogenous respiration curve, the actual growth rate is low. The excess sludge drawn from the process has undergone complete nitrification and additional treatment of this excess sludge is not required or actually possible. These excess solids may be wasted directly to, and will dry rapidly with little or no odor on open sludge drying beds, dried by mechanical means, or stored in holding tanks or in excedss sludge lagoons for later disposal. For normal climatic conditions, sludge drying beds may be sized based on one square foot per population equivalent.

OPERATION OF PLANTS

The theory, design and operation of the Oxidation Ditch process make it possible for this process to provide effective secondary treatment with a minimum operational adjustment or upset by shock loadings. Operation with a highly efficient rotor aerator keeps power costs to a minimum and complete nitrification is provided as a bonus. The proof of this performance can be observed by reviewing actual operating data from several plants.

Table 1 of the Appendix lists operating data collected by Region VII of the Federal EPA (1) covering three of the Oxidation Ditch plants included in their study. The Data includes both summer and winter operation.

The Nixa, Missouri, plant was designed for 0.412 MGD flow and 204 mg/l BOD with operation starting in 1971. The current average daily flow rate is reported to be 0.159 MGD. Seymour, Missouri, was designed for 0.255 MGD flow and 204 mg/l BOD with operation starting in 1973. Current average daily flow is reported to be 0.225 MGD. Battle Creek, Nebraska, was designed for 0.21 MGD flow and 215 mg/l BOD and operation of this plant began in 1968. This plant is fully loaded with current average daily flow at the designed figure of 0.21 MGD.

All three of these plants were designed based on a ditch loading of 13.5 lbs of BOD per thousand cubic feet. This data illustrates operation of partly loaded to fully loaded installations. BOD and suspended solids removals more than meet effluent requirements for secondary treatment regardless of the ditch loading. It should be noted the summer data on the Battle Creek, Nebraska, plant was taken with the return sludge pump being closed and inoperative. The data also shows that complete nitrification is possible at various loadings and during summer and winter operation. All three plants used sludge drying beds for their excess sludge and this sludge is pumped directly from the final tank to the sludge drying beds with little or no apparent odor and with no additional treatment.

Data on the Berthoud, Colorado Oxidation Ditch plant appears in Table 2 of the Appendix. This plant consists of a single ditch fitted with four (4) 11'-0" Cage Rotors and a 55'-0" diameter final settling tank. The plant sits at about 5000' elevation and is in the northern part of Colorado and does experience considerable cold weather operation. Actual operation of this plant started in 1973. The plant design was 0.9 MGD flow and 163 mg/l BOD and a ditch loading of 13.5 lbs BOD per thousand cubic feet. For 1975 this plant produced an average BOD reduction of 96.5 percent and a suspended solids reduction of 95.4 percent. It should be noted that the average BOD and suspended solids in the effluent were less than 5 mg/l.

West Liberty, Ohio operating data is shown in Table 3 of the Appendix. This Oxidation Ditch plant consists of dual ditches each fitted with a 16' long Cage Rotor and dual 24'-0" diameter final tanks. Operation of this plant commenced late 1972. Plant design was for a flow of 0.5 MGD and a strength of 122 mg/l BOD or 510 lbs of BOD per day. This plant was designed to handle a very weak waste. Operating data for the year 1974 shows that the flows to the plant varied considerably depending upon the amount of rainfall. Actual operation was at loadings even less than were originally set forth in the design. Based on the operating data, the average loading to this ditch was 1.27 lbs of BOD per thousand cubic feet. Normally there is concern with very weak wastes that it's not possible to get the high reductions expected from the stronger waste. Even with this very weak waste, the plant provided average BOD and suspended solids reductions of 97 percent.

The Oxidation Ditch plant at Somerset, Ohio consists also of dual ditches and dual final tanks. This plant was put into operation in 1965. Operating data was collected by composite samples under a U.S. demonstration grant to Ohio University, Athens, Ohio under the direction of H.M. Kaneshige. Some of the data is shown in Table 4 of the Appendix illustrating operation over a 9 month period which includes operation right through the winter. The total plant design is for a flow of 0.287 MGD and a strength of 298 mg/l or 715 lbs of BOD per day. Each ditch has a volume of 28,600 cubic feet with a design loading of 12.5 lbs of BOD per thousand cubic feet and is equipped with two (2) 8' long Cage Rotor assemblies. There are also two (2) 20' diameter final settling tanks, each fitted with return sludge air lifts.

During the test period, only one ditch was being operated and the data shown in Table 4 represents a fully loaded operation. The results show the ditch carried a mixed liquor solids concentration ranging from 5,228 to 7,812 mg/l. The percentage of volatile solids ranged from 50 to 70 percent. Nitrogenous reduction shows that almost 100 percent of the ammonia was converted to nitrates. No difference is shown in this nitrification between the summer and the winter operation. The table shows an average BOD reduction of 97.1 percent and a reduction of suspended solids of 95 percent. This plant is doing an exceedingly excellent job.

Oxidation Ditch plants are in operation on domestic wastes for populations as low as 150 persons and as high as 33,000 persons. At present, there's an Oxidation Ditch under construction to handle 8 MGD and there are several other large plants in the design stage above this 8 MGD figure. The interest in and use of the Oxidation Ditch has expanded since the original introduction here in the States. The reasons are that the Oxidation Ditch does provide effective secondary treatment with BOD_5 and suspended solids reductions of 90 to 98 percent and nitrification of ammonia and organic nitrogen is virtually complete in summer and winter operation. The Oxidation Ditch will perform effectively with a minimal amount of personal operation and minimum power costs.

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5. P.G.H. Walker, Rotor Aeration of Oxidation Ditches, Water and Sewage Works, June 1962.
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Table 1. Operating data.

Nixa, Missouri	Avg. Previous EPA Eff. Data	% Rem.	Current Winter Eff. Performance	% Rem.
Flow MGD	0.058	--	0.029	--
BOD_5 mg/l	3.4	99	3	98.4
COD mg/l	15.2	98.3	21	96.0
NFS mg/l	1.3	99	<5	97.8
NH_3-N mg/l	0.92	98.2	<0.5	98.4
H_2O Temp. C.	17.4	--	8.3	--
Seymour, Missouri				
Flow MGD	0.196	--	0.107	--
BOD_5 mg/l	5.1	96	4.5	97.9
COD mg/l	15.2	94.8	48	87.5
NFS mg/l	1.8	98	11	94.7
NH_3-N mg/l	0.42	98.1	<0.5	98.4
H_2O Temp. C.	16.4	--	8.7	--
Battle Creek, NE.				
Flow MGD	--	--	0.21	--
BOD_5 mg/l	13	87.7	14.6	93.4
COD mg/l	19	86.4	54	90.1
NFS mg/l	8	96	22.3	93.4
NH_3-N mg/l	--	--	1.23	94.5
H_2O Temp. C.	--	--	4.7	--

Table 2. Town of Berthoud, Colorado, average monthly operating data for 1975.

		INFLUENT					EFFLUENT							
Month	Flow	Temp. (F)	pH	Settl. Solids	BOD	Susp. Solids	Temp. (F)	pH	Settl. Solids	D.O.	BOD	Susp. Solids	% Red BOD	% Red S.S.
JAN.	.403	51	7.6	7.0		147	44	7.3	0	3.7		6.0		95.9
FEB.	.406	49	7.7	6.5	173	130	44	7.3	0	3.6	4.5	1.2	97.3	98.0
MAR.	.412	48	7.6	4.5	171	110	46	7.2	0	3.9	4.8	3.0	97.2	94.4
ARPIL	.466	53	7.5	6.8	192	220	46	7.5	0	3.8	5.6	6.6	97.0	96.9
MAY	.548	54	7.1	5.0	206	130	58	7.4	0	3.2	9.3	6.3	95.4	95.1
JUNE	.714	59	6.9	4.3	104	93	61	7.4	0	1.1	6.6	5.1	93.6	94.5
JULY	.807	64		3.0	88	79	64		0	3.0	3.0	3.0	96.5	96.1
AUG.	.829	65	7.3	1.9	70	88	66	7.1	0	3.7	2.8	7.0	96.0	92.0
SEPT.	.670	62	7.4	4.3	105	104	63	7.1	0	3.5	3.2	2.5	96.4	97.5
OCT.	.552	60	7.1	4.4	137	137	57	6.9	0	3.4	4.3	5.0	96.8	96.2
NOV.	.488	58	7.1	6.6	158	113	50	6.9	0	3.7	4.2	3.8	97.3	96.6
DEC.	.464	56	7.2	4.5	153	150	47	6.8	0	3.4	3.3	11.3	97.8	91.5
AVG.	.563	57	7.3	4.9	142	125	54	7.2	0	3.5	4.3	5.1	96.5	95.4

Notes: Settleable Solids = ml/liter
Flow is in mgd.

Suspended Solids & BOD is mg/l.
Temperature is of liquid.

Table 3. West Liberty, Ohio, operating data, 1974.

WEST LIBERTY, OHIO OPERATING DATA 1974

FLOW mgd		SUSPENDED SOLIDS mg/l		5 - DAY BOD mg/l		DISSOLVED OXYGEN mg/l
MONTH	Average/Day	Raw	Final	Raw	Final	Final
January	.404	52	3	36	1.6	7.8
February	.379	52	5.5	55	2.4	7.2
March	.305	52	3.7	41	3.3	6.5
April	.324	33	1.7	32	1.8	6.3
May	.208	58	1.5	65	1.4	4.6
June	.124	142	2.9	67	1.2	4.8
July	.098	74	.6	56	1.0	4.5
August	.107	175	3	58	.8	4.5
September	.128	82	2.2	48	.9	5.1
October	.092	86	1.6	62	1.3	5.0
November	.079	70	1.0	60	.9	5.5
December	.115	62	2.1	53	1.5	6.0
Total	2.363	938	28.8	633	18.1	67.8
Average	.196	78	2.4	52	1.5	5.6

Table 4. Somerset, Ohio, Oxidation Ditch Plant.

DATE	BOD mg/l	BOD Eff. mg/l	BOD % Red	SUSPENDED SOLIDS Raw mg/l	SUSPENDED SOLIDS Eff. mg/l	SUSPENDED SOLIDS % Red	Mixed Liquor mg/l	Mixed Liquor % Vol.	NITROGEN mg/l NH3 Raw	NITROGEN mg/l NH3 Eff.	NITROGEN mg/l NO3 Eff.
6/20/66	189	6	96.8	288	11	96.3	6602	53	47	0.1	33
6/29/66	302	6	98.0	477	8	98.3	6618	54	38	0	34
7/11/66	221	14	93.6	286	10	96.5	6962	54	30	0.2	20
7/20/66	166	6	96.4	184	6	96.8	7138	52	19	0	29
7/31/66	234	4	98.3	335	14	95.9	6665	50	34	0.1	25
8/18/66	301	5	98.3	456	20	95.5	6602	50	35	0.1	40
9/7/66	266	3	98.8	406	19	95.3	7410	48	45	0.1	39
9/27/66	191	3	98.4	258	5	98.1	7005	50	41	0.1	26
10/11/66	232	3	98.8	350	8	97.7	7250	51	37	0	34
10/25/66	212	3	98.5	505	14	97.4	7575	53	45	0.1	45
11/15/66	156	5	96.8	219	16	92.7	7720	63	22	0.1	20
11/29/66	272	4	98.5	350	7	98.0	6382	59	48	0.1	37
12/6/66	165	4	97.7	222	21	90.6	6530	65	18	0.1	42
12/27/66	325	7	97.8	469	19	96.0	6570	63	42	0.7	35
1/10/67	272	7	97.4	400	20	95.0	6788	65	43	0.1	45
1/23/67	350	11	96.8	492	35	92.8	7812	69	34	0.1	52
2/7/67	132	7	94.7	218	15	93.2	7038	70	22	0	22
2/17/67	229	19	91.7	225	18	92.0	7038	67	30	0.1	23
3/24/67	162	8	95.2	204	19	90.7	5228	65	17	0.1	19
3/30/67	151	6	96.1	176	16	91.0	5442	65	17	0.1	22

Land Treatment for Roosevelt, Utah

*Gilbert R. Horrocks, P.E.**

INTRODUCTION

During the early 1970s oil exploration and development increased rapidly in the Uintah Basin, spreading from the Western Colorado oil fields into the Uintah and Duchesne Counties in Utah. Roosevelt City, located in Duchesne County near the Uintah County line, serves as a commercial center for the surrounding area and was heavily impacted by the influx of new people. Population increased from 3,372 in the 1970 census to an estimated 6,400 in early 1975.

The existing wastewater treatment facility is a facultative lagoon system at five stabilization ponds with a design capacity of approximately 4,000 people. This facility is seriously overloaded resulting in discharge of unsatisfactory effluents, odors and aesthetic problems to encroaching development.

Faced with the requirements to provide a high level of treatment to conform with the new state and federal requirements, as well as to increase its wastewater treatment capacity to accommodate future flows from a rapidly increasing population, Roosevelt City initiated a facilities planning study.

DESIGN CONDITIONS

Population

The most important economic factors affecting the future population of Roosevelt City is the oil industry, agriculture and tourism. The total potential yield of the Uintah Basin crude oil fields could amount to as much as 1 billion barrels, which would make this area one of the ten largest on-shore discoveries in the United States.

The single most important mineral resource of the Uintah Basin is the oil shale formations. Total oil in Uintah Basin shale is estimated to be between 900 and 1300 billion barrels. Some of the richer deposits are estimated to be more than 25 feet thick and to contain at least 25 gallons of crude oil per ton of oil-bearing rock. The total oil content of these richer shale deposits, which are located in the southern half of Uintah County and Duchesne County, has been estimated at 100 billion barrels.

Construction of the two prototype oil-shale facilities, to be located about 50 miles southeast of Roosevelt City, is expected to begin in 1978 and continue for about three years.

The following table gives the projected population for the Rooselvelt area assuming no significant shale oil developments during the design period, and projections assuming a moderate development of the shale-oil resources. It should be noted that a moderate shale-oil development would increase the 1995 design population of the study area by a factor of 2.67.

ROOSEVELT AREA POPULATION PROJECTIONS

Year	Projected Population: NO Oil-Shale Development	Projected Population: MODERATE Oil-Shale Development
1975	7,000	7,000
1980	8,500	10,000
1985	10,500	15,000
1990	11,000	24,000
1995	12,000	32,000

Because of the unpredictable nature of the Uintah Basin economics and the impact it will have on the study area economics, it is extremely difficult to make an accurate prediction of the population growth. For the purposes of design of new sewage treatment facilities, it is proposed that the population projections assuming no significant oil shale development be used. However, it is essential that any proposed treatment facility have the flexibility for expansion without major modifications.

Because of the strong possibility of the oil-shale development, and the long lifetime of sewer mains and interceptors, it was felt that some consideration for the projected population impact of these oil-shale developments must be taken into consideration. In the design of the new interceptor sewers, it was proposed that a design population of 22,000 be used (average of projections assuming no oil-shale industry and assuming full-scale development).

Wastewater Flow and Characteristics

The present average per capita sewage flow for Roosevelt City appears to be about 100 gallons per day, which is typical for communities of its size with metered water consumption. However, as the size of the city and the average family income increases and the life style becomes more complex, the average per capita sewage flow is likely to increase. Comparisons of the sewage treatment system alternatives and designs for the proposed treatment system, and the proposed interceptor system were based on a per capita sewage flow of 125 gallons per day. The design peak flow rate was assumed to be four times the design average flow rate.

The average daily organic loadings for the influent raw wastewater for Roosevelt City has been estimated

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using 0.17 pounds of BOD_5 per person per day (present and future). Under design conditions this amounts to 2,040 pounds of BOD_5 per day for the influent raw wastewater.

Water Quality and Wastewater Discharge Standards

Any proposed system must be capable of meeting the 1977 discharge requirements and must have provision for increasing the treatment efficiency to conform with the 1980 and the 1983 discharge requirements.

A summary of these discharge and water quality requirements is as follows:

Date for Compliance	Requirement	30 Day Limitation
June 30, 1977	State Interim Discharge Requirement	BOD_5 - 25 mg/l, 85% removal SS = 25 mg/l, 85% removal Fecal coliform = 200/100 ml
Jul 1, 1977	EPA Secondary Treatment	BOD_5 = 30 mg/l, 85% removal SS = 30 mg/l, 85% removal Fecal coliform = 200/100 ml
Jun 30, 1980	State Interim Discharge requirement	BOD_5 = 10 mg/l, 90% removal SS = 10 mg/l, 90% removal Fecal coliform = 20/100 ml
Jul 1, 1983	EPA Best Practicable Treatment	Nitrification*
Dec 31, 1983	State Class "C" Water Quality Standard	BOD_5 = 5 mg/l in receiving stream

*Possible exclusion for wastes with a temperature less than 20° C.

Receiving Waters

Dry Gulch Creek with an annual average flow of about 50 cfs receives the discharge from the present lagoon system. Water is diverted at various downstream points for irrigation purposes.

WASTEWATER TREATMENT SYSTEM ALTERNATIVES

Principal Constraints

The most important requirements for any new wastewater treatment system for Roosevelt City is that it have the capability for having its population capacity increased beyond its design capacity without major modifications; be capable of meeting the 1977 discharge requirements with provision for increasing the treatment efficiency to conform with the 1980 and 1983 discharge requirements; be compatible with area land use planning; minimize adverse environmental impacts; be politically and financially implementable.

Alternatives Considered

Many wastewater treatment alternatives were considered in a preliminary evaluation, and of these only six merited a detailed evaluation.

1. Complete Containment Lagoons
2. Aerated Lagoon with Winter Storage and Land Disposal
3. Facultative Lagoon with Winter Storage and Land Disposal
4. Mechanical-Biological Treatment Plant
5. Physical-Chemical Treatment Plant
6. No Action

All of these alternatives have the capability of meeting the constraints given above. A semi-detailed description of each of these is given below.

1. Complete Containment Lagoon. This facility would be designed such that the anticipated total inflow during the design year would be equal to the net evaporation plus the seepage from the ponds. The lagoon would consist of three or more cells, the number of ponds being dictated by economic and operation considerations, with the primary or receiving pond sized to keep the BOD_5 loading below the odor producing threshold. Increasing the number of ponds would also provide a smaller wind-fetch.

2. Aerated Lagoon with Winter Storage and Land Disposal. This system would consist of two aerated cells designed with a detention time and aeration capacity such as to achieve 85 percent reduction in the BOD of the influent raw wastewater. Effluent from the second aeration cell would be discharged into the winter storage pond. The winter storage pond would be designed to retain all of the wastewater for 180 days during the colder months. During the late spring, summer, and early fall the treated wastewater would be withdrawn from the winter storage pond, chlorinated, and spray irrigated at a land disposal site. The withdrawal rate would coincide with the cover crop evapotranspiration demand and the soil moisture content.

3. Facultative Lagoon with Winter Storage and Land Disposal. For this alternative, primary-secondary treatment would be accomplished by a series of shallow facultative ponds. Again the design average BOD reduction would be 85 percent before the wastewater was discharged into the winter storage pond. The operation of the chlorination facility and the spray irrigation system would be the same as described for Alternative 2.

4. Mechanical-Biological Treatment Plant. Of the mechanical-biological systems that are capable of meeting the effluent limitations required for new facilities, the system which appears to be the most economical and the least susceptible to operational problems for small installations would consist of a) comminutors; b) grit chambers; c) primary clarifiers; d) redwood media biological tower; e) high rate aeration cell; f) final clarifiers with sludge return to tower; g) final filter; h) chlorination facility; and i) sludge treatment and disposal facilities.

5. Physical-Chemical Treatment Plant. A physical-chemical plant would consist of: a) comminutors; b) grit chambers; c) primary clarifiers; d) rapid mix chamber with chemical feed system; e) flocculation; f) final clarifiers; g) final filter; h) chlorination facility; and i) sludge treatment and disposal facilities.

6. No Action. Under this alternative the existing treatment facility would be retained in operation and would not be modified, expanded or improved. The facility is presently overloaded. The effluent does not conform with the requirements soon to be enforced by the State of

Utah. After July 1, 1977, continued operation of the existing facility would be unlawful. As the population increases, as it has rapidly done so over the past few years, the efficiency of the existing system will undoubtedly worsen.

Another serious limitation of the No Action Alternative is the rapid development of the area around the existing lagoon system. To allow proper growth of the city and to alleviate the problems associated with the unsightly condition of the lagoon facility and the associated odors, there is a severe need for a relocation of the treatment facility.

The requirement by law that the Roosevelt City wastewater treatment system comply with the new state and federal regulations coupled with the serious need to provide an increased treatment capacity and to relocate the treatment facility, makes this alternative unviable.

Preliminary Cost Comparisons

The following table gives the preliminary cost estimates for each of the six treatment alternatives. It should be noted that alternatives four and five were much more costly than any of the other alternatives, both in terms of initial construction costs and equivalent total annual costs. On the basis of these preliminary cost estimates and the demonstrated unviability of Alternative 6, it was concluded that detailed comparisons would be required of Alternatives 1, 2, and 3 only. Alternatives 4 and 5 offer no significant advantages to justify the higher cost and increased operational complexity.

COMPARISON OF APPARENT THREE BEST WASTEWATER TREATMENT ALTERNATIVES

General

The three alternatives which are considered to be the apparent best alternatives are: 1) Complete Containment Lagoons, 2) Aerated Lagoon with Winter Storage and Land Disposal, 3) Facultative Lagoon with Winter Storage and Land Disposal. Each of these alternatives would be considered by the State of Utah, Bureau of Environmental Health to be complete containment system, since there would be no direct discharge to any surface water course.

Functional Comparisons

In terms of the alternatives ability to meet the necessary effluent discharge limitations, each of these alternatives would perform adequately since there would be no direct discharge in any case. However, more careful management would be required for Alternatives 2 and 3 in order for them to function properly.

Environmental Comparisons

Alternative 3 is considered the most environmentally sound of the three alternatives, with the following justifications: 1) where suitable land is obtainable, land disposal of domestic secondary effluent is perhaps the best method of tertiary treatment available and where land prices are low, it is often the least expensive. The biologically active soil-plant system provides almost

TREATMENT SYSTEM ALTERNATIVES
COST COMPARISONS

	Alternative	Estimated Total Capital Costs	EPA Share of Capital Costs	Local Share of Capital Costs	Net Annual O & M Costs		Effective Total Annual Cost (20 yrs, 7%)	Actual Total Annual Local Costs	
					Initial	Design		Initial	Design
1	Complete Containment Lagoons	1,433,000	948,750	484,250	5,250	5,450	140,770	33,480	33,680
2	Aerated Lagoon With Winter Storage and Land Disposal	1,233,000	858,750	374,250	13,750	11,990	129,390	33,570	33,810
3	Facultative Lagoon with Winter Storage and Land Disposal	1,343,000	941,250	401,750	2,480	760	128,530	25,900	24,180
4	Mechanical-Biological Plant	1,955,000	1,451,250	503,750	47,500	54,400	235,700	76,870	83,770
5	Physical-Chemical Plant	2,300,000	1,710,000	590,000	67,000	95,100	298,400	101,400	129,500
	No Action*	0	0	0	3,000	3,000	3,000	3,000	3,000

*Does not meet project objectives

complete renovation of wastewater, including removal of eutrophication inducing nutrients. The following table presents an estimate of the efficiency of land disposal for treatment of secondary effluent.

ESTIMATED EFFECTIVENESS OF LAND DISPOSAL TECHNIQUES

	Spray Irrigation	Overload Runoff	Rapid Infiltration Ponds
BOD	99	80	99
SS	99+	80	99
N	80-90	80	80
P	99	80	90
Heavy Metals	99	10-30	95
Organic Cpds.	99	50	90
Viruses and Bacteria	99+	90	99+
TDS			
Cations	75	30-50	50-75
Anions	0-50	0-10	30-50

Source: Eugene B. Welch and Demetrois E. Spyridakis, TREATMENT PROCESSES AND ENVIRONMENTAL IMPACTS OF LIQUID WASTE DISPOSAL ON SOIL, Fourth Environmental Engineers, Conference, Montana State University, February 1973.

A large portion of the applied water (as much as 50 percent under design conditions) would be renovated by the oil-plant system and returned to groundwater. Although an equal amount of the water might be returned to groundwater as seepage from a completed containment lagoon, the soil column through which the water would pass would not be biologically active and would not provide as effective renovation; 2) the nutrients in the wastewater would be utilized beneficially for the production of crops; 3) the proposed land disposal site contains no unique biological, geological, archaelogical, historical, or aesthetic values and thus the conversion of this land to productive, income producing agricultural land would appear to be environmentally beneficial; 4) the capacity of the system could be increased at less cost than for either of the other alternatives; 5) net energy and resources consumption would be less than for Alternative 2. As much as 650,000 killowatt-hours of power per year would be required for Alternative 2 under design conditions; 6) Alternative 3 is more aesthetically pleasing than Alternative 1 because of the smaller total pond area; 7) the net annual cost to Roosevelt City would be less for this alternative.

Economic Comparison

On an effective total annual cost basis, Alternative 3 appears to be the most economical. The net annual cost to Roosevelt City would be about 35 percent more for either Alternative 1 or 2 than for Alternative 3.

Selection of Apparent Best Alternative

Alternative 3 was selected as the apparent best alternative with the following justifications:

1. It appears to be the most environmentally sound alternative
2. It has the least effective total annual cost
3. Local annual cost would be considerably less than for Alternatives 1 and 2

DESIGN FACTORS

Successful design of a land application system depends upon the specific site available and proper application of the principals of environmental engineering, hydrology, soil science, agriculture, geology, and land use planning.

Wastewater Quality

Complete chemical analyses of the existing lagoon effluents were obtained to determine whether they contained any constituents which would affect permeability of the soil, be toxic to crops, excessive salinity or boron or excessive heavy metals which might have an adverse impact. No ions were found in excess of the recommended level for the maximum design application rate of 60 inches per year.

Site Selection

The proposed land application site was selected while considering such criteria as: 1) elevation differences between the site and the collection system to avoid having to pump; 2) sufficient area available for facultative lagoons, winter storage and irrigated land required; 3) land availability and cost; 4) environmental impact; 5) compatible with the area zoning and land use plan.

Local representation of not only Roosevelt City but the surrounding areas were consulted and their recommendations received regarding the treatment process and treatment site. The site selected best met the above criteria.

Soil Characteristics

The proposed irrigation site had previously been mapped by the local Soil Conservation Service soil scientist and was shown to be suitable for irrigation and the crops planned. Additional samples were taken in the root zone (top 5 feet) and to depths of 14 feet to check the permeability and water table level.

The results of the physical and chemical analysis of the soils investigation showed the soils to be deep, homogeneous, uniform, loamy sands with high permeabilities and low sensitivity to any of the common chemical constituents of the wastewater. There was no evidence of water table above 10 feet.

Crops

Field evaluation and consultation by a Plant/Soil Scientist from the Utah State University Extension

Service was utilized to make recommendations regarding suitable crops for the area. Productivity studies were made by the Plant/Soil Scientist.

A local advisory group composed of farmers from the area evaluated the proposed site, cropping plans and economics of the proposed irrigated farm operation. Both the Plant/Soil Scientist and the farmers advisory group felt that the farming operation could be economically successful and recommended alfalfa as the main crop with rotation of grain as necessary to reestablish the alfalfa.

State Division of Health Evaluation

The proposed site together with all criteria and data were reviewed by the Utah State Division of Health and found to be acceptable. It is not anticipated that there will be any contamination of underground water in the area and that sufficient buffer zone is available to prevent aerosols from escaping the irrigation area.

PROPOSED SYSTEM

Evaluation of the above design factors resulted in the following design (See Figure VI-A).

Raw wastewater will be transported to the treatment site through a 36 inch interceptor and pass through a Parshall flume (2 foot throat) into an inlet structure. The wastewater then passes through a 15 inch and 24 inch inlet pipe to the primary cell of the lagoon.

The State of Utah maximum allowed BOD_5 loading for any lagoon cell is 40 lbs per acre per day. This requirement dictates the size of the primary cell for the proposed lagoon, which has a total mid-depth water surface area of 51 acres and a working depth of 6 feet. The theoretical detention time under design conditions will be 66 days. The primary effluent will then pass into the first of two secondary cells.

The two secondary cells will be equally sized and will increase the detention time under design conditions to 120 days. The average reduction in BOD_5 will be 85 percent or more. The treated wastewater will then pass into the winter storage pond.

The winter storage pond will have a mid-depth water surface area of approximately 55 acres and a working depth of 15 feet. The pond will be designed to retain all the treated wastewater, under design flow conditions, from the middle of October to the middle of April. The wastewater will be discharged from the winter storage pond during the irrigation season through the chlorine contact chamber.

The chlorine contact chamber consists of 800 feet of 6 foot diameter aluminum pipe submerged in the winter storage pond adjacent to the inside toe of the north dike of the storage pond. This pipe will be laid such that the top of the pipe will be at roughly the same elevation as the pond bottom. The contact chamber will be sized for a one hour minimum detention time under design conditions.

The chlorine solution will be introduced into the contact chamber just upstream of a 2 foot diameter orifice plate, which will be located near the upstream end of the chamber. The orifice plate and two sets of baffles, which will be located 10 to 20 feet respectively downstream of the orifice plate, will provide effective mixing. The disinfected wastewater will be withdrawn from the contact basin at the downstream end by five short couple constant-speed lineshaft turbine pumps. The chlorine feed rate will be coordinated with the pumping rate. The upstream end of the chlorine contact chamber will be covered with a mechanical screen to prevent debris from entering the chamber and damaging the booster pumps. The winter storage pond will be pumped empty once each year and overflow from the secondaries stopped, so that chlorine contact basin might be cleared and inspected.

The pump station will consist of a wet well, five manually controlled pumps with surge release valves, an electrical control system to synchronize operation of the pumps and the spray system shut-off valves, and adequate housing. The chlorination equipment will be housed in a separate room of the same building.

The spray irrigation system will consist of center pivoted, electrically propelled spray irrigation units. These spray units will be turned on and off by remotely controlled, motor operated shut-off valves. The land disposal area will include a total wetted area of about 268 acres, which will be divided into 5 sub-areas. Under normal operating conditions each sub-area would receive its weekly allocation of water during a 48 hour period or less and then no water would be applied to that particular parcel for 5 days.

The cover crop will be harvested 3 or 4 times each year. Any nutrients not supplied in sufficient quantities by the wastewater will be applied as needed. Yearly check of soil conditions will be required to insure a proper nutrient balance.

The irrigated area will be surrounded by a 200 foot buffer zone, with a 4 foot high fence on the outside border of the buffer zone. As an added precaution to prevent aerosols from escaping the spray area, the spray units will be shut down during high winds.

Dikes will be constructed where needed to minimize the possibility of surface runoff from the spray area. Test wells will be installed around the perimeter of the spray area to allow testing of the groundwater quality during system operation.

ANTICIPATED ANNUAL OPERATION AND MAINTENANCE COSTS FOR PROPOSED WASTEWATER TREATMENT SYSTEM

	Initial O & M Costs	Design Year O & M Costs
Lagoons and Winter Storage Pond	$ 3,800	$ 3,800
Chlorination Facility	10,200	17,320
Land Disposal System	-11,250	-20,360*
Net O & M	$ 2,480	$ 760

*Negative Costs indicate that cover crop gross sales exceed total year expenditures.

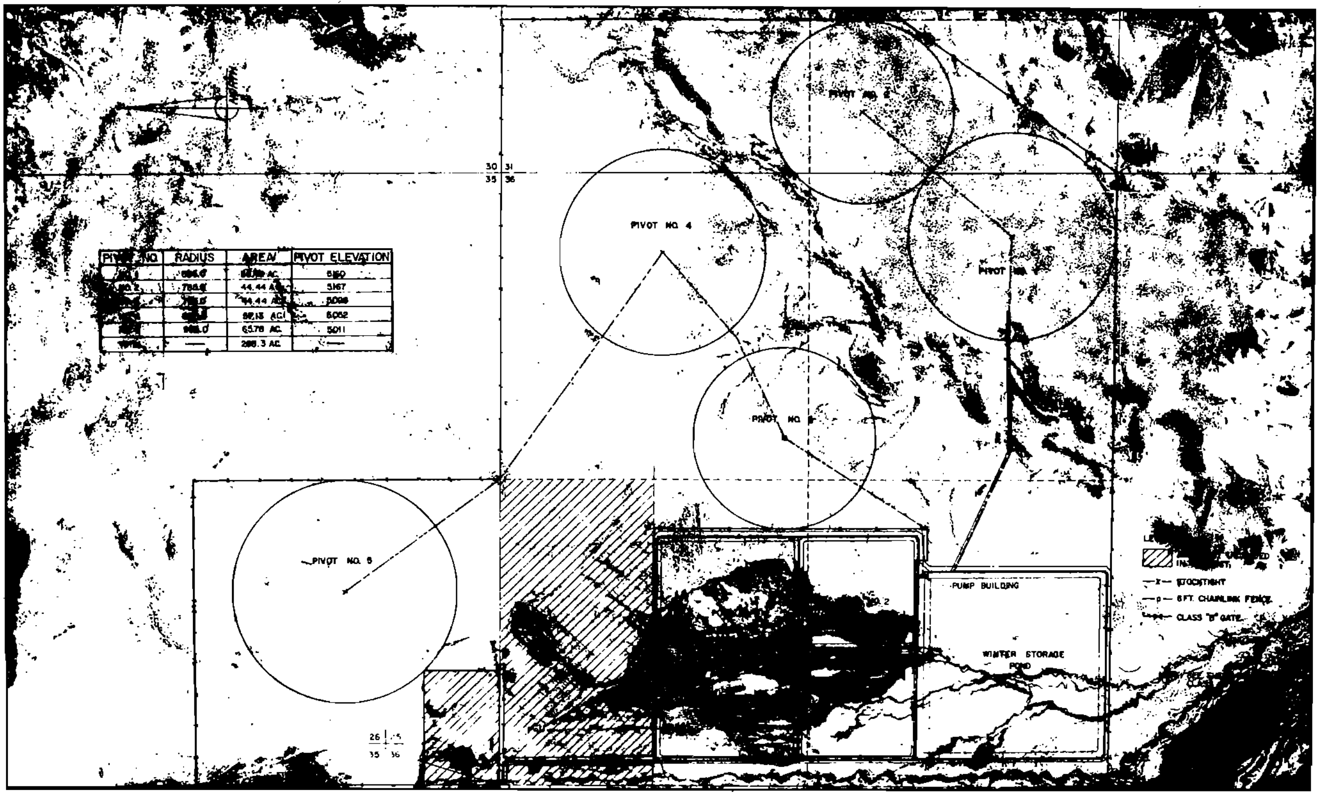

Figure VI-A. Experience in Treatment Plant Design, Wastewater Land Application (Irrigation), and Lagoon Design.

Chlorination of Waste Stabilization Lagoon Effluent

*Bruce A. Johnson, Jeff Wight, J.H. Reynolds, and E.J. Middlebrooks**

Waste stabilization lagoons have been used for many years to provide adequate treatment for domestic sewage. However, as a result of more stringent state and federal discharge standards, there are serious doubts about the ability of many existing lagoons to meet new requirements. This is particularly true with respect to bacterial reduction. Therefore, chlorination has been and is being considered as a means of upgrading lagoon effluents to meet bacteriological discharge standards.

There is, however, evidence that chlorination of wastewater high in organic nitrogen content may be accompanied by adverse effects. Among the concerns are toxicity of chlorinated compounds (Zillich, 1972), increases in biochemical and chemical oxygen demands (Echelberger et al., 1971, Han, 1972), effects on suspended solids (Dinges and Rust, 1969; Kott, 1973), and increases in chlorine demand (Kott, 1971; White, 1973). To add to the knowledge concerning the chlorination of waste stabilization lagoon effluent, this study was undertaken with the primary objective of operating field scale chlorination facilities to evaluate lagoon effluent chlorination practices under varying seasonal conditions. This evaluation included determinations of the chlorine residual concentrations necessary to reduce bacterial populations to acceptable levels and of the effects of temperature, suspended solids, ammonia, chemical oxygen demand, and sulfide on chlorination practices.

Improving disinfection efficiency by filtering lagoon effluent through an intermittent sand filter prior to chlorination was also investigated. The results from this study were used to develop a mathematical model, which was then used to construct a series of design and operation curves to aid in selecting chlorine doses necessary for various levels of disinfection at different qualities of lagoon effluent.

MATERIALS AND METHODS

The Logan, Utah, wastewater stabilization lagoons were selected as the site for this study. Because of the relatively high bacteriological quality of the final lagoon effluent, the facilities were constructed with capabilities of treating either primary or secondary lagoon effluent. Four systems of identically designed chlorine mixing and contact tanks, each capable of treating 50,000 gallons per day, were constructed. Using recommendations presented by Collins, Selleck, and White (1971), Kothandaraman and Evans (1972) and Marske and Boyle (1973), the chlorination systems were constructed to provide rapid initial mixing followed by chlorine contact in plug flow reactors. A serpentine flow configuration, having a length to width ratio of 25:1, coupled with inlet and outlet baffles, was used to enhance plug flow hydraulic performance. The chlorine mixing and contact tanks are illustrated in Figure 1. Dye studies similar to those conducted by Deaner (undated) were used to determine average detention times for each contact tank. The theoretical detention time for each tank was 60 minutes, white the actual detention time for the four tanks averaged 49.6 minutes.

Three of the four chlorination systems were used for directly treating primary and secondary lagoon effluent. The effluent treated in the fourth system was filtered through an intermittent sand filter prior to chlorination. Filtered lagoon effluent was also used as the solution water for all four chlorination systems. Appropriate quantities of chlorine gas were mixed with solution water by use of

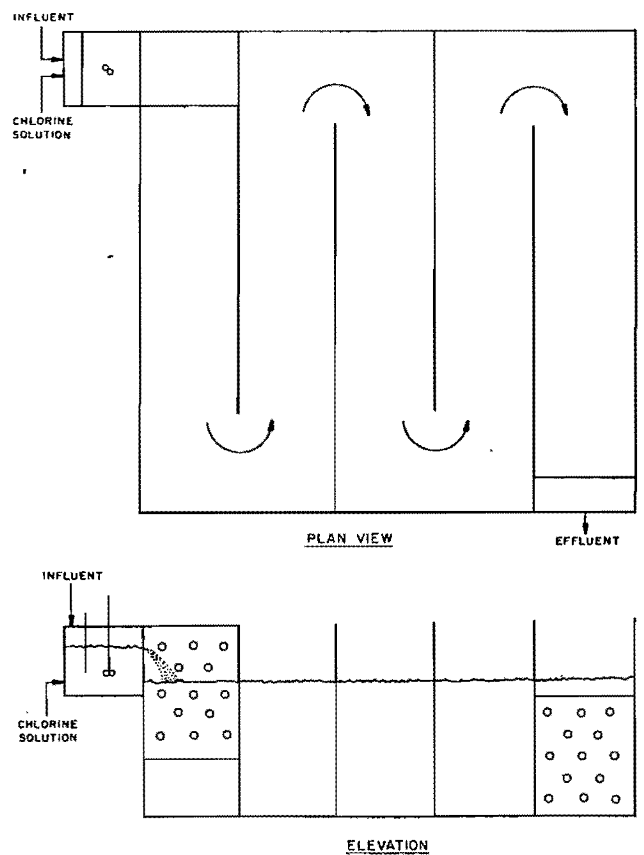

Figure 1. Chlorine mixing and contact tanks.

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vacuum operated diffusers prior to introducing solution lines into chlorine mixing tanks and exposing the main flow of lagoon effluent to chlorine. A schematic diagram of the chlorination operation is presented in Figure 2.

Chlorination of lagoon effluent began on August 6, 1975 and continued until August 24, 1976. Samples were collected at least twice a week throughout the study period except between December 1975 and February 1976. No samples were collected during that period because of pipeline freeze-up. Samples were collected just prior to chlorination as well as at points corresponding to residence time of 17.5, 35.0, and 49.6 minutes in each contact tank. Chlorine doses were varied between .25 and 30 mg/l. In addition to chlorinated samples, other samples were collected from the influent and effluent of the lagoon systems and from the effluent from each cell in the system. This was done to characterize the performance of the lagoon system and to assist in determining how to adjust chlorination practices to compensate for seasonal fluctuations in lagoon performance.

The chlorinated samples were analyzed bacteriologically for MPN total and fecal coliforms (TC and FC). Five tubes were used for each dilution. Membrane filter total and fecal coliforms were also determined on all unchlorinated samples. Additional water quality analyses included ammonia (NH_3-N), biochemical oxygen demand (BOD_5), dissolved oxygen (DO), total and soluble chemical oxygen demand (TCOD and SCOD), sulfide ($S^=$), suspended solids (SS), volatile suspended solids (VSS), pH, temperature, and turbidity. Free and combined chlorine residuals (FCl and CCl) were also measured for all chlorinated samples using the amperometric titration method. With the exception of the sulfide, all samples were collected and analyzed using recommended procedures outlined in APHA Standard Methods (1971). Sulfide was analyzed using a method described by Oria (undated).

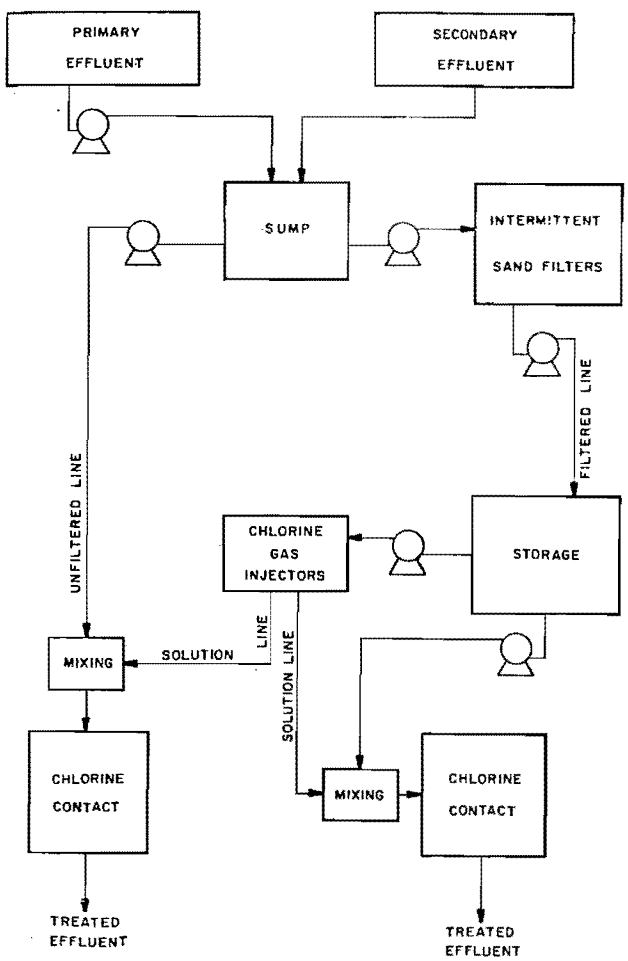

Figure 2. Experimental chlorination schematic.

In addition to the field study as described, laboratory studies were also conducted. These studies were performed to assist in describing relationships between chlorine and other wastewater consitutents.

RESULTS AND DISCUSSION

In studying chlorination practices of waste stabilization lagoon effluent, it was found that ammonia, organic nitrogen, sulfide, suspended solids, and chemical oxygen demand were the most sensitive parameters of concern. Evaluation of these water quality characteristics were made to appraise chlorine demand and disinfection and to develop design and operation curves.

Ammonia and Organic Nitrogen

Reactions between chlorine and ammonia (NH_3-N) in wastewater are significant due to the formation of chloramines and subsequent reduction in disinfection capacity. This reduction of disinfection capacity represents an effective chlorine demand. Breakpoint chlorination is also of interest as a possible means of removing ammonia from wastewater and producing free chlorine residual, a more effective disinfectant. Results of this study show that in most cases there was little reduction of ammonia concentrations and disinfection was accomplished by combined chlorine residual. In 6 percent of the data, the chlorination breakpoint was reached and free chlorine residual produced. However, it was determined that breakpoint chlorination is almost never necessary for achieving satisfactory disinfection and that adequate disinfection of lagoon effluents can be achieved with combined chlorine residual in less than 30 minutes contact time. As a means of removing ammonia, the theoretical breakpoint curve was found to be of limited applicability for wastewater chlorination. The shape of the breakpoint curve was found to be highly variable as evidence indicates that interactions between chlorine and organic nitrogen influence breakpoint chlorination. Attempts to identify the quality and quantity factors which determine the shape of the breakpoint curve in wastewater were largely unsuccessful due to insufficient data and the complex nature of breakpoint reactions in wastewater.

Sulfide

During certain periods of the year, when ice covered the lagoons, it was found that up to 1.8 mg/l sulfide was produced in secondary lagoon effluent. Under these conditions, up to seven times more chlorine dose was required to produce the same chlorine residual resulting during other times of the year. It was determined that chlorine reacts with hydrogen sulfide according to the following reactions.

$$HOCl + H_2S \rightarrow H_2SO_4 + 4HCl \quad \text{(1)}$$
$$4HOCl + H_2S \quad HCl + H_2O + S \quad \text{(2)}$$

For sulfide concentrations between 1.0-1.8 mg/l it was determined that 3.6 moles of chlorine were required for each mole of sulfide oxidized. Sulfide production in the lagoons continued until shortly after spring overturn, when sulfide concentrations quickly disappeared. Close surveillance of chlorine doses and residuals must be maintained during this period.

Suspended Solids

In evaluating interactions between chlorine and suspended solids (SS) (mostly composed of volatile suspended solids (VSS)) it was found that increases in SS do not necessarily coincide with increases in chlorine demand. Regression analyses performed between applied chlorine dose and total chlorine residual for varying ranges of SS were inconclusive, largely because of scatter in the field data. However, it was observed that chlorine does have some effect on SS concentrations. From field data, reductions in SS of up to 40 percent were observed following chlorination. However, accumulations of solids in the contact tanks were also observed, indicating that a large portion of SS reduction was the result of settling. Laboratory tests were conducted on algae suspensions to determine if some reduction of SS was caused by chemical interaction with chlorine. The results, as shown in Figure 3, do indicate reductions of SS, along with increases in turbidity, with increasing chlorine doses. These changes are probably the result of chlorine oxidizing suspended solids into soluble material and breaking large particles into smaller ones. The same tests, performed on solids suspensions not compound of algae, did not show these trends. This indicates that changes in SS are determined by the composition of the material in suspension, as well as by concentrations of SS and chlorine.

Chemical Oxygen Demand

In considering the chlorination of waste stabilization lagoon effluent, concern has been expressed that the lysis of algae cells would cause an increase in chemical oxygen demand (COD). Results of laboratory and field studies indicate that chlorine has little, if any, affect on total COD. However, in laboratory studies using algae suspensions, it was found that increases in soluble COD were produced with increasing concentrations of total chlorine residual for contact periods of up to two hours. The same trends were not observed for field data using total chlorine residual. However, when only data involving free chlorine residual was evaluated, increases of soluble COD were observed for unfiltered effluent. The correlations between changes in soluble COD (SCOD) and free chlorine residual are given in Figure 4. The same trend, however, was not observed for filtered data, probably because of lower SCOD concentrations initially and fewer data points.

Chlorine Demand

The level of coliform reduction expected for a given chlorine dose is determined by the chlorine residual remaining after a specified contact period. The chlorine residual is determined by the chlorine demand exerted by a particular quality of lagoon effluent. For this study, the chlorine demand was generally found to be almost 50 percent of the applied chlorine dose. The principle exception to this was observed during periods of sulfide production when the chlorine demand was much higher. In evaluating chlorine demand, it was observed that the demand increases with increasing contact time. It was also observed that for a given chlorine dose, less demand was exerted by filtered effluent than by unfiltered effluent, as illustrated in Figures 5 and 6. This can be attributed to the removal of chlorine demanding material as a result of filtration. Generally, it was determined that the exertion of chlorine demand can be expressed as a function of total chemical oxygen demand.

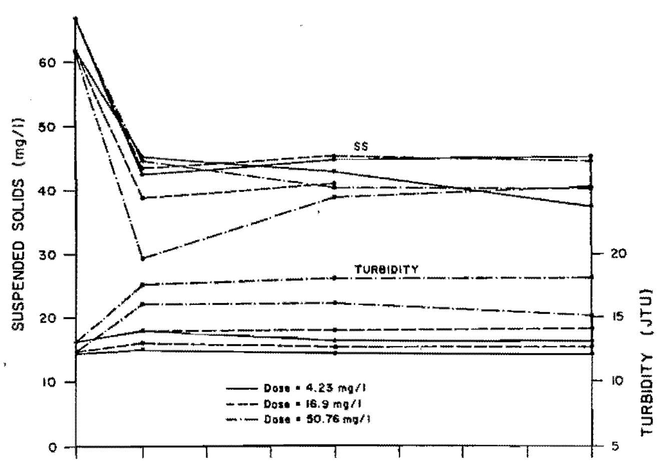

Figure 3. Effects of chlorine on SS and turbidity (April 9-10, 1976).

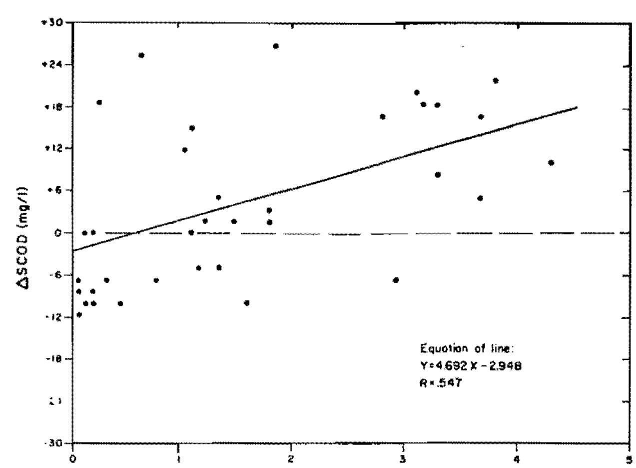

Figure 4. Changes in soluble COD when free chlorine residual is present in unfiltered lagoon effluent.

Disinfection

As expected, the field data shows that increases in total chlorine residual produce increased total and fecal coliform reductions for both filtered and unfiltered lagoon effluent. Statistically significant reductions in coliform concentrations were also observed with increasing chlorine contact times. It was also found that less total chlorine residual was required to produce a given level of disinfection for filtered effluent. On the average, the chlorine residual required for a given level of disinfection was 42 percent less for total coliform and 23 percent less for fecal coliform in filtered effluent as compared to unfiltered effluent. As an example, a comparison between the chlorine residual required at different times for total coliform reduction in both filtered and unfiltered lagoon effluent is presented in Figures 7 and 8. Similar trends were observed for fecal coliform reduction.

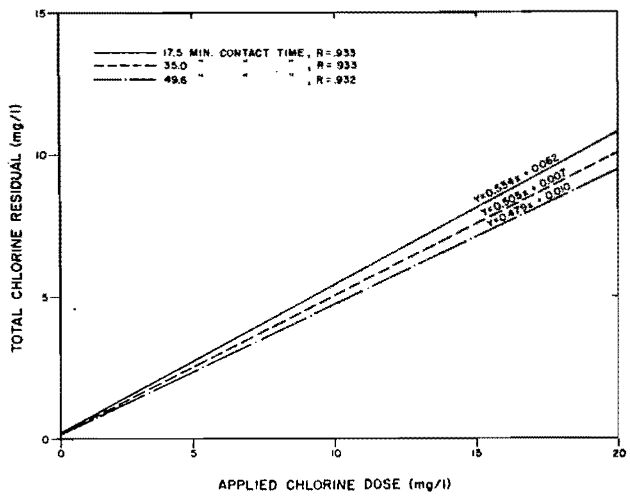

Figure 5. Summary of total chlorine residual remaining after application of chlorine dosage using filtered lagoon effluent at various chlorine contact times.

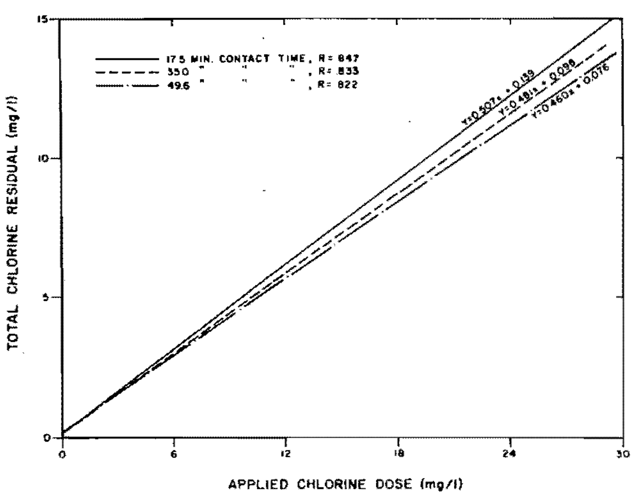

Figure 6. Summary of total chlorine residual remaining after application of chlorine dosage using unfiltered lagoon effluent at various chlorine contact times.

Design and Operation Curves

The data derived from this study was used to develop a mathematical model, largely based upon kinetic rate expressions. The model was then used to construct a series of curves to aid in the design and operation of lagoon effluent chlorination facilities. Referring to Figures 9 through 15, an example may best illustrate how these curves are to be applied.

Assume that a particular lagoon effluent is characterized as having a fecal coliform concentration of 10,000/100 ml, 0 mg/l sulfide, 20 mg/l COD (TCOD), and a temperature of 5°C. If it is necessary to reduce the fecal coliform counts to 100/100ml, a combined chlorine residual sufficient to produce 99 percent bacterial reduction must be obtained. If an existing chlorine contact chamber has an

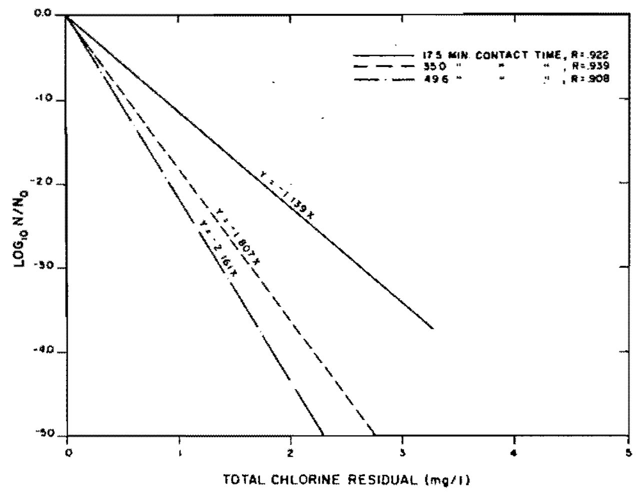

Figure 7. Summary of total coliform removal efficiency, using filtered lagoon effluent, as a function of total chlorine residual at various chlorine contact times.

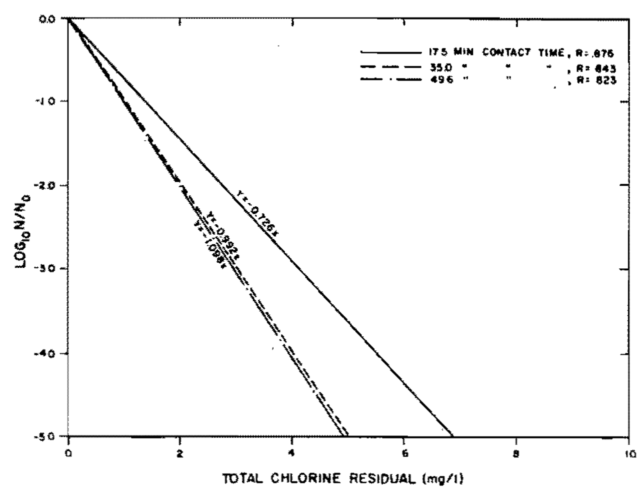

Figure 8. Summary of total coliform removal efficiency, using unfiltered lagoon effluent, as a function of total chlorine residual at various chlorine contact times.

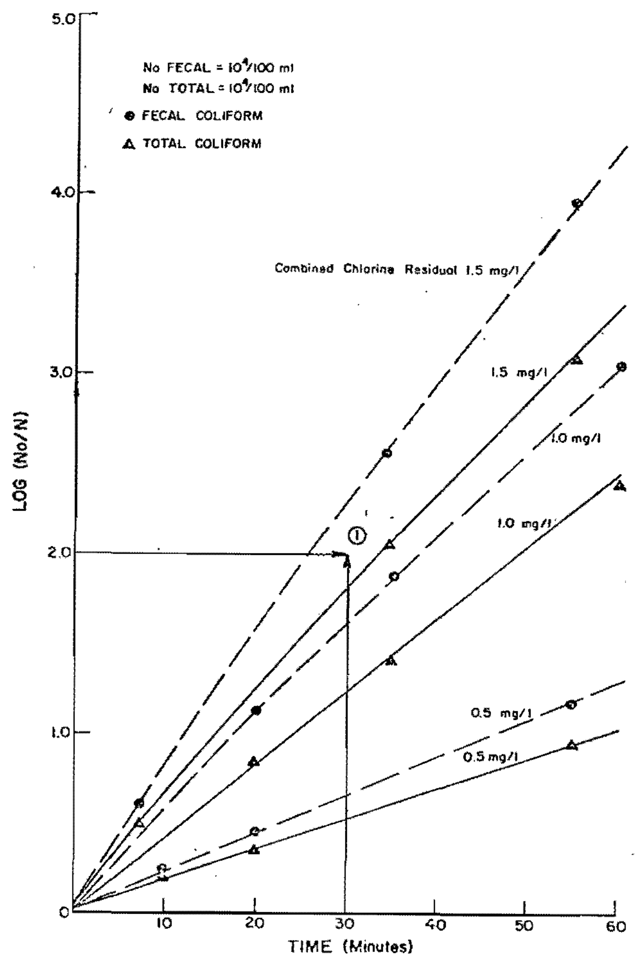

Figure 9. Combined chlorine residual at 5°C for coliform = 10^4/100 ml.

average residence time of 30 minutes, the required chlorine residual is obtained from Figure 9. A 99 percent bacterial reduction corresponds to log (N_0/N) equal to 2.0. For a contact period of 30 minutes, a combined chlorine residual of between 1.0 and 1.5 mg/l is required to produce that level of fecal coliform reduction. Upon interpolating, the actual chlorine residual is determined to be 1.30 mg/l. This is indicated by part 1 in Figure 9.

Going to Figure 10, it is determined that if a chlorine dose produces a residual of 1.30 mg/l at 5°C, the same dose would produce a residual of 0.95 mg/l at 20°C. This is because of the faster rate of reaction between TCOD and chlorine at the higher temperature. This is indicated by point 2 in Figure 10. For an equivalent chlorine residual of 0.95 mg/l at 20°C and 20 mg/l TCOD, it is determined from Figure 11 that the same chlorine dose would produce a residual of 0.80 mg/l if the TCOD were 60 mg/l. This is because higher concentrations of TCOD increase the rate of chlorine demand. Point 3 in Figure 11 corresponds to this residual. The chlorine dose required to produce an equivalent residual of 0.80 mg/l at 20°C and 60 mg/l TCOD is determined from Figure 12. For a chlorine contact period of 30 minutes, a chlorine dose of 2.15 mg/l is necessary to produce the desired combined residual as indicated by point 4 in Figure 12. This dose will produce a reduction of fecal coliform from 10,000/100 ml to 100/100 ml within 30 minutes at 5°C and with 20 mg/l TCOD.

If, in the previous example, the initial sulfide concentrations was 1.0 mg/l instead of 0 mg/l, it would be necessary to go directly from Figure 9 to Figure 13. Here, a chlorine residual of 1.30 mg/l at a TCOD of 20 mg/l and a temperature of 5°C is converted to an equivalent chlorine residual of 1.10 mg/l for a TCOD of 60 mg/l.

This is represented by the point 5 in Figure 13. Going to Figure 14, which corresponds to an initial sulfide concentration of 1.0 mg/l, it is determined that a chlorine dose of 6.65 mg/l is necessary to produce an equivalent chlorine residual of 1.1 mg/l after a contact period of 30 minutes. Point 6 in Figure 14 corresponds to this dose. The sulfide remaining after chlorination is determined to be 0.44 mg/l from Figure 15 as indicated by point 7.

SUMMARY

From this study it was determined that disinfection of waste stabilization lagoon effluent can generally be achieved with relatively low doses of chlorine and in contact times of less than 50 minutes. The chlorine demand was found to be less than reported in other literature during most of the year. Generally, it was found that the chlorine demand was about 50 percent of the applied dose during all times of the year except when hydrogen sulfide was produced. During that period, the chlorine demand was found to be as high as 85 percent. Combined chlorine residuals of between 0.5 to 1.0 mg/l were found to be adequate in reducing fecal coliforms below the discharge standard of 200/100 ml. This residual is produced by a chlorine dose of between 2-3 mg/l, except during periods of hydrogen sulfide production when a dose of 7-8 mg/l is required.

Chlorination of these algae laden waters was accompanied by few adverse effects. Soluble COD was observed to increase in the presence of free chlorine residual. Increases in turbidity and reductions in SS were also observed for high chlorine doses. However, it was rarely necessary to chlorinate at high enough doses for these responses to have any major repercussions. Breakpoint chlorination was observed to be of minimal importance in providing adequate disinfections. Filtering of lagoon effluent through intermittent sand filters prior to chlorination was found to reduce chlorine demand and enhance disinfection efficiency.

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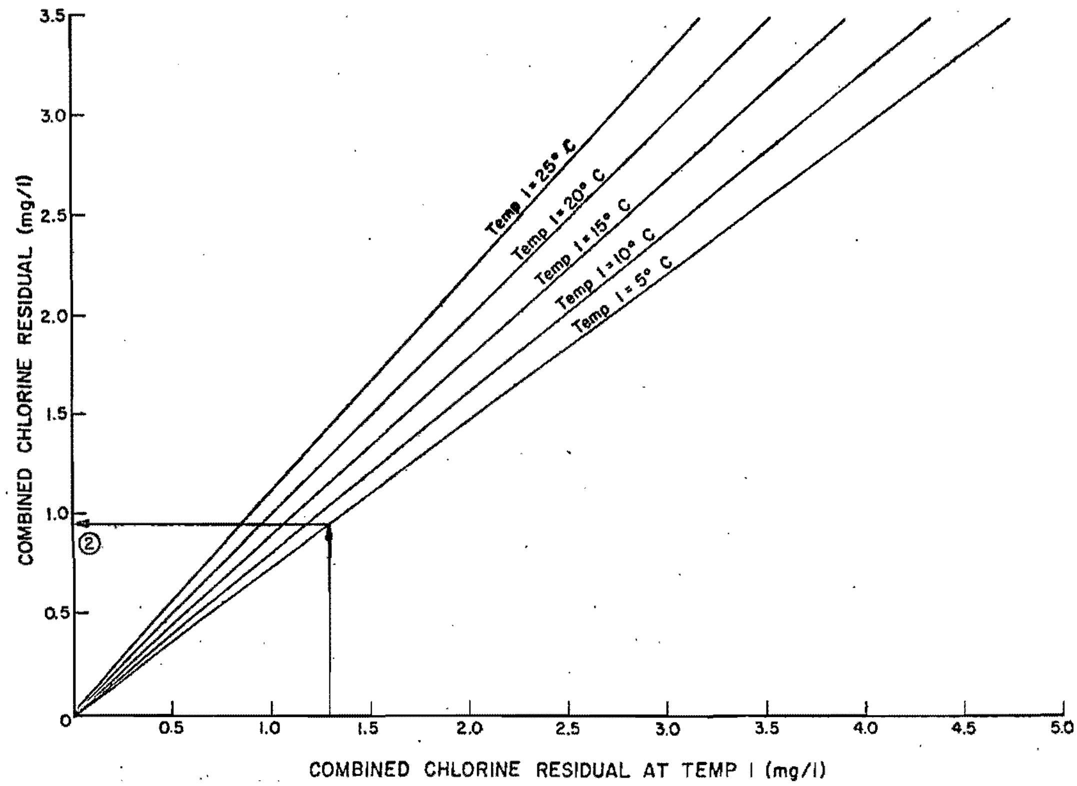

Figure 10. Conversion of combined chlorine residual at Temp 1 to equivalent residual at 20°C.

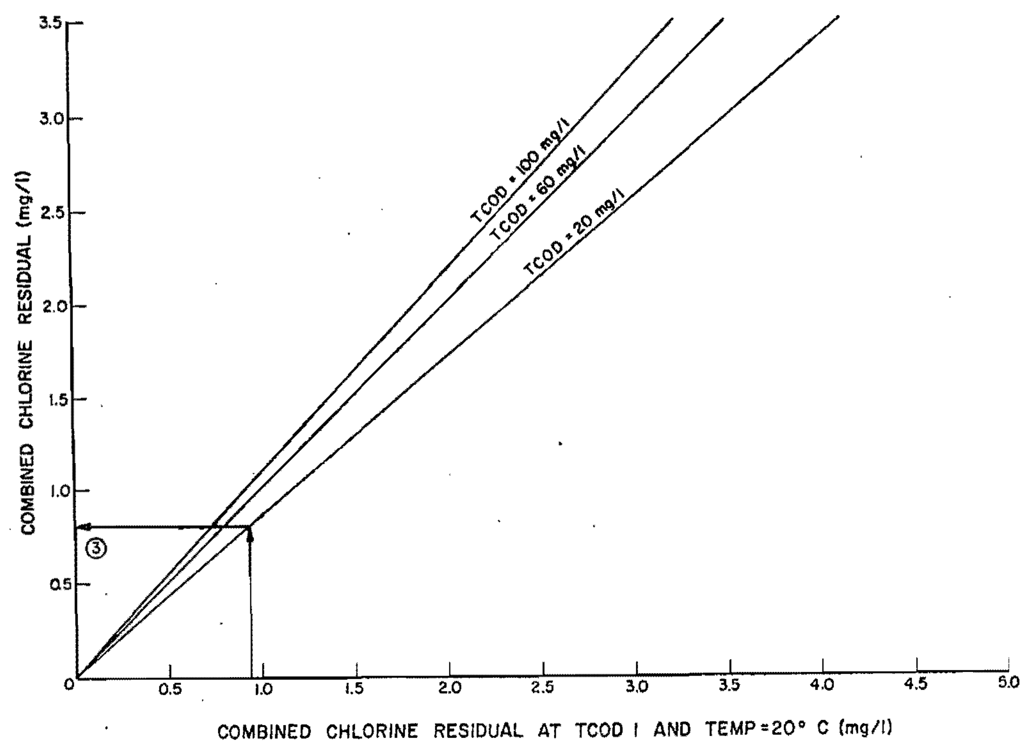

Figure 11. Conversion of combined chlorine residual at TCODl and 20°C to equivalent residual at 20°C and TCOD = 60 mg/l.

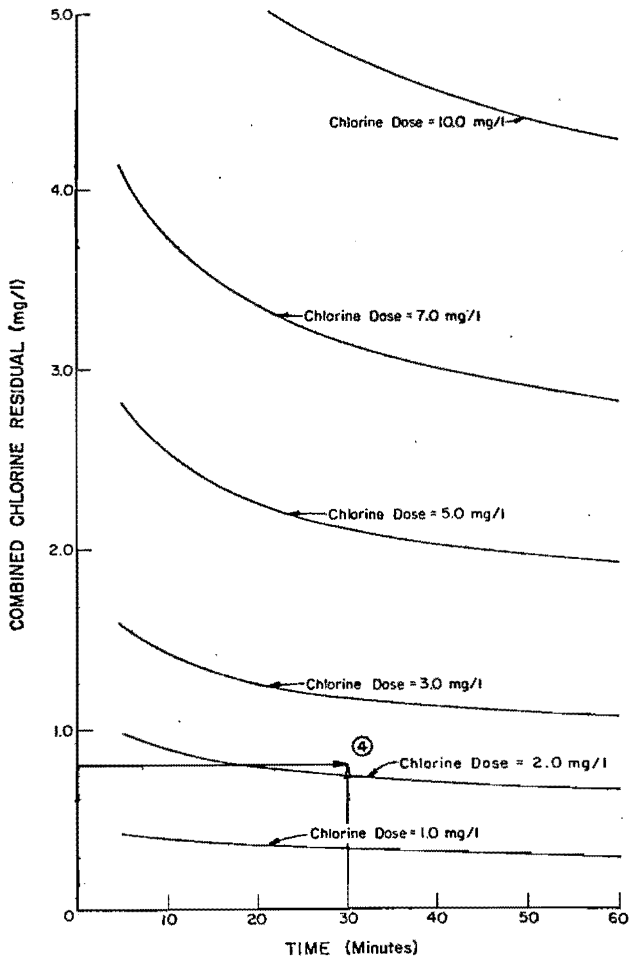

Figure 12. Determination of chlorine dose required for equivalent combined residuals at TCOD = 60 mg/l and Temp. = 20°C.

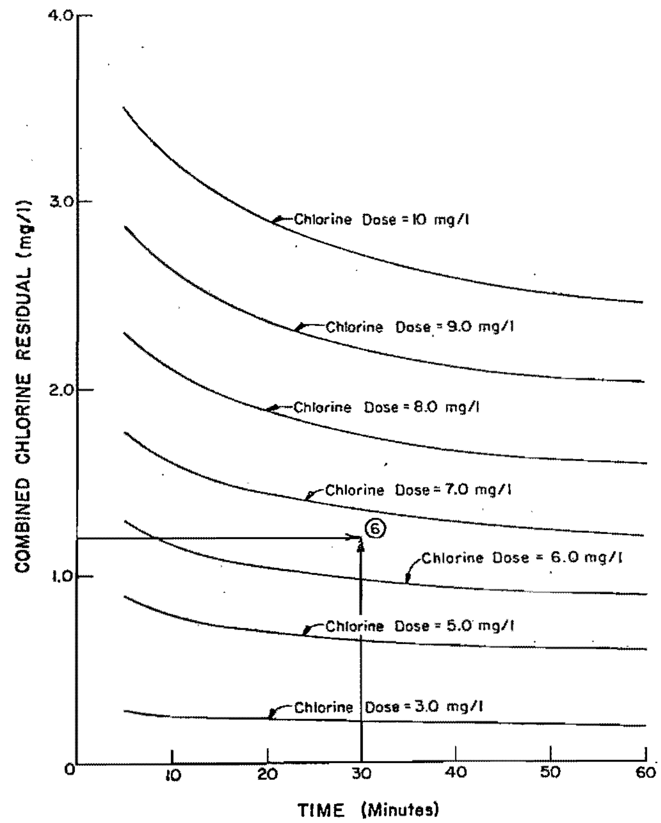

Figure 14. Determination of chlorine dose required when $S^{=}$ = 1.0 mg/l, TCOD = 60 mg/l, and Temp. = 5°C.

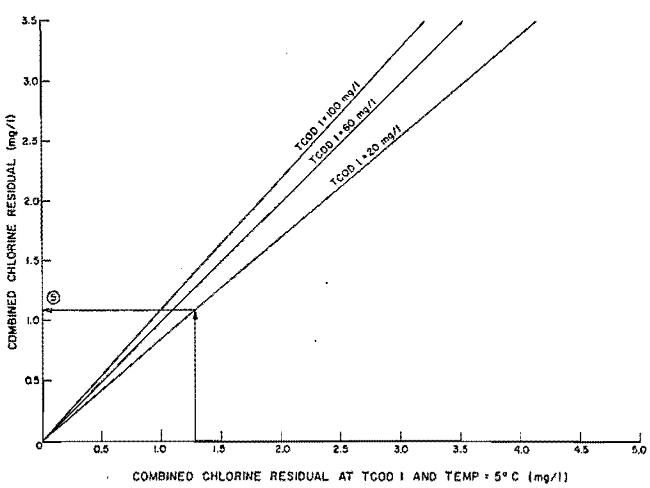

Figure 13. Conversion of combined chlorine at 5°C and TCOD1 to equivalent residual at 5°C and TCOD = 60 mg/l.

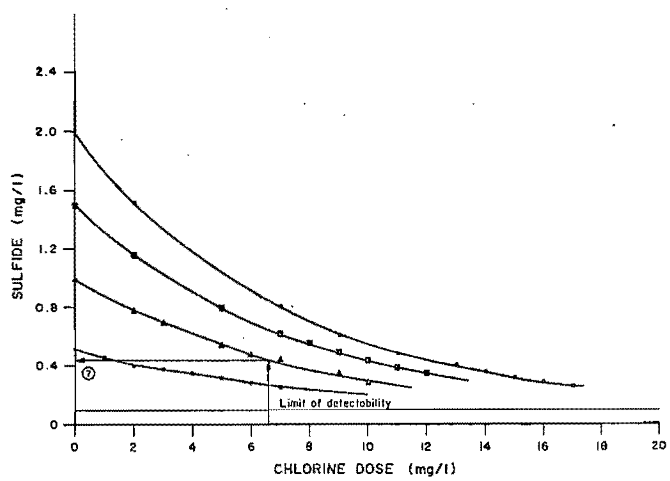

Figure 15. Sulfide reduction as a function of chlorine dose.

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Performance Evaluation of a Seven Cell Lagoon System for BOD_5, SS, and Fecal Coliform

*Ralph E. Swiss, James H. Reynolds, Christine A. Macko, E. Joe Middlebrooks**

INTRODUCTION

Wastewater stabilization lagoons have provided acceptable, low cost, efficient wastewater treatment for nearly 5,000 communities in the United States. However, with the implementation of the Water Pollution Control Amendments of 1972 (PL 92-500) stringent secondary discharge standards have been established. It is possible that waste stabilization lagoon systems may not be capable of satisfying these new discharge requirements. At present, very little data exist which adequately describe the yearly performance of waste stabilization lagoon systems.

The general objective of this study was to determine the yearly performance of a seven cell facultative waste stabilization lagoon system treating domestic wastewater from a community with a population of 471 persons and to compare this actual performance with existing federal discharge standards, State of Utah discharge standards, criteria used to design the lagoon system and to evaluate existing design equations.

Twenty-four hour composite samples of the raw sewage influent to the lagoon system and the effluent from each pond in the system were collected twice each week for approximately 13 months. In addition, these same samples were collected for four 30 consecutive days (once each season) during the same 13 month period. The samples were analyzed for biochemical oxygen demand (BOD_5) and suspended solids. Fecal coliform and bacteria were monitored with grab samples. In addition, influent and effluent daily flowrates, air temperature, wind, evaporation, and solar radiation were recorded.

The results indicate that the system did not exceed the federal biochemical oxygen demand requirement of 30.0 mg/l at any time during the study. However, it failed to meet the 85 percent biochemical oxygen demand (BOD_5) removal requirement 4 of the 13 months studied. The system also satisfied the State of Utah biochemical oxygen demand requirement of less than 10.0 mg/l 8 of the 13 months studied. The system was able to meet the federal suspended solids requirement of less than 30.0 mg/l 10 of the 13 months studied. It also satisfied the State of Utah suspended solids discharge requirement of less than 10 mg/l 8 of the 13 months studied. However, it failed to meet the federal 85 percent suspended solids removal requirement 5 of the 13 months studied. The system never exceeded the federal or State of Utah fecal coliform bacteria discharge standard.

In general, the loading on the lagoon exceeded the criteria used to design the system. Application of the data to existing design equations indicated that the equations were not adequate to predict overall performance.

ACKNOWLEDGEMENTS

Special thanks is given to the mayor of Corinne, Utah and his staff for their help in the conducting of this study.

This study was conducted in fulfillment of Environmental Protection Agency Contract Number 68-03-2060.

PROCEDURES

Study Location

The study was conducted at the Corinne Waste Stabilization Lagoon System, Corinne, Utah. The City of Corinne is located in Box Elder County in the Northwestern portion of Utah. The community has a population of 471 persons (1970 Census) and no major industry. It is predominantly a rural farming community with a few residents commuting to surrounding industries outside the Corinne area.

Lagoon System

The Corinne City Wastewater Lagoon System was constructed during 1970 and began discharging in the spring of 1971. A flow diagram of the system is shown in Figure 1. The facility consists of seven facultative cells connected in series. None of these cells are mechanically aerated and comminution is the only pretreatment prior to the raw sewage entering the primary cell.

The system was designed according to State of Utah requirements for waste stabilization pond design in 1970 (Sudweeks, 1970). The original design calculations were based on a design population of 700 people, a design flowrate of 265,000 liters/day (70,000 gal/day), assuming a raw sewage strength of 0.077 kg BOD_5/person/day (0.17 lbs BOD_5/person/day) and a flowrate of 378.5 liters/person/day (100 gal/person/day). The design organic load

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was 36.2 kg BOD_5/ha/day (32.2 lbs BOD_5/acre/day) with a winter theoretical total hydraulic detention time of 180 days. Thus, the total surface area of the system is approximately 3.86 ha (9.53 acres). The average depth of all the ponds is approximately 1.22 meters (4 ft).

The comminutor is located at a pump lift station located approximately 152.4 meters (500 feet) from the primary lagoon. Also located at the pump lift station is the influent flow recorder. The influent flow meaning device is a 20.32 cm (8 inch) Palmer Bowlus Flume coupled to a Stevens Model 61-R[1] continuous flow recorder.

The effluent flowrate from the final pond (final system effluent) was monitored with a 45 degree V-notch weir coupled with a Stevens Model 61-R continuous flow recorder.

Sample Collection and Analysis

The location of each sampling station is shown on Figure 1 and described in Table 1. Automatic 24-hour composite samplers were located at Stations 2 through 7. A flow proportional 24-hour composite sampler was located at Station Number 1 (raw sewage influent).

[1]Leupold and Stevens, Inc., Box 688, Beaverton, Oregon.

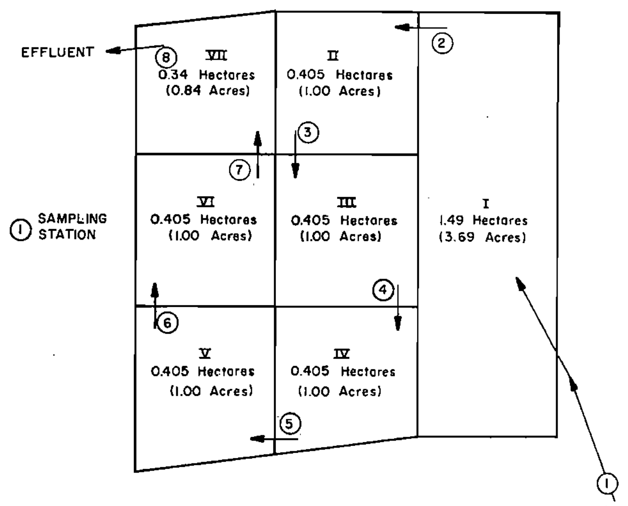

Figure 1. Flow diagram of Corinne City Wastewater Lagoon System.

Sampling began January 23, 1975 and continued until January 31, 1976. Samples were collected every third day on a rotating schedule except for a 30 day period each season when samples were collected daily for 30 consecutive days. All samples were collected between 5:00 a.m. and 10:00 a.m. At the Utah Water Research Laboratory, Logan, Utah, the composite samples were analyzed for BOD_5, and suspended solids. Grab samples were substituted for the composite samples when the automatic composite samplers failed to function properly. This occurred on less than 10 percent of the samples.

All chemical analyses performed at the UWRL and by the EPA laboratory followed the methods and procedures described in Methods for Chemical Analysis of Water and Wastewater (EPA, 1974).

In addition to the composite samples, grab samples for fecal bacteria analysis were collected at each station. Bacteria samples were analyzed at the UWRL for fecal coliforms using the membrane filter technique. All analyses were performed according to the methods described in Standard Methods (1971). The methods and media used are tabulated in Table 2.

All samples were transported from the study site to the Water Quality Laboratory, Utah Water Research Laboratory, Utah State University, Logan, Utah. Transportation required approximately 45 minutes, all samples were transported in their collection containers and shielded from sunlight. The bacterial samples were iced during transportation.

Meterological Data

Precipitation, wind speed, temperature (maximum, average, minimum), pan evaporation and solar radiation (total incident radiation) was collected at weather stations near Corinne and published in Climatological Data (NOAA, 1975, 1976). All information except that relating to evaporation and solar radiation was obtained from the Corinne reporting station located 1.6 kilometers (1 mile)

Table 1. Description of sampling stations for Corinne City Wastewater Lagoon System.

Sampling Station Number	Station Description
1	Raw wastewater influent to system; pump lift station (a totalizer flow meter is located at this point)
2	Effluent from Cell I
3	Effluent from Cell II
4	Effluent from Cell III
5	Effluent from Cell IV
6	Effluent from Cell V
7	Effluent from Cell VI
8	Effluent from Cell VII and also final effluent from the Lagoon System (a totalizer flow meter is located at this point)

from the treatment facility. Evaporation data were furnished by the Bear River Refuge reporting station located 16 kilometers (10 miles) from the study site. Solar radiation data were obtained from the solar radiation station, located at Utah State University in Logan, Utah, 32 kilometers (20 miles) from the Corinne site.

Hydraulic Data

Flow rates and total volumes of wastewater entering and leaving the lagoon system were recorded at Stations Number 1 and Number 8. Flow patterns and detention times were determined by injecting rhodamine B dye into the influent of each pond and monitoring the effluent of each pond for dye concentration.

Dye samples were analyzed on a Turner[1] model 111 fluorometer using a 568 μm primary filter and a 590 μm secondary filter. The meter was calibrated according to procedures outlined by Buttes (1969). Dye dispersion curves were plotted using the temperature-corrected readings. These curves were analyzed using the techniques provided by Marske and Boyle (1973).

Data Analyses

Statistical calculations were performed according to methods provided by Sokal and Rohlf (1969).

RESULTS AND DISCUSSION

Seasonal Performance

General

All of the ponds are designated by pond number. The data for a given pond represent the quality of the effluent water from that particular pond. Pond Number 7 is the final pond in the system, its effluent is, therefore, the effluent for the entire system and is generally designated as "Effluent" rather than "Pond 7." "Influent" represents the incoming raw sewage wastewater from the City of Corinne.

Hydraulic performance

The average monthly influent and effluent daily flowrates are recorded in Table 3 and shown graphically in Figure 2. The average monthly daily flowrate varied from 1,029,645 liters/day (272,033 gal/day) in October to 253,667 liters/day (67,019 gal/day) in December. The yearly average influent daily flowrate was 693,724 liters/day (183,282 gal/day). This represents a per capita hydraulic load to the system of 1472.9 liter/person/day (5,574.93 gal/person/day). The yearly average daily flowrate exceeds the hydraulic design flowrate by 2.62 times.

Dye studies were conducted on each lagoon to determine the actual hydraulic residence time in the system. The results are recorded in Table 4. The actual total hydraulic residence time for the system was found to be 88.3 days which is 49.1 percent of the 180 day requirement of the State of Utah. These dye studies were from December 1975 to July 1976 and thus represent the condition existing at that time.

In summary, the hydraulic load to the lagoon system exceeded the design hydraulic flowrate by 2.62 times. This excessive hydraulic loading is most likely due to

[1]Turner Fluorometer, Model 111, by G. K. Turner Associates, Palo Alto, California.

Table 2. Methods and media used for the bacteriological analyses.

	Method	Media (Manufacturer)	Die-off/Lagoon Study
Fecal coliforms	Standard Methods (APHA, 1971) Section 408 B	m-FC broth (BBL 11365)	Die-off and Lagoon Study

Table 3. Monthly average influent and efluent daily flowrate in liters per day.

Month	Influent l/day	Effluent l/day
January 1975	408,947	51,729
February	799,695	196,305
March	929,051	740,698
April	782,083	503,394
May	638,222	335,014
June	665,630	455,173
July	876,413	296,612
August	957,753	265,310
September	862,348	296,490
October	1,029,645	294,954
November	489,139	530,831
December	253,667	225,192
January 1976	325,817	176,703
Average	693,724	336,031

liters x 3.785 = gallons

Table 4. Residence times.

Pond No.	Dye Study Date	Theoretical Residence Time	Actual Residence Time	Dye Dispersion Chart Figure No.
1	12-8-75/1-30-76	77.2 days	35.1 days	A-1
2	5-10-76/6-23-76	9.1 days	8.5 days	A-2
3	6-23-76/7-31-76	9.7 days	6.7 days	A-3
4	5-10-76/6-23-76	10.4 days	8.1 days	A-4
5	6-23-76/7-31-76	11.3 days	9.0 days	A-5
6	5-10-76/6-23-76	12.2 days	8.8 days	A-6
7	6-23-76/7-31-76	16.4 days	12.1 days	A-7

Note: Theoretical residence times calculated from flows for the same periods during the year 1975. Correction was made for evaporation effect on flows in the latter ponds.

Correction was added to each pond for evaporation.

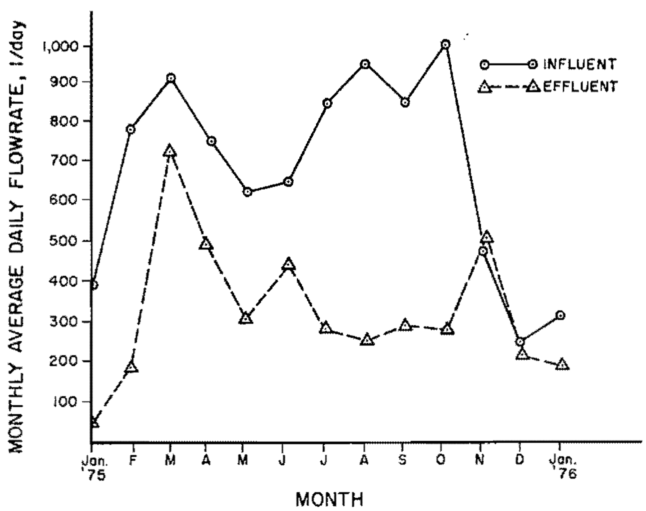

Figure 2. Monthly average influent and effluent daily flowrate.

groundwater infiltration from agricultural irrigation and stormwater inflow into the sewage collection system. The yearly average effluent flowrate from the system is only 48.4 percent of the yearly average influent to the system.

Biochemical oxygen demand (BOD_5)

The monthly average biochemical oxygen demand (BOD_5) performance for the lagoon system is reported in Table 5 and illustrated in Figure 3 for each pond in the system.

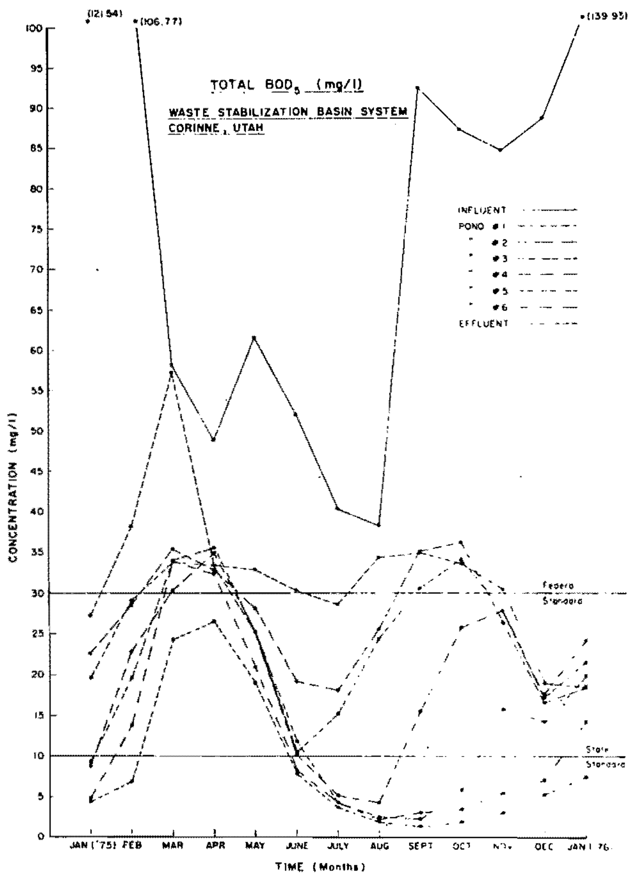

Figure 3. Monthly average biochemical oxygen demand (BOD_5) performance of the Corinne Waste Stabilization Lagoon System.

The influent monthly average BOD_5 ranged from a maximum of 139.93 mg/l to a minimum of 40.26 mg/l, with a mean of 74.62 mg/l. During the winter period, when the influent flowrate was low, 2.5 x 10^5 1/day (0.067 MGD), the BOD_5 concentration of the influent was high (121 mg/l). The summer period was characterized by a diluted influent BOD_5. This trend of summer dilution and winter concentration of the influent flow was evidenced in many of the parameters.

Spring thaw was accompanied by hydraulic mixing in all of the ponds. This period, February to May, was characterized by rising influent flows with a corresponding dilution of the influent BOD_5, and a resuspension of winter-settled organic materials by the overturning pond water. The monthly average effluent BOD_5 concentration of Pond Number 1 was much higher (57.3 mg/l) than was the monthly average effluent BOD_5 concentration of the other ponds (see Figure 3). The other six ponds had average monthly effluent BOD_5 concentrations ranging from 26.5 mg/l (Pond Number 7) to 35.5 mg/l (Pond Number 5). Summer temperatures and the uptake of spring-mixing-generated organics supported an increase in treatment efficiency resulting in a marked drop in the BOD_5 concentrations of all the pond effluents except the effluent from Pond Number 1. Late summer and the end of the irrigation season caused the influent to become more concentrated. Following this concentrating was an increase in the BOD_5 level of the effluents from all of the ponds except Pond Numbers 6 and 7. Effluent BOD_5 concentration from all the ponds continued to rise throughout the fall until the colder weather caused the ponds to destratify and mix again. This fall overturn increased the effluent BOD_5 rise which was then followed by a sharp decline in effluent BOD_5 concentrations (see Figure 3).

Statistical analysis of the data indicated that the effluent BOD_5 concentrations from Pond Numbers 5, 6, and 7 were statistically alike. This would indicate that no significant difference in BOD_5 treatment is achieved by Pond Numbers 6 and 7 over that accomplished by Pond Number 5. However, the actual yearly mean concentration for Pond Number 7 was 8.91 mg/l, a decrease of 6.82 mg/l over the 15.73 mg/l yearly mean concentration for Pond Number 5. Also, Pond Number 7 was the only pond which remained below the federal standard (30 mg/l) throughout the entire year.

The only pond to comply with the Utah State standard of 10 mg/l was Pond Number 7. However, it exceeded the

Table 5. Monthly average biochemical oxygen demand (BOD_5)of the Corinne Waste Stabilization Lagoon System.

TOTAL BOD(5) (MG/L)		MONTHLY AVERAGES						
MONTH	INFLUENT	POND 1	POND 2	POND 3	POND 4	POND 5	POND 6	EFFLUENT
JAN-75	121.54	27.23	22.52	19.65	8.78	9.23	4.79	4.27
FEB	106.77	38.09	28.53	29.07	22.30	19.68	13.88	6.87
MAR.	58.12	57.33	35.34	33.72	33.23	33.98	33.87	24.27
APR.	48.86	33.46	32.80	33.39	34.97	35.55	32.33	26.53
MAY	61.52	32.82	28.05	25.27	25.25	25.09	20.97	19.00
JUNE	51.82	30.22	19.41	10.49	10.10	11.85	8.23	7.91
JULY	40.26	28.60	18.07	13.21	5.15	4.26	4.27	3.75
AUG.	38.21	34.23	25.59	24.43	4.35	2.52	2.34	1.98
SEP.	92.48	34.94	35.17	30.60	15.56	2.39	3.11	1.40
OCT.	87.16	33.56	36.20	34.18	25.77	5.83	3.41	1.89
NOV.	84.65	30.46	26.40	27.74	27.76	15.82	5.47	3.12
DEC.	88.42	19.00	16.69	17.08	17.62	14.34	7.04	5.39
JAN-76	139.93	18.62	18.59	21.55	24.11	19.86	14.29	7.51

Utah State standard during the spring overturn period for nearly four months (February to May 1975). All of the other ponds exceeded the state standard for longer periods of time.

The BOD_5 treatment efficiency ranged from 47.7 percent during April 1975 to 97.8 percent during October 1975. The yearly average treatment efficiency was 88.06 percent. The system failed to satisfy the 85 percent removal requirement of PL 92-500 during four of the 13 months studied. However, as discussed earlier, the system never exceeded a final effluent BOD_5 concentration of 30 mg/l. The ability of the system to satisfy the 85 percent removal requirement appears to be more dependent on the raw sewage influent BOD_5 concentration than on the final system effluent BOD_5 concentration.

The organic loading on the primary cell (Pond Number 1) of the system is shown in Table 6. The organic load ranged from 15.1 kg BOD_5/ha/day (13.4 lbs BOD_5/acre/day) to 60.2 kg BOD_5/ha/day (53.6 lbs BOD_5/acre/day). The yearly average organic load to the primary cell was 33.6 kg BOD_5/ha/day (29.9 lbs BOD_5/acre/day). The system was designed for an organic load of 36.2 kg BOD_5/ha/day (32.2 lbs BOD_5/acre/day). On a yearly basis the system was not organically loaded beyond the design capacity. However, during three of the 13 months studied, the organic loading rate did exceed the design capacity. Each of these three months (February, September, October) were during periods of the year when the lagoon system should have been less able to assimilate the overload. However, during each of these months the final effluent BOD_5 concentrations were less than 10 mg/l. Thus, it appears that the 36.2 kg BOD_5/ha/day (32.2 lbs BOD_5/acre/day) used to design the system was at least adequate and may be somewhat conservative.

Table 6. Average monthly organic loading rate on the primary cell (Pond Number 1) of the Corine Waste Stabilization System.

Month	Average Organic Loading (kg/ha/day)
January 1975	33.4
February	57.3
March	36.2
April	25.6
May	26.3
June	23.1
July	23.6
August	24.6
September	53.5
October	60.2
November	27.8
December	15.1
January 1976	30.6
Yearly Average	33.6

kg/ha/day x 0.89 = lbs/acre/day

In summary, the BOD_5 influent to the Corinne system was effectively reduced to levels acceptable to the federal standard (i.e. 30 mg/l), and to the state standard (10 mg/l) the majority of the time. Effluent BOD_5 levels were subject to mixing conditions both in spring and fall causing all pond effluents except Pond Number 7 effluent to reach unacceptable levels. Winter effluent BOD_5 concentrations were acceptable with respect to the federal and state standards. These concentrations were also lower than effluent BOD_5 concentrations during other portions of the year. Finally, statistical analysis showed little improvement in BOD_5 removal beyond that attained by Pond Number 5.

Suspended solids (SS)

The monthly average suspended solids concentration for the raw sewage influent and the effluent for each pond in the system is reported in Table 7 and illustrated in Figure 4.

The monthly average raw sewage influent suspended solids (SS) concentration ranged from 39.12 mg/l in August to 119.76 mg/l in January 1976 with a yearly average of 71.3 mg/l. The raw sewage influent suspended solids concentration was closely related to the raw sewage influent flowrate (see Figure 2). As the raw sewage influent flowrate increased the influent raw sewage suspended solids concentration tended to decrease. In general, the raw sewage influent suspended solids concentration was less than expected for a typical domestic sewage.

The final effluent monthly average suspended solids concentration (Pond Number 7) varied from 2.53 mg/l in September to 179.24 mg/l in April with a yearly average concentration of 33.69 mg/l. The yearly average final effluent suspended solids concentration (i.e. 30.2 mg/l) is somewhat misleading in that during eight of the 13 months studied the monthly average final effluent suspended

Table 7. Monthly average suspended solids performance of each pond in the Corinne Waste Stabilization Lagoon System.

SUSPENDED SOLIDS (MG/L) MONTHLY AVERAGES

MONTH	INFLUENT	POND 1	POND 2	POND 3	POND 4	POND 5	POND 6	EFFLUENT
JAN-75	91.53	46.10	27.83	34.23	14.74	8.71	13.38	9.15
FEB	72.82	61.28	49.76	54.49	49.21	30.75	20.53	12.53
MAR.	75.99	55.25	53.22	57.66	58.13	63.66	101.51	73.69
APR.	55.79	68.96	80.89	87.67	106.75	139.17	152.03	179.24
MAY	73.07	74.12	72.17	63.33	70.83	64.03	65.01	64.93
JUNE	61.89	55.81	50.04	15.30	13.71	24.37	12.80	9.36
JULY	42.58	64.74	40.36	26.14	5.99	7.64	5.46	3.92
AUG.	39.12	86.76	70.57	51.55	5.23	5.67	3.47	3.46
SEP.	44.70	95.38	100.94	74.36	26.73	4.84	4.96	2.53
OCT.	106.97	85.26	84.33	68.60	39.22	9.18	9.29	3.51
NOV.	78.21	66.37	57.95	53.95	44.77	23.00	16.82	5.26
DEC.	65.06	26.76	28.52	25.13	34.18	24.47	12.80	9.02
JAN-76	119.76	18.10	20.63	22.51	26.65	32.65	20.90	16.07

solids concentration never exceeded 10 mg/l and in fact, during only three of the 13 months studied did the monthly average final effluent suspended solids concentration exceed 30 mg/l (see Figure 4 and Table 7).

The peak monthly average final effluent suspended solids concentration (179.24 mg/l in April, 1975) occurred during the spring overturn. The effluent suspended solids concentration for all of the ponds in the system increased significantly during this period. Although the same phenomenon occurred during the fall overturn period (November), the increase in effluent suspended solids concentration was not as dramatic as it was during the spring overturn. The influence of the fall overturn was found in the effluent suspended solids concentrations measured by Pond Numbers 1, 2, 3, and 4. The effluent from Pond Number 1 had the highest fall concentration (95.38 mg/l). A sharp decline was seen in the effluent from Pond Numbers 1 through 4 at the onset of cold weather. The suspended solids concentrations plotted in Figure 4 correspond very closely to the effluent BOD_5 concentrations plotted in Figure 3.

Statistical analysis indicated that there was no significant difference (95 percent level) in the effluent suspended solids concentration from Pond Numbers 4, 5, 6, and 7. Thus, statistically, no additional suspended solids removal occurred beyond Pond Number 4. However, inspection of Figure 4 clearly illustrates that Pond Numbers 5, 6, and 7 did provide meaningful suspended

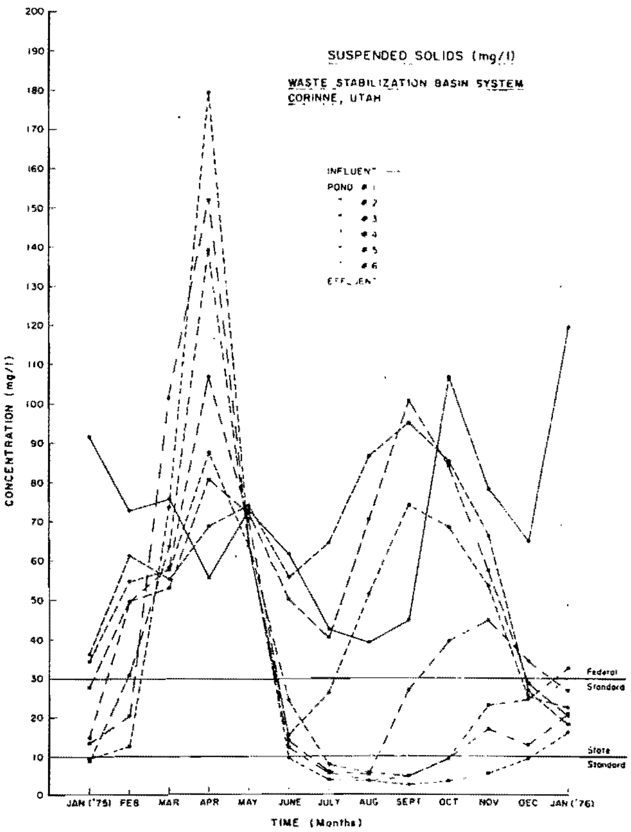

Figure 4. Monthly average suspended solids performance of each pond in the Corinne Waste Stabilization Lagoon System.

solids removal during September, October, November, and December. Thus, it appears that the additional ponds did provide a measure of protection during the fall overturn period.

The suspended solids performance of the system with respect to both federal (PL 92-500) and State of Utah requirements is illustrated in Figure 4 and reported in Table 8. The final effluent suspended solids concentration was 30.2 mg/l which is slightly in violation of the federal standard of 30.0 mg/l. However, the federal standard is based on the monthly average effluent suspended solids concentration and as reported earlier during only three of the 13 months studied was the monthly average effluent suspended solids concentration greater than 30.0 mg/l. Table 8 indicates the suspended solids removal efficiency of the system. The yearly average suspended solids removal efficiency was only 51.47 percent. However, the system failed to remove 85 percent of the raw sewage influent suspended solids concentration during only five of the 13 months studied. The system satisfied the State of Utah's effluent suspended solids standard of 10 mg/l during eight of the 13 months studied. During the summer months (June to September, 1975) of peak algal activity, the final effluent suspended solids concentration averaged 3.3 mg/l. This indicates that algae were not a problem in satisfying discharge requirements during the summer months for this particular system.

Fecal coliform

The results of 12 months of monitoring the fecal coliform die-off in the waste stabilization lagoon system located in Corinne, Utah are shown in Figure 5.

In general, the fecal coliform number in each of the 3 lagoons plotted stayed within two log scales throughout the year. The most noticeable exceptions occurred in January and February under ice cover when the numbers increased by one order of magnitude in each pond. In June, the numbers decreased by one log scale. These effects were most noticeable in the secondary and tertiary ponds. These ponds received relatively stable flows compared to the primary pond which was subject to large variations in flow and influent fecal coliform concentrations.

The two overturn periods were reflected in the fecal coliform counts during March and October. During the

Table 8. Treatment efficiency of the Corinne Waste Stabilization Lagoon System with respect to suspended solids (SS).

Month	Monthly Average SS Influent (mg/l)	Monthly Average SS Effluent (mg/l)	Treatment Efficiency (%)
January 1975	91.53	9.13	90.0
February	72.82	12.53	82.7
March	75.99	73.69	3.0
April	55.79	179.24	-221.3
May	73.07	64.93	11.1
June	61.89	9.36	84.8
July	42.58	3.92	90.8
August	39.12	3.46	91.2
September	44.70	2.53	94.3
October	106.97	3.51	96.7
November	78.21	5.26	93.3
December	65.06	9.02	86.1
January 1976	119.76	16.07	86.6

first overturn period in early March, fecal coliform counts increased. The period between overturns showed decreased numbers of fecal coliforms with the count increasing during the second overturn period and dropping again afterwards. Increased counts during overturn could be the result of the circulation of organisms which had settled. Increased nutrient concentrations during overturn could also decrease die-off by providing substrates for these organisms. The effects of fall overturn are not readily apparent in the fecal coliform data collected for Pond Numbers 4, 5, 6, and 7.

The mean monthly fecal coliform bacteria concentration for the raw sewage influent and the effluent from each pond in the system is illustrated in Figure 5. At no time during the 13 month study period did the final effluent fecal coliform concentration exceed the federal effluent fecal coliform standard of 200 colonies/100 ml, or the State of Utah effluent fecal coliform standard of 20 colonies/100 ml. As Figure 5 indicates, both the federal and State of Utah effluent discharge requirements for fecal coliform were satisfied after the wastewater had passed through the third pond in the system. The coliform removal in the Corinne system is due solely to natural forces since disinfection of the effluent is not practical.

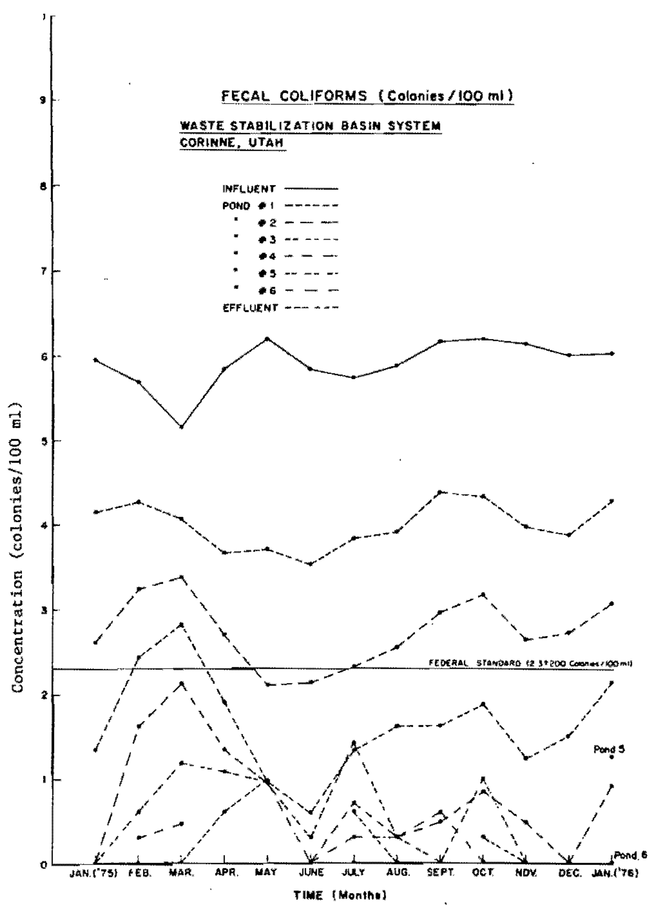

Figure 5. Mean monthly fecal coliform bacteria concentrations of the raw sewage influent and effluent from each pond in the Corinne Waste Stabilization Lagoon System.

Fecal coliform die-off in the 7 cell Corinne system appears to be a function of detention time or cell number. The large primary cell reduces the fecal coliforms by two orders or magnitude. Each of the smaller succeeding cells reduce the count by one order of magnitude. The last 3 cells showed very few fecal coliforms consistently throughout the year (average<20 colonies/100 ml).

Performance Summary

General

All of the parameters examined for the Corinne system were reduced in concentration by the lagoon system. Reduction percentages for the parameters are listed in Table 9. Also included is a recording of the statistically suggested most effective pond number and the corresponding percent reduction in concentration.

On a yearly basis the Corinne waste stabilization system provided 88 percent removal of the incoming BOD_5, 51 percent removal of the suspended solids and 99.99 percent removal of the fecal coliforms.

Table 9 also contains the statistical recommended number of ponds in series for effective treatment at the 95 percent confidence level. It can easily be recognized that the maximum number of effective ponds depends on the parameter in question.

All of the parameters examined at the Corinne system showed either direct or indirect affect from the hydraulic mixing forces during the spring and fall of the year. Winter altered the operation of the lagoons and some build occurred in the BOD_5 and SS. However, despite winter conditions effluent stabilization continued.

Most of the summer period was characterized by low levels for all of the parameters. Fall produced a second hydraulic mixing (fall overturn). Lower temperatures and light levels served to curtail the fall peak and forced an alteration of activity to a winter state. Ice formation on the surface of the lagoons in November and December, 1975, sealed the lagoons until spring thaw and the beginning of another yearly cycle.

In general, the raw sewage influent suspended solids concentrations was less than expected for a typical domestic sewage.

Satisfying federal and State of Utah discharge standards

The biochemical oxygen demand (BOD_5) concentration of the final effluent from the Corinne Waste

Table 9. Performance summary and recommended number of ponds to achieve federal standards (PL 92-500) for the Corinne Waste Stabilizaion Lagoon System.

Parame.er	Influent mg/l	Final Effluent mg/l	Total Reduction %	Number of Ponds Recommended	Measured Quality From Recommended Number of Ponds: Effluent mg/l (yearly average)	Measured Quality From Recommended Number of Ponds: Reduction % (yearly average)
BOD	74.62	8.91	88.06	5	15.74	78.92
SS	69.42	33.69	51.47	5	36.49	47.44
FC[a]	928673.	2.40	99.99	4	17.40	99.99

[a]FC in colonies/100 ml.

Stabilization Lagoon System never exceeded the federal standard of a 30 day arithmetic mean concentration of less than 30.0 mg/l, or the 7 day arithmetic mean concentration of less than 45.0 mg/l. The monthly average final effluent BOD_5 ranged 1.40 mg/l to 26.53 mg/l. However, it did not satisfy the requirement for 85 percent BOD_5 removal 4 of the 13 months studied. The system satisfied the State of Utah BOD_5 final effluent standard of less than 10 mg/l 10 of the 13 months studied.

The suspended solids concentration of the final effluent from the system exceeded the federal standard of a 30 day arithmetic mean concentration of less than 30.0 mg/l 3 of the 13 months studied. The federal standard requiring a final effluent 7 day suspended solids concentration of less than 45.0 mg/l was consistently exceeded during the spring overturn (i.e. March, April, May). However, after the spring overturn period, the 7 days studied was not exceeded. The system satisfied the 85 percent removal requirement of the federal standard 8 of the 13 months studied. In addition, it satisfied the State of Utah 30 day average effluent suspended solids requirement of less than 10.0 mg/l 8 of the 13 months studied.

The system never exceeded the federal effluent discharge fecal coliform bacteria standard of less than 200 colonies/100 ml during the entire study, nor did it exceed the same effluent standard for the State of Utah even though disinfection was never practiced. In addition, the system never exceeded the State of Utah's total coliform bacteria studied of 2000 colonies/100 ml during the entire study.

CONCLUSIONS

Based on the results of 13 months of the performance of the Corinne Waste Stabilization Lagon System, Corinne, Utah, the following conclusions can be made.

1. The yearly average daily hydraulic influent flowrate of 693,724 liters/day (183,282 gallons/day) exceeded the design hydraulic flowrate by 2.62 times.

2. The actual hydraulic residence time in the system was 88.3 days. This was 49.1 percent less than the 180 day hydraulic residence time required by the State of Utah.

3. The organic strength of the raw influent sewage was less than a typical domestic sewage. The monthly average influent biochemical oxygen demand (BOD_5) concentration to the system ranged from 40.26 mg/l to 139.93 mg/l with a yearly mean concentration of 74.62 mg/l.

4. On a yearly basis the system was not organically overloaded. However, the organic load did exceed the design organic load of 36.2 kg BOD_5/ha/day (29.9 lbs BOD_5/acre/day) several times during the study. The average monthly organic loading on the primary cell ranged from 15.1 kg BOD_5/ha/day (13.4 lbs BOD_5/acre/day) to 60.2 kg BOD_5/ha/day (53.6 lbs BOD_5/acre/day) with a yearly average of 33.6 kg BOD_5/ha/day (29.9 lbs BOD_5/acre/day).

5. The final effluent biochemical oxygen demand of the system never exceeded the Federal Secondary Treatment Standards. The monthly average final effluent biochemical oxygen demand (BOD_5) concentration ranged from 1.40 mg/l in August to 26.55 mg/l in April with a yearly average concentration of 8.91 mg/l.

6. The ability of the system to satisfy the 85 percent biochemical oxygen demand (BOD_5) removal requirement of the Federal Secondary Treatment Standards appears to be more a function of the influent BOD_5 concentration rather than the effluent BOD_5 concentration. The system failed to satisfy the 85 percent biochemical oxygen demand (BOD_5) removal requirement of the Federal Secondary Treatment Standards four of the 13 months studied.

7. The final effluent biochemical oxygen demand (BOD_5) concentration satisfied the State of Utah requirement of less than 10.0 mg/l, ten of the 13 months studied.

8. Statistical analysis indicated that no significant (95 percent level) biochemical oxygen demand (BOD_5) removal occurred beyond the fifth pond in the seven pond series.

9. The raw sewage influent suspended solids concentration was less than that expected for a typical domestic sewage. Monthly average raw sewage influent suspended solids concentrations ranged from 39.12 mg/l in August to 119.76 mg/l in January with a yearly average concentrations of 71.3 mg/l.

10. The final effluent monthly average suspended solids concentration of the system satisfied the Federal Secondary Treatment Standards ten of the 13 months studied. The monthly average final effluent suspended solids concentration ranged from 2.53 mg/l in September to 179.24 mg/l in April with a yearly average concentration of 33.69 mg/l.

11. The ability of the system to satisfy the 85 percent removal of suspended solids requirement of the Federal Secondary Treatment Standards appears to be more a function of the influent suspended solids concentration rather than the effluent suspended solids concentration. The system exceeded this requirement five of the 13 months studied.

12. The final effluent monthly average suspended solids concentrations satisfied the State of Utah effluent suspended solids discharge requirement of less than 10.0 mg/l eight of the 13 months studied.

13. Statistical analysis indicated that no significant (95 percent level) removal of suspended

solids occurred beyond the fifth pond in the seven pond series.

14. The system was very efficient at removing fecal coliform bacteria even though disinfection was not practiced. At no time did the final effluent exceed the Federal Secondary Treatment Standards or the State of Utah discharge requirement for fecal coliform bacteria. Although fecal coliform bacteria removal continued throughout the system, the monthly average effluent fecal coliform bacteria concentration of the fourth pond never exceeded 200 colonies/100 ml.

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