January 1979

Proceedings of the Utah Water Pollution Control Association, 1979 Annual Meeting

Kathleen E. Bayn

PROCEEDINGS OF THE

UTAH WATER POLLUTION CONTROL ASSOCIATION

1979 ANNUAL MEETING

April 12-13, 1979
Salt Lake City, Utah

Compiled by
James H. Reynolds and Kathleen E. Bayn

College of Engineering
Utah State University
Logan, Utah 84322

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UTAH WATER POLLUTION CONTROL ASSOCIATION

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President Elect, *James E. Charnholm*
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College of Engineering
Utah State University
Logan, Utah 84322

April 1979

PREFACE

This volume is the third published proceedings of the Utah Water Pollution Control Association's Annual Meeting. The Technical Program of the 1979 Annual Meeting was divided into four separate sessions. The opening session (Session I) provided an overview and direction for the entire meeting. Session II-A: Plant Safety, and Session III-A: Plant Safety, centered on the need for establishing appropriate safety procedures at wastewater installations. Session II-B: Selected Topics, was devoted to technical presentations of special merit and interest to the local environmental engineering profession. Session III-B: Interim Upgrading, considered various alternatives for upgrading wastewater facilities on an interim basis. Session IV: Energy Conservation, included a panel discussion on various methods for reducing energy consumption at wastewater treatment facilities.

For the second year, a Fellowship Breakfast was held the second day of the Annual Meeting. Although not included in these Proceedings, Mr. Hal Goble, Goble Sampson Associates and Mr. Jim Marsh, Assistant Basketball Coach at the University of Utah, were featured speakers at this breakfast.

The Technical Program Committee is deeply grateful to Mrs. Kathy Bayn and Mrs. Kathy Eck for their persistence, dedication, and technical skill in preparing the manuscript of these Proceedings.

James H. Reynolds
Program Chairman

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1958-59	Blaine Liston[1]	Provo		
1959-60	Lyle S. Ford	Salt Lake City		
1960-61	Emil Meyer	Salt Lake City		
1961-62	Harold A. Linke, Jr	Salt Lake City		
1962-63	Howard B. Kelly	Brigham City		
1963-64	Maurice N. McKendrick	Salt Lake City		
1964-65	William W. Anderton	Salt Lake City		
1965-66	Melvin D. Iverson	Salt Lake City		
1966-67	J. Linden Chapman	Heber City		
1967-68	Ludvig D. Olsen	Bountiful		
1968-69	Robert R. Emerson	Salt Lake City		
1969-70	Kenner B. Clayton	Provo		
1970-71	Calvin K. Sudweeks	Salt Lake City	Norman B. Jones	Logan
1971 [2]	Norman B. Jones	Logan	Clifford N. Stutz	Salt Lake City
1971-72	Clifford N. Stutz	Salt Lake City	Donald E. Burns	Salt Lake City
1972-73	Donald E. Burns	Salt Lake City	Floyd J. Erickson	Syracuse
1973-74	Floyd J. Erickson	Syracuse	John R. Call	Salt Lake City
1974-75	John R. Call	Salt Lake City	Lloyd S. Mulvey	Salt Lake City
1975-76	Lloyd S. Mulvey	Salt Lake City	E. Joe Middlebrooks	Logan
1976-77	E. Joe Middlebrooks	Logan	Garth R. Morgan	Salt Lake City
1977-78	Garth R. Morgan	Salt Lake City	LaVere B. Merritt	Provo
1978-79	LaVere B. Merritt	Provo	James E. Charnholm	Salt Lake City

	VICE-PRESIDENT		SECRETARY-TREASURER	
1957-58	Blaine Liston[1]	Provo	Howard M. Hurst	Salt Lake City
1958-59	Lyle S. Ford	Salt Lake City	Howard M. Hurst	Salt Lake City
1959-60	Emil Meyer	Salt Lake City	Howard M. Hurst	Salt Lake City
1960-61	Harold A. Linke, Jr	Salt Lake City	William W. Anderton	Salt Lake City
1961-62	Howard B. Kelly	Brigham City	William W. Anderton	Salt Lake City
1962-63	Maurice N. McKendrick	Salt Lake City	Roscoe W. Godfrey	Salt Lake City
[3]1963-64	Keith Maloney[1]	Tooele	Roscoe W. Godfrey	Salt Lake City
1964-65	Melvin D. Iverson	Salt Lake City	Ludvig D. Olsen	Bountiful
1965-66	J. Linden Chapman	Heber City	Ludvig D. Olsen	Bountiful
1966-67	Ludvig D. Olsen	Bountiful	Robert R. Emerson	Salt Lake City
1967-68	Robert R. Emerson	Salt Lake City	Lloyd S. Mulvey	Salt Lake City
1968-69	Kenner B. Clayton	Provo	Lloyd S. Mulvey	Salt Lake City
1969-70	Calvin K. Sudweeks	Salt Lake City	Clyde M. Hopkins	Salt Lake City
1970-71	Walter W. Brock	Salt Lake City	Clyde M. Hopkins	Salt Lake City
1971 [2]	Jack H. Petersen	Salt Lake City	Ted C. Earl	Brigham City
1971-72	Floyd J. Erickson	Syracuse	Lyle S. Ford	Salt Lake City
1972-73	Richard C. Hansen	Salt Lake City	David B. Brinton	Salt Lake City
1973-74	Lloyd S. Mulvey	Salt Lake City	David B. Brinton	Salt Lake City
1974-75	E. Joe Middlebrooks	Logan	Norman B. Jones	Logan
1975-76	Jack H. Petersen	Salt Lake City	Norman B. Jones	Logan
1976-77	LaVere B. Merritt	Provo	Norman B. Jones	Logan
1977-78	Glenn W. Turpin	Murray	Jay B. Pitkin	Salt Lake City
1978-79	Willard E. DeVault	Salt Lake City	Jay B. Pitkin	Salt Lake City

[1]Deceased
[2]Feb. - Sept. 1971
[3]William W. Anderton elected 4/19/63 to serve unexpired term of Keith W. Maloney

FEDERATION DIRECTORS

1957-60	Lynn M. Thatcher	Salt Lake City
1960-63	Howard M. Hurst	Salt Lake City
1963-66	Marland L. Davidson	Ogden
1966-69	Harold A. Linke, Jr	Salt Lake City
1969-72	Melvin D. Iverson	Salt Lake City
1972-75	Grant K. Borg	Salt Lake City
1975-78	Roscoe W. Godfrey	Salt Lake City
1978-81	E. Joe Middlebrooks	Logan

UTAH WATER POLLUTION CONTROL ASSOCIATION DIRECTORS

1965-66

Lyle S. Ford	Salt Lake City
Robert R. Emerson	Salt Lake City
Kenner B. Clayton	Provo

1966-67

Lyle S. Ford	Salt Lake City
Floyd J. Erickson	Syracuse
Kenner B. Clayton	Provo

1967-68

Norman B. Jones	Logan
Floyd J. Erickson	Syracuse
William W. Anderton	Salt Lake City

1968-69

Norman B. Jones	Logan
Robert L. Cook	Salt Lake City
Phil S. York	Salt Lake City

1969-70

Jack W. Pratt	Salt Lake City
Robert L. Cook	Salt Lake City
Walter W. Brock	Salt Lake City

1970-71

Jack W. Pratt	Salt Lake City
Donald E. Burns	Salt Lake City
Howard B. Kelly	Brigham City

*1971

Clyde W. Montgomery	Heber City
Donald E. Burns	Salt Lake City
James Rooney	Salt Lake City

1971-72

Clyde W. Montgomery	Heber City
Glenn W. Turpin	Salt Lake City
Richard C. Hansen	Salt Lake City

1972-73

E. Joe Middlebrooks	Logan
Glenn W. Turpin	Salt Lake City
John R. Call	Salt Lake City

1973-74

E. Joe Middlebrooks	Logan
LaVere B. Merritt	Provo
Emil Meyer	Salt Lake City

1974-75

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LaVere B. Merritt	Provo
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1975-76

Garth R. Morgan	Salt Lake City
LaVere B. Merritt	Provo
David B. Brinton	Salt Lake City

1976-77

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Dean Wheadon	Provo
David B. Brinton	Salt Lake City

1977-78

Gale H. Larson	Logan
James E. Charnholm	Salt Lake City
Willard DeVault	Salt Lake City

1978-79

James Faulkner	Murray
John J. Wheelwright	Woods Cross
William A. Luce	Salt Lake City

*Feb. - Sept. 1971

WATER POLLUTION CONTROL FEDERATION AWARDS

1968 - Philip F. Morgan Medal - Certificate of Merit
Marland L. Davidson - Ogden

1969 - Harrison Prescott Eddy Medal
E. Joe Middlebrooks - Logan

1971 - Quarter Century Operator's Club
Clyde M. Hopkins - Salt Lake City

1975 - Membership Increase - $100 Cash Award
Utah Water Pollution Control Association (62%)

1977 - Life Member
Clifford N. Stutz - Salt Lake City - (Joined 1934)

WATER POLLUTION CONTROL FEDERATION
AWARDS

ARTHUR SIDNEY BEDELL AWARD

1960 - Lynn M. Thatcher . Salt Lake City
1963 - Grant K. Borg . . Salt Lake City
1967 - Howard M. Hurst . . Salt Lake City
1970 - Lyle S. Ford . . . Salt Lake City
1973 - Calvin K. Sudweeks . Salt Lake City
1976 - Norman B. Jones . . . . . Logan

WILLIAM D. HATFIELD AWARD

1960 - Emil Meyer . . . . Salt Lake City
1963 - Marland L. Davidson . . . . Ogden
1965 - Ludvig D. Olsen . . . Bountiful
1968 - Floyd J. Erickson . . . Syracuse
1971 - Lloyd S. Mulvey . . Salt Lake City
1974 - Leslie E. Roberts . . . . Provo
1977 - Garland J. Mayne . . Pleasant Grove

OUTSTANDING OPERATOR AWARD

	Plants under 5 MGD Design Capacity	Plants over 5 MGD Design Capacity
1973	Glenn W. Turpin Murray City, UT	
1974	Clyde W. Montgomery Heber City, UT	Willard E. DeVault Salt Lake City, UT
1975	James E. Wade St. George, UT	Wallace A. Bowden S.L.C. Suburban Sanitary Dist. #1 Salt Lake County, UT
1976	Gary C. Hales South Davis Sewer Improvement Dist. Woods Cross, UT	Rodney D. Ivie Granger-Hunter Sewer Improvement Dist. Salt Lake County, UT
1977	Donald E. Stark South Davis Sewer Improvement Dist. Woods Cross, UT	Gary D. Holmes Cottonwood Sanitary Dist. Salt Lake County, UT

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Glenn W. Turpin
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[1] - Honorary
[2] - Deceased

NOTE: Above list may be incomplete. Please report omissions to the Association Secretary.

WATER POLLUTION CONTROL FEDERATION

AWARDS

BEST OPERATED PLANT AWARD

Under 5 MGD Design Capacity		Over 5 MGD Design Capacity
South Davis (South Plant) Sewer Improvement Dist. Ludvig D. Olsen, Supt. Woods Cross, UT	1973	
South Davis (South Plant) Sewer Improvement Dist. Ludvig D. Olsen, Supt. Woods Cross, UT	1974	Central Weber Sewer Improvement Dist. Marland L. Davidson, Supt. Ogden, UT
South Salt Lake City Lloyd S. Mulvey, Supt. So. Salt Lake, UT	1975	North Davis Sewer Improvement Dist. Floyd J. Erickson, Mgr. Syracuse, UT
South Salt Lake City Lloyd S. Mulvey, Supt. So. Salt Lake, UT	1976	South Davis, (North Plant) Sewer Improvement Dist. Ludvig D. Olsen, Supt. Woods Cross, UT
South Davis (South Plant) Sewer Improvement Dist. Woods Cross, UT	1977	Central Weber Sewer Improvement Dist. Ogden, UT

OUTSTANDING PLANT SAFETY AWARD

1973

Central Weber
Sewer Improvement Dist.
M. L. Davidson, Supt.
Ogden, UT

1974

Central Weber
Sewer Improvement Dist.
M. L. Davidson, Supt.
Ogden, UT

1975

(No Award)

1976

Salt Lake City
Water Reclamation Plant
Jack H. Petersen Supt.
Salt Lake City, UT

1977

Granger - Hunter
Sewer and Water Improvement Dist.
Gordon Beals, Supt.
Salt Lake County, UT

OUTSTANDING INDUSTRIAL WASTEWATER MANAGEMENT

1974

Phillips Petroleum Co.
Henry H. Lee, Mgr.
Woods Cross, UT

1975

(No Award)

1976

Metals Processing, Inc.
Clark Romney, Pres.
Salt Lake City, UT

Cheveron Oil Co.
J. K. Murray, Refinery Mgr.
North Salt Lake, UT

1977

Pepperidge Farm, Inc.
Richmond, UT

PUBLIC SERVANT AWARD

1973

Hon. Frank E. Moss
U.S. Senator (D-Utah)
Salt Lake City, UT

1976

KUTV - Channel 2
Salt Lake City, UT

COLLECTION SYSTEM AWARD

1977

Under 5 MGD Capacity	Over 5 MGD Capacity
City of Murray Corp. Richard Buck Murray, UT	S.L.C. Suburban San. Dist. #1 Dean MacNeil Salt Lake City, UT

SEWERAGE FACILITIES CONSTRUCTION

*James D. Clise**

Under Utah law and the Federal PL 92-500, the State Division of Health and EPA share responsibility for the control of water pollution. A most important part of the responsibility deals with municipal sewage treatment plants. As the state's designated water pollution control agency, the Division of Health is also charged with Utah's responsibilities in the sewer facilities construction grants program. This puts us in a position of dealing on a day to day basis with EPA in matters relating to the design, funding, construction, and operation of municipal sewerage facilities.

Utah law establishes a Water Pollution Committee made up of a variety of citizens, with the responsibility of developing a state-wide program to control water pollution. The committee has the authority to adopt regulations; to establish water quality and effluent standards, to accept and administer federal grants; and to issue and enforce orders necessary to abate or control water pollution problems.

The committee is also authorized to review plans and issue permits for the construction or modification of any sewage treatment facility or effluent outfall line, and to control any action of a municipality which would increase the volume or strength of sewage effluent beyond that which was approved in the construction permit. It is the responsibility of the Division of Health to provide the staff to administer the programs and implement the regulations developed by the Water Pollution Committee.

Utah's Water Pollution Control Act was passed in 1953. Since that time the State has developed and implemented standards for water pollution control, and specifically for sewage treatment, which remain among the highest in the nation. The minimum acceptable level of treatment for discharged sewage effluent is polished secondary. The objective being to obtain a consistent effluent of at least secondary quality with the reliability associated with polishing filters. Utah standards require disinfection of sewage effluent, as opposed to the standards of many states which merely require reduction of coliform organisms to levels of stream standards. The objective being to assure, as far as possible, the destruction or inactivation of all human pathogens prior to discharging sewage effluents into waters of the state. Utah standards have for years addressed the reuse of wastewater, establishing a minimum level of secondary treatment for effluent to be reused for various irrigation purposes.

Is is worthy of note that Utah's progress in water pollution control has been achieved with very real participation of the public, through the many people who have served on the Committee and participated in the hundreds of hearings that have been held over the past 26 years since the Water Pollution Control Act was passed.

It is also somewhat ironic to realize that Utah has problems in our dealings with EPA in matters relating to water quality standards, required levels of sewage treatment, disinfection of sewage effluent, and public participation requirements.

Several of these problems, and the efforts being made to resolve them by the Division of Health and the Denver EPA office, are worthy of specific discussion.

As part of the Federal Water Pollution Control Program, EPA administers the sewerage facilities construction grants. Congress has been appropriating about $4.5 billion a year for sewer facilities construction, making this the biggest public works program in the country. Utah receives about $22 million a year in grants.

The money that has been allocated by Congress under the present Water Pollution Control Act has been for the specific purpose of providing secondary levels of treatment at all municipal sewage treatment plants. Levels of treatment higher than secondary can be funded only when such higher levels of treatment can be shown to be necessary to meet water quality standards. The polishing of secondary effluent as required by Utah regulations is not always necessary to meet established water quality standards. It is required to add a margin of reliability to secondary plants, and to provide for easier and more reliable disinfection of effluent for public health reasons. EPA has determined that polishing filters constitute treatment beyond secondary, and that they are not eligible for funding unless Utah can show them to be necessary for attainment of water quality standards.

This means that any municipal plant in Utah is required by State standards to provide polishing facilities which, in most instances, are not eligible for construction grant assistance.

The polished secondary problem is further complicated by an additional EPA policy decision. Every two years each state is required to develop a "Needs Inventory". This inventory is a list of all sewerage and sewage treatment needs, in various categories, and is used by Congress to develop the allocation formula to distribute grant funds among the states. The approximate costs needed to provide secondary treatment at all municipal facilities are placed in a category which received very high priority. Advanced treatment needs, or needs higher than secondary, are placed in a lower priority category. EPA has informed Utah that if we require polishing filters on a secondary treatment plant, the entire facility, not just the polishing filters, must be placed in the advanced waste treatment category. The possible result is that Utah's sewage treatment needs will receive lower priority than needs of the states with lower standards, and Utah will receive a smaller portion of the appropriated funds.

In order to resolve or reduce the problem, Utah regulations have been changed to allow approval of secondary sewage treatment without polishing filters in less critical areas at the discretion of the Water Pollution Committee. In other instances, the Committee will approve postponement of installation of polishing filters until construction grant funds are available for their construction. In these instances, secondary plants must be designed to facilitate the addition of filters in the future.

EPA has tentatively agreed to fund polishing filters in those instances where the Division of Health can substantiate their need. Hopefully, with the glimmer of consideration EPA is beginning to give to protection of public health, this consideration of need can be extended beyond the need to attain or protect water quality standards.

Another problem area has to do with chlorine residuals in discharged sewage effluents. EPA has no disinfection or bacterial standards. Discharged effluents merely cannot result in violation of the coliform standard assigned to the receiving waters. For public health protection, Utah requires disinfection of sewage prior to discharge. By definition, disinfection means the inactivation or destruction of all pathogenic organisms. Chlorination to this level results in two things: first, it results in no coliform count in the effluent because, theoretically at least, coliforms are killed more readily by chlorination than are some of the more resistant pathogens; second, it results in higher levels of chlorine residual being discharged in the effluent. Our recent concerns with the effect of discharged chlorine residuals upon fish and other biological forms, and the very low levels of allowable chlorine contained in NPDES permits, make the increased levels of chlorine needed for disinfection an undesirable result. The problem is further complicated by the omission of filters on secondary treatment plants, because the higher concentrations of organic matter in unpolished effluent necessitates higher chlorine concentrations to attain disinfection. Hopefully, the disinfection - chlorine residual problem has been resolved by the realization by EPA that disinfection is a legitimate water quality need as determined by the State Health Agency, and that resulting undesirable levels of chlorine justify funding of dechlorination facilities.

A third problem area resulting from conflicting State-EPA requirements has to do with land application of sewage. Utah has a long standing requirement that before sewage effluent can be used for irrigation purposes, it must be subjected to at least secondary treatment. Higher levels of treatment, including disinfection, are required before sewage effluent can be used in any location where the public may come in contact with it.

EPA is promoting consideration of land treatment as a less costly method of sewage treatment. EPA has no pretreatment requirements prior to applying sewage to land for treatment. Under proper conditions of isolation, primary effluent, or even raw sewage, could be applied to land for land treatment.

Utah's blanket requirement for secondary treatment prior to land application ran head-on into the EPA land treatment effort. The problem was further magnified when the EPA policy was changed, whereby land treatment is not only an accepted method of treatment, but is now required to be considered in every instance as part of the cost effective analyses.

Utah regulations have been changed to allow less than secondary pretreatment prior to land treatment, dependent upon the isolation of the facility, and at the discretion of the Water Pollution Committee.

One other area of concern, particularly to design engineers, is the EPA policy dealing with alternative methods or innovative technology. In addition to requiring consideration of land treatment as a less costly alternative to conventional treatment, EPA is encouraging development and utilization of other less costly innovative methods of sewage treatment.

The concept of alternative methods has far reaching possibilities. It not only addresses alternatives to our conventional sewage treatment processes - such as land treatment as opposed to mechanical plants - but also extends to sewage collection systems. In the past, Utah has funded very few sewage collection systems. Some states include funding for collectors in every sewerage project. Under the present EPA Policy, collector systems are eligible for funding only under very restrictive conditions. Collectors can be funded only to serve existing problem areas, where septic tanks have failed, and then only after a thorough survey of the area establishes the fact that properly installed and maintained on-site systems will not function properly. The policy encourages consideration of all alternatives to the collection and centralized treatment of sewage, including establishing sewer authorities to service on-site systems, including holding tanks.

The policy relating to innovative technology encourages development on innovative, less costly methods of collection and treatment. To add incentive to the use of innovative treatment processes, the EPA grant, which is normally 75 percent of the eligible cost, can be increased to 85 percent. Where innovative systems fail to provide the anticipated results, an EPA grant equalling 100 percent of the replacement cost is available.

The Utah Water Pollution Committee has assessed these EPA incentives as a possible invitation to disaster, and adopted a policy intended to assure that only sound, well engineered innovative systems will be installed.

Utah's regulations have required that substantial data from full scaled operating facilities be provided in the application for a permit to install any treatment facility other than those of conventional design. To address the EPA Policy, Utah's policy has been changed to allow innovative systems providing the proposal has been evaluated and recommended by a registered professional engineer. In making such an evaluation, the criteria must include the anticipated performance in meeting effluent and water quality criteria; the proposal's equivalence to previously accepted processes; and the owner's ability to finance, maintain and operate the system. The responsibility of demonstrating process viability rests with the person requesting approval.

In all of our dealings with EPA, particularly in the Construction Grants Program, the Division of Health finds itself in a mediator role, attempting to resolve the differences between recognized local needs and EPA nationwide policy. In fact, we see this as our most important function in our efforts to assist localities in obtaining needed sewerage capacity. Quite often the processing of grant applications by the state, and subsequent processing of proposals by EPA, results in a duplication of efforts and what would appear to be unnecessary delay in the construction of needed facilities. Congress has addressed this problem, and the Clean Water Act has been amended to allow more of the processing of construction grants to take place within the states with a reduction of activity at the federal level.

Under this amendment each state is authorized to set aside 2 percent, or $400,000 - whichever is greater, of the state's construction grants allocation to fund a construction grants management program. The set aside money comes into the state in the form of a program grant from EPA, paying for specific responsibilities of the construction grants program assumed by the state.

To implement the Construction Grants Management Program, the state must enter into a basic agreement with EPA outlining the organization capabilities of the state, and which is used as the basis for delegation agreements for specific parts of the construction grants program.

The Division of Health has developed the basic agreement and held the required public hearing on the proposal. It is anticipated the initial agreement will be signed by July 1, 1979, with full delegation of the Construction Grants Program to occur over the next 18-24 months.

It is anticipated that inflation reduction resulting from increased processing of grants will more than offset the costs associated with the Construction Grants Management Program.

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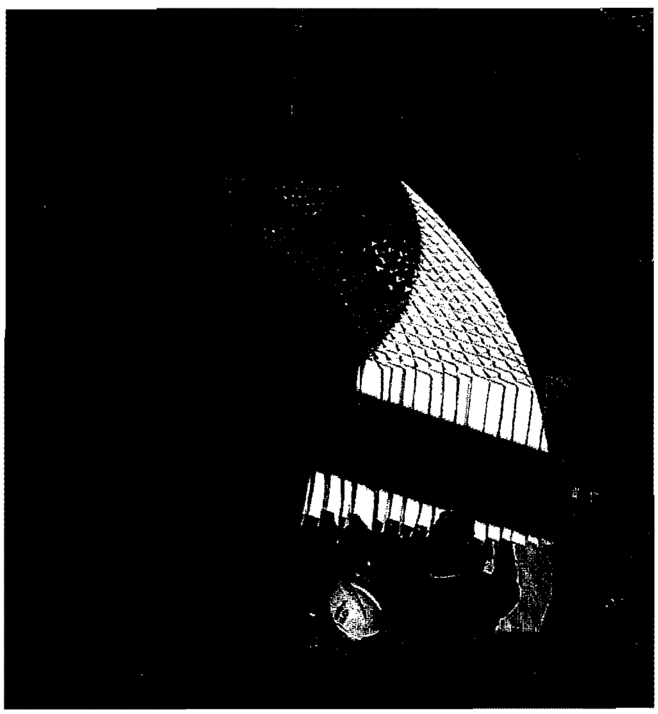
* *James D. Clise is the Deputy Director of Health, for the Environmental Health Services Branch, Salt Lake City, Utah.*

Tubes to fit



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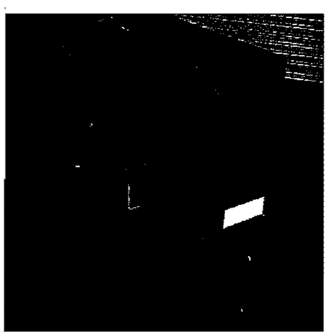
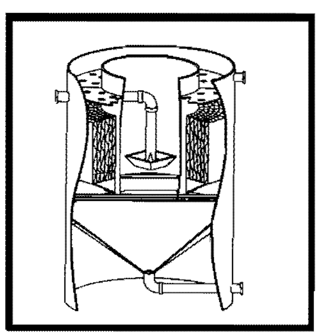
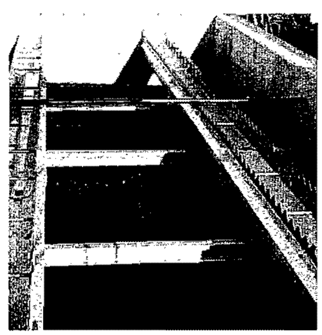


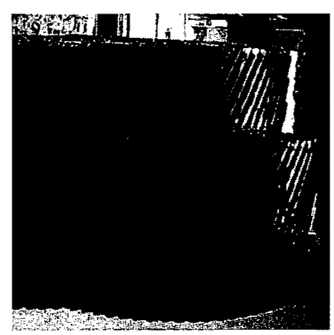
Pilot Tube Settler Unit



High-Rate Gravity Separator



Settling Basin With Tubes



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OVERVIEW OF EPA'S APPROACH TO INNOVATIVE AND ALTERNATIVE TECHNOLOGY ASSESSMENT

*James O. Brooks**

INTRODUCTION

Past experience has shown that the traditional method of providing public wastewater treatment facilities was to design gravity collection sewers that conveyed all wastewaters to a central, conventional type treatment plant. This approach has been a traditional method for several reasons:

- The systems are tried and proven.
- Usually more cost-effective because of economics of scale.
- Greater acceptance by government authorities, the general public, and engineers.

Congress in passing the "Federal Water Pollution Control Act Amendments of 1972" (Public Law 92-500) recognized the need to study and evaluate alternative waste management techniques such as land treatment or other designs which would allow, to the extent practicable, the application of technology at a later date which would provide for reclaiming or recycling of wastewater. Because of the emphasis placed on land treatment and subsequent EPA policy stressing the use of land treatment processes, land disposal of effluents and sludge from treatment processes has been on the increase in the United States.

On December 27, 1977, the "Clean Water Act of 1977" (Public Law 95-217) amending the FWPCA was signed by President Carter. This Act clearly established the intent of Congress to meet national water quality goals through greater use of wastewater treatment systems that provided for reclamation and reuse of wastewater and wastewater constituents and the more efficient use of energy and resources. In order to achieve these goals the Act provided for the identification and use of alternative and innovative wastewater treatment technology. To encourage government authorities, planners, and engineers to fully evaluate and utilize innovative and alternative wastewater treatment processes and techniques, the Act provided the following incentives:

- Financial assistance increased from 75 percent to 85 percent for grants made after September 30, 1978, and before October 1, 1981. This 10 percent increase must be obligated from set aside funds that total two percent of a State's allotment of funds for fiscal years 1979 and 1980, and three percent for fiscal year 1981. Also included in the above total percentages, an amount not less than one-half of one percent of the funds allotted to a State for each of the above fiscal years must be expended for 10 percent increase in the Federal share for construction of projects utilizing innovative processes or techniques.
- Treatment works that propose innovative or alternative technology with life cycle costs not exceeding by more than 15 percent the life cycle cost of the most cost effective alternative may be funded if determined to be in the public interest.
- States may give higher priority to innovative and alternative treatment projects.
- The Act authorizes that for each fiscal year beginning on or after October 1, 1978, four percent of a State's allotment be set aside to provide 75 percent grant assistance to communities having a population of 3500 or less. These funds must be expended for alternatives to conventional sewage treatment works. This set aside is mandatory for all states with a rural population of 25 percent or more as determined by the Bureau of Census.
- The Act provides an "insurance policy" for risks through a provision for 100 percent grant assistance for replacement or modification of treatment works which utilized innovative or alternative processes and techniques and are unable to meet design performance specifications unless failure can be attributed to negligence.

The Environmental Protection Agency has promulgated rules and regulations and developed guidelines designed to aid Federal and State authorities in achieving the goals of the Act.

INNOVATIVE WASTEWATER TREATMENT PROCESSES AND TECHNIQUES

Innovative wastewater treatment processes and techniques are developed methods which have not been fully proven under the circumstances of their contemplated use and which represent a significant advancement over the State of the Art in terms of meeting the national goals of cost reduction, increased energy conservation or recovery, greater recycling and conservation of water resources (including preventing the mixing of pollutants with water), reclamation or reuse of effluent and resources (including increased productivity of arid lands), improved efficiency and/or reliability, the beneficial use of sludges or effluent constituents, better management of toxic materials or increased environmental benefits.[1]

Since innovative wastewater treatment processes and techniques are generally limited to new processes and improved application of alternative technologies, it is difficult to define universal criteria for determining innovative design. The criteria used by EPA to determine innovative wastewater treatment

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processes and techniques are as follows:[1]

1. The life cycle cost of the treatment works is at least 15 percent less than that for the most cost-effective alternative which does not incorporate innovative wastewater treatment processes and techniques (*i.e.*, is no more than 85 percent of the life cycle cost of the most cost-effective noninnovative alternative).

2. The net primary energy requirements for the operation of the treatment works are at least 20 percent less than the net energy requirements of the least net energy alternative which does not incorporate innovative wastewater treatment processes and techniques (*i.e.*, the net energy requirements are no more than 80 percent of those for the least net energy noninnovative alternative).

3. The operational reliability of the treatment works is improved in terms of decreased susceptibility to upsets or interference, reduced occurrence of inadequately treated discharges and decreased levels of operator attention and skills required.

4. The treatment works provides for better management of toxic materials which would otherwise result in greater environmental hazards.

5. The treatment works results in increased environmental benefits such as water conservation, more effective land use, improved air quality, improved groundwater quality, and reduced resource requirements for the construction and operation of the works.

6. The treatment works provide for new or improved methods of joint treatment and management of municipal and industrial wastes that are discharged into municipal systems.

The first two criteria, cost and energy reduction, are to be used in determining if conventional concepts of treatment can be classified as innovative. The other four criteria specify improved benefits without providing quantitative levels of conformance. Alternative technology may qualify as innovative if any one of the six criteria is met. Figures 1 and 2 provide the procedure for determining the classification and the funding decision methodology to be used in the evaluation of projects proposing innovative and alternative technology.

In the past, planners in developing facility plans have had the tendency to analyze only a narrow range of alternatives biased toward time proven conventional designs. With new emphasis being placed on innovative concepts, a broad range of alternatives that utilize recycle, reclamation, improved application, and energy recovery or provide for significant cost savings must be evaluated in the Step 1 planning process. In order to accomplish these broader planning objectives, greater effort is required in concept development and the formulation of innovative alternatives along with a higher level of discipline in the systematic classification and screening of alternatives.

ALTERNATIVE WASTEWATER TREATMENT

Alternative wastewater treatment processes and techniques are proven methods which provide for the reclaiming and reuse of water, productively recycle wastewater constituents or otherwise eliminate the discharge of pollutants or recover energy.[1]

The adjective *alternative* is defined by Webster as "offering a choice or possibility of one of two things." In the context of the Clean Water Act this means a choice between conventional wastewater treatment systems and systems that reclaim and reuse wastewater, recover energy, and/or eliminate the discharge of pollutants.

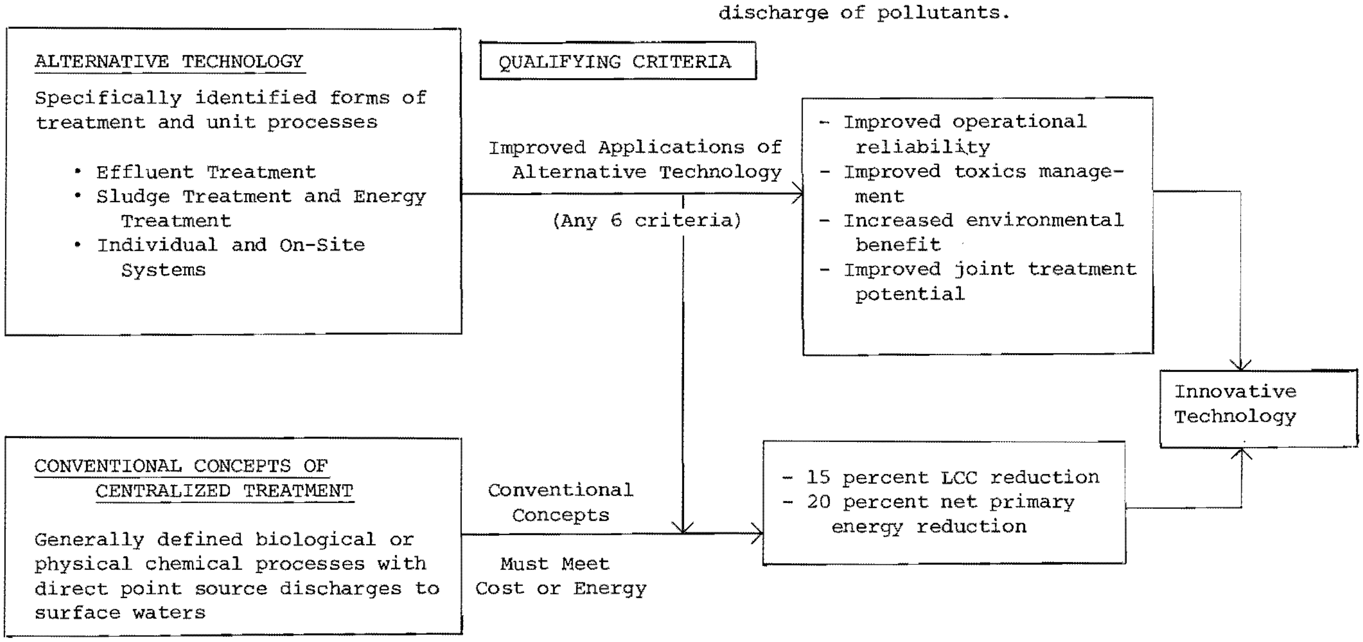

Figure 1. Classification of innovative and alternative technology.[2]

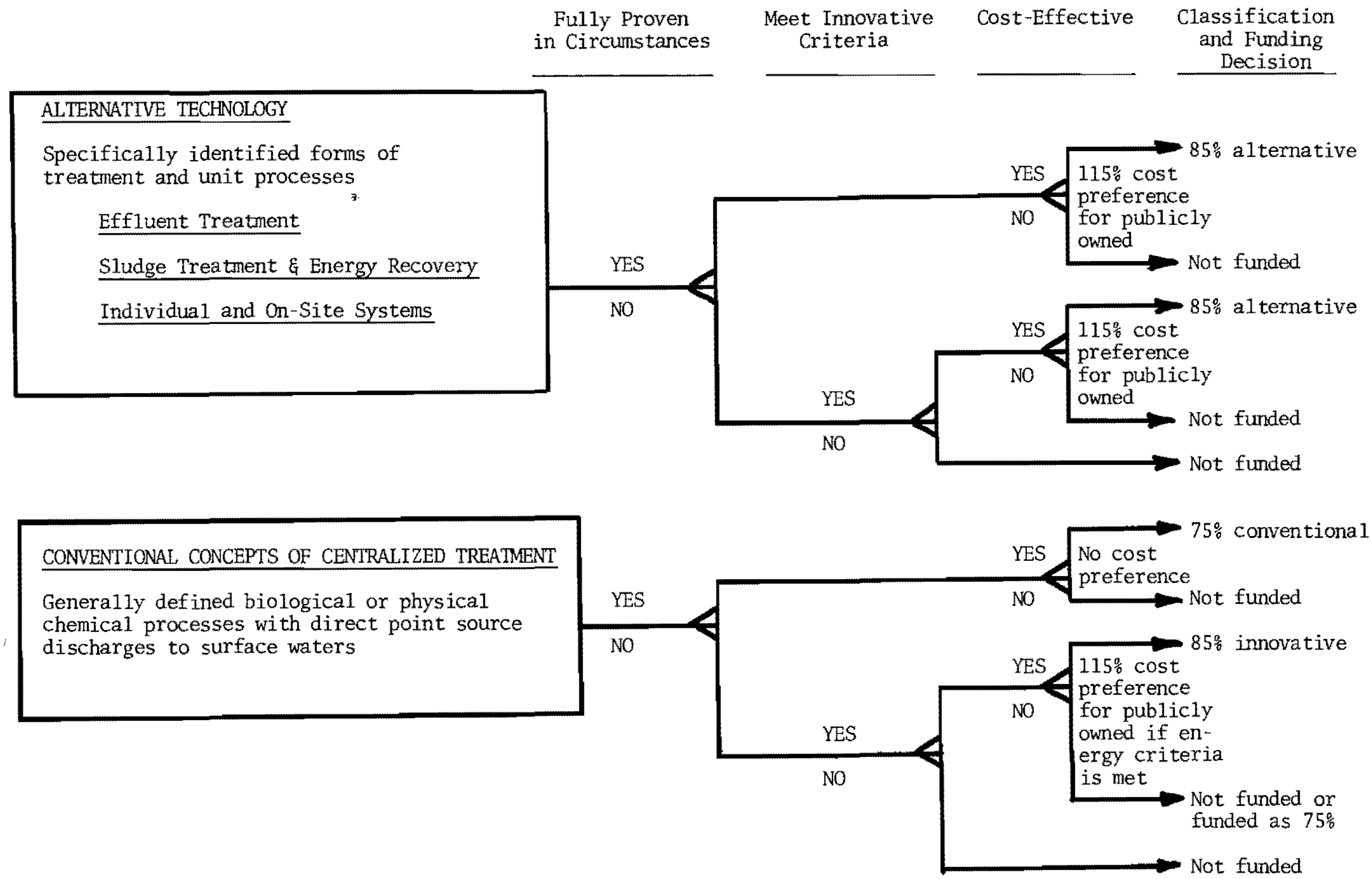

Figure 2. Innovative and alternative technology decision technology.[2]

Alternative processes and techniques include the following:

For wastewater effluents -

- land treatment
- aquifer recharge
- aquaculture
- silviculture
- direct reuse (non potable)
- horticulture
- revegetation of disturbed land
- containment ponds
- treatment and storage prior to land application
- preapplication treatment required prior to land application

For sludges -

- Land application for horticulture,
- silviculture, or
- agricultural purposes (includes processing by composting or drying)

For energy recovery -

- Anaerobic digestion facilities if more than 90 percent of the methane gas is recovered and used as fuel
- Self-sustaining incineration if the energy recovered and productively used is greater than that required to process the sludge to usable state.
- co-disposal of sludge and refuse

Other on-site and individual wastewater treatment systems which are alternative to collection and conventional treatment and discharge are also included in the alternative classification. These include:

- on-site treatment
- septage treatment
- alternative collection systems for small communities

Although alternative techniques are defined as proven methods, any of the above listed alternatives which can meet any one of the six criteria for determining innovative design is eligible for the innovative classification.

COST EFFECTIVENESS ANALYSIS OF INNOVATIVE AND ALTERNATIVE SYSTEMS

The only change in the cost effectiveness analysis normally used for analyzing wastewater treatment systems life cycle cost is the added 15 percent cost preference (allows for 115 percent times the cost of the least cost alternative) for projects proposing innovative or alternative processes and techniques. Proposed designs may include a mix of conventional and non-conventional processes, components, and equipment. For these cases, the analysis must identify all cost in each category (conventional, alternative, and innovative) indicating if the 115 percent preference applies. Table 1 summarizes the approach for determining cost effectiveness and project grant eligibility.

Table 1. Cost effectiveness and grant increase project portion eligibility.

Project Preference or Eligibility	Portion of Total Project That Is Eligible[1]	
	For project portion less than 50 percent of total project	For project portion greater than 50 percent of total project
115 percent cost effectiveness preference for Innovative and Alternative (I&A) Technology	Only I&A Portion	Entire Project
75 percent to 85 percent grant increase for Innovative or Alternative (I&A) Technology[2]	Only I&A Portion	Only I&A Portion

[1] Project eligibility is based on present worth cost of total project eligible portions excluding sewer related costs except for projects qualifying as alternatives for small communities (a municipality with a population of 3,500 or less or a highly dispersed section of a larger community).

[2] Conventional concepts of treatment qualifying as innovative under the energy criteria must meet the overall 115 percent cost effectiveness criteria to be eligible for funding.

SUMMARY

It is EPA's overall objective to provide the regulations and guidelines required to achieve the goals of the Clean Water Act of 1977 without interrupting the momentum of the construction grant program. Success will depend on planners, engineers, and regulatory agencies' reviewers to demonstrate the highest possible standards of engineering excellence and judgment in development and screening of innovative and alternative technology applications.

REFERENCES

1. Federal Register, Vol. 43, No. 188, September 27, 1978.

2. Draft Innovative and Alternative Technology Assessment Manual, Municipal Environmental Research Laboratory Office of Research and Development, U.S. EPA, Cincinnati, Ohio, 1978, EPA-430/9-78-009.

3. The Clean Water Act showing changes made by the 1977 Amendments, Serial No. 95-12, U.S. Government Printing Office, Washington, 1977.

WASTEWATER OPERATOR TRAINING IN UTAH
1978 STATUS REPORT

*Stephen E. Moehlmann**

Progress in operator training in Utah has been made along two fronts in 1978. First, fifteen workshops covering a variety of topics were held at different locations throughout the State. Secondly, Utah Technical College at Provo (UTC/P) with the help of the Utah Water Pollution Control Association, Utah State Division of Health, and Utah State University developed a one year certificate and a two year Associates of Applied Science degree program for operators of wastewater collection systems and treatment facilities.

UTAH ENVIRONMENTAL SYSTEMS OPERATORS TRAINING PLAN

THIRD PROGRESS REPORT - 1978

In 1978, the Utah Environmental Systems Operations Training Plan (UESOTP) continued to make progress towards implementation of its goal of increased competency and qualifications in operations personnel who are involved in the operation, maintenance and management of facilities and systems for public water supply, wastewater control, solid waste control and air pollution control.

The membership of UESOTP was expanded by the addition of a representative from the Utah Environmental Health Association.

The education committees of the professional associations represented in UESOTP served as subcommittees for selection and coordination of the training activities in their fields. The sponsorship of UESOTP was transferred to UTC/P from Utah State University.

Of the 53 training activities coordinated through UESOTP, 16 were held in Salt Lake City and 37 were held in other locations throughout Utah. A total of 1220 participants attended these training activities: 289 attended fifteen wastewater training activities, 609 attended twenty-nine public water supply training activities, 270 attended six solid waste training activities and 72 attended four air pollution training activities.

UTAH TECHNICAL COLLEGE AT PROVO PROGRAM DEVELOPMENT

A two year Associates of Applied Science degree program in Environmental Technology has been developed at UTC/P. UTC/P was officially assigned the role of environmental control training by the Utah State Board of Higher Education on February 28, 1978. The Environmental Technology program involves the fields of wastewater control, public water supply, solid waste control and air pollution control.

Prior to the development of the degree program at UTC/P, operator training had been sponsored by the Utah State Division of Health, the professional associations in the State, the League of Cities and Towns, and the State Universities.

Dr. Roger Plothow, director of the Continuing Education-Extended Day Program, accepted the responsibility of developing the Environmental Technology program. He assembled an advisory committee composed of staff members of UTC/P, the State Division of Health, elected officials and professionals. This committee recommended the formation of a technical advisory committee composed of professionals to provide advice, coordination and leadership necessary to develop the degree program. Members of both committees are voluntary and are not paid for their work.

To best represent the professions covered by the degree program, the T.A.C. included members of the education and advancement committee of the professional associations in the State and the Utah State Division of Health. The professional organizations represented are the Utah Water Pollution Control Association, the Intermountain Section of the American Water Works Association, the Utah Chapter of the Governmental Refuse Collection and Disposal Association, and Utah Environmental Health Association. The T.A.C. has been used to determine the scope of the program, develop the training approach, classify objectives and courses, and establish degree requirements.

The T.A.C. decided the program would offer an Environmental Technology degree with specializations. The specializations are wastewater treatment, wastewater collection, wastewater laboratory, public water supply treatment, public water supply distribution, public water supply laboratory, air pollution, solid waste and environmental laboratory.

UTC/P proposed two curriculum alternatives to the T.A.C. The first alternative divided the subject matter according to process units. The second alternative divided the subject matter according to process control tasks. The T.A.C. chose the second alternative as best suited for the operators in Utah and as offering the greatest flexibility.

Due to the size of the T.A.C. and the widely divergent backgrounds, it was decided to split the original T.A.C. into a separate T.A.C. for wastewater, public water supply, air pollution, solid waste and environmental laboratory.

An evaluation system was established for the objectives for each course and the courses required for each degree specialization. The "need to know" criteria developed by the Association of Boards of Certification (ABC), and listed in the October 15, 1978, Status Report of the Joint Training Coordinating Committee, were chosen for the objectives. These criteria were used to insure coordination between the training and certification programs.

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The ABC modular exam structure has grouped the objectives into four main categories: general, support systems, unit processes and process control, and technical supervision/management.

Both the objective and course evaluations were accomplished with the classification project. The classification project has been completed for wastewater and public water supply.

A matrix-type worksheet was used for the classification project. It listed the objectives and course descriptions. Each profession (wastewater treatment, wastewater collection, public water supply treatment and public water supply distribution) was divided into the three work areas: operations, maintenance, and laboratory. The rating scale, (see Table 1) was listed at the bottom of each page. The rating scale was based on competency levels and approximate ABC grade levels. A number was assigned to each rating for ease in data tabulation.

Table 1. Rating scale.

Rating	Competency	Descriptions	ABC Level
0		(Not applicable to job)	
1	Apprentice	Lowest level of technical ability	1
2	Journeyman	Intermediate level of technical ability	2
3	Master	Highest level of technical ability	3
4	Supervisor	Supervision and/or management	4

The wastewater T.A.C. and public water supply T.A.C. have each held a half day workshop to fill in the review sheets. Each T.A.C. invited additional participants with technical competence to participate in the workshop.

Guidelines for completion of the review sheets and equipment lists for support systems, wastewater treatment, wastewater collection, public water supply treatment and public water supply distribution were distributed to each participant for use during the workshop.

Each reviewer completed the review sheets according to his own specialization.

Food and refreshments were served during the workshops. Reviewers were not paid for their work, travel or other expenses. The total cost of the workshops was less than $100.

The results of the classification project have been used to develop course guides and to select courses for each degree specialization.

In addition to objective and course selection, the classification project results have been used to determine the objectives and courses required for each competency level of the work areas for each degree specialization.

A cooperative project between the training and certification programs is necessary to further refine this project and establish definite guidelines for each competency level. A project of this type would improve coordination between the training and certification programs.

These same results could be used to generate detailed job descriptions for each occupation and competency level in the plants or systems. This could result in the coupling of job advancement with training and/or certification. This would enhance the training and certification programs, and improve the operations of the plants and systems by tying pay raises to increased skill levels. As each objective or group of objectives is mastered as verified by successful completion of coursework or certification, the employer would give the employee a pay raise for his increased competency.

The State Apprentice Council might be involved to certify completion of the apprentice level training.

The next step in the classification process is to assess the importance of each objective in the different competency levels. To measure their importance, the T.A.C.'s will be asked to assign the percentage of time each employee at the various competency levels in the various occupations works on the objective. For example, how much time is spent cleaning screens, recording flow data, etc., by the apprentice operator. It will generate data on the relative importance of the objectives at each competency level and how much time should be spent on the subject matter in the courses.

In addition to the program development, UTC/P has made application to EPA for a $500,000 construction grant to build a training center on the Orem Campus under section 109B of PL 92-500. The application has been approved by the State and is waiting approval by the Region VIII.

The center will include a classroom, laboratory, shop, library and storage areas for mobile equipment. The library will contain training material which may be loaned to the treatment plants for use in their training programs.

UTC/P plans to use a variety of instructional methods for their delivery systems. These will include regularly scheduled classes at the training center, workshops at the center and throughout the State, and apprenticeship and self paced training to be conducted on-site. A particularly interesting approach will be to offer courses at treatment plants as part of their training programs. This will allow the enrolled operators to receive college credit while learning how to operate their own plant.

The instructional staff of the college will be supplemented by experienced and qualified operators and others with technical expertise who will serve as part time instructors. These operators will be registered by UTC/P to instruct specified courses.

OPERATOR TRAINING AND START-UP OF PROVO'S ACTIVATED SLUDGE PLANT

*Ronald J. Bergland, P.E.**

The transition from construction to operation of today's complex wastewater treatment facilities is extremely difficult. In an effort to smooth out this difficult period, operator training and start-up programs are now being implemented. This paper describes the operator training and start-up program which is being provided at Provo's new $16.7 million wastewater treatment plant.

PLANT

The Provo Wastewater Treatment Plant is a fairly large and complex facility. It is designed to process 80,000 cubic meters (21 million gallons) of wastewater each day and to produce an effluent which meets Utah's "Polished Secondary" effluent requirements of 10/10. This performance is achieved by adding activated sludge and dual-media filtration to the existing trickling filter plant.

Process flow diagrams for the plant are shown in Figure 1. The liquid stream processes consist of: screening and grit removal, followed by sedimentation and biofiltration, followed by sedimentation and activated sludge (nitrification), followed by sedimentation and gravity filtration, followed by disinfection. All solids are anaerobically digested prior to dewatering and drying in gravel beds. A dissolved air-flotation thickener is provided to concentrate all waste activated sludges prior to digestion.

This facility also possesses 144 square meters (1,550 square feet) of laboratory. Not only does this lab perform the function of effluent monitoring, it also provides data for use in process control. With three types of biological unit processes on-site, generating accurate process data is vital to the consistent and stable operation of the entire facility.

In order to achieve desired performance, both in terms of effluent quality and reduced costs, automation is provided throughout the facility. For example, the air flow rate from the blowers can be set to follow changes in either the influent flow rate or a time scaled program.

A central monitoring station is also provided so that operators can effectively maintain a "tight rein" on the vital functions of processes and equipment distributed over the 200,000 square meters (50 acres) of plant site. This equipment indicates, totalizes, and records all critical process data such as influent flow rate, chlorine residual, and waste sludge flows. Also, an annunciator panel is provided to alert operators as to status of all equipment. When a function deviates from its normal operational limits an alarm is triggered. These features make it possible to detect the early warning signs of process problems and provide the lead time so necessary to take remedial action which will prevent a problem from damaging effluent quality.

PEOPLE

As one might expect, a large complex plant requires a sizable staff to operate it. A staffing evaluation was conducted and it was determined that as many as 20 full-time people could be required. With this in mind, the city authorized these 20 positions but elected to fill only 18 positions at start-up. The remaining two positions could then be filled at a later day should the need arise.

Though the existing organization chart worked well with 5 people, when the staff was expanded to 20 people a complete restructuring was required. The reorganization involved dividing responsibilities into three areas; operation, maintenance and laboratory. A supervisory position was then assigned to each area with overall responsibility still belonging to the Plant Superintendent.

When it came time to hire these people, it was found that the demand exceeded supply. Recruitment efforts were extended to several states, not only just to find eligible candidates, but, also in hopes of obtaining operators with activated sludge experience. After a great deal of effort which was at times extremely discouraging, a full staff was obtained. However, due to limited availability of experienced operators, most individuals had at best a "text book" knowledge of wastewater and some experience with primary treatment. This factor placed more importance on the effectiveness of the "on-the-job-training" which was to follow during start-up.

PLAN

In order to bring this plant on-line in an orderly manner, we divided our thrust into three phases; construction coordination, operator training, and start-up assistance. For the first and last phase, we acted in a consulting capacity, for the second phase we became educators.

With the overall goal of effluent quality in mind the following objectives were developed for each phase:

Construction - Make the plant work
Training - Make the people work
Start-up - Make the people and the plant work together.

The tasks required to meet these objectives and results obtained at Provo are described in the subsections which follow.

CONSTRUCTION

Construction began on the treatment plant in June, 1976, and by summer of 1977 the facilities were about 50 percent complete. It was at this early date our efforts began. It was now time to begin the planning necessary to determine just what it was going to take

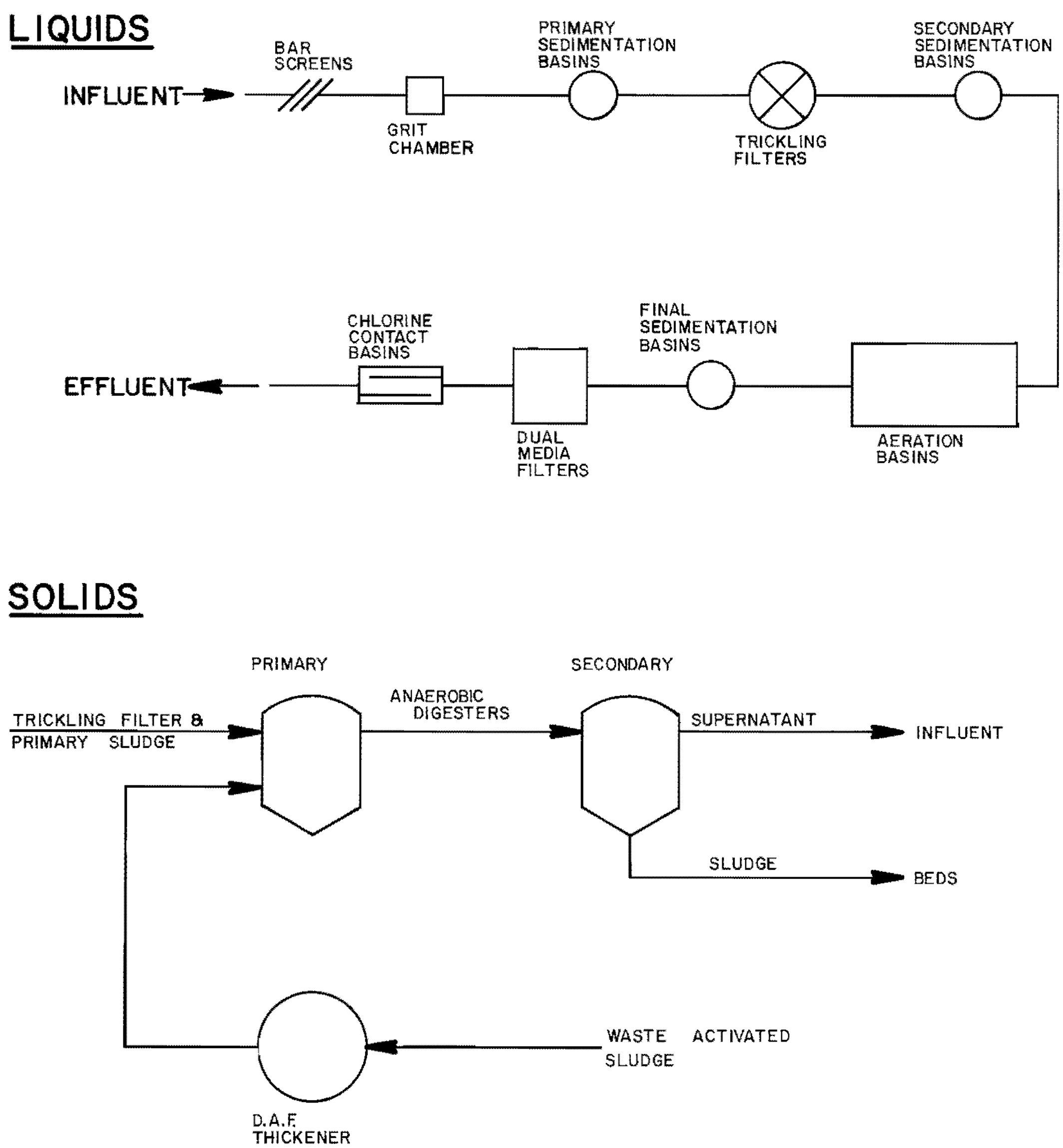

Figure 1. Process flow diagrams.

to put these facilities into service. Through meeting with city and contractor, a time schedule which showed critical events leading to plant start-up was developed. Items such as hiring operators, completion date of Operation and Maintenance Manual, sequence of start-up, and budget were included. This work was faithfully updated on a monthly basis. For this planning to be of any value, regular revisions are mandatory. This planning effort brought all parties to pull together toward their common goal. It was especially helpful at Provo since numerous units had to be sequenced in and out of service so that construction tie-ins could be made.

Also, at this early stage in the project, it was timely to begin work on a preventive maintenance program. A considerable effort was required and it was important to determine just what it was going to take to keep equipment running. Hence, we worked with the city to select the right system for their needs. We also helped them obtain expert assistance in selecting lubricants and determining application requirements.

Also during this period a considerable effort was devoted to planning and scheduling testing. Prior to requiring a facility to treat wastewater, it is important to determine if all meters, pumps, and other equipment work as specified. If sequence of start-up is carefully planned, most units can be checked out by contractor, supplier, electrician, and engineer without the additional constraints of requiring that unit to treat wastewater. For example, at Provo we were able to test all systems in the digester-gas mixer, boiler, heat exchanger, recirculation pumps, etc. - using treated effluent instead of raw sludge.

With the planning behind us and with the assurance that the equipment would work when we get to the point where the plant must perform, we were now ready to divert our attention back to the personnel.

TRAINING

The training effort was of two types, formal and informal. The formal sessions were lectures and they were intended to familiarize the operators with the purpose of each unit process, the name and location of all facilities, and with how to find information in the Operation and Maintenance Manual. The informal sessions were conducted when a unit was running and its purpose was to provide the operator with a "hands-on" sense for how units should look, sound, and feel *prior* to their being required to make these units perform treatment functions.

At Provo, the formal training began in February, 1978, and continued through May, 1978. A total of 40 sessions were delivered to 20 people. Three to four sessions were delivered during each week that training was conducted. This was done to minimize travel expenses for instructors. Also, each session was delivered first to one group in the morning and then repeated to another group in the afternoon. This allowed classes to be smaller and provided operator coverage of the plant at all times. Each session was comprised of one-half to one hour in the classroom with the Operation and Maintenance Manual and two to two and one-half hours in the field looking at the facilities.

Special sessions were conducted for supervisory personnel. While the regular sessions provided more of the "how to do things", these special sessions concentrated on process control and "why things are done."

When it came time to start units the supervisors were integrated into the "hands-on" start-up team which also included contractor, supplier, electrician, and engineer. We then worked with the supervisors to develop routine operating and checking procedures for each unit. When initial operational stages had passed and operation was more or less normal, the supervisors then passed this information along to the front-line operators as *they* showed them how the unit worked.

START-UP

At Provo the first unit brought into service was the laboratory. This was a completely new facility for this plant and as such took some time to establish. Also, early start-up provided the additional lead time that was needed so that background data could be developed for start-up. When it is time to start a unit one does not want to be questioning the suspended solid or dissolved oxygen analyses, that must all be history.

As soon as the lab was generating data we worked with plant personnel to get this information into a usable form. The Operation and Maintenance Manual provided the basic record system. After a few changes in forms the data were being accumulated in a form which allowed analysis. It is so important, for any given day, to be able to glance through the records and see just what is happening in all processes. It is also of equal importance that one can look at a process and be able to see trends during a period of days.

Finally in July, 1978, all was ready and the new digesters were started. They were filled with primary effluent and heating was begun. During the two-week period it took to bring the digesters up to temperature, all other process equipment was checked out and tested. When a temperature of 35°C (95°F) was reached, a feeding program was begun and pH, volatile acids, and alkalinity analyses were performed daily. The raw sludge feed was gradually increased, watching volatile acids at all times, and finally gas production was achieved after about one month.

Construction sequence required us to next start the aeration basin and divert all flow through it so that existing facilities could be taken out-of-service and connected into the new plant. We initiated operation by passing a minimum flow through one aerator and one final sedimentation basin. When the system was full, blowers and return sludge pumps were turned on. Each day the amount of activated sludge in the system was measured and feed rate adjusted accordingly. Additional units were added as required. When full flow was achieved the tie-in work began.

As soon as the tie-ins were completed, the flow to the activated sludge was reduced to "starve" it into nitrification. Complete nitrification was achieved in less than a week's time. At this point the activated sludge unit was placed in a holding pattern until other units and equipment could be made operable. A feeding program was established to sustain the nitrifying culture and to build to a level to handle full flow at the same rate as construction was completed.

Finally, in mid-September, full flow was achieved and the process was stabilized so that pilot work could commence for selection of filter media.

The pilot work began in October, 1978, and was completed in February of 1979. During this period, all processes were "fine tuned" except the activated sludge. It was considered not prudent to make process changes which could adversely affect the results of downstream pilot work.

Tables 1 and 2 display the process parameters and effluent data which were achieved during the last three months. As one can see the plant is very near to meeting 10/10 requirements at this time, even though the gravity filters are not yet operational. We are hopeful that with the "fine-tuning" currently in progress, additional performance can be obtained from the Activated Sludge Process.

Table 1. Current operating parameters[1].

MCRT = 20 days
F/M = 0.1 gm BOD/mg MLSS

[1]Average values for period Dec. 1, 1977 through Feb. 28, 1978.

Table 2. Current effluent data[2].

BOD =	12 mg/ℓ
TSS =	8 mg/ℓ

[2]Average values for period Dec. 1, 1977 through Feb. 28, 1978.

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TIMPANOGOS: A CASE HISTORY IN LARGE OXIDATION DITCH DESIGN

*Dennis K. Wood, P.E.**

Utah's polished secondary effluent requirements are among the most stringent in the country. To meet these, new treatment plants should maximize removal efficiencies and reliability while minimizing construction and operating costs. Oxidation ditches coupled with filtration often meet these criteria for small communities. However, facilities planning revealed the process to be applicable for the larger 0.33 m^3/sec facilities for the Timpanogos Special Service District. These facilities (excepting effluent filters) serving the communities of Alpine, American Fork, Lehi, and Pleasant Grove are now under construction. This paper will recount the key decisions made in designing one of the largest oxidation ditch treatment plants in the United States.

The flow schematic for the Timpanogos facilities is shown on Figure 1. Preliminary treatment will consist of screening, then grit removal in aerated grit chambers. The wastewater will subsequently be pumped to the oxidation ditches. The ditches operate in the extended aeration mode of the activated sludge process. Primary clarification is eliminated, and the process is operated at a mean cell residence time (MCRT) of 20 to 100 days. The characteristic features of the oxidation ditch are the closed loop or "racetrack" flow pattern and the brush rotors. The rotors aerate the mixed liquor by surface agitation and subsequent air entrainment. The rotors also keep solids in suspension by maintaining velocities greater than 0.3 meters per second. Following the final clarifiers, the treated wastewater will be disinfected before discharge to Utah Lake. Gravity filters have been designed and will be constructed to meet Utah's future polished secondary treatment standard.

DITCH CONFIGURATION

Oxidation ditches are usually shallow earthen basins lined with concrete or gunite. The ditch is dug into the ground as a matter of economics. The lining is supported on undisturbed earth. Groundwater conditions prevented this construction at Timpanogos. During the early summer months, the groundwater level can rise to near the surface. Since only half of the four ditches will be needed during the first year of operation, the empty linings would be subject to flotation. The uplift could cause displacement or breakup of a conventional lining system.

Alternatives included constructing the ditches entirely above ground or designing structures with foundation piles that could resist the uplift forces. Plant hydraulics allowed the first alternative at Timpanogos.

In addition to the high groundwater, the soil conditions at the site are poor. The estimated settlement under the ditches was excessive, and it appeared that foundation piles might still be required. However, investigations revealed that soil presettlement by preloading or surcharging was feasible. The surcharging was accomplished by importing approximately 94,000 cubic meters of fill. The fill was piled over the proposed ditch location for six months. The area settled an average of 0.2

FIGURE I

FLOW SCHEMATIC

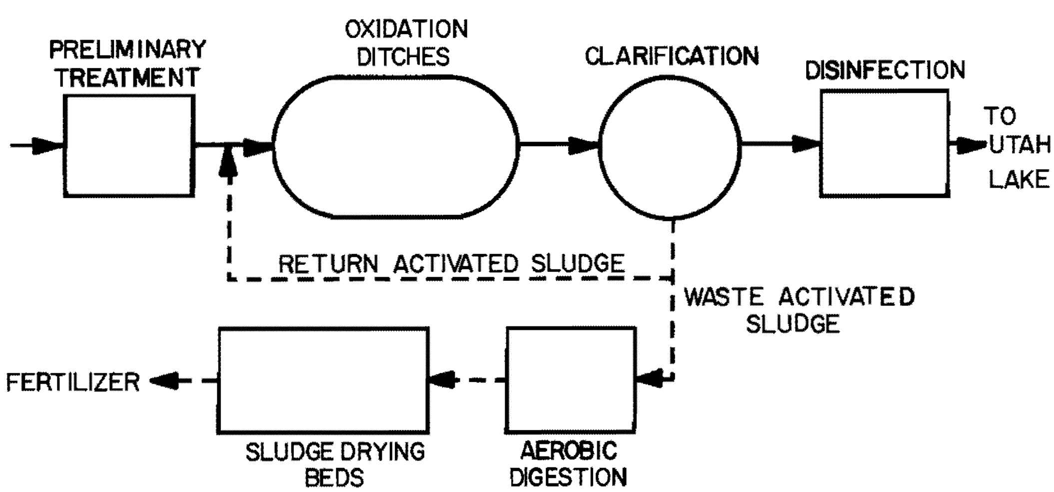

Figure 1. Flow schematic.

meters, and settlement had nearly stopped by the end of the period. No soil rebound has been measured.

The imported fill was subsequently used to construct sludge bed embankments and to raise portions of the site above the 100-year flood level. The material therefore played two key roles. Estimated construction savings with this method were approximately $1,000,000.

The final ditch configuration consists of four basins with vertical common wall construction. The "racetrack" or oval loop configuration has been retained. Turning baffles at each end of the basins are provided to distribute the velocities around the end turns. This prevents solids deposition. The water depth of the ditches is 2.44 meters. Deflection baffles downstream of the brush rotors will help distribute velocities over the basin depth and prevent surface waves.

NITRIFICATION

With recycle flows, the organic loading on the ditches will be only 173 grams five-day biochemical oxygen demand (BOD_5) per day per m^3 (10.8 pounds per day per 1,000 cubic feet). This low loading coupled with long MCRT's will naturally favor the growth of nitrifying organisms. Nitrification is a characteristic of the extended aeration activated sludge mode. The occurrence can be an advantage because of:

1. Reduced chlorine dosages as negligible chloramines are formed, and
2. A lower oxygen demand is exerted on the receiving water.

However, nitrification can result in several operational problems. Nitrification is an autotrophic process and destroys alkalinity as follows:

$$NH_4^+ + 2HCO_3^- + 2O_2 \rightarrow NO_3^- + 2CO_2 + 3H_2O$$

As the buffering capacity of the bicarbonate alkalinity is destroyed, the mixed liquor pH could drop below optimum limits. It is very important to characterize the wastewater before the ditch process is selected. If the waters do not contain sufficient alkalinity, addition of lime or other base may be required. This could reduce or eliminate any economic advantage of the ditch concept. At Timpanogos, sufficient alkalinity is present for complete nitrification.

Secondly, nitrification exerts a significant oxygen demand. The brush rotors should be designed to provide sufficient oxygen to:

1. Satisfy the BOD,
2. Raise the dissolved oxygen content of the influent wastewater from 0 to 1 mg/ℓ, minimum, and
3. Allow complete nitrification during the warmer summer months.

At Timpanogos, the rotors have been designed to supply 2.73 grams oxygen per gram BOD_5 removed to satisfy the above demands. Again, complete influent wastewater characterization is stressed.

CLARIFIER DESIGN

Mixed liquor concentrations in oxidation ditches are often maintained at levels reaching 8,000 mg/ℓ. This level affects secondary clarifier design. Solids loading can control over hydraulic surfacing loading in determining needed surface area.

As an example, consider a plant designed and operated as follows:

Plant Flow	3,785 m^3/d (1 MGD)
Mixed Liquor Suspended Solids (MLSS)	8,000 mg/ℓ
Return Activated Sludge Rate	50 percent
Clarifier Overflow Rate	24.4 m^3/m^2/day (600 gpd/sf)

Though the surface overflow rate is that often cited for activated sludge, the solids loading rate for these conditions is 293 kilograms per m^3 per day (60 pounds per day per square foot). This is more than twice the range of 98 to 146 kilograms per m^3 per day (20 to 30 pounds per day per square foot) often recommended as a general, though not rigorous, guide. Clearly, the solids factor should be considered. Either clarifiers should be built larger or the ditches must be operated at lower MLSS concentrations. The lower concentration will also lower the process MCRT. This approach is also feasible if it is recognized that the waste activated sludge will be less stable at a lower MCRT, and the total mass of waste solids will be greater. Adequate digestion facilities are needed, and this approach was adopted for Timpanogos.

Further, with the high MLSS, the sludge blanket can be quite deep. The blanket can be drawn down by increasing the return activated sludge flow. However, this can be a waste of pumping energy. The clarifiers at Timpanogos have been designed with a 3.66 meter sidewater depth to provide more storage and a lower sludge blanket from the surface.

DIGESTION AND SOLIDS HANDLING

With the long MCRT associated with the ditch process, the percent of volatile suspended solids (VSS) of the waste activated sludge can be reduced by as much as 50 percent. This reduction is due to endogenous respiration occurring in the ditch. The VSS is too low for the operation of anaerobic digesters, and aerobic digestion facilities are being constructed at Timpanogos.

The facilities have been sized to provide 30 days solids retention time to comply with Part III of the State Code of Waste Disposal Regulations. While 30 days may be required to adequately stabilize primary sludge, trickling filter humus, or conventional waste activated sludge, the long period would not be needed for extended aeration sludge. The process itself produces a very stable sludge. In the course of our design, we contacted over 35 plants and only 5 of these had any digestion provided. In most cases, the waste sludge was applied directly to land.

In lieu of the solids retention criteria, it is recommended that the State consider adopting an alternate criteria of oxygen uptake rate for aerobically digesting sludge from extended aeration plants. The digestion facilities at Timpanogos will be operated by measuring the oxygen uptake rate of the sludge. Uptake rate 0.5 to 1.0 mg/ℓ of oxygen per hour per gram is considered stable, and the aeration will be reduced at this point to save power. The digested, dried sludge will be stored for one year before it can be used for fertilizer.

Thickening facilities for the wasted activated sludge are not normally constructed in conjunction with extended aeration activated sludge processes. Therefore, the aerobically digested sludge is expected to have a solids content of only 1.0 to 1.5 percent. Compared to digested sludge from a conventional trickling filter plant, twice the liquid volume must be handled. This can affect solids handling subsequent to digestion.

At Timpanogos, 0.46 m^2 (5 square feet) of sludge drying bed area per capita has been provided to account for the larger liquid volume expected. Two types of beds are being constructed. One-third of the bed area will have a sand and gravel filter and underdrain system. These beds will be used during the warmer months. The remaining area consists of deeper beds with no underdrains. These are for cold weather use. The sludge may be left to dry in these beds for an extended time throughout the summer. A decant system can supernate clear liquid back to the plant to reduce drying time.

Increased sludge bed area was cost-efficient at this site to handle the increased sludge volume. Sufficient land is available, and operation and maintenance costs of the beds are low. Other sludge handling methods may be dictated under other situations.

DESIGN CRITERIA

Design criteria for the Timpanogos facilities are given in Table 1. Construction cost of the facilities is $7,527,000.

SUMMARY

High groundwater and poor soil conditions at the site greatly affected the configuration of the ditches. Instead of the conventional earthen lined ditch, the ditches were built entirely above ground with vertical common walls. Nitrification, concomitant with the ditch process, can affect design and operation. Sufficient wastewater alkalinity is needed to satisfy the nitrifying organism requirements to prevent pH drop, and additional oxygenation capacity is required. The clarifier design should take into account the high MLSS carried in the ditches. Solids loading criteria should be considered in sizing the clarifiers. Additional sidewater depth is justified. Aerobic digestion has been selected in conjunction with the oxidation ditches. Sludge bed area must be determined with regard to the low solids concentration and high volume.

Table 1. Design criteria.

Parameter	Value
Peak Month Flow (m^3/s)	0.33
Influent BOD_5 (mg/ℓ)	140
Influent Suspended Solids (mg/ℓ)	140
Oxidation Ditches	
Total Volume (m^3)	25,738
Detention Time (hrs)	21.5
BOD_5 Loading (g/m^3/d)	173
Clarifiers	
Overflow Rate (m^3/m^2/d)	
Sidewater Depth (m)	3.66
Chlorine Contact Basins	
Volume (m^3)	918
Contact Time (min)	46
Contact Time In Outfall (min)	12
Aerobic Digesters	
Volume (m^3)	5,890
Solids Retention Time (days)	30
Sludge Beds	
Area (m^2)	24,247
Area Per Capita (m^2/Cap)	0.46

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MUNICIPAL SLUDGE IRRADIATION AND REUSE

*Scott B. Ahlstrom**

ABSTRACT

The disinfection of municipal sludge by radiant energy is being investigated at Boston's Deer Island Electron Research Facility and at Sandia Laboratories in Albuquerque, New Mexico. The Deer Island facility uses a particle accelerator to produce high-energy electrons capable of destroying pathogenic organisms. At Sandia, Cesium-137 is used as the radiation source. Sludge irradiation produces a highly disinfected product. It appears to be economically competitive with other processes achieving a comparable degree of disinfection. Furthermore, the sludge product is of greater potential value since it may be offered for sale in higher, more restrictive, markets. However, short-range application of the process will be limited due to questions regarding the acceptance of sludge irradiation and the necessity of producing a highly disinfected sludge product.

INTRODUCTION

Municipal sewage sludge reuse alternatives must be evaluated in terms of protection of human health. A major worry in the reuse of sludge concerns the fate of viruses and pathogens. To minimize public contact with pathogenic organisms, some states prohibit residential use of food crop application of sludge unless pathogen reduction beyond that achieved during mesophilic anaerobic digestion is provided.

A market survey of Washington, D.C. metropolitan area indicated that the private residence owner constitutes the largest potential market for sludge reuse [*Urban Services Group, Inc.*, 1976]. A similar condition probably exists in other large metropolitan areas with limited access to land suitable for sludge application. If additional pathogen reduction is practiced in a controlled reproducible manner, sludge products may be able to enter currently restricted markets.

The following methods are identified by the U.S. Environmental Protection Agency as capable of achieving the required additional pathogen reduction [*Federal Register*, 1977]:

- Pasteurization for 30 minutes at 70°C (158°F).
- High pH treatment, typically with lime, at a pH greater than 12 for three hours.
- Long-term storage of liquid-digested sludge for at least 60 days at 20°C (68°F) or 120 days at 4°C (39°F).
- Complete composting at temperatures above 55°C (131°F) as a result of oxidative bacterial action and curing in a stockpile for at least 30 days.
- Both gamma and high energy electron ionizing radiation under various application procedures including combination treatment with thermal conditioning and oxygenation.

The last of these methods is referred to as sludge irradiation-the topic of this paper.

The idea of disinfecting sewage by high-level radiant energy was first documented toward the end of the last century and has been repeatedly researched during the last 25 years. In 1973, the first commercial sewage irradiator started operation in West Germany. Presently in the United States, high energy electrons from particle accelerators and penetrating gamma rays from radioactive materials have been applied to sludge. This paper will discuss the degree of disinfection achieved by radiation treatment as well as its physical and chemical effects. The sludge irradiation systems being investigated in the United States will be described and estimated irradiation costs and product values assessed. The paper concludes with a discussion of the probability of incorporating sludge irradiation into a municipal sewage treatment system.

SLUDGE DISINFECTION

The most common form of sludge stabilization, mesophilic anaerobic digestion, significantly reduces the number of pathogenic organisms in sludge. Many cities further treat the digested sludge by air-drying. This treatment technique, however, is not severe enough to disinfect the sludge. As an example, the egg of the parasitic roundworm *Ascaris* is one of the most abundant parasitic species in sludges. The layered structure of the *Ascaris* egg and the composition of its shell allow it to survive the sewage treatment process. *Ascaris* and other organisms which withstand sewage treatment have the potential for remaining viable and infective.

Experiments indicate that irradiation can be an extremely effective form of sludge disinfection. A dose of 10,000 to 100,000 rads (a rad being a unit of absorbed energy) will destroy parasites and their larvae and eggs. Bacteria and viruses are more resistant. Levels of radiation necessary for various effects are shown in Figure 1. Factors influencing the destruction of organisms by ionizing radiation include the rate at which the dosage is delivered, nature and concentration of the target organisms, medium in which the organisms exist, temperature during irradiation, and the availability of oxygen. Oxygen added prior to and during irradiation serves to sensitize the organisms and greatly increases pathogen destruction. Likewise, a synergistic effect between heat and radiation has been observed.

Investigations conducted by *Brandon* [1976] indicate that an absorbed dose of one megarad (10^6 rads) at a temperature of 23°C (74°F) with oxygenation will essentially eliminate coliforms, fecal streptococci and salmonella. Work by *Sinskey, et al.* [1976]

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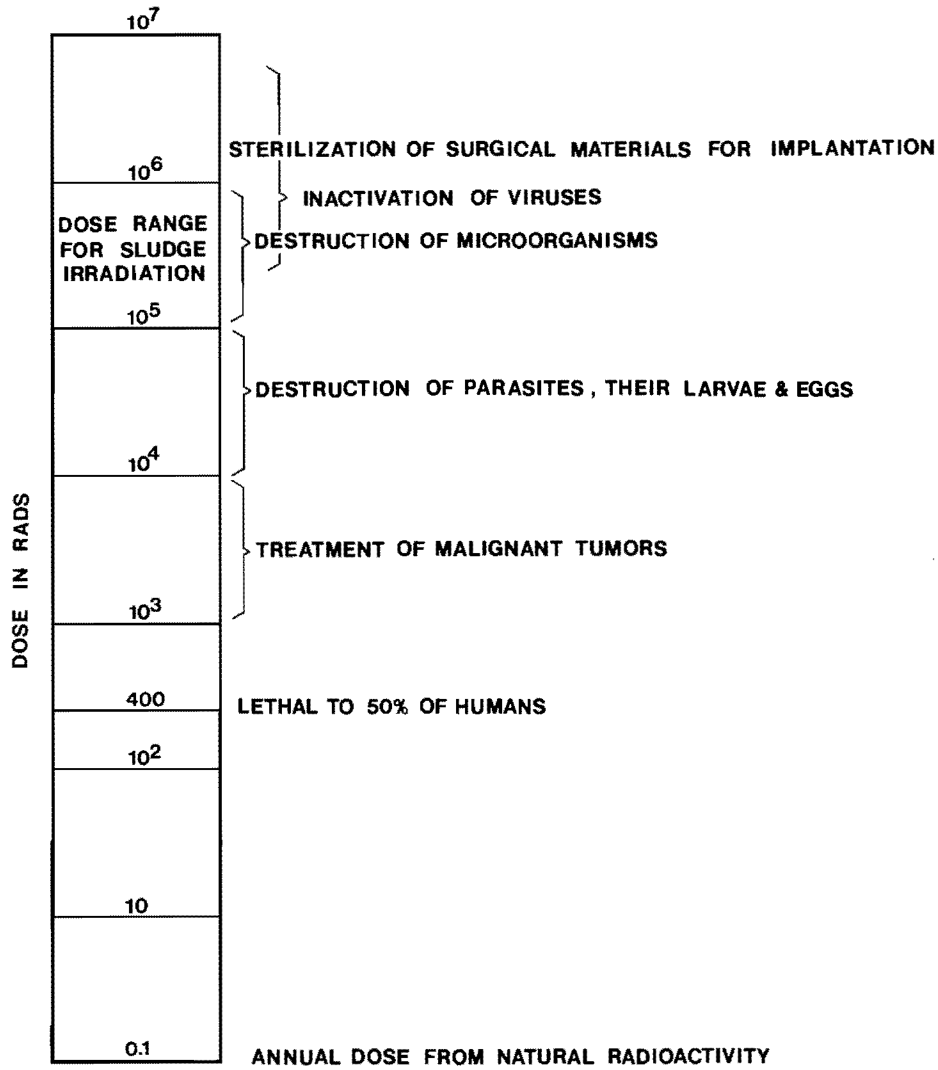

Figure 1. Effects of various radiation doses [Sinskey, et al., *1976;* Hang, et al., *1977;* Techbriefs, *1978*].

demonstrates that coliform bacteria, salmonella and shigella are essentially eliminated at a dose of 200 kilorads. Higher doses are required for equivalent destruction of fecal streptococci. Some inactivation of the viral content of liquid sludge is also achieved at a dose of 400 kilorads [*Sinskey, et al.,* 1976]. Larger doses may be required for inactivation of virus adsorbed to sludge solids.

Current sludge irradiation systems typically use doses ranging from 300 kilorads to one megarad. The future dose level for adequate disinfection will depend on the availability of ionizing radiation and the degree of disinfection which is economically attractive from a public health point of view.

In addition to the initial reduction of pathogen levels in sludge, regrowth must be controlled. Contamination of irradiated sludge with untreated material can lead to serious problems with growth of pathogenic bacteria to fairly high levels. One method to minimize contamination is to dry and bag the sludge product prior to radiation treatment.

PHYSICAL/CHEMICAL EFFECTS OF SLUDGE IRRADIATION

The effects of irradiation on sludge dewaterability, odor, and on toxic chemicals and metals contained in the sludge are described here. First, however, it is important to note that using radiant energy for disinfection does not make sludge radioactive. Sludge irradiation can be compared to a hospital X-ray. Both the sludge and patient are irradiated, but neither becomes radioactive.

DEWATERABILITY

Ionizing radiation appears to enhance the dewaterability of sewage sludges. However, studies recently completed at Sandia Laboratories in Albuquerque, New Mexico, demonstrate that radiation treatment is not as effective as chemical additives in increasing sludge filterability. Furthermore, the combined use of radiation and organic polymer conditioners show no significant improvement in filterability over the use of polymer alone

[*Techbriefs*, 1978].

ODOR

Preliminary odor analyses conducted at Sandia Laboratories have failed to show any reduction in odor of either liquid digested or liquid raw sludge following an absorbed dose of one megarad [*Morris, et al.*, 1977]. Studies at Battelle-Pacific Northwest Laboratories, in Richland, Washington, report that irradiated sludge odor levels are worse than sludge not receiving radiation treatment [*Ahlstrom and McGuire*, 1977].

TOXIC CHEMICALS AND HEAVY METALS

Trump [1978] reports that irradiation may destroy toxic chemicals in sewage sludge. For example, PCB dissolved in water to the limits of saturation or in a 0.5 percent soap solution is destroyed by an irradiation dose of 400 kilorads. Investigations are continuing on the effect to pesticides, herbicides and certain carcinogenic compounds.

No reduction in heavy metal content has been observed from radiation treatment. It is expected, however, that heavy metal concentrations in sludge will be reduced as regulations prohibiting toxic metals in industrial effluents are implemented. If heavy metals are not reduced, they may severely limit any benefit achieved by additional disinfection.

EXISTING SLUDGE IRRADIATION SYSTEMS

In the United States, research involving sewage sludge disinfection by radiant energy is underway at the Deer Island Wastewater Treatment Plant near Boston, Massachusetts, and at Sandia Laboratories in Albuquerque, New Mexico. The sludge disinfection systems developed at these locations differ significantly.

DEER ISLAND ELECTRON RESEARCH FACILITY

At the Deer Island facility, incoming sludge is passed through a screener to remove excessively large particles and then passed through a comminutor. It is pumped at a steady flow into a vault and spread in a thin layer on a rotating stainless steel drum (Figure 2). A high-energy electron beam generated by a beta particle accelerator sweeps back and forth across the full width of the drum and irradiates the sludge layer.

A thin layer of sludge must be produced due to the limited penetration of the electron beam. The sludge layer is typically 1.2 m wide and 2 mm thick. It passes through the electron beam in about 0.05 second and receives a 400 kilorad dose. The irradiated material is then pumped out for subsequent purposes. The Deer Island facility can process 0.0044 m^3/sec (100,000 gpd) of sludge at various moisture contents.

SANDIA IRRADIATOR FOR DRIED SEWAGE SOLIDS (SIDSS)

The SIDSS shown in Figure 3 is designed to process dry, digested and composted sewage sludge in bulk or bags. The unique bucket conveyor which transports the sludge past the radiation source has several unusual features.

The buckets are supported by a heavy link chain that is extended to allow the buckets to turn sharply around corner sprockets without contact (Figure 4). At the radiation source and loading area, the chain sections collapse to allow the buckets to come together. The collapsed configuration at the irradiation zone permits efficient use of the gamma source. Overlapping lips on the buckets prevent spillage when loading.

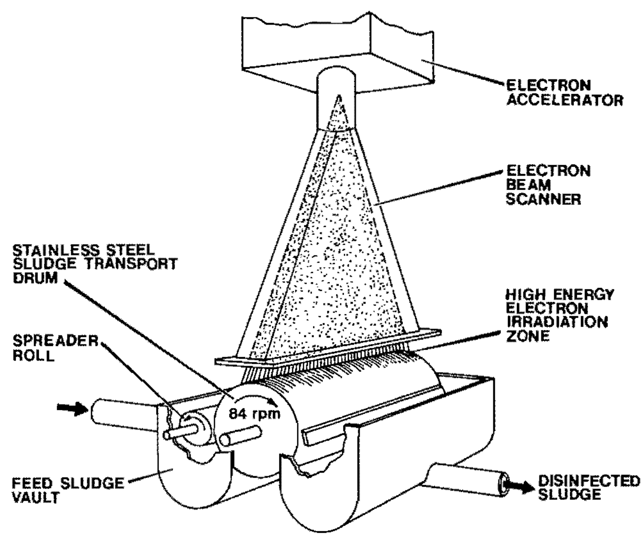

Figure 2. Sludge irradiation process used at Deer Island electron research facility.

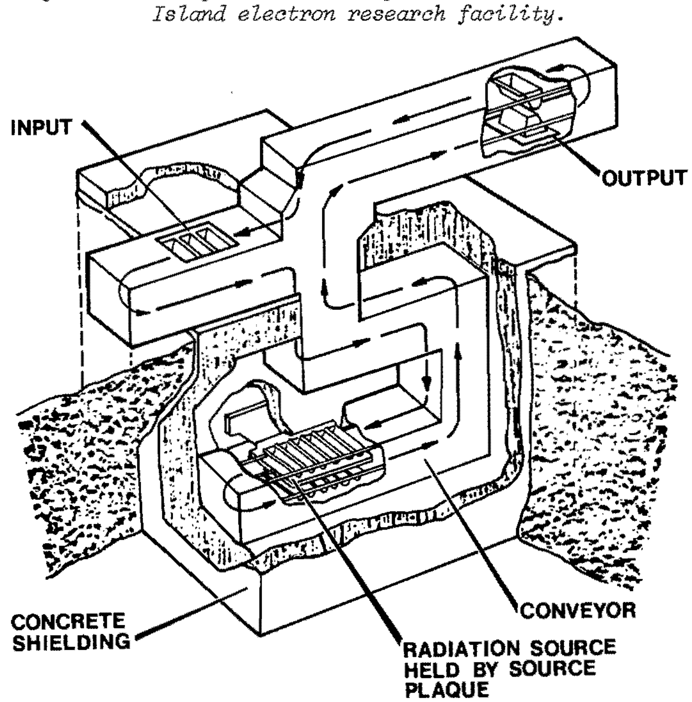

Figure 3. Cutaway of Sandia irradiator for dried sewage solids.

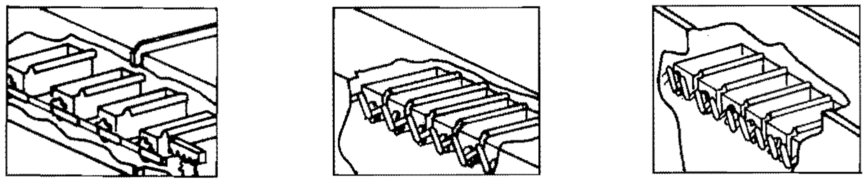

Figure 4. Detail of conveyor buckets used in the Sandia irradiator.

The irradiator can process 7,258 kg/day (eight short tons per day) at a one megarad dose. Each bucket holds bulk material 20 cm (8 inches) deep or two 18 kg (40-pound) bags. The sludge is irradiated from both above and below for a uniform dose distribution. Normal operating speed for the conveyor is approximately 10 cm per minute (4 inches per minute).

The radiation source consists of capsules containing cesium-137 in the form of cesium chloride. Cesium-137 is one of the products resulting from uranium and plutonium fission and is recovered from fuel reprocessing plants. It is an extremely "hot" gamma ray source and has a half-life of 30.2 years. The capsules are fabricated by Rockwell International at the Waste Encapsulation and Storage Facility in Richland, Washington. Large amounts of cesium-137 are currently available from reprocessing of military wastes. The SIDSS could provide a market for these radioactive wastes. Additional cesium-137 can be obtained by reprocessing commercial power reactor fuels.

ECONOMICS OF SLUDGE IRRADIATION

Preliminary cost estimates from the Deer Island facility indicate that sludge with five percent solids can be irradiated for about $18.70 per dry metric ton [*Trump*, 1978]. *Morris, et al.* [1977] reported that the estimated cost for disinfecting dry sludge with the SIDSS was $19.80 per dry metric ton for a 100-ton per day facility. Bagging the sludge would increase the price by an additional $17 per dry metric ton [*Morris, et al.*, 1977]. Sludge irradiation costs at the SIDSS and Deer Island facility should not be compared since different assumptions were used in the cost derivations.

Ahlstrom and McGuire [1977] evaluated the cost of sludge irradiation based on the SIDSS concept. Sludge disinfection processes having similar pathogen-reducing ability were compared. Thirteen treatment systems were evaluated, each consisting of stabilization when applicable, dewatering and drying processes. The study concluded that radiation of composted sludge produces a product of similar quality at less cost than any treatment option for situations where highly disinfected, dry sludge is required.

It is important to note that irradiation costs are dependent on the moisture content of the sludge. Significant economic advantages are obtained by increasing the solids content prior to irradiation.

PRODUCT VALUE

The value of sludge products varies widely and depends on the method of reuse. The economic value of a sludge-based soil conditioner can be estimated from price levels for commercial fertilizer or receipts of current sales. Presently 20 percent of the sludge produced in the United States is reused as a soil conditioner [*Morris, et al.*, 1977].

Table 1 indicates that sludge-based soil conditioners may vary in value from $5 to $66 per metric ton based on price levels for commercial fertilizer. This evaluation assumes that all nutrients have a value, which may or may not be the case. For example, if the sludge is applied to satisfy only the nitrogen requirement, then attendant phosphate and potash should have no dollar value. This evaluation, however, provides no credit for the slow release characteristics of sludge-based fertilizer, for the micro-nutrients in sludge, or for its soil conditioning capability.

During 1975 and 1976, public opinion surveys were conducted for the U. S. Environmental Protection Agency to determine user acceptance of composted sewage sludge [*Ettlich*, 1976]. The results indicated that the upper price limit for bulk sewage sludge would be $4.50 to $11 per metric ton and for packaged sewage sludge about $66 per metric ton at the point of sale, based on West Coast price levels. The surveys also indicated that dry sludge properly packaged and promoted on the retail market can produce a price competitive with similar retail products.

If radiation processing permits sludge-based fertilizer to penetrate more profitable markets, then a credit equal to the increased product value can be claimed to offset a portion of the sludge treatment costs.

THE FUTURE

Research and demonstration projects involving sludge irradiation will probably continue for the next few years. Efforts are underway at Sandia to locate a suitable wastewater treatment facility where a 23-metric ton per day dry sludge irradiator can be constructed. Sandia also has a facility proposed at the Albuquerque Wastewater Treatment Plant No. 2 that will irradiate wet sewage sludge (moisture content $\geq$ 90 percent). This research may produce volumes of data; however, the actual

Table 1. Dollar value of nutrients in 907 kg (2,000 lbs) dry sewage sludge [Haug, et al., *1977*].

Nutrient Content	Value of Nutrients in Sludge[1]		
	High	Medium	Low
Low (N = 2.0 percent, P_2O_5 = 1.1 percent, K_2O = 0.12 percent)	$ 9.80	$ 7.30	$ 4.80
Medium (N = 3.3 percent, P_2O_5 = 5.3 percent, K_2O = 0.4 percent)	34.00	26.40	18.70
High (N = 5.0 percent, P_2O_5 = 9.2 percent, K_2O = 2.4 percent)	61.70	48.20	34.70

[1] Assumptions: One-third of the total N would be immediately available to crops while all of P_2O_5 and K_2O would be available.

Fertilizer prices are as shown below:

Nutrient	Price Range (Dollars/kg)		
	High	Medium	Low
N (Nitrogen)	$0.66	$0.44	$0.22
P_2O_5 (Phosphate)	0.55	0.22	0.33
K_2O (Potash)	0.26	0.22	0.18

implementation of sludge irradiation into a municipal reuse program depends upon two major factors:

1. The degree to which the public and technical community accept this form of sludge disinfection as a viable alternative and,
2. The necessity of entering sludge reuse markets where high-levels of disinfection are required.

The degree of public resistance to sludge irradiation is difficult to gauge. The public has generally accepted equivalent types of radiation treatment facilities for sterilization of pharmaceuticals and medical disposables. If sludge irradiation is considered comparable to this type of radiation processing, public opposition may be minimal. Some resistance seems inevitable, however, when radioactive isotopes are involved.

Technical acceptance will depend on process reliability, control and economics developed and demonstrated at various pilot plants. Presently the methods most often applied for additional pathogen reduction are heat treatment and composting. Irradiation appears to offer some economic advantages over heat treatment. Furthermore, the ability of composting to continuously produce a pathogen-free product has been questioned by some regulatory officials. As additional data is gathered, these concerns will be evaluated by the technical community and irradiation placed in proper perspective.

The necessity of entering residential and other higher markets for sludge reuse will occur primarily in large metropolitan areas. Limited land availability for sludge application will encourage entrance into these markets. The necessity of achieving high-level disinfection for continued sludge application to agricultural land may result from stricter state and federal disposal regulations.

In summary, sludge irradiation is a technically feasible, high-level disinfection process. It appears to be economically competitive with other processes producing similar levels of disinfection, the pathogen reduction achieved allows sludge to enter presently restricted markets and thereby potentially increases the sludge product value. However, serious questions regarding its acceptance and the necessity of high level disinfection remain. Presently, these questions appear to be of sufficient magnitude to severely restrict the application of sludge irradiation within the next five to ten years.

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THE CENTRAL WEBER SEWER IMPROVEMENT DISTRICT INFILTRATION/INFLOW SITUATION

*J. T. Jacobs, M. L. Davidson, W. A. Luce**

Infiltration into sanitary sewer lines is not new to the Central Weber Sewer Improvement District (CWSID). The problem began about 1890, when the first sewers were installed in Ogden City, now part of the Sewer District. Many of the early sewers were installed with open joints. The intent was that the sewers serve both as sanitary sewers and groundwater drains. Even though tight sewer joints were later required, the procedures and materials used were still inferior by today's standards.

In 1953 the CWSID was organized and in 1963 a study of the infiltration/inflow situation in the District was conducted. Summer infiltration in 1963 was estimated to be 33 MGD, and annual average infiltration to be 23 MGD, while flow at the plant was 42 MGD in the summer and 28 MGD for an annual average. In 1963 Ogden had 77 percent of the District population and contributed 90 percent of the total District infiltration.

As part of the 201 facilities planning process, the infiltration/inflow situation has been studied further in the District, as required by EPA under the Infiltration/Inflow Analysis guidelines.

The District, which is located in Weber County, Utah, covers an area of over 44 square miles and includes the cities of Ogden, South Ogden, North Ogden, Washington Terrace, Riverdale, Harrisville, and Pleasant View. The population of the District in 1963 was 92,300. In 1975 the population was 103,700, and the 1978 population was estimated at 109,000. This is an 18 percent increase since 1963.

Average annual flows at the District treatment plant increased from 28 MGD in 1963 to 41 MGD in 1975. Average annual infiltration/inflow increased from 23 MGD to 28 MGD from 1963 to 1975, flows at the treatment plant and the estimated infiltration/inflow flows both increased at rates higher than the population for that period.

Another indicator of the I/I problem is culinary water use. In 1975, 7.5 billion gallons of culinary water were supplied to the cities within the District. The District's sewage treatment plant processed 14.8 billion gallons of water in 1975, or nearly twice the amount of culinary water supplied.

The magnitude of the problem is further demonstrated in Figure 1 which shows a sewage flow-precipitation hydrograph for 1975 and Figure 2 shows a hydrograph for the period 1972-1976. These figures show the dramatic flow increases experienced each year at the plant. The summer peaks correspond with the irrigation season, which runs from May to October.

During the I/I Analysis, infiltration/inflow rates were determined using existing treatment plant data and municipal water use records. A minimum of field work was required during this stage, since ample existing data were available. It was estimated by use of this data that the average annual infiltration in the CWSID was 28 MGD in 1975, with a summer peak between 35 and 40 MGD.

A cost-effective analysis was carried out as part of the I/I Analysis. Figure 3 shows graphically the results of the cost-effective analysis. This analysis used projected infiltration/inflow rates and estimated that 15 MGD of the District infiltration/inflow could be cost effectively removed.

With this information, an EPA grant was awarded the District to undertake an intensive investigation of the infiltration/inflow situation within the District. This study was to be done under the Sewer System Evaluation Survey guidelines prepared by the EPA.

The Survey was divided into two phases. The first phase was to include flow monitoring and manhole and sewer inspection, and the second phase was to include cleaning, television inspection, and the final report. The first phase has been completed and its results will be discussed herein.

The District was divided into 41 Areas with sewer lengths ranging from 0.43 to 23.22 miles, and 100 Sub-areas with lengths from 0.14 to 8.36 miles. Complete 24-hour flow measurements for all Areas were carried out during the summer of 1977, winter of 1977-78, spring of 1978, and summer of 1978. Sub-area flow measurements were made during the summer of 1977, winter of 1977-78, spring of 1978, and twice during the summer of 1978. Flow measurements were also obtained along the Wall Avenue sewer during the summer of 1978.

Electronic depth measuring devices and velocity meters were used to obtain instantaneous flow measurements and to calibrate portable flowmeters for continuous measurement procedures. Flumes and weirs were used very little due to the fact that most of the locations used for flow monitoring had sewers 18-inches in diameter and larger. Flumes and weirs of this size are somewhat difficult to work with in standard size manholes.

As mentioned, portable continuous flowmeters were calibrated by means of the instantaneous velocity and depth measurements which were taken. The slope and roughness coefficient of the channel at the monitoring locations were not used in any of the flowrate calculations. This eliminated the error which could be introduced into the calculations due to incorrect slope information or inaccurate roughness coefficient assumptions.

Table 1 shows a comparison of the flows at the treatment plant, during the periods in which field

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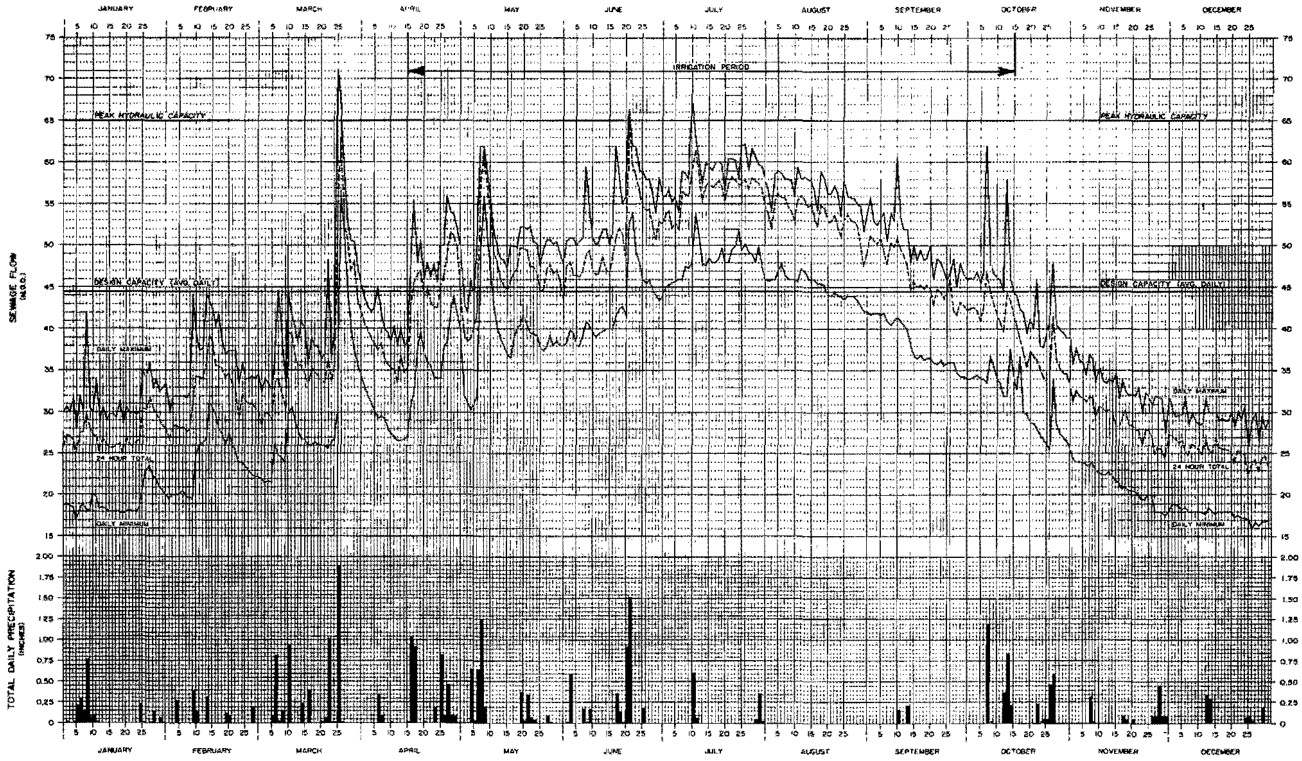

Figure 1. Sewage flow - precipitation hydraograph, 1975, 201 facilities planning project.

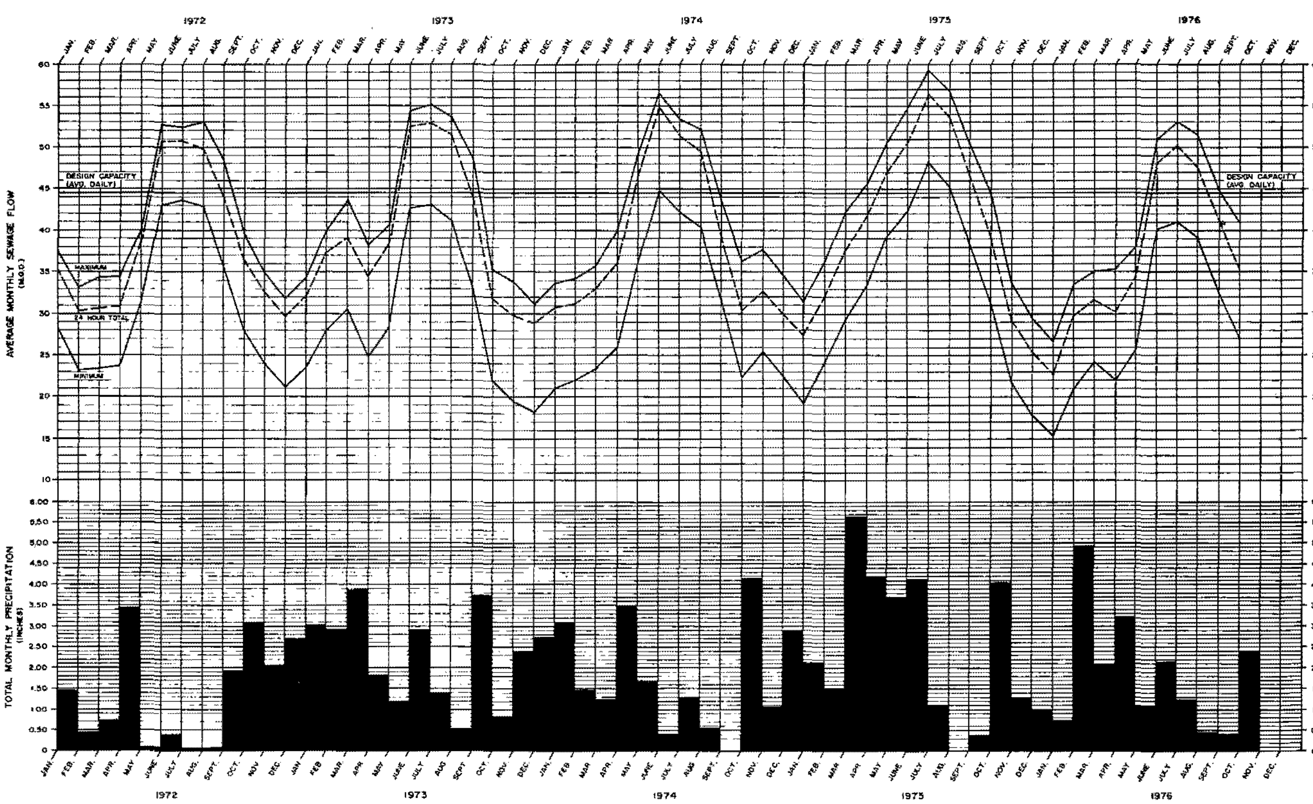

Figure 2. Sewage flow - precipitation hydrograph, 1972-1976, 201 facilities planning project.

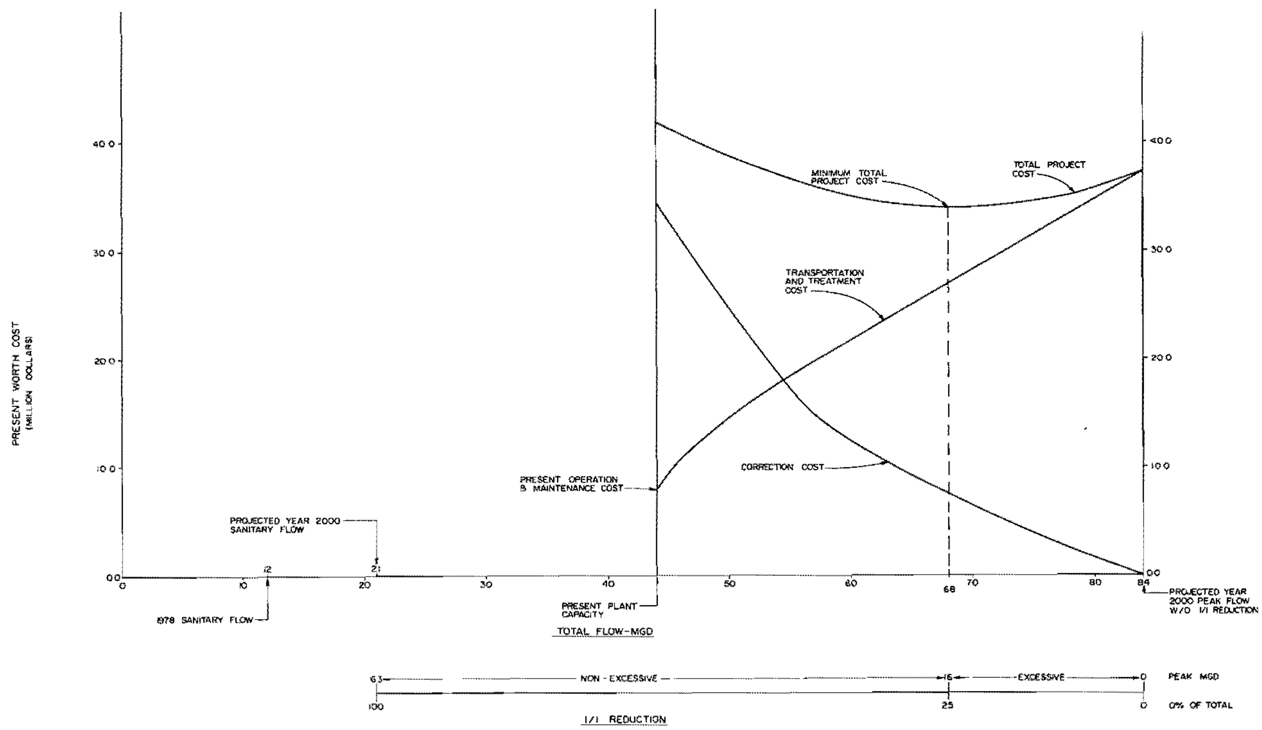

Figure 3. Infiltration/inflow analysis, excessive I/I cost curve, 201 facilities planning project.

Table 1. Flow data comparison (flows in MGD).

Time of Measurement	Plant Data			Field Data			
	Average Minimum	NDF	I/I	Area Total	Wall Average	Total I/I	Percent Difference I/I Flows
Summer 1977	30.9	4.1	26.8	24.0	3.5	27.5	2.6
Winter 1977-78	16.7	3.8	12.9	13.2	0.5	13.7	6.2
Spring 1978	27.0	3.8	23.2	25.0	1.0	26.0	12.1
Summer 1978	40.5	4.1	36.4	31.8	4.5	36.3	0.3

measurements were taken, with the flows measured in the field at thirty different locations. NDF signifies Nighttime Domestic Flow or that portion of the minimum daily flow attributed to domestic sources. The table shows field data to be well within reasonable accuracy limits of less than 10 percent for a system of this size.

Table 2 shows the municipal infiltration breakdown for the District. Incremental I/I is expressed in gallons/dia-in/mi/day and includes service laterals. The table indicates that during the summer only Pleasant View has an incremental I/I rate less than the EPA standard of 1500. The flow monitoring indicated that 12 percent of the District system has in incremental I/I rate less than 1500.

Manhole and sewer inspection was carried out during the summer of 1978. Over 3500 manholes and approximately 200 miles of sewer were included in this task. Almost 60 percent of the manholes inspected showed signs of existing or evidence of past leaks. Problem areas indicated by data collected during this task correlated well with the areas the flow monitoring task indicated to be problem areas.

The District also experiences an inflow problem during storms or heavy runoff periods. No single major cross connections or sources of this inflow were located during the study. Most of the problems of this type were isolated during an earlier study and have since been corrected. The major source of inflow now entering the system was determined to be

Table 2. Municipal infiltration breakdown (flows in MGD).

Municipality	Summer 1977 Total I/I	Summer 1977 Incr. I/I	Summer 1977 % of Dist. Total	Winter 1977-78 Total I/I	Winter 1977-78 Incr. I/I	Winter 1977-78 % of Dist. Total	Spring 1978 Total I/I	Spring 1978 Incr. I/I	Spring 1978 % of Dist. Total	Summer 1978 Total I/I	Summer 1978 Incr. I/I	Summer 1978 % of Dist. Total
Ogden	18.63	7,484	78	10.37	4,144	78	18.77	7,500	75	25.54	10,206	80
So. Ogden	1.40	2,899	6	0.89	1,836	7	1.95	4,023	8	1.66	3,425	5
No. Ogden	1.41	3,719	6	0.64	1,688	5	2.17	5,724	9	1.89	4,985	6
Wash. Terr.	0.68	2,580	3	0.34	1,290	2	0.60	2,276	2	0.53	2,011	2
Riverdale	1.10	5,166	4	0.61	2,865	5	1.03	4,838	4	1.38	6,482	4
Harrisville	0.41	4,253	2	0.25	2,594	2	0.37	3,840	1	0.52	5,395	2
Pleas. View	0.24	1,099	1	0.10	458	1	0.15	686	1	0.25	1,144	1
Total District	23.97	5,765		13.20	3,175		25.04	6,025		31.77	7,640	
Total District w/Wall Ave. Sewer	27.47*	6,440		13.70*	3,210		26.04*	6,105		36.27	8,500	

* Estimated

manhole lids. The stormwater does not seem to enter the system through the holes in the manhole lids, but between the lid and ring. Calculations indicated that the amount of water entering the collection system during storm events could easily enter by means of the approximately 4800 manholes in the District.

At the completion of the first phase of the survey, the original cost-effective analysis was revised. The revision indicated that 16 MGD of infiltration/inflow could be removed cost effectively. The Sub-area were then listed from highest incremental I/I rate to the lowest. According to the cost-effective analysis, the cost-effective incremental I/I rate was 4960. It was recommended that internal inspection be carried out in all those Sub-areas with an incremental I/I rate of 4960 or greater. This ultimately included over 160 miles of sewer.

Phase II of the Survey will include the cleaning and internal television inspection of those Sub-areas determined to contain excessive infiltration/inflow. Recommendations for methods of rehabilitation will be made upon completion of Phase II of the Survey.

NEW SEWER LOCATION IN BUILT-UP AREAS

*Harold R. Linke, P.E.**

INTRODUCTION

SINCE THE BEGINNING

The first new sewer located in a built-up area was built in Boston by Francis Thrasher in 1704. At the time the streets were made of cobblestone and the houses employed wooden drains to rid their cellars of wastewater. The digging permit issued by the City government indicated the sewer was "not only a General good and Benefit by freeing the Street from the Usual annoyance with Water and myer by the Often Stoppage and breaking of Small wooden Trunks or drains... but a more particular benefit to Neighborhood as a Common Shore [sewer] for draining of the Cellars and conveying away their wastewater."[1]

Ever since Thrasher's first sewer American engineers have fought the problems of built-up areas.

BUILT-UP AREAS

For the purposes of this paper, a built-up area is any area with a paved highway right-of-way and one or more existing underground or above ground utilities in the right-of-way.

These areas are targets for new sewer construction for many reasons. Sewer lines may need replacement because they have decayed or corroded with time. The sewer may need increased capacity because of growth in the area or a new sewer may be needed because on-site waste systems such as septic tanks begin to fail.

GENERAL

It is most unusual to come across a built-up right-of-way which has a convenient place to construct a sewer. To begin with, sewers are very inconvenient to construct. They have dozens of constraints on their placement. Second, built-up rights-of-way usually have several other utilities underground, on the surface, and above the right-of-way to contend with.

People who like to do things in an orderly way favor the "corridor" approach to utility design where each utility using the public right-of-way has a specific typical location.

When things get particularly congested, especially on wide streets, the designer may resort to duplicate sewers on either side of the street to provide service. On winding rural roads and on the now-stylish curved subdivision streets, designers are experimenting with curved sewer alignment which conforms to the road and other utilities. In severe cases it has been necessary to move existing utilities in order to accommodate a sewer and, in some more severe cases, it has been necessary to put sewers in front yards or back yards of homes abutting the right-of-way in order to provide service.

This paper will discuss all of these placements along with their advantages and disadvantages and then speak about some special construction techniques in areas where sewers must cross freeways, rivers, bridges, canals, railroads, and narrow-angle crossings.

BASIC CONSTRAINTS ON SEWER LOCATION

SEWER OUTLET

Because of the economic necessity to use gravity flow wherever possible, the elevation of the sewer outlet in relation to the basements and dwellings to be served is in the main key to sewer placement.[2] Usually the outlet location is fixed in space and a designer must negotiate the hills and valleys to get to the area to be served, staying in a public right-of-way wherever possible. Most constraints appear because of the need for gravity flow, the need to serve basements, other utilities in the right-of-way, safety, foundation, and uncertainty as to exact location of other utilities. The following three tables represent the most important constraints in sewer grade and sewer alignment. Because the sewer is the deepest utility, almost all other utilities will have to be crossed with main lines and lateral lines in the street.

Table 1. Constraints to grade of a new sewer.

- Depth of existing basements and drains.
- Probable depth of basements which may be constructed in the future.
- Topography of the surrounding areas (Can all of tributary area be served by a future gravity sewer connecting to the new sewer?).
- Topography of the area immediately around the sewer (Can basements on the down-hill side be served?).
- Can laterals be extended to each probable building site without directly interfering with an existing utility (and once the laterals are extended can subsequent utilities such as storm sewer or water be constructed without interfering with the laterals?).
- Can future main line connections be made to the sewer at a manhole and without interfering with existing or probable utilities?
- Contamination. Are sewers separated from water lines in accordance with health codes?[3,4]
- Foundation. Will the material under the pipe be suitable to support the pipe and can the material placed over the pipe provide a suitable foundation for other utilities and the street surface? Are special foundation considerations necessary because the sewer is quite deep?
- The sewer must be placed below the probable frost

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line.

- The sewer should, whenever possible, be placed above the free water surface to avoid foundation and infiltration problems.
- The depth should be so designed that a safe trench could be constructed according to OSHA standards for the placement of the pipe even though conducting a safe trenching operation is the responsibility of the contractor.
- The grade and alignment should be such that when considered together, other utilities, particularly gas, water, and power conduits, are not likely to be cut or ruptured, endangering the safety of the workers.
- Once these constraints have been met, the economics of the sewer should again be studied to determine if there are more economical configurations which still meet the previous constraints. Does the sewer authority have sufficient funds and expectation of revenue to make this sewer line extension economical?

Table 2. Constraints to alignment.

- Other utilities (see Table 3).
- The sewer should avoid, wherever possible, survey monuments for streets, area reference markers, and section lines.
- The alignment should be such that wherever possible, the minimum number of lateral road crossings need be made.
- The alignment should be on or immediately adjacent to the public right-of-way for ease of maintenance.
- The alignment should be away from waterlines to avoid contamination.
- Where possible the sewer lines should be located so as to avoid restraint of traffic and need to cut asphalt or concrete pavement.
- The alignment should be in an area where surface storm water is not likely to enter the manhole. Where ditches or gutters are involved, offset manholes can be used to place the top of the manhole on drained ground.
- The alignment should be such that the trenching operation can be safely conducted without extreme hazard from traffic or danger from rupture or breaking of other utilities.
- The alignment should be reviewed for sensitivity to inaccurate records on existing utilities. If, for example, a gas line is two feet from its designated location, will sewer construction be slowed or stopped and realignment be necessary?
- Again, the economics of the alignment should be studied to determine if other alignments would satisfy the constraints at a lower cost. Are sufficient funds available to construct the sewer?

Table 3. Other utilities common in built-up areas.

Water	Underground vaults
Storm drain	Private pipelines
Gas	Landscape
Paving	Landscape sprinkling system
Street lighting	Curb and gutter
Irrigation	Sidewalk
Electric service	Fences
Telephone	Trolleys
Railroad	Subways

APPROACHES TO SEWER LINE LOCATION

UTILITY CORRIDORS

A utility corridor is a reserved area typical to each urban street for a particular utility. Many engineering and economic studies have been made to determine what the optimum placement of utilities will be within a street. As you might imagine, the optimum depends on many variables, among them capital costs, construction costs, length and size of the line, depth and width of the trench, soil types, labor rates, number of service connections, expected life of facilities, frequency of maintenance, acts of nature such as storms and floods, motorist delay costs, increased vehicle operating costs, public safety impact, aesthetic costs, and costs of finance and inflation. Such a study was performed by a joint committee of the American Public Works Association and the American Society of Civil Engineers.[5] In their generalized study, two location plans were evaluated against one another. Essentially the study indicated that in light traffic areas location plan A with utilities spaced evenly across the highway was an economical alternative for the particular circumstances assumed. If the highway traffic is heavier, location B with utilities clustered under medians and sidewalks becomes a more attractive alternative (see Figures 1 and 2). Needless to say, many variables can affect the outcome of the study.

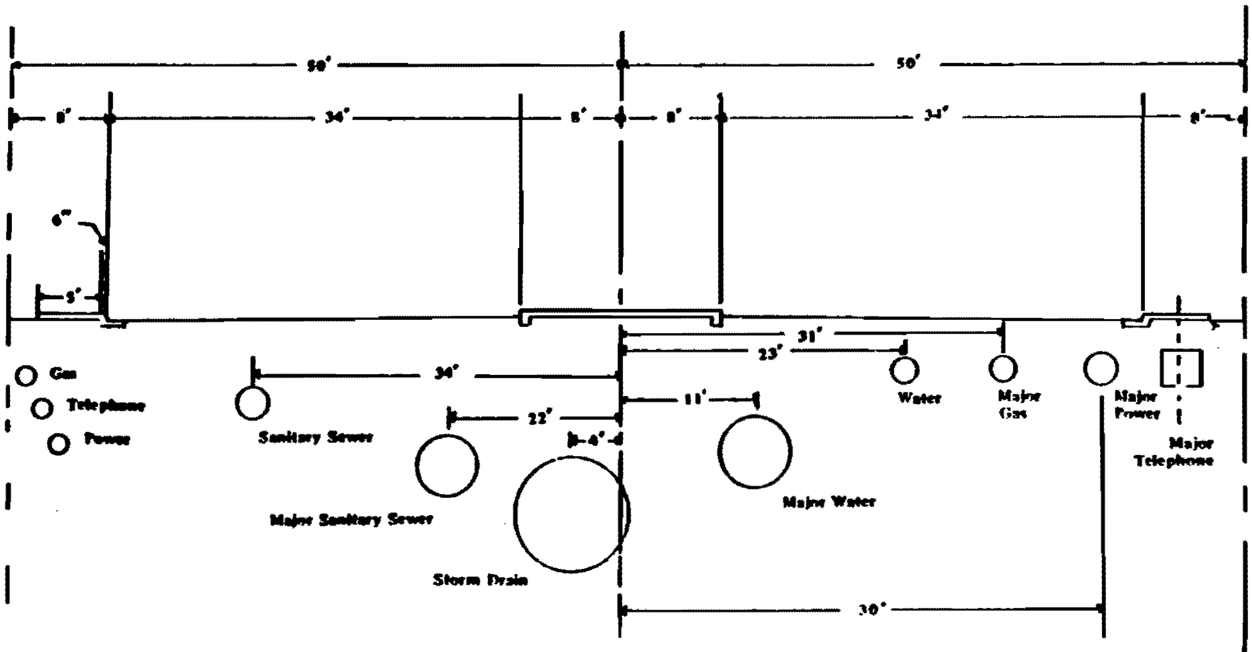
Typical corridor plans are shown in Figure 3 for Salt Lake County, Figure 4 for Phoenix, Arizona, and Figure 5 for Montreal, Quebec.

ADVANTAGES OF CORRIDOR ALIGNMENT

Where the streets are straight, corridor alignment provides a predictable location for utilities; a definite advantage where maintenance has to be done. This approach prevents one utility from using two corridors and making placement of additional utilities difficult. Safety is increased since traffic interference is designed to be a minimum in the economic analysis. The overall long-run cost to the area residents is lower in the long- and short-run even though some utilities may bear a disproportionate burden of cost to achieve this savings.

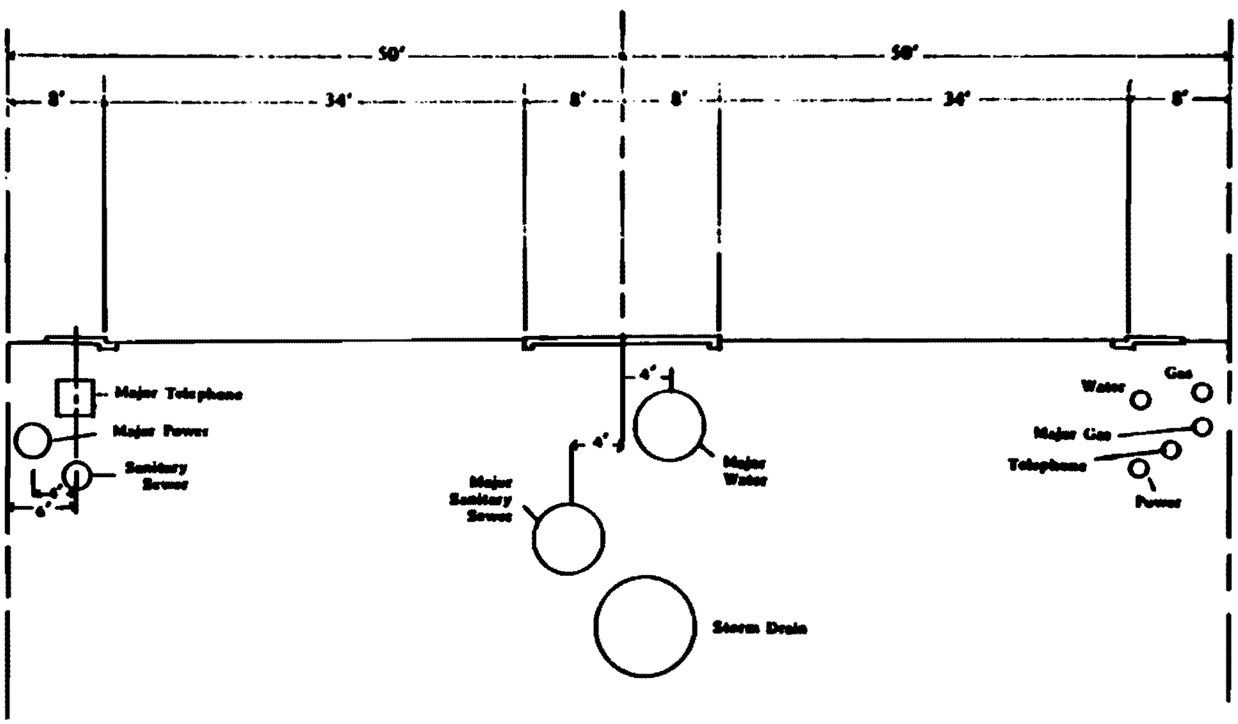
DISADVANTAGES OF CORRIDORS

Corridors are generally expensive to design initially and there may be some conflict over standards



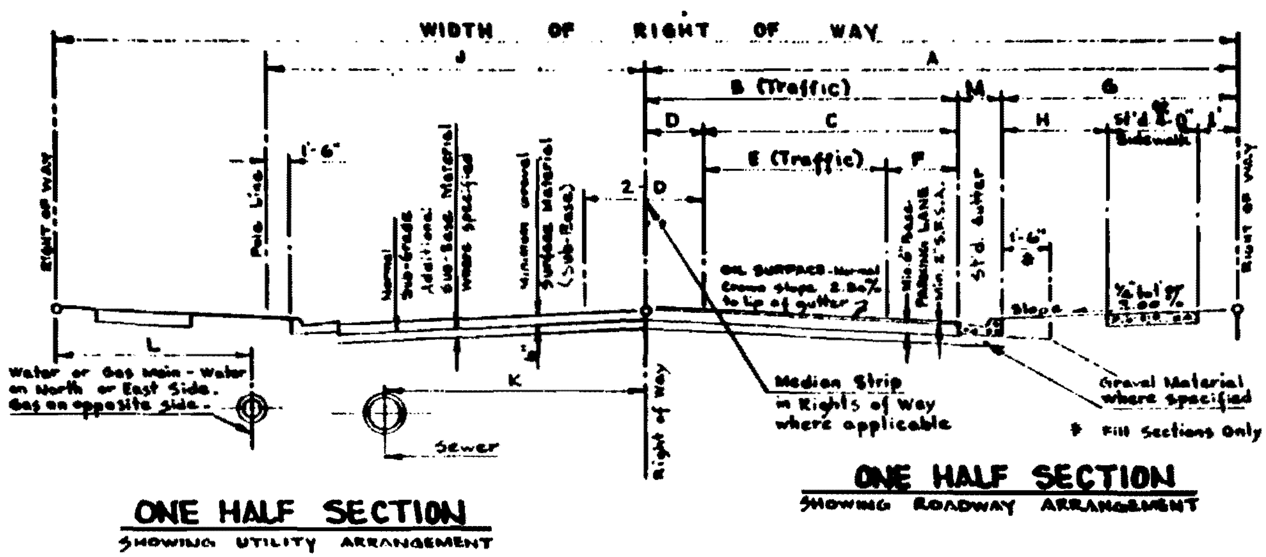
Note: vertical distances and line dimensions are not to scale

Figure 1. Location Plan A.



Note: vertical distances and line dimensions are not to scale

Figure 2. Location Plan B.



STANDARD ROADWAY SECTION

SCHEDULE OF DIMENSIONS

SECTION		A	B	C	D	E	F	G	H	J	K	L	M
1	40 feet (A)	20'	12'-6"				7'-6"	5'		16'-6"	5'	2'	2'-6"
2	40 " (B)		12'-6"				7'						"
3	50 "	25'	12'-6"				7'-6"	10'	5'	16'-6"	5'	7'	"
4	60 "	30'	17'-6"				7'-6"	10'	5'	21'-6"	9'-6"	7'	"
5	66 "	33'	24'-6"				8'	10'	5'	24'-6"	10'-6"	7'	"
6	80 "	40'		25'	2'-6"	17'	8'	10'	5'	31'-6"	17'-6"	7'	"
7	100 "	50'		32'-6	5'	24'-6	8'	10'	5'	41'-6"	27'-6"	7'	"

(A) No sidewalks.
(B) Section 2 is unbalanced on the center line to permit a sidewalk on one side only. Dimensions are to be determined by the Planning Commission and County Surveyor when this section is approved.

NOTE:

Dimension "G" to be 8'-6" or 1'-6" when property bounds state roads, only when state right of way is 50' or more. Dimension "G" to be determined by the State Road Commission when property bounds state roads.

Figure 3. Typical corridor plans for Salt Lake County.

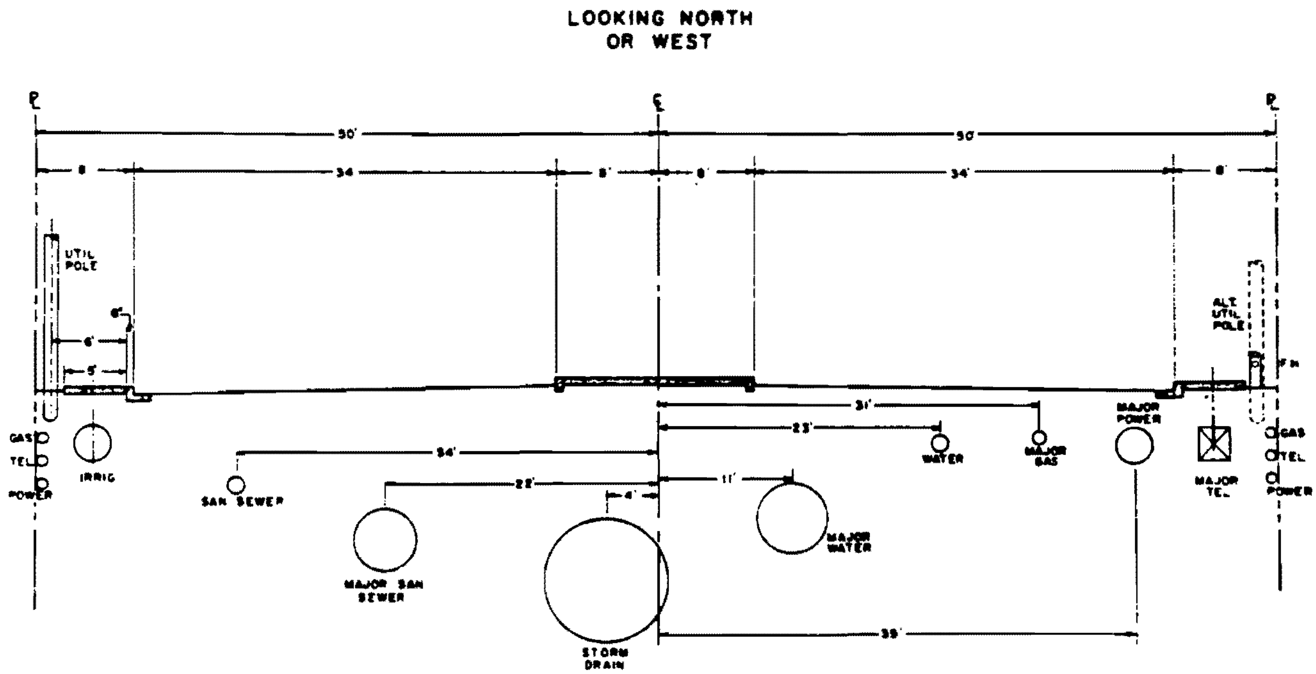


Figure 4. Location standards, major street, City of Phoenix, Arizona.

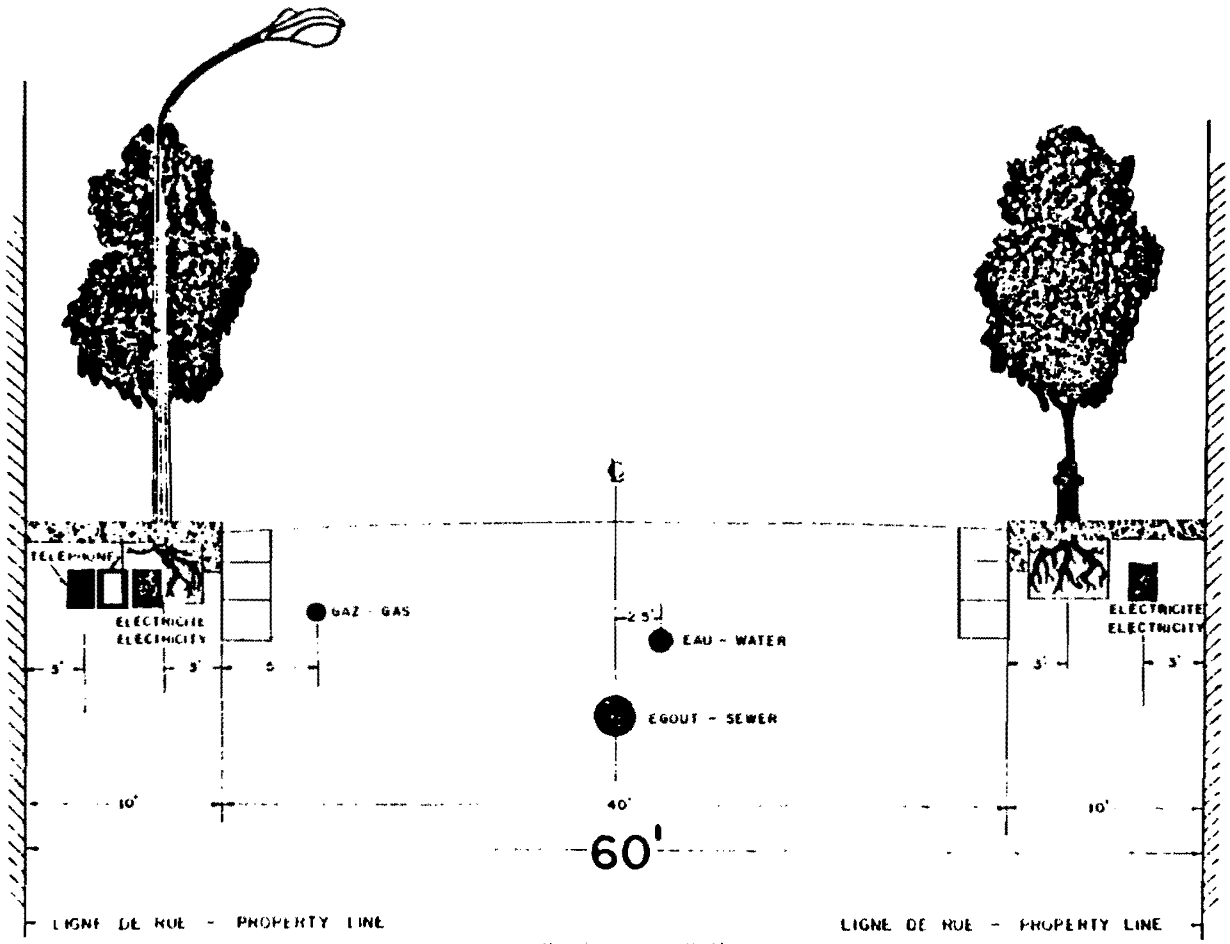


Figure 5. City of Montreal, Quebec.

among the participants. Any utilities which are placed at great depth or have other utilities placed above theirs in the trench may pay extra for unfavored locations. The corridor is difficult to use when the streets are curved as in subdivisions or on long winding lanes because sewer and storm sewer locations should generally be straight-line. Special problems are posed by roads with utilities constructed before the corridor design was adopted and roads which are of unusual configuration or width.

DUPLICATE SEWERS

Where the right-of-way is built-up and the road carries a heavy amount of traffic, or in situations where the road is unusually wide, duplicate sewers to serve each side of the road can be advantageous. An example of such a sewer is on State Street between 10600 South and 11000 South. State Street, besides being a heavy traffic route, is underlain by a 12-inch thick concrete slab from the previous highway. Since highway departments generally require whatever material is encountered be replaced, construction of numerous laterals along State Street becomes much more expensive than constructing one or two augered crossings and branching secondary or duplicate sewers from these crossings.

ADVANTAGES OF DUPLICATE SEWERS

Duplicate sewers make it easier to maintain traffic flow. Less paving of the street is required and the operation is generally safer. The cost is more than that of a single sewer but usually less than alternatives with long-length laterals or with frequent laterals for sewer service.

DISADVANTAGES OF DUPLICATE SEWERS

Duplicate sewers can create grade problems. The main sewer on the low side of the street becomes the "outlet" for the sewer on the opposite side. Unless the low-side sewer is quite deep, there can be service problems on the uphill side of the street. There is more sewer to maintain and infiltration/inflow is more likely. In a corridor plan, duplicate sewers can use one extra corridor which might otherwise be available for other utilities.

CURVED ALIGNMENT

In some areas it may be possible with permission to design a sewer using a curved alignment. The Utah State Code[3] indicates that sewer should be placed on a straight alignment but allows for submission of alternative designs where conditions warrant. It is important to be sure variances are allowed by state and local officials before a curved alignment is attempted.

It is easy to see why a curved alignment is attractive. Many country roads wind to-and-fro and more and more it is the fashion to have streets in subdivisions curved for aesthetic reasons. A curved sewer has a slightly greater cost than a straight-line sewer because of the extra labor involved in survey layout and alignment checking by the contractor.

A curved sewer is built from straight pipe and the curve is accomplished by deflecting the joints between one and three degrees each. It is important to carefully check the joint and gasket dimensions to be sure the pipe can withstand joint deflection over long periods of time and gaskets will properly seat.

ADVANTAGES OF CURVED ALIGNMENTS

The curved alignment can follow a curved street and conform with other utilities such as gas and water which are generally placed on curved alignment in their usual corridors. There is a better chance of avoiding traffic tie-ups and keeping off paving.

DISADVANTAGES OF CURVED ALIGNMENTS

Curved alignments need special approval in most states by the State Health Department, which may put an additional delay in the design. Inspection is more difficult since straight-line sighting through the pipe is not possible. Inspection may require a television survey to verify joint integrity and cleanliness of the pipe. Survey and layout is more complex and location of the pipe is more difficult once it is in place. It is desirable to place a metallic tape in the trench above nonmetallic pipes so that the sewer alignment can be located with a metal detector. A curved alignment is more likely to restrict solids at the joints than is a straight-aligned pipe. Cleaning is more difficult for the same reason. Infiltration is more likely in a curved alignment because of the joint deflections. Should an excess load be placed on a joint, there is less safety factor in the seal.

MOVING EXISTING UTILITIES

Occasionally it is impossible to locate a sewer within an existing right-of-way without moving some utilities. Some utilities are easier than others to move and repair. Examples are overhead lines, power poles, and landscape. More difficult are the surface utilities such as curb, gutter, sidewalk, and paving. The most difficult are the underground utilities such as storm sewers, gas lines, and water lines.

ADVANTAGES OF MOVING EXISTING UTILITIES

Moving utilities usually allows the sewer line to stay in the right-of-way. Usually a utility can be selected for movement which avoids conflict with traffic both for the sewer and the moved utility. Disadvantages are that the utility to be moved may be interrupted and can create scheduling difficulties for installing the sewer. The project is larger and therefore the expense is larger and more coordination is required.

BACK-YARD AND FRONT-YARD SEWERS

Occasionally it is impossible to locate a utility in an existing right-of-way and a new right-of-way must be maintained either in front of or behind the lots to be served. Sometimes such a sewer is desirable from a grade standpoint. An example is Eastridge Subdivision in Sandy, Utah. This subdivision has lots which are quite low on one side of the street with a sharp drop-off near the back of the lot. The sewer line was placed along the back lot line to receive flows from these homes. The sewer connects into the main line at the perpendicular street at the end of the back lot line. Occasionally front yard sewers are required where streets are unusually narrow and there is extreme utility congestion or traffic. Usually the sewer can be placed on the main right-of-way after buying a construction easement five to fifteen feet wide.

ADVANTAGES OF OFF RIGHT-OF-WAY SEWERS

Generally there is not traffic disruption and the sewer line does not interfere with other utilities or paving. There is less frost hazard to the sewer line since frost does not penetrate as deeply away from paved surfaces. Safety of the installation is quite good since the sewer line is away from traffic.

DISADVANTAGES OF BACK AND FRONT YARD SEWERS

Any foray into someone's front yard is full of problems, particularly when landscaping must be repaired or steps or special walls must be replaced. There is more maintenance involved in connection and laterals since every connection or operation on the pipe occurs on or near private property and all surface improvements must be restored. Manhole location is more difficult since homeowners generally insist manholes either not be on their property or be located according to their instructions. Construction is more difficult since the contractor must contact many owners to arrange sewer construction and restoration. Extra costs may be required for television monitoring of the construction before and after completion.

SPECIAL CROSSINGS

There are many special techniques for crossing existing utilities in the right-of-way. A few of these are listed and examples from the author's experience are described.

INVERTED SIPHON

The interceptor line from the Sandy, Utah, Sewage Treatment Plant is at grade with the Jordan River. A special inverted-siphon structure utilizing six 10-inch sewer lines connecting to bulkhead structures on either side of the river was used to avoid interference with the flow pattern of the river. The barrels are constructed with adjustable headgates which channel the flow to one barrel at a time depending on the incoming flow. For instance, at half capacity, three of the six barrels would be flowing. Special access for cleaning of the structure is provided at either end.

FREEWAY CROSSINGS

One typical freeway crossing and casing is in the 215-foot long crossing of Interstate 15 for the Applied Digital Data Company Plant in Draper. The casing pipe which was jacked under the freeway is oversized to allow for a larger carrier pipe to be installed at some future date.

CROSSING RIVER ABOVE GRADE

Where crossing a river is required and the crossing is above the grade of the river, a bridge structure with a carrying pipe can be used. The pipe crossing the river must be designed to carry live and dead loads of sewage and the pipe as well as seismic and settlement loads from the abutment structures.

CANAL CROSSINGS

In most cases in Salt Lake County canals are accomplished by open cut if the canal is not running or by augering underneath the canal during the irrigation season.

RAILROAD CROSSINGS

Railroads generally require that crossings be accomplished by protecting the carrier pipe within a casing pipe for the area under the entire right-of-way of the railroad. Common right-of-way width is 100 feet.

NARROW-ANGLE CROSSINGS

Any time a narrow-angle crossing of another utility can be avoided, it should be, for the benefit of both utilities. Sometimes in the cases of railroads and canals, such crossings are impossible and must be accomplished by supporting the existing utility during construction or by tunneling or augering a casing under the utility for the entire critical portion of the work. Where railroad crossings are involved, it is usually advisable to make as near a right angle crossing as possible even if this requires extra pipeline and manholes.

SUMMARY

In design of new sewers in built-up areas it is worthwhile to use a check-list like the one at the beginning of this paper for design or review of a design. Wherever possible, corridors should be established on engineering economic considerations and thoughtful design practices. Corridors may not be efficient or economical for all utilities involved but the overall cost to the public is less than first-come-first-served placement of utilities where no corridor is designated. The author suggests use of the alternatives for sewer placement.

As a parting note, it is extremely important to keep accurate records of the sewer line once it is placed to assure that future users of the right-of-way can be accommodated and not be needlessly penalized or jeopardized by inaccurate record keeping.

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OPERATIONAL AND MAINTENANCE REQUIREMENTS FOR INTERMITTENT SAND FILTERS USED TO UPGRADE LAGOON EFFLUENT AT MOUNT SHASTA, CALIFORNIA, MORIARTY, NEW MEXICO, AND AILEY, GEORGIA

*J. S. Russell, J. H. Reynolds, and E. J. Middlebrooks**

INTRODUCTION

In the past few years intermittent sand filters have gained acceptance as a viable means of upgrading waste stabilization lagoon effluent to meet secondary treatment standards. Research at Utah State University has established that intermittent sand filters can produce quality effluent and suggested operational and maintenance guidelines to enhance the success of the filters [*Harris, et al.*, 1977; *Hill, et al.*, 1977]. The main theme of these O & M guidelines was the ability of the filter to pass the applied wastewater. When the filtering ability is impaired as the result of plugging, one of two options can provide rejuvination of the filter.

The first option is to rake the surface of the filter (top 2-4 cm of sand). The second option is scraping the filter. Cleaning is accomplished by removing the top 2-4 cm of sand that has become heavily laden with filtered material. The combination of raking and scraping optimizes the cost of rejuvinating the filter bed.

The objective of this paper is to present the results of a sixteen month study of operation and maintenance practices at three full-scale lagoon-intermittent sand filter systems.

PROCEDURES

The study monitored the operational and maintenance requirements of three full-scale lagoon-intermittent sand filter facilities located at Mount Shasta, California, Ailey, Georgia, and Moriarty, New Mexico, for a period of over one year (Jan. 1976 to Apr. 1977). These facilities were selected to represent a range of design flow, and design variations (see Table 1) used in different geographic and climatic regions of the United States.

The data is a culmination of both operator reported and research team observed operations. The research team observed operation and maintenance requirements during three 30-day periods at each site. These observation periods represent three different seasons in relation to the geographic location of each facility. The operator of each facility recorded the operational and maintenance data during the periods between visits by the research team.

Table 1 presents the design criteria used for the three facilities. Of the facilities monitored, the largest and most complex facility was located at Mount Shasta, California (Figure 1). The design dry weather flow rate of 0.7 MGD is treated with three one-acre filters (six one-half acre filter sections). Mount Shasta is located at an elevation of 3000 ft in northern California and experiences a mild climate.

The Moriarty, New Mexico, facility illustrated in Figure 2 was designed to treat 0.2 to 0.4 MGD. Moriarty is situated on the high plains of New Mexico at 6200 ft above sea level and experiences a semi-arid climate.

Figure 3 illustrates the Ailey, Goergia, facility which was designed for a flow rate of 0.08 MGD. This small flow rate is characteristic of a small community of 400 people. Ailey is located on the eastern coastal plain at near sea level and experiences a humid climate.

RESULTS AND DISCUSSION

Table 2 presents a summary of the observed operation and maintenance data collected from the three facilities. The category of "Normal Operation" is a brief statement of the normal design operation mode prescribed by the design engineer for each facility. "Operational Variations" is a statement of acceptable variations or deviations from the engineer's design operational mode to accommodate unique operational problems or criteria. The categories listed as Tour #1, Tour #2, and Tour #3 present a summary of the operation of the intermittent sand filters as observed by the research team during the three monitoring periods at each facility. Included in these brief statements are the most significant problems observed during each period at each facility.

The final category is a comment addressing the overall operation of the intermittent sand filters and the major problems identified from the observations made by the research team.

A summary of required maintenance for the intermittent sand filters is presented in Table 3. The Mount Shasta facility required the most manpower of the three facilities studied. This is probably due to the relatively large size of the facility. The Ailey and Moriarty facilities both were estimated to require one man-year for operation and maintenance. The general trend indicates that less than the design estimation of manpower requirements was actually required at each facility.

The annual costs for the operation and maintenance requirements at the three facilities are presented in Table 4.

Capital cost estimates for the systems were derived from bid summary sheets provided by the design engineer or contractor. The cost per 1000 gallons of filtrate was calculated from annual costs based on a seven percent interest rate and 20 year service life.

Annual operating and maintenance costs for the three sites visited during the study were calculated using the observed maintenance and operating costs.

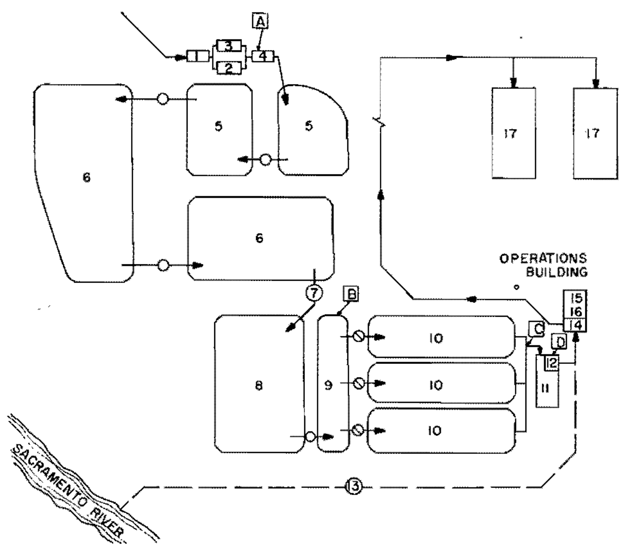
* *Mr. Russell is a research associate at the Utah Water Research Laboratory, Utah State University, Logan, Utah.*

*Table 1. Facility design criteria.**

Facility	Mount Shasta Water Pollution Control Facility	Moriarty, N.M. Wastewater Treatment Facility	Ailey, Georgia Sewage Treatment Plant
Design Flow Rate	Dry weather 0.7 MGD Wet weather 1.2 MGD	0.4 MGD maximum 0.2 MGD normal	0.080 MGD
Lagoons Number/type/size/depth/capacity			
Primary	2/aerated/1.4 acres/10 ft/3.2 MG	2 aerated/0.4 acre/10 ft/1.0 MG	1/facultative/5.5 acres/3.5 ft/4.8
Secondary	2/aerated/4.2 & 3.4/5 ft/3.4 & 2.8 MG	2/facultative/1.2 acres/3 ft/1.1 MG	1/facultative/0.75 acre/3.0 ft/0.8
Total retention time	12 days to 20.5 days	10 days to 20 days	70 days
Filters			
Number/area of each filter	3/1.0 acre (6/0.5 acre)	8/0.082 acre	2/0.14 acre
Design loading rate	0.7 MGAD	0.6 MGAD	0.4 MGAD
Media			
Sand			
Effective size (mm)	0.37	0.20	0.50
Uniformity coefficient	5.1	4.1	4.0
Depth	24 inches	24 inches	30 inches
Underdrain			
Layer depth/gravel size	18 in./coarse, 4 in./medium 3 in./fine, 3 in./coarse sand	4 in./pea gravel, 4 in./¾ inches remaining depth of 1½ inches gravel	3 in./coarse sand, 3 in./pea gravel, 3 in./medium, coarse
Available freeboard	24 inches	18 inches	18 inches
Dosing system	Time controlled butterfly valves	Automatic siphon (25,000 gal)	Float actuated valves (18,000 gal)
Distribution system	18 inch diameter manifolds with sixteen concrete splash pads	Six redwood distribution troughs (replaced with gravel splash pads)	3-6 inch diameter lateral plastic perforated pipes
Disinfection			
Type	Gas chlorinators	Tablet type	Gas chlorinators
Contact time	63 minutes	Variable with flow	One hour
Other Points of Interest	Summer discharge is pumped to a drain field above the city	Disinfection is accomplished prior to application to the filters	Use of two lift stations are required within the facility because of flat topography

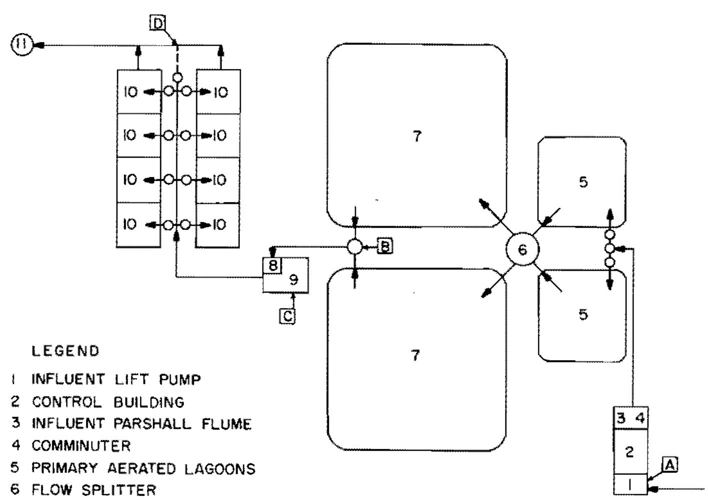
*Conversion to SI units

*1 MGD = 3785 m^3/d
1.0 MGAD = 9.360 $m^3/ha \cdot d$
1 MG = 3785 m^3
1 acre = 0.405 hectare
1 ft = 0.30 m
1 inch = 2.54 cm



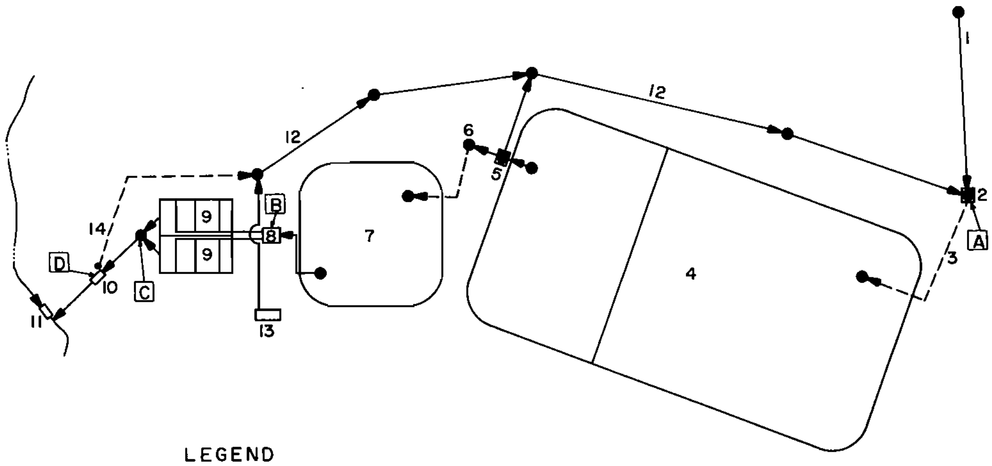
LEGEND
1 BAR RACK
2 COMMINUTER
3 BYPASS BAR RACK
4 INFLUENT PARSHALL FLUME
5 PRIMARY AERATED LAGOONS
6 SECONDARY AERATED LAGOONS
7 LAGOON SYSTEM PARSHALL FLUME
8 BALLAST LAGOON
A LAGOON INFLUENT SAMPLING POINT
B LAGOON EFFLUENT SAMPLING POINT
9 DOSING BASIN
10 INTERMITTENT SAND FILTERS
11 CHLORINE CONTACT BASIN
12 EFFLUENT PARSHALL FLUME
13 RIVER DISCHARGE LINE
14 OUTFALL PUMP STATION
15 CHLORINATOR / SULFONATOR
16 AERATION BLOWERS
17 WATER RECLAMATION SYSTEM
C FILTER EFFLUENT SAMPLING POINT
D CHLORINATED FILTER EFFLUENT POINT

Figure 1. Mount Shasta, California, water pollution control facility process flow diagram.



LEGEND
1 INFLUENT LIFT PUMP
2 CONTROL BUILDING
3 INFLUENT PARSHALL FLUME
4 COMMINUTER
5 PRIMARY AERATED LAGOONS
6 FLOW SPLITTER
7 SECONDARY FACULTATIVE LAGOONS
8 TABLET CHLORINATOR
9 DOSING BASIN WITH AUTOMATIC SYPHON
A LAGOON INFLUENT SAMPLING POINT
B LAGOON EFFLUENT SAMPLING POINT
10 INTERMITTENT SAND FILTERS
11 DISCHARGE OUTFALL LINE
O MANUAL VALVES
C CHLORINATED LAGOON EFFLUENT SAMPLING POINT
D FILTER EFFLUENT SAMPLING POINT

Figure 2. Moriarty, New Mexico, wastewater treatment facility process flow diagram.



LEGEND

1 INFLUENT MAIN LINE
2 LIFT STATION #1
3 FORCED MAIN
4 OXIDATION POND
5 FLOW SPLITTER
A LAGOON INFLUENT SAMPLING POINT
B LAGOON EFFLUENT SAMPLING POINT
6 LIFT STATION #2
7 POLISHING POND
8 DOSING BASIN
9 INTERMITTENT SAND FILTERS
10 CHLORINE CONTACT CHAMBER
C FILTER EFFLUENT SAMPLING POINT
D CHLORINATED FILTER EFFLUENT SAMPLING POINT
11 IN-STREAM PARSHALL FLUME
12 SEWER RETURN LINE
13 CONTROL BUILDING
14 CONTACT CHAMBER DRAIN LINE

Figure 3. Ailey, Georgia, sewage treatment plant process flow diagram.

Table 2. Summary of intermittent sand filter operation.

Operation Category	Mt. Shasta, California Water Pollution Control Facility	Moriarty, New Mexico Wastewater Treatment Facility	Ailey, Georgia Sewage Treatment Plant
Filter System Normal Operation	Normal operation (May 1 to Oct 31) Timer controlled automatic dosing. Hydraulic loading rate variable. Filter cycling--1 day load and 2 days rest.	Automatic dosing of four filters with dosing siphon. Hydraulic loading rates dependent on discharge rate from lagoon system, disinfection prior to filtration.	Automatic alternate dosing of the two filters. Hydraulic loading rate is dependent on lagoon discharge rate.
Operational Variations	Filters to be used in the winter only when the lagoon effluent doesn't meet discharge requirements.	During extremely cold weather when filter freezing can occur filters should be bypassed.	During dry weather, filter operation is restricted to ratio of 1 to 1 relative to discharge rate and stream flowrate.
Tour #1	Simultaneous use of two filters with 4 doses per day per filter providing 6 hours rest between doses. Problems: frozen filters, high hydraulic loading.	Dosing siphon malfunction resulted in unequal dosing cycles. Use of two filters simultaneously. Problems: short run time.	Normal operation.
Tour #2	Parallel systems, slow sand and intermittent sand filter operation. Problems: excessive sand removal during cleaning.	Dosing siphon functioning properly. Manual alternating of two filters simultaneously resulting in reduction of loading rate.	Malfunction in automatic dosing device resulted in manual dosing part of the tour. Normal operation resumed upon repair.
Tour #3	Four daily doses per filter, filter cycling 1 day load and 2 days rest. Problems: excessive loading, poor filter condition due to mixing of top 20-25 cm of sand.	Same as Tour #2, using all eight filter sections provides one day loaded and three days rest for each filter group.	Normal operation. Problems: high loading rates.
Comments	20-25 cm of sand removed in two cleanings. Filter loading to provide constant flow to disinfection system.	Problems: short service life due to wind blown soil layer, calcium carbonate precipitation.	System performed without major incident.

Table 3. Summary of reported and observed maintenance.

Job Description	Mount Shasta WPCF	Moriarty WWTF	Ailey STP
Daily operation and maintenance (daily monitoring)	(1.0 hr) × 7 days × 52 wks = 365 hrs	(1.0 hr) × 7 days × 52 wks = 365 hrs	(0.5 hr) × 5 days × 52 wks = 130 hrs
Filter cleaning	54[1] hrs	28[1] hrs	None
Filter raking	12 hrs raking[1] 16 hrs mixing[1]	13 hrs	22 hrs
Filter weed control	N.A.	None	26 hrs
Miscellaneous maintenance	N.A.	11 hrs	None
Grounds maintenance	42 hrs	8 hrs	28 hrs
Total reported man-hours	489 man-hours	425 man-hours	206 man-hours
Design estimated manpower requirements	2.4 man-years[2]	1 man-year[2]	1 man-year[2]
Actual reported manpower input	2.0 man-years[3]	0.28 man-year[2]	0.14 man-year[2]

[1] Man-hours with mechanical assistance.
[2] Assuming 1500 man-hours = 1 man-year.
[3] Considering extra assistance for filter cleaning and weekend monitoring.

Table 4. Annual costs for operation and maintenance at Mount Shasta, California, Moriarty, New Mexico, and Ailey, Georgia ($/1000 gallons of filtrate).

Cost Category	Locations		
	Mt. Shasta	Moriarty	Ailey
Filter Capital Cost	0.19	0.12	0.20
Filter O & M Cost	0.04	0.04	0.02
Filter (Total)	0.23	0.16	0.22

The observed operations at the Mount Shasta facility exhibited the greatest degree of variability of the three systems monitored. The major source of the variations was caused by operator-induced changes in operation or accidental breakdown of equipment. The relative size of the Mount Shasta facility can be used to rationalize the magnitude of operational problems. The major problem identified was the operator's reluctance to let the system run on automatic controls.

The Moriarty system provided good service after some initial start-up problems were solved. The major problem experienced in the early phase of the study at the Moriarty facility was the result of wind-blown soil that accumulated on the surface of the filters. The accumulated soil in combination with photosynthetically induced calcium carbonate precipitation cited by *Reynolds, et al.* [1974] was responsible for the shortening of filter run lengths from two months to five days.

When the soil was removed, the filters provided excellent filter run lengths. The filters provided up to four months of service with only one raking required. Winter operation of these filters experienced no problems. With ambient temperatures as low as 20°F (-7°C), the filters were in use with only a thin layer of ice forming on the surface of the water as it passed through the sand. The thin layer of ice would be melted by the solar radiation during daylight hours.

The Ailey system exemplified the ability of a simple system to be operated with a minimum of operational and maintenance requirements. The only problem encountered was a prolific growth of weeds on the surface of the sand during the summer months. The removal of the weeds was performed manually with the aid of garden rakes. The Ailey filters provided up to seven months of service without requiring any maintenance. Both filters at the Ailey facility operated well over one year without having any sand removed or required raking. The weeds were removed only three times during one year of operation.

SUMMARY

An overall look at the maintenance requirements illustrates that the Moriarty and Ailey facilities both provided excellent service with a minimum of manpower required. On the other hand, the Mount Shasta facility maintenance requirements were substantial.

The cost of operation and maintenance at the Mount Shasta and Moriarty facilities was estimated to be $0.04 per 1000 gallons of filtrate. The lowest cost of operation and maintenance was demonstrated by the Ailey facility with a cost of $0.02 per 1000 gallons of filtrate.

The general conclusion is that a simple system with diligent observation and operation can provide excellent service at a minimum of cost.

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SALT LAKE CITY'S WASTEWATER SAFETY PROGRAM

*Jack H. Petersen**

An important issue which must be dealt with is the degree of accident rates in the wastewater collection and treatment industry. The wastewater collection and treatment industry has one of the highest accident rates among all industrial occupations.

There are many reasons for the high accident rates. Very few treatment facilities have established extensive employee training and safety programs. Some of the excuses are lack of personnel, lack of funds, and lack of interest.

Most all accidents are caused by human error, about 90 percent. Hazardous conditions and faulty equipment make up the other 10 percent. This emphasizes the need for reduction of this factor by having a training program which promotes safety awareness. There are two main components: safety meetings and safety inspections.

Several years ago, I realized that our plant needed an organized safety program. Until then the informal get-togethers and safety talks had seemed enough, but they were falling short of our needs. To satisfy our needs and develop and implement a comprehensive safety program, I established a safety committee for our plant. Operators and maintenance personnel make up the committee. Their duties include inspection of the facility and the personnel to determine the working conditions and working habits. Twice a month the committee reports its findings to me, Ray (Assistant Superintendent), and Willy (Operations Supervisor). Violations are noted, solutions discussed, and we set a timetable to make the corrections. Minutes of each meeting are typed and put in our safety book. This book is very valuable to us because it documents our violations and corrections, and can be used as the basis for personnel action. The committee has the power to issue citations and time off without pay for repeated violations. This policy includes all personnel at our plant, even management. To indicate how well the safety program has been accepted, no citations have been issued yet.

It is important to me that the men police themselves. Safety begins with them, and the program is rightly theirs. This gives you a quick insight into our safety program.

One of the first and most important duties of the Safety Committee is to establish safety rules. The following safety rules were written by the safety committee chairman at our plant, Hyrum Frank, and adopted by the safety committee.

1. All electrical work shall be done by qualified electricians. NO EXCEPTIONS.
2. There shall be NO water sprayed around or above any electrical centers.
3. Unauthorized persons shall not use welding, cutting, and brazing equipment.
4. Wrist watches, rings, or other jewelry shall not be worn on the job where they constitute a safety hazard.
5. Hard hats are for your own protection, and they shall be worn in the following places:
   a. Entering and working in manholes.
   b. Overhead protection - means hard hats shall be worn when working on scaffolds where overhead hazards exist, and anytime any work is done overhead.
   c. Working in Grit Channel or any other area where overhead hazards exist.
6. Hard-toes (safety shoes) shall be worn at all times during working hours.
7. Good housekeeping shall be kept throughout the plant at all times. An excessively littered and dirty work area will not be tolerated, as it constitutes an unsafe and hazardous condition of employment.
8. No unauthorized personnel shall use any equipment without permission.
9. Goggles or safety glasses and gloves shall be worn when grinding, chipping, and wherever an eye injury can occur.
10. NO SMOKING in hazardous locations.
11. Get help when handling heavy material and use appropriate lifting or carrying devices available.
12. Use safe tools only.
13. Safety precautions when using flammable or toxic materials (proper ventilation, no smoking, rubber gloves).
14. Know fire extinguisher locations and proper type and usage. Keep extinguisher areas clear of material or debris.
15. One man shall never work alone on energized equipment that operates at or above 440 volts.
16. DO NOT attempt to clean on or near operating equipment without shutting it down.
17. DO NOT enter hazardous areas without adequate training, personal protection, assistance, or authorization.
18. DO NOT climb out on Clarifier grease skimmer ramp.
19. Nobody is to climb into Launders. Use designated stairway ONLY.
20. As of this date, February 10, 1977, there shall be more than two men on a shift. When only two men are on Operator's shifts (afternoon and midnight shifts), another man shall be called in.
21. There shall be no long hair and long sideburns (over two inches long.) There shall be absolutely NO BEARDS.

Safety in the Salt Lake City wastewater collection and treatment facilities is an integral part of good operations and produces the following plus values:

1. All work being performed in an efficient manner without being interrupted by accidents.

** Jack H. Petersen is Deputy Water Reclamation Superintendent, Salt Lake City Corporation.*

2. Employees knowing and following safe work practices.
3. Receiving favorable publicity because of good safety records.
4. Costs being controlled by the elimination of unnecessary expense of accidents.
5. Good employee relations and low turnover.

There are seven basic elements of a good safety program:

1. Management Leadership
2. Assignment of responsibility
3. Maintenance of safe working conditions
4. Establishment of safety training
5. *Accident analysis and records*
6. Medical and first aid
7. Acceptance of personal responsibility of employees

Action in the area of safety must begin at the management level, and every supervisor in the organization must accept this fact.

With respect to knowledge, the old saying "experience is the best teacher," has a great deal of merit; but safety knowledge is one exception. Preparing safety rules can help employees avoid injury and benefit from the experience of others.

Employee group safety meetings, when properly planned and conceived, can be effective in reducing employee injuries, improving employee morale, and stimulating an employee esprit de corp, their cost will be offset by reduction in insurance premiums, improved efficiency, reduction in operation failures and improved employee and public relations.

ALGAE REMOVAL FROM STABILIZATION POND EFFLUENT VIA MICROSTRAINING

*L. S. Barker, B. T. Hicken, R. P. Bishop**

INTRODUCTION

The City of Burley, Idaho, initiated efforts to improve effluent quality from its stabilization lagoons by completing a wastewater facilities plan [*CH2M Hill, Inc.*, 1976]. The plan, completed in 1976, recommended that improvements to the city treatment facilities be designed and constructed in two phases. Phase I included an aerated pond (which preceded the existing lagoons and provided increased loading capacity in order to handle an expanding population base), and a chlorine disinfection facility. Phase I facilities are scheduled for startup in March 1979. Phase II improvement involved the more difficult task of selecting an algae removal process to upgrade treated effluent to secondary standards.

An addendum to the facilities plan [*CH2M Hill, Inc.*, 1977] examined several algae removal techniques including intermittent sand filtration, chemical addition (alum), air flotation and filtration, and phase isolation. The addendum recommended chemical addition air flotation and filtration as the alternative offering the greatest performance reliability. However, as a result of a desire to find an alternative with low operation and maintenance costs, the city decided to pilot test a microstrainer algae removal system [*Barker*, 1978]. This paper summarizes the results of pilot testing two manufacturers of microstrainers and includes an economic comparison of microstraining vs. chemical addition, air flotation, and filtration.

LITERATURE REVIEW

Microstrainers have not proven historically a reliable method for separating algae from stabilization lagoon effluent [*Middlebrooks, et al.*, 1974; *Brown and Caldwell*, 1974; *Dryden and Stern*, 1968]. Perhaps the main reason has been that mesh openings for microstrainers have traditionally ranged between 23 and 60 microns while the algae found in lagoons is often as small as several microns in size. New polyester fabric, however, has reduced the mesh opening to one micron, thus allowing microstrainers to remove algae.

A recent study in Scottsbluff, Nebraska, [*Envirex*, 1978] found that domestic stabilization pond effluent containing from 35 to 98 mg/ℓ (average 67 mg/ℓ) total suspended solids (TSS) could be lowered to an average of nine mg/ℓ by microstraining, using units having one micron filter cloth. Similarly, stabilization pond effluent containing 21 to 61 mg/ℓ (average 41 mg/ℓ) was lowered to an average of 14 mg/ℓ at Gehring, Nebraska [*Envirex*, 1978]. Western Nebraska has a climate similar to that in Burley so that the successful operating results from Nebraska should be applicable to Burley.

Envirex has conducted numerous other successful microstrainer pilot tests on domestic lagoon effluent [*Kormanik and Cravens*, 1978; *Cravens and Kormanik*, 1978]. Pilot tests in Alabama, Georgia, Missouri, Oklahoma, and South Carolina all produced microstrainer effluent TSS concentrations less than 22 mg/ℓ. Table 1 summarizes results from the Envirex testing program.

Table 1. Envirex microstrainer pilot results.[1]

Site	Lagoon Effluent TSS (mg/ℓ)	Microstrainer Effluent TSS (mg/ℓ)
Greenville, Alabama	44	12
Adel, Georgia	69	9
Cumming, Georgia	26	6
Blue Springs, Missouri	64	22
Owasso, Oklahoma	58	15
Camden, South Carolina	126	19

[1] *Kormanik and Cravens*, 1978; *Cravens and Kormanik*, 1978.

Union Carbide Corporation pilot tested a one micron microstrainer on their Seadrift, Texas, lagoon effluent and averaged only a 36 percent removal of TSS, without polymer addition [*Union Carbide Corporation*, 1979]. However, extremely heavy rainfall during the test period may have upset the ecological balance of the lagoons and produced abnormal test conditions.

METHODS AND PROCEDURES

MICROSTRAINER EQUIPMENT

Two microstrainer pilot units were installed adjacent to the secondary cell of the Burley stabilization pond system where tests were conducted during September, October, and November of 1978. One of the microstrainers was manufactured by Zurn Industries, Inc. and the other by Envirex, Inc.

Both microstrainers had a 1.219 m (4-foot) diameter by 0.61 m (2-foot) long drum type configuration. The drums were covered with polyester strainer cloth having a one micron pore size. The microstrainers used electric motors to rotate the strainer drums. Portable pumps were used to withdraw water from the stabilization pond and transport it to the microstrainer units.

The microstrainers removed suspended solids by passing wastewater through a strainer cloth which retained solids greater than one micron. A continuous, pressurized backwash spray of effluent cleaned the strainer cloth. A trough captured and carried away backwash water. In a full-scale microstraining facility, the concentrated solids in the backwash water would be recycled to the primary cell of the

* *Sheldon Barker is an engineer with CH2M Hill, Inc. in Boise, Idaho.*

Burley stabilization pond system.

SAMPLING

Sampling was performed on a daily grab basis with the Zurn unit, and by an automatic sampler with the Envirex unit. Samples were taken of the microstrainer influent, effluent and backwash waters. All three locations were sampled simultaneously.

SAMPLE ANALYSES

The test parameter of prime interest was total suspended solids. Other test parameters were pH, dissolved oxygen, and temperature. All tests are in accordance with procedures as outlined in *Standard Methods* [1975].

PHYSICAL-MECHANICAL ANALYSES

Physical-mechanical parameters investigated were backwash water flow rate and pressure, drum speed, drum submergence, and drum loading rate. These parameters are vital to evaluation of the technical economic feasibility of a full-scale microstraining system.

Visual observations were made of the microstrainer influent, effluent, and backwash waters.

RESULTS AND DISCUSSION

MICROSTRAINER OPERATION - ENVIREX

The Envirex microstrainer functioned without mechanical breakdown. A mild chlorine solution added to the backwash unit prevented clogging of the straining cloth. The half-hour chlorine treatments every three to five days removed accumulated slime growth. The backwash nozzles on the Envirex unit were a nonclog design and functioned without stoppage.

As shown in Figure 1, the Envirex microstrainer was consistently able to produce effluent with a TSS content of 20 mg/ℓ or less, well below the limit for secondary treatment. However, the influent to the microstrainer was less than the 30 mg/ℓ limit 65 percent of the time. Equally important is the fact that when influent TSS concentration peaked far above the 30 mg/ℓ level, as high as 80 mg/ℓ, the effluent TSS concentration remained less than 20 mg/ℓ.

The performance of the Envirex microstainer is encouraging. However, some conditions during the test period make it impossible to conclude, based on the Burley results alone, that large concentrations of single cell algae can be removed from the Burley stabilization pond effluent by microstraining.

One condition was the predominance of daphnia in the influent wastewater and sparse populations of algae. It can be concluded that daphnia, rather than algae, composed the major portion of the suspended solids removed.

Another condition, which resulted from unavoidable Phase I construction activities, was the diverting of raw sewage into the secondary stabilization pond at a location approximately 12 m (40 feet) from the suction line to the microstrainer. The microstrainer was processing a wastewater atypical of algae laden secondary pond water.

There is reason to believe, however, that under normal operation of the stabilization pond system, with typical secondary pond water having algae as the major TSS component, microstraining could be effective in lowering suspended solids concentrations to 30 mg/ℓ or less. The algae forms observed to be predominant in the Burley wastewater stabilization ponds are larger than the one micron microscreen mesh. With proper sealing of the microscreen drum, the only particles passing through the screen should be small amounts of colloidal particles, juvenile algae, or fragments of dead algae cells. Visual observation of the Envirex microstrainer operation in Burley gave a qualitative indication that algae was being removed.

MICROSTRAINER OPERATION - ZURN

The Zurn microstrainer was plagued with operational problems. After about one week of use, the filter cloth on the Zurn unit became blinded with slime growth, severely restricting the unit flow capacity. Backwash spray nozzles required frequent removal for cleaning. Blinding of the strainer cloth with slime growths indirectly caused poor suspended solids removal. When the screen was blinded, the headloss became great enough for influent water to spill into the overflow bypass and into the effluent chamber. The effluent analyzed from the machine had a suspended solids content very near that of the influent, as shown by test results summarized in Figure 2. Another report has concluded that with periodic chlorination, the Zurn unit may have performed comparably to the Envirex machine [*CH2M Hill, Inc.*, 1979].

It was also observed that daphnia were found in effluent from the Zurn microstrainer at various times when the unit was not hydraulically overloaded. Because daphnia are much larger than the one micron screen size, a leak probably existed in the rubber gasket between the rotating filter drum and effluent chamber. A leak in this seal would provide a pathway for large suspended solids to bypass the strainer element.

TEST SCALE-UP

No attempt to scale-up results of the Zurn test has been made because of the lack of positive data. The Envirex test, however, provided operational parameters to be used in preliminary design. As stated in the Envirex summary report [*Cravens*, 1978], a hydraulic application rate of 1.02 liters per second/meter^2 (1.5 gpm/sq ft), backwash rate of 0.145 liters per second per lin meter (0.7 gpm/lin ft), 54 percent drum submergence, and a constant 0.305 m (12-inch) headloss through the strainer resulted in successful operation. Using these factors, four 3.05 m (10-foot) diameter by 4.88 m (16-foot) long microstrainers would be required to treat Burley's design flow of 8,517 meter^3/per day (2.25 MGD). The microstrainer would be capable of operating with 0.61 m (2-foot) headloss during peak TSS loading conditions.

A schematic design of the total Burley wastewater facility is shown in Figure 3.

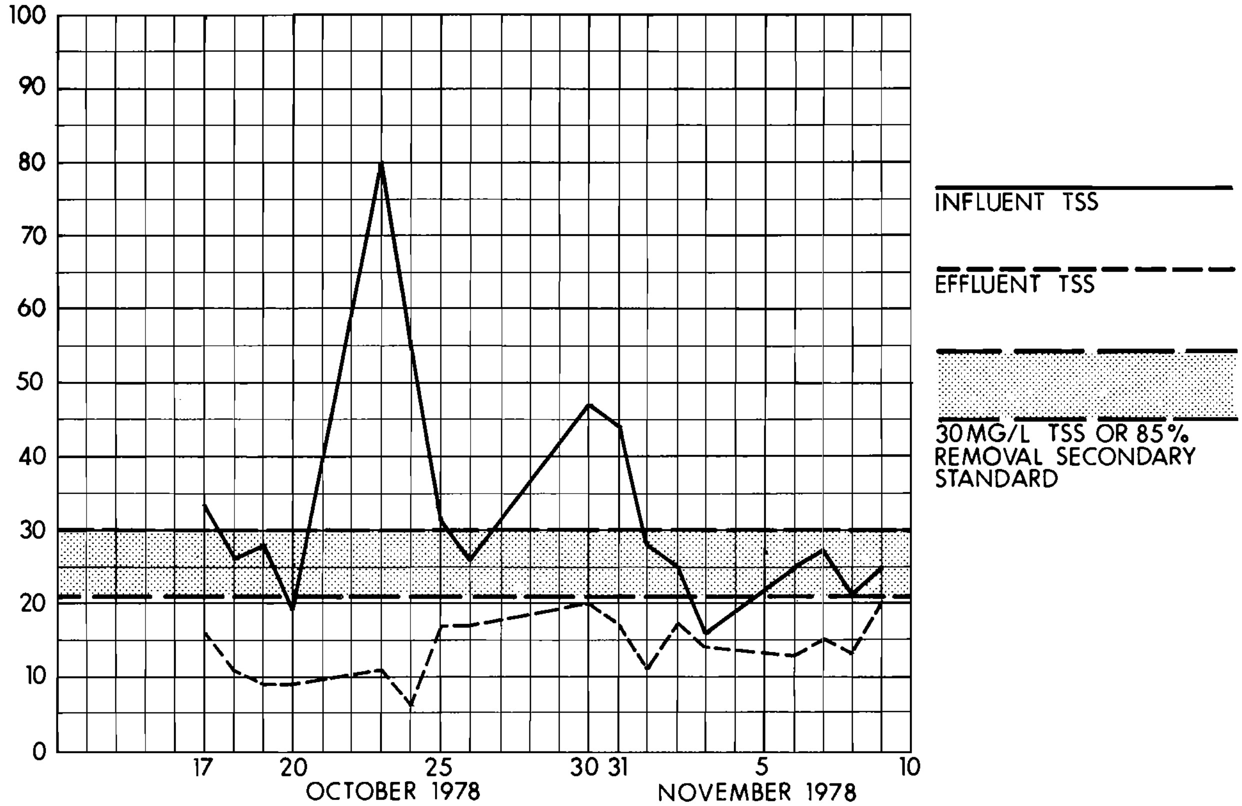

Figure 1. Total suspended solids removal with Envirex microstrainer.

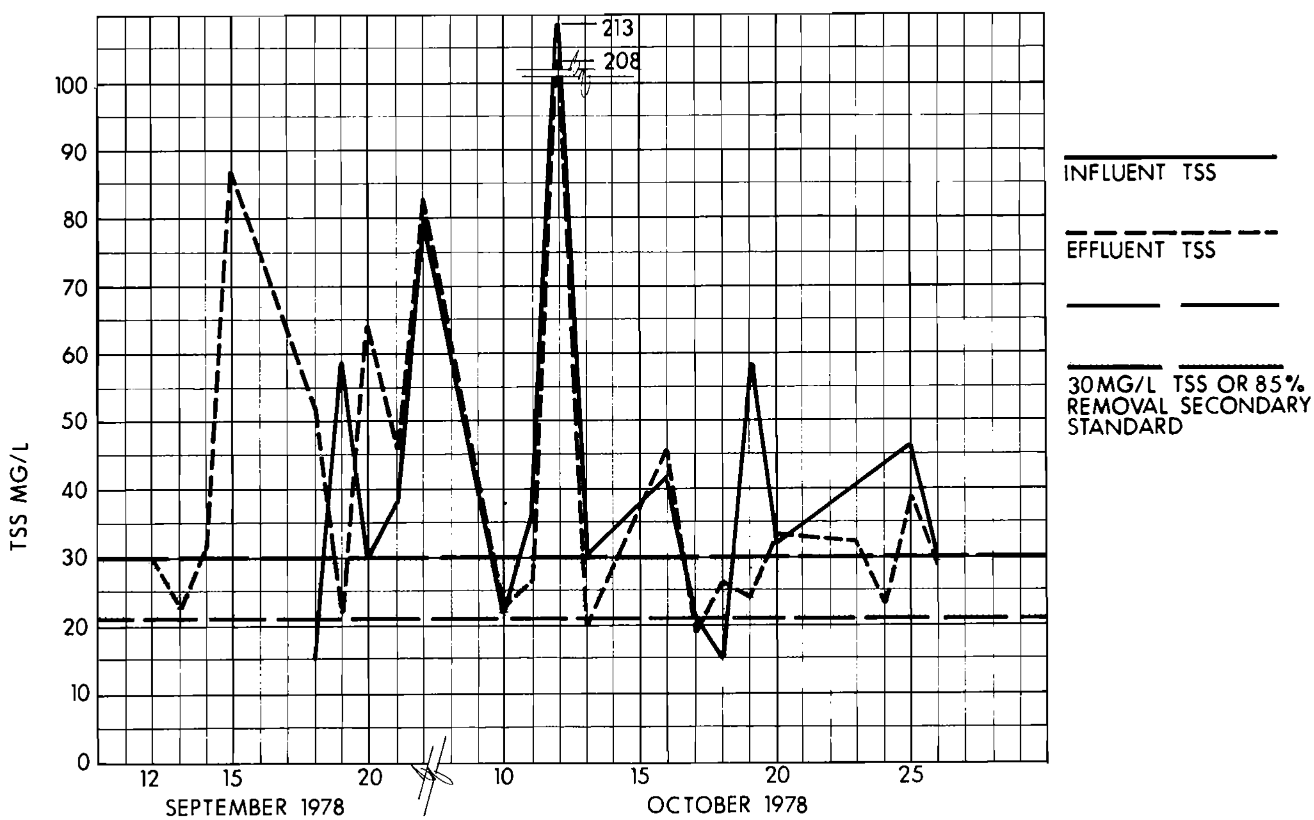

Figure 2. Total suspended solids removal with Zurn microstrainer.

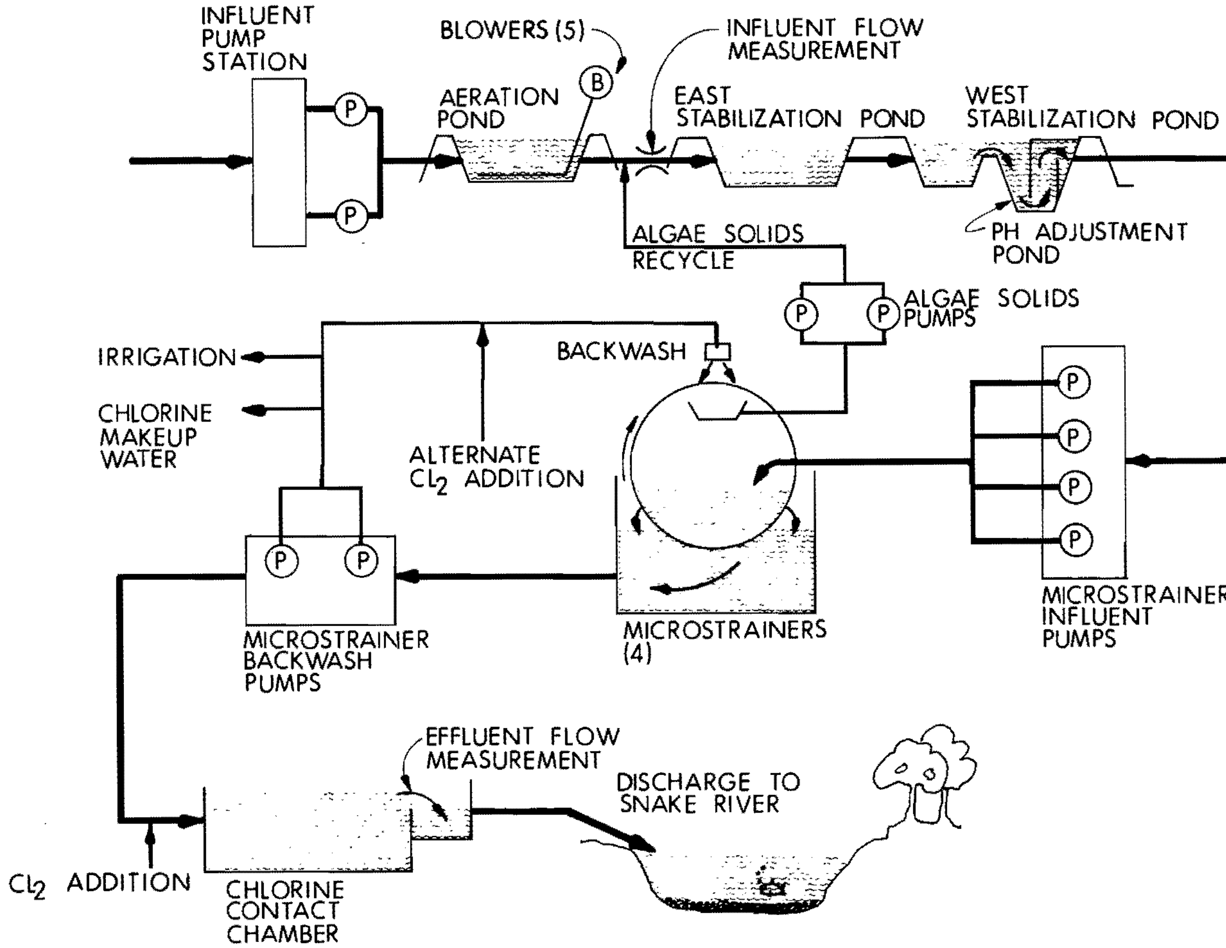

Figure 3. Schematic diagram of Burley wastewater treatment facilities.

ECONOMICS: MICROSTRAINER-AQUACULTURE VERSUS CHEMICAL TREATMENT WITH FILTRATION

In the Facilities Plan [*CH2M Hill, Inc.*, 1976] and Addendum No. 1 [*CH2M Hill, Inc.*, 1977] chemical treatment (alum addition) and air flotation with filtration was found to be the cost-effective treatment alternative. However, results from this literature review and pilot testing indicates that microstraining is an effective alternative and should be economically compared with the previous cost-effective alternative.

Examination of Tables 2 and 3 show that a slight capital cost savings may be realized if a microstrainer system is used instead of chemical treatment, air flotation and filtration. Within the accuracy of the estimates, however, the capital costs should be considered equal.

A definite cost advantage with the microstrainer alternative is apparent when operation and maintenance (O&M) costs are examined. As shown in Tables 4 and 5, the annual O&M cost for chemical treatment, air flotation and filtration is 2.0 times greater than the O&M cost for the microstrainer alternative. The cost savings is realized because the uncomplicated nature of the microstrainer requires comparatively low operator attention and skill level, thus saving labor costs. Also, no costly chemical additives are required for the microstrainer alternative. An added benefit to the microstrainer alternatives is a 25 percent savings in primary energy.

Table 2. Capital cost estimate, microstrainer alternative treatment.[1]

Bond and Insurance	$ 15,000
Move-in and Temporary Facilities	5,000
Microstrainers	499,900
Pumping-Secondary Effluent, Backwash, Waste Algae	82,400
Treatment Building	227,300
Irrigation	3,600
Sampling Equipment	4,000
Yard Piping	20,400
Landscaping	6,000
Electrical	60,500
Subtotal	$ 924,100
Contingency and Engineering - 35 percent	323,400
TOTAL CAPITAL COSTS	$1,247,500

[1] ENR-CCI = 3256; June 1980

Table 3. Capital cost esminate: chemical treatment, air flotation, and filtration.[1]

Bond and Insurance	$ 15,000
Move-in and Temporary Facilities	5,000
Flocculation Basin	30,900
Air Flotation Unit	231,100
Mixed Media Filtration	130,100
Sludge Pumping	19,500
Yard Piping	40,600
Sludge Lagoons	71,800
Chemical Handling and Storage	19,000
Decant Pump Station	20,400
Treatment Building	215,100
Pump Station	103,000
Irrigation	3,600
Sampling Equipment	4,000
Landscaping	6,000
Electrical	64,000
Subtotal	$ 979,100
Contingency and Engineering - 35 percent	342,700
TOTAL CAPITAL COSTS	$1,321,800

[1] ENR-CII = 3256; June 1980

Table 4. Operation and maintenance cost estimate: microstrainer alternative.[1]

Labor[2]	$10,800
Power[3]	10,500
Equipment Repair and Maintenance[4]	15,000
Subtotal	$36,300
Annual Equipment Replacement Sinking Fund	43,700
TOTAL ANNUAL OPERATION AND MAINTENANCE COST	$80,000

[1] All costs are based on an assumed ENR-CII of 3256, June 1980.
[2] Hourly wage at $8/hour including fringes and benefits.
[3] Electrical power costs computed at $0.02/kwh.
[4] Equipment repair and maintenance at three percent of original major equipment cost.

Table 5. Operation and maintenance cost estimate: chemical treatment, air flotation and filtration.[1]

Labor[2]	$ 33,300
Electric Power[3]	14,000
Equipment Repair and Maintenance[4]	7,200
Chemical Cost[5]	68,100
Sludge Removal	3,900
Subtotal	$126,500
Annual Equipment Replacement Sinking Fund	33,600
TOTAL ANNUAL OPERATION AND MAINTENANCE COST	$160,100

[1] All costs are based on an assumed ENR-CII of 3256.
[2] Hourly wage at $8/hour including fringes and benefits for two operators.
[3] Electrical power costs computed at $0.02/kwh.
[4] Equipment repair and maintenance at three percent of original major equipment cost.
[5] Liquid alum delivered to Burley at $0.182/kg ($0.087/lb).

On a net present-worth basis, microstraining is 34 percent less costly than chemical treatment, air flotation and filtration over a 20-year design life. Table 6 shows the salvage values used in part in determining the net present-worth values for the two treatment alternatives. Table 7 shows the net present-worth of the microstrainer alternative to be $1,886,200 vs. $2,851,300 for chemical treatment, air flotation, and filtration.

Table 6. Estimated salvage values.

Item	Microstrainer Aquaculture	Chemical Treatment With Filtration
Equipment Replacement Sinking Fund[1]	$600,200	$461,500
Land	14,600	14,600
Structures	205,800	216,600
SALVAGE VALUE (at End of 20-year Design Life)	$820,600	$692,700

[1] 6⅞ percent interest.

Table 7. Estimated net present worth.[1,2]

	Microstrainer Aquaculture	Chemical Treatment With Filtration
Capital Cost	$1,247,500	$1,321,800
Operation & Maintenance	855,800	1,712,700
Salvage	- 217,100	- 183,200
NET PRESENT WORTH	$1,886,200	$2,851,300

[1] 20-year amortization.
[2] 6⅞ percent interest.

CONCLUSIONS

1. Pilot tests at several locations have shown microstraining to be an effective means of polishing domestic stabilization pond effluent.

2. The microstrainer pilot test in Burley does not, in itself, prove algae suspended solids can be adequately removed by microstraining.

3. Test conditions in Burley, rather than any failure in process equipment precluded the complete evaluation of algae suspended solids removal by microstraining.

4. Weekly or more frequent treatment of the microstrainer filter cloth with a mild chlorine solution is necessary to prevent blinding with slime growths.

5. Because of advances in microstrainer technology (*i.e.*, development of the one micron filter cloth), and because algae typically found in stabilization ponds are larger than one micron, microstrainers should now be effective in removing algae suspended solids from stabilization

pond effluent.

6. A microstrainer treatment system is 34 percent less costly than a chemical treatment, air flotation and filtration system over a 20-year design life.

7. A 25 percent savings in primary energy may be realized if microstrainer treatment is used rather than chemical treatment, air flotation and filtration.

8. Four 3.05 m (10-foot) diameter by 4.88 m (16-foot) long microstrainers with one micron pore size filter cloth are required to treat Burley's design flow of 8,517 meter^3/day (2.25 MGD).

ACKNOWLEDGMENT

We appreciate the assistance of Mr. Bob Martin, Water and Sewer Superintendent, and Mr. Rod Smith, treatment plant operator, of the city of Burley, for their help in completing the pilot work and analyzing results.

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UPGRADING TRICKLING FILTERS

A CASE STUDY OF THE WASTEWATER TREATMENT PLANT AT BEDFORD, OHIO

*Gary A. Jones**

FORWARD

In 1972, Congress set into motion a comprehensive program to restore and maintain the Nation's rivers and lakes by passage of amendments to the Federal Water Pollution Control Act. The recent Clean Water Act of 1977 reaffirmed this commitment by adopting additional amendments which strengthened a number of provisions of the law.

A major element of the country's clean water strategy is to improve the quality of the effluent discharged from municipal wastewater treatment works. Federal funds for the construction of municipal wastewater treatment works provide the cornerstone on which the municipal program is built. With the availability of large amounts of Federal grant funds, there may be a tendency to choose capital intensive and more complex newer technology. This is not to say that such technologies will not be needed to cost-effectively achieve many of our objectives. However, certain "tried and true" systems such as trickling filters can also play an important role in these efforts.

Trickling filters offer advantages of lower energy needs and relative ease of operation. This report presents operating results from an existing trickling filter plant which was successfully upgraded to meet increased demands on volume of waste treated and quality of effluent. Also included are the results of a number of other "upgrade" operations which on the whole proved very successful in meeting their objectives. From these results it can be seen that, when properly designed, constructed, and operated, trickling filters are an alternative which is worthy of further consideration in meeting the discharge requirements of the law [*Pierce*, 1978].

The operating system described in this paper details the upgraded wastewater treatment plant at Bedford, Ohio. In 1974 and 1975, the existing single-stage trickling filter plant was converted to a two-stage trickling filter operation. The new high-rate second-stage filter used plastic media supplied by The BFGoodrich Company. The following sections describe the sewer system, flow through the new plant and operation of the trickling filters. Finally, information is presented which compares operation of the improved Bedford plant with a number of other upgraded facilities for which operating data is available.

SEWER SYSTEM

The total city area of Bedford is approximately 1215 ha (3000 acres) of which 50 percent is tributary to the sanitary sewer system. The first system of sewers including a waste treatment plant was constructed in Bedford in the year 1913 to serve the then "built-up" or central portion of the village. This plant was soon overloaded and as a result was abandoned and new facilities were constructed in 1937-38 with a capacity of 4163.5 m^3/day (1.1 mgd) to serve 11,000 persons. After 13 years, the new plant had reached its design capacity and an enlargement of the facilities was completed in 1952 which provided a capacity of 8327 m^3/day (2.2 mgd) based on a population of 18,200 plus an equivalent population of 3,900 persons for industrial waste. This system was based on treatment provided by a single stage rock trickling filter. The enlargement discussed in this paper was constructed in 1974-75 and provides for a capacity of 12,112 m^3/day (3.2 mgd).

FLOW THROUGH PLANT

Raw wastewater enters the plant through a 0.762 m (30 inch aerial sewer of 35,768 m^3/day (9.45 mgd) maximum capacity and discharges into an outlet chamber.

Total flow is measured at the downstream end of the aerial sewer. Flows up to a maximum peak of 30,280 m^3/day (8 mgd) are directed to the plant for treatment through a flow regulating device which allows peak flow up to 24,224 m^3/day (6.4 mgd) (200 percent of average) to receive primary treatment. Surplus flow is routed directly to secondary treatment.

Primary treatment includes screening and grit removal followed by primary settling tanks to remove settleable solids, scum and floating debris.

Following primary settling the primary wastewater effluent is directed to a dosing (head) tank where it is mixed with any flows in excess of 24,224 m^3/day (6.4 mgd) which have been re-routed by way of the flow regulator. Under average flow conditions, 7,002 m^3/day (1.85 mgd) will be directed to the existing (rock) trickling filter while the remaining 5,488 m^3/day (1.45 mgd) flows to the pumping station which combines the trickling filter effluent to be pumped to the oxidation tower. The layout of the rock filter and the plastic oxidation tower provides series operation (double filtration) of 57 percent of the flow at average conditions, 12,491 m^3/day (3.3 mgd), and 38 percent at peak 23,656 m^3/day (6.25 mgd). The normal maximum design flow over the oxidation (plastic media) tower is 24,224 m^3/day (6.4 mgd) [dosing rate 20.35 ℓ/min m^2 (0.5 gpm/s.f.)] and the normal maximum design flow over the existing filter is 9,084 m^3/day (2.4 mgd).

The filter pumping station consists of three chambers. The first chamber is the pumping station wet well referred to previously, equipped with three pumps each having a 12,112 m^3/day (3.2 mgd) capacity; the second chamber will receive the underflow from the oxidation tower. The two chambers will be interconnected by two pipes with flap valves. This arrangement will provide makeup to the pumping

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station during low flows and allow the pumps to operate at the rated capacity.

The flow from the last chamber in the pumping station is divided proportionally and directed to the two clarifiers. The overflow from the clarifiers is then discharged to the microstraining facilities followed by chlorination and final discharge to Wood Creek.

The grit from the grit chamber is discharged into a dewatering bed. It can be removed by hauling to final disposal and the separated water is returned to the waste flow system.

The sludge from primary and secondary settling is drawn off and flows into a thickener. The overflow from the thickener is directed to the filter pump station. The thickened sludge is removed by pumping to the primary digester. The supernatant from the primary and secondary digesters is treated by the way of a wet oxidation process reactor using chlorine where it is stabilized, coagulated and deodorized and returned to the plant influent. The digested sludge is dewatered by vacuum filtration or sand drying beds. Dried sludge is disposed of by trucking to a disposal site or making it available for soil conditioning.

Trapped solids from the microstrainers are recycled into the waste flow ahead of the oxidation tower.

Phosphorus removal is affected by use of ferric chloride, alum, or sodium aluminate additions to the influent or to the effluent from the trickling filters.

TRICKLING FILTERS

The waste treatment plant has been designed for the reduction of BOD using an aerobic biological system. This process includes an oxidation tower which is a modification of the trickling filter type installation plus a rock trickling filter to provide parallel treatment at high flow and two step aeration at low and medium flow conditions.

Maintaining adequate flow through the oxidation tower and rock filter is important. The media in both filters must be continually wetted and in the case of the oxidation tower, pumping rates must be adequate to result in a dosing rate of 20.35 ℓ/min m^2 (0.5 gallons per minute per square foot) of filter area. In the rock filter a minimum flow is required at all times, to feed the biological growth and maintain it in an active state.

At the dosing tank the flow from the primary settling tanks is divided so that 57 percent of the flow is discharged to the rock filter at average conditions. The balance flows to the pump station well, mixes with the rock filter effluent and is pumped to the oxidation tower distributor mechanism. At maximum design flow, 23,656 m^3/day (6.25 mgd), only 38 percent of the flow receives two-stage treatment. At less than average flow conditions, the application rate to the oxidation tower media is assured through make-up from the mix chamber by way of a flap gate which opens when the water level in the pump well is lower than that of the mix chamber allowing recycle of the treated water back through the tower.

UPGRADE COMPARISONS

In a report by D. M. Pierce [*Pierce*, 1978], information on 68 single-stage and 20 two-stage filter plants was summarized. These data provide a good basis for predicting levels of performance of single (or parallel) filter systems compared with two-stage (series) operations. Comparisons are provided in the statistical probability plots presented in Figures 1 and 2. These plots show a normal distribution of BOD values in both single and two-stage systems with a definite statistically significant difference in performance of the two systems. The probable value for the single-stage system is 83 percent removal compared with 89-90 percent for two-stage. It may be further noted that there is 90 percent probability that BOD removals will be 74 percent or higher for single-stage systems and 82 percent or higher for two-stage systems. The probability curve for removal of suspended solids at single-stage plants is visually identical with the BOD curve. Furthermore, there is no statistically significant difference in removal of suspended solids at single-stage and two-stage plants. No significant difference was observed at the 90 percent probability level with a spread of 85 percent to 87.5 percent removal at the most probable (50 percent) value.

The values for Bedford, as indicated on Figures 1 and 2, fall in the upper levels of performance for both periods of single-stage or parallel operation (1965-1975) and two-stage operation (1976-1978). These exceptional levels of performance reflect to some degree the practice of chemical addition as a part of the tertiary treatment at the Bedford plant.

Table 1 gives the operating averages for the Bedford plant for the period 1965 to 1978. The years 1965 through 1975 are classified as "single-stage" operation and the years 1976 through 1978 as two-stage operation. Although operating data for three years is far from conclusive, it appears to point out improvement in overall plant operation with marked improvement in the removal of BOD.

SUMMARY

It is hoped that the information presented in this paper points out the potential operational abilities of trickling filters in wastewater treatment plants. The use of new and existing trickling filters in conjunction with proper plant design and operational practices can provide acceptable treatment levels of domestic and industrial wastewaters at very reasonable costs. Although outside the scope of this report, a list of references is included for obtaining comparative costs for construction and operation of most types of wastewater treatment facilities. A thorough examination of these reports can serve as a testament to the cost effective construction costs and operation of trickling filter plants.

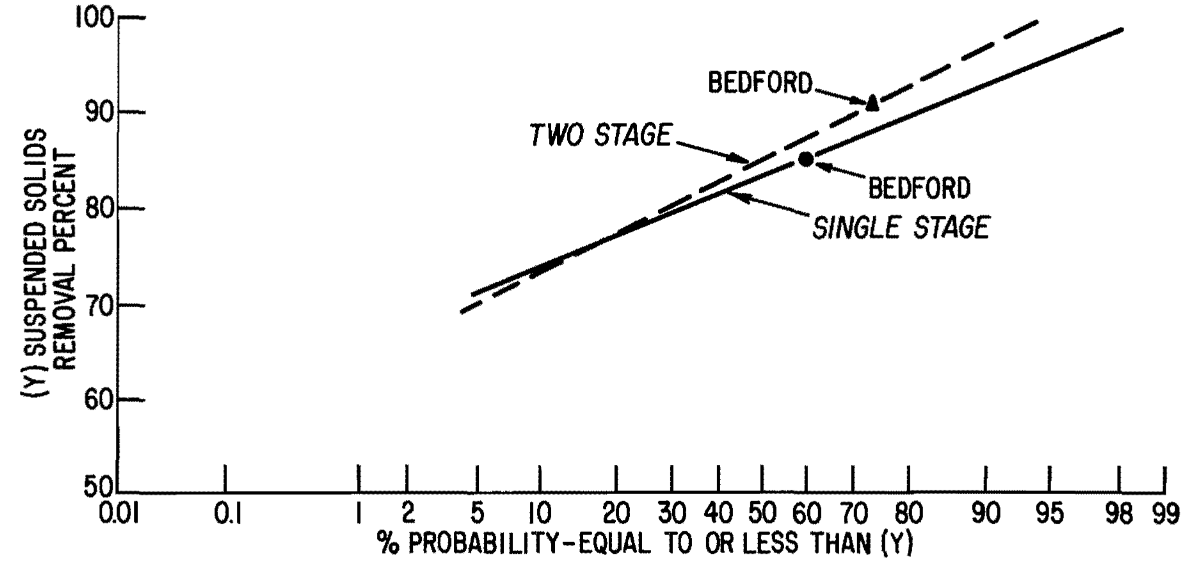

Figure 1. Comparison of single stage vs. two stage trickling filter plants in removal of suspended solids [Pierce, *1978*].

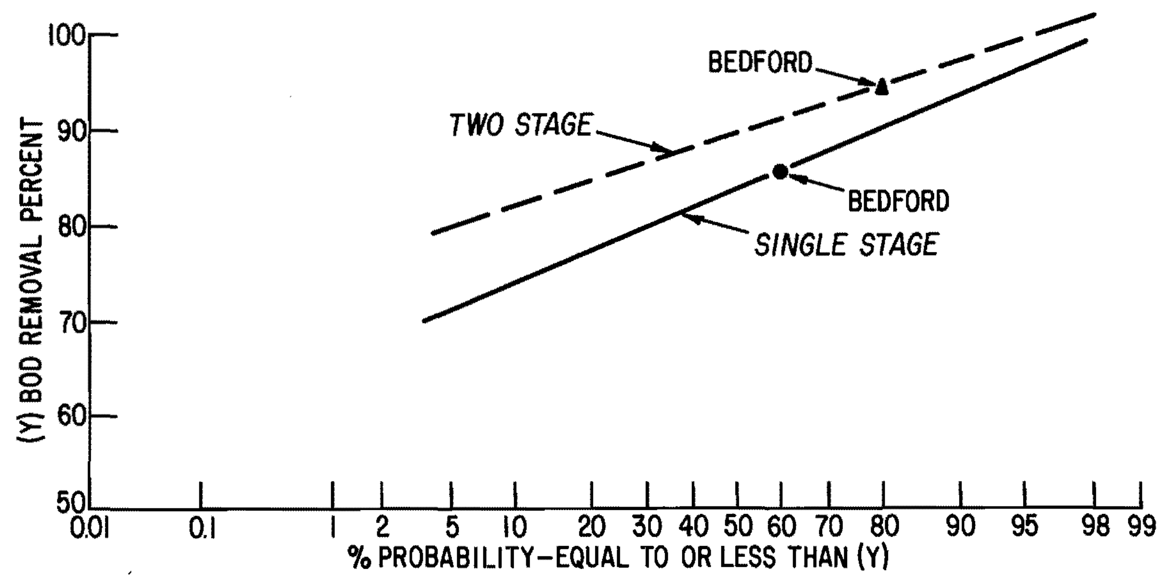

Figure 2. Comparison of single stage vs. two stage trickling filter plants in removal of BOD_5 [Pierce, *1978*].

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Table 1. Operating averages at Bedford treatment plant (1965-1978).

Year	Flow mg/day	SS raw	SS final	SS % removal		BOD raw	BOD final	BOD % removal	
65	2.4	129	25	81		140	30	79	
66	2.29	147	30	80		168	30	82	
67	2.17	166	30	82		165	30	82	
68	2.49	194	24	88		185	33	82	
69	3.09	144	29	80		148	33	78	
70	2.91	149	10	93		246	24	90	
71	2.74	147	13	91		275	26	91	
72	3.52	124	13	90	11 year average 85 percent	140	23	84	11 year average 84 percent
73	3.18	124	11	91		94	11	88	
74	2.88	96	18	81		63	12	81	
75	2.77	106	19	83		112	17	85	
76	2.42	150	9	94	3 year average 91 percent	251	13	95	3 year average 94 percent
77	2.54	116	12	90		214	11	95	
78	2.28	102	12	88		194	14	93	

DESIGN AND APPLICATION OF STATIC SCREEN DEVICES FOR PRIMARY CLARIFICATION, STORM WATER TREATMENT, AND SLUDGE DEWATERING

*Kurt I. Grover**

Screening devices, both static and mechanical, have played an integral part in wastewater treatment for many years. Application of screening equipment varies widely and has inherently rested upon the ability of screens to function as efficient liquid/solids separators. As technology continues and new developments are made screens are being applied to very specific separation tasks within the wastewater treatment process.

The scope of this report shall be limited to the discussion of two types of static or stationary screening methods: (1) *Cross Flow Screening* as applied to primary clarification and combined sewer-storm water treatment, and (2) *Restricted Drainage Dewatering* as applied to the dewatering of municipal wastewater sludges. Each method of screening shall be discussed upon the basis of application, operation, performance, and adaptability to facility upgrading.

CROSS FLOW SCREENING

Cross flow screening describes a method of high rate liquid/solids separation typified by a static screening device known as a sieve or sieve screen. A sieve is comprised of two basic components, a frame and headbox assembly and profile wire screen, as shown in Figure 1.

In operation, influent enters the sieve through a rear inlet nozzle and into a headbox area where turbulence is reduced with the aid of an internal baffle. Flow continues upward, passes uniformly over a curved weir and onto an acceleration plate where it is thinned and increases in velocity toward the inclined profile wire screen. During acceleration elongated particles and stringy materials align themselves with the flow thus simplifying their removal. As the flow passes onto the screen, free water is stripped from beneath the stream and solids begin to mass together and roll down the face of the screen due to residual kinetic energy. The solids stall towards the end of the screen where additional drainage takes place. These solids will, however, be continually discharged off the end of the screen by new oncoming material. The water which is removed by the screen falls to the base of the unit, passes through the outlet nozzle and on to further treatment.

The profile wire screen is the main functional component of a sieve. It is a two part type 304 stainless steel screen constructed of individual profile wires and cross rods, as shown in Figure 2. The profile wires are continuous formed wires consisting of alternating loops and triangular shapes. Assembly is accomplished by passing round cross rods through the loops and tightening the profile wire together. The loops also act as spacers to maintain accurate slot openings between the triangular shapes. The resulting screen is strong, uniform, puncture proof and virtually non-clogging due to the triangular profile.

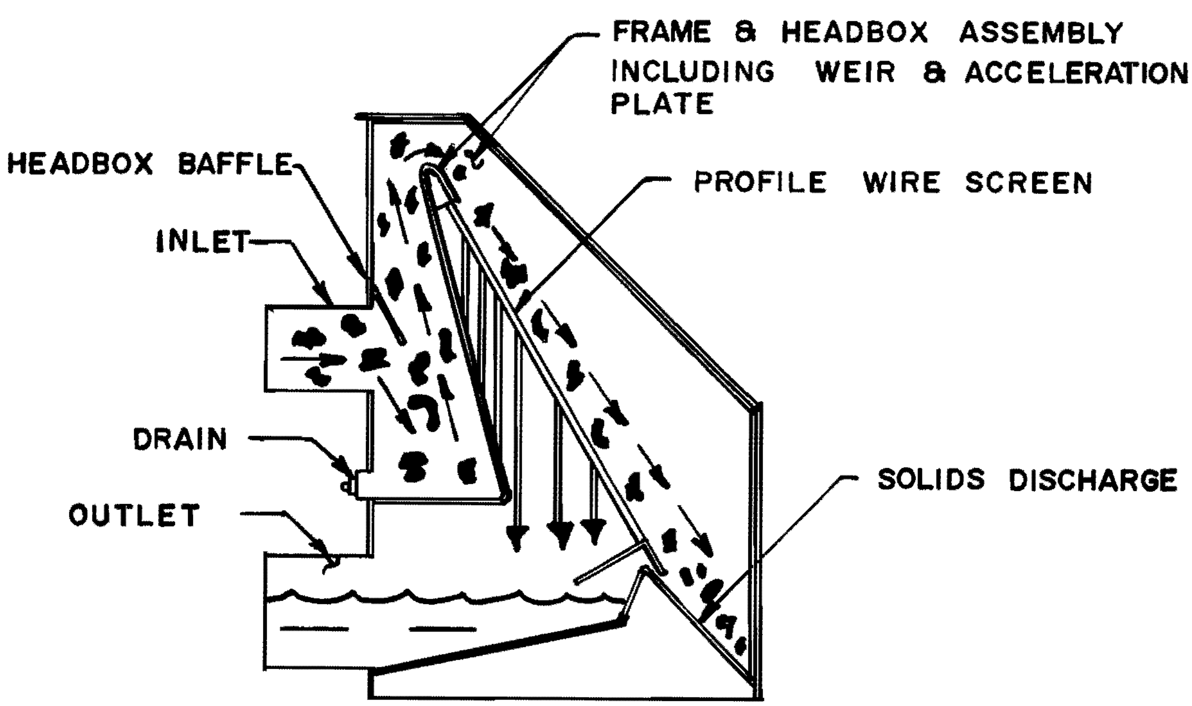

Figure 1. Cross flow screening unit or sieve.

Figure 2. Profile wire.

Inclined profile wire screens, as applied in sieves, achieve unusually high fluid removal due to a phenomenon known as the Coanda effect or wall attachment effect [*Kadosch*, 1964]. Fluid hydraulically attaches itself to the face of the profile wires, is carried through the slot openings and is deposited behind, off the apex of the triangular profile (Figure 3).

Figure 3. Coanda effect.

Sieves were first successfully applied to municipal wastewater treatment as an alternative to primary clarification and comminution in the late 1960's [*Ginaven*, 1970]. Hundreds of sieves have since been installed as primary clarifiers.

One six foot wide sieve unit, fitted with a standard profile screen six feet wide and four feet long with 1.5 mm (0.060 inch) slot openings will handle nominal primary flows of 1.0 MGD and peak flows of 1.5 MGD. Multiple units are used for larger flows, while smaller units (2, 3, 4, or 5 foot wide) are applied to lesser flows. Figure 4 shows a two foot sieve handling a nominal flow of 0.25 MGD (175 GPM).

Sieves used in lieu of conventional primary clarification remove 90 percent of all floatables, 35 percent of suspended solids, including non-biodegradable particulate, and typically reduce BOD levels by 30-35 percent. The action of the screening process also increased DO levels by up to 3 mg/ℓ which, in turn, causes grease separation in the effluent receiving chamber and preconditions the effluent for aerobic biological treatment. Depending upon influent characteristics 0.75 - 1.30 cubic meters of solids are removed for every one million gallons of flow. Screenings typically range from 15 to 20 percent dry solids.

Sieves have proved an economical means of facility upgrading as either a temporary measure to alleviate overloaded conditions or as a permanent installation. Sieves offer treatment in a minimum amount of space, and can normally be installed with minimal disruption to an existing plant and at a low initial cost. The use of sieves also leaves open existing primary tanks for other uses including secondary clarification, sludge holding, flow equalization, etc. When used as a temporary measure, sieves lend themselves to removal and reinstallation when more permanent upgrading is accomplished.

The main operational requirement of a sieve and important design consideration is head. Sieves have no moving parts nor operational power requirements, operation is accomplished by the potential energy present in the flow. Units can be gravity fed, however, if sufficient head in unavailable (72 inches required) auxiliary pumping is required either before the sieve, via force main, or afterwards to the plant. If pumping is accomplished after screening, the pumps are protected by the absence of heavy solids.

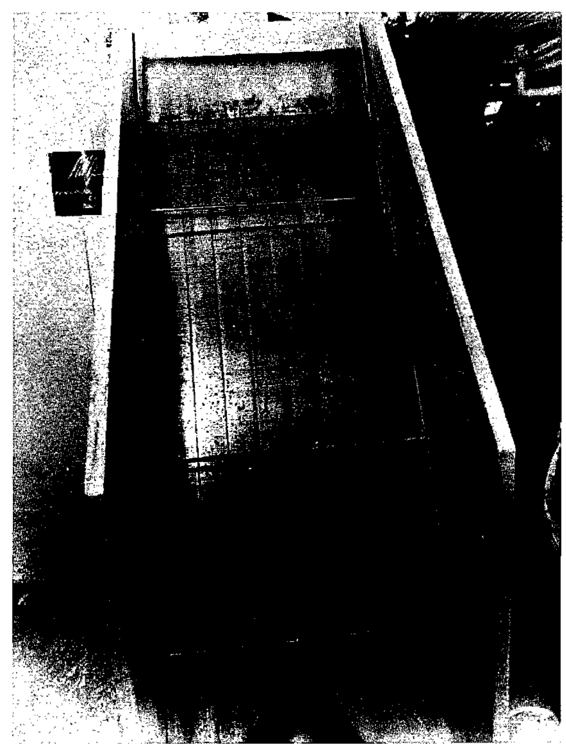

Figure 4. Wedgewater sieve shown in operation. Manufactured by the Hendrick Fluid Systems Division.

The sieve has also proved applicable in other areas of the treatment process with the most notable being combined sewer-storm water treatment. Storm flows can be many times greater than a plant's design capacity, making it impossible to treat conventionally. By the use of sieves as primary screens, large flows can be handled economically with one six foot unit handling peak loads of 1.5 MGD. In operation, over 90 percent of all floatables and 30-35 percent suspended solids and BOD are removed. The screenings generated are sent to further treatment and sieve effluent lagoon either further treatment at low flow periods in the treatment process, chlorinated and discharged, or direct discharge in extreme flow periods.

RESTRICTED DRAINAGE DEWATERING

Restricted drainage dewatering is a patented process covering an improvement to conventional gravity dewatering of sludges. The equipment employing the theory is marketed by the Hendrick Fluid Systems Division, Carbondale, Pennsylvania, under the trade name Wedgewater Filter Bed. The theory of operation is dependent upon the ability to control the rate of fluid extraction from a sludge and thus optimize gravity separation and control media and sludge blinding.

The components of the system include a watertight rectangular tank (steel or concrete), a false floor media of profile wire panels constructed of type 304 stainless steel, panel support structure and controlled drainage valve, all as shown on Figure 5.

In operation, "support water" is run into the tank to approximately 0.5 inch above the false floor. Sludge is then introduced, after conditioning, via a splash plate which prevents solids from being forced through the screen. The support water, which can be plant effluent, acts as a cushion to the incoming sludge and prevents media blinding due to premature fluid extraction. When the tank is filled, controlled drainage commences. By controlling the rate of drainage under the media, sludge porosity can be maintained and efficient gravity separation accomplished. As fluid extraction continues the sludge will become progressively thicker due to increased differential head pressure created by escaping effluent. The fluid level will, after a period of time (15 minutes to 2 hours), drop below the level of the media and be evacuated. Under normal conditions fluid will continue to drip out of the sludge for 8 to 12 hours as flocs fracture and the sludge compresses due to its own weight.

The normal dewatering cycle for the system varies from 24 to 72 hours depending on sludge type, plant conditions, and final solids concentration required. Additional dewatering will continue for this period of time due to evaporation, however; the rate of evaporation will vary depending on weather conditions and if the units are outdoors or covered.

Evaporation, although one of the most variable factors in gravity sludge dewatering, is enhanced by the basic restricted drainage design. In conventional sand drying beds, not only must water be evaporated from a sludge but because of capillary action water from underlying wet sand is drawn upward replacing water evaporated from the sludge, thus extending drying time. With the presence of a false floor, this phenomenon is not encountered and water must be evaporated only from the sludge. Also, in comparison to sand drying beds approximately $\frac{1}{6}$ to $\frac{1}{10}$ of the area is required for dewatering with an installation, in operation for 8 years, which reduced drying bed area by 23 to 1.

Table 1 shows typical design information for the Wedgewater Filter Bed process and Table 2 shows a performance chart as compiled at the Rollingsford, New Hampshire, Wastewater Treatment Facility on excess activated sludge drawn from a holding tank.

Table 1. Wedgewater filter bed capabilities.

Type Sludge	Loadings (lb/ft^2)	Percent Dry Solids (initial)	(final after 24 hrs)
Raw primary	1 - 3	.5 - 8	10 - 15
Aerobically digested	.75-1.5	.5 - 3	8 - 12
Anaerobically digested	1 - 3	.5 - 6	12 - 18
Excess Activated	1 - 2	.5 - 4	8 - 14

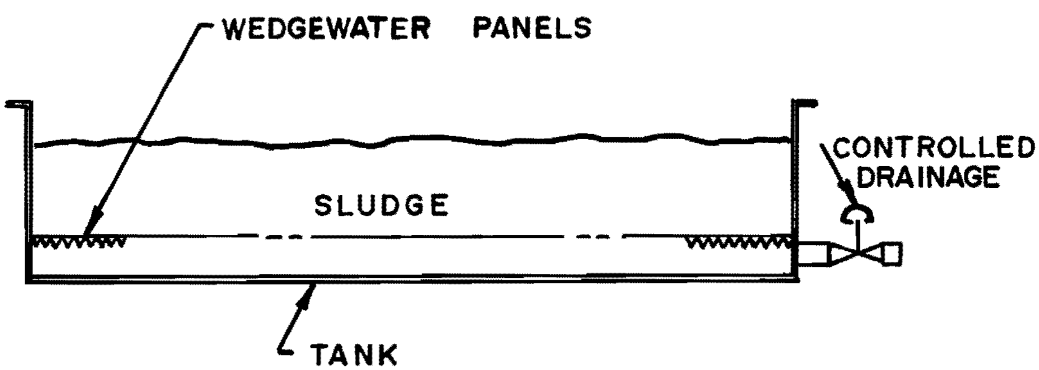

Figure 5. Wedgewater filter bed.

Table 2. Wedgewater filter bed performance chart.

Test No.	Loadings (lb/ft^2)	Percent Dry Solids (initial)	(final after 24 hrs)
1	1.25	2.9	9.35
2	1.25	2.8	8.8
3	1.25	2.8	9.4
4	1.10	2.5	9.3
5	.90	2.0	10.4
6	.90	1.9	9.0

Following dewatering the sludge cake must be removed from the media. The means of removal varies with the system type and configuration. Smaller plants using prefabricated steel units remove sludge manually, while larger plants with larger beds can use small front end loading equipment. With the use of hydraulically dumped steel units, the bed is tilted to an angle of 80 degrees and sludge slides off into a receiving vessel. Following cake removal, the media is hosed down and made ready for the next cycle.

The process has also proved to be an economical and effective means of upgrading existing sludge drying beds due to its adaptability to various tank configurations. Upgrading is accomplished by converting sand drying bed area to the appropriate area of Wedgewater Filter Bed. Converting an existing sand bed is carried out by pouring a concrete floor in the sand bed, thus forming a concrete tank. The tank is then fitted with controlled drainage valve, media supports, hold down strips and profile screen media.

Figure 6 shows a converted sand drying bed covered by a green house structure. This particular modification was performed by the plant staff. It is usually only required to convert part of a plant's sand drying bed area due to the difference in system efficiencies. The remaining area is thus left open for other uses or as a back-up dewatering system.

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Figure 6. Wedgewater filter bed sandbed conversion, Clarks Green, Pennsylvania.

RECENT DEVELOPMENTS IN THE APPLICATION OF TUBE SETTLERS TO EXISTING CLARIFIERS

*C. L. Meurer**

The modern tube settler has developed over a period of nearly a century. Patents were granted for laminar flow sedimentation devices as early as the turn of the century. For example, a patent filed in 1909[1] describes a settling device with a multiplicity of concentric cones to produce a shallow basin laminar flow separator. The patent anticipates an amazing number of concepts used today by plate and tube manufacturers. More specifically, the patent describes fundamentally the modern upflow tube settler in that it describes a device composed of a multiplicity of baffled plates disposed at an angle of 60° to the horizontal in order to achieve counter-current gravity drainage of solids. Many other patents describe various types of tubes and plate-separators.

The use of the tube settler is expanding at a rapid rate today due to increased needs for economy, space conservation design and high quality effluent. The tube settler is by far the most viable tool a designer can use to meet these needs.

Tube settlers are a simple device which enhance the clarification process in many ways. The most obvious advantage they offer is that the average settling distance of a particle is reduced from several feet to less than two inches. Since tube settlers operate at extremely low Reynolds numbers (typically less than 50), the flow through them is laminar and particle settling is unhindered by the random currents always present in conventional clarifiers. This latter advantage makes tube settlers a highly predictable device. Since they are in effect a collection of tiny ideal clarifiers, they may be incorporated in sedimentation basins with a great deal of confidence.

A new development (patented) has occurred which allows a prospective user of tube settlers the opportunity of easily testing an existing basin to determine the benefits of adding tubes and to establish better design criteria than was possible before. These pilot units are self contained tube modules with troughs, pumps, and pontoons which float on the surface of the actual basin being tested (Figure 1). Although a very recent development, this system of pilot testing has been used on a number of existing secondary clarifiers and the results appear promising (Table 1). The results for each clarifier tested vary somewhat and complete data verifying overall clarifier performance with pilot unit performance are not yet available; however, the results obtained by the tube pilot unit are similar to results obtained in full scale secondary clarifier upgrades. These data show that hydraulic loadings on secondary clarifiers can be increased as much as 100 percent while decreasing effluent suspended impurities by as much as 50 percent. The use of these pilot units in the future should improve the predictability of performance in upgraded clarifiers and aid in the establishment of design criteria.

Another recent development in tube settler technology allows greater flexibility in the installation of settlers in a wide variety of existing clarifiers. Sixty degree tube settlers can now be provided in an unlimited variety of complex curved shapes (patent pending). The use of these shapes solves a number of difficult problems involved in the installation of tubes in existing clarifiers.

Table 1. Results of pilot tests on secondary clarifiers in Utah and Colorado.

Site #1 (Activated Waste)*	BOD mg/ℓ	SS mg/ℓ
Plant Sample (.7 gpm/sq ft)	15	18
Pilot Sample (1.4 gpm/sq ft)	7	5
Site #2 (Trickling Filter)		
Plant Sample (.63 gpm/sq ft)	N.A.	54
Pilot Sample #1 (.63 gpm/sq ft)	N.A.	34
Pilot Sample #2 (1.26 gpm/sq ft)	N.A.	37
Site #3 (Trickling Filter)		
Plant Sample (425 gpd/sq ft)	31	44
Pilot Sample #1 (1400 gpd/sq ft)	17	20

* Test site names have been withheld to protect clients.

One of these problems occurs due to the fact that the hydraulics of a conventional clarifier are being altered somewhat when tube modules are added. The flow through the tubes must be drawn in a uniform manner and generally clarifiers have only peripheral launders. Since the annulus of tube modules is generally placed a maximum of two to three feet below the water surface to allow sufficient clearance below the tube modules, the launder is close to the tops of the tubes at the periphery of the clarifier and quite a distance from the tubes toward the center of the annulus. Because of this additional radial, launders are usually provided. A center baffle is also required to eliminate short-circuiting around the tube modules. The launders and baffles necessitate the use of complex support structure which often must extend deep into the clarifier. These structures often interfere with the scraper mechanism which requires that the entire design be compromised until a practical solution is obtained.

A more rational approach of great simplicity is now available by utilizing complex curved tube modules (Figure 2). The modules are curved to match the appropriate radius and tapered to form a truncated cone with a flat, horizontal peripheral rim. This arrangement is placed in the basin such that the outer ring of modules is lowest in the basin and the inner ring of modules just contacts the water surface. The tube tops are more nearly equidistant from the discharge weirs for improved hydraulic distribution and the clearance between the tube bottoms and the floor of the basin becomes greater toward the center of the

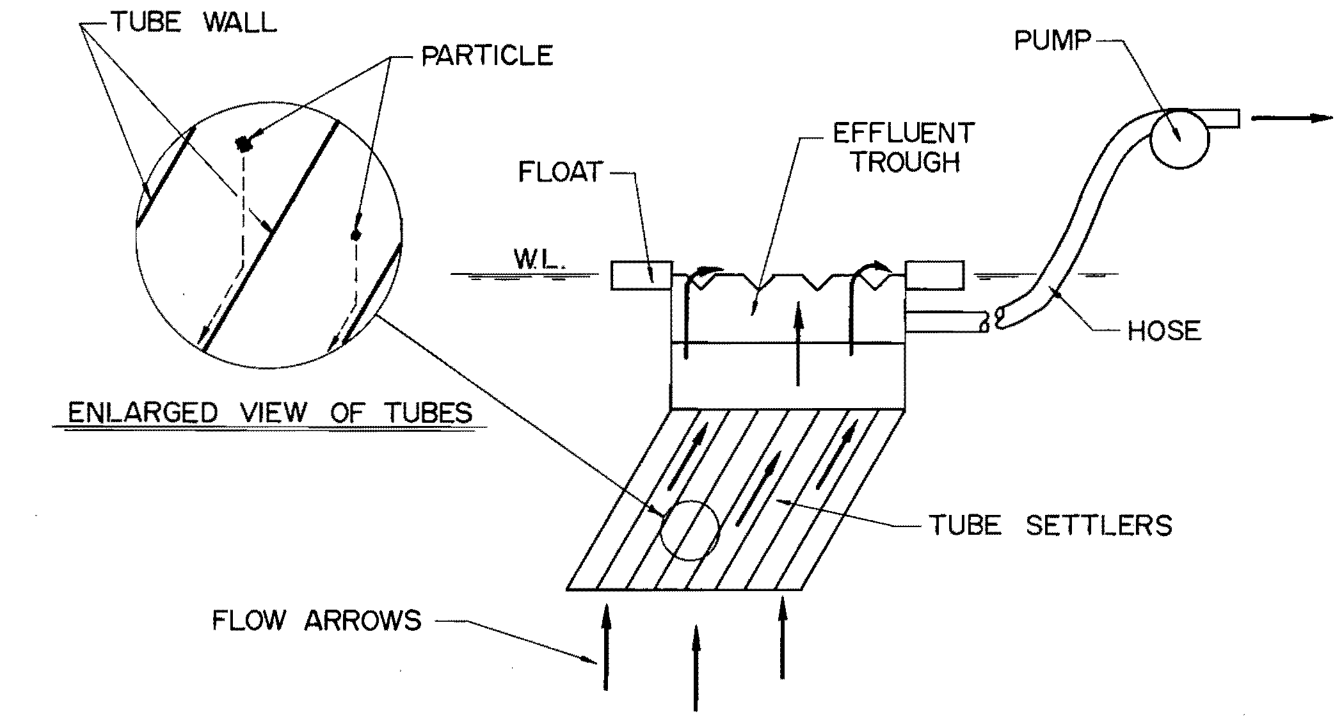

Figure 1. Tube settler pilot unit.

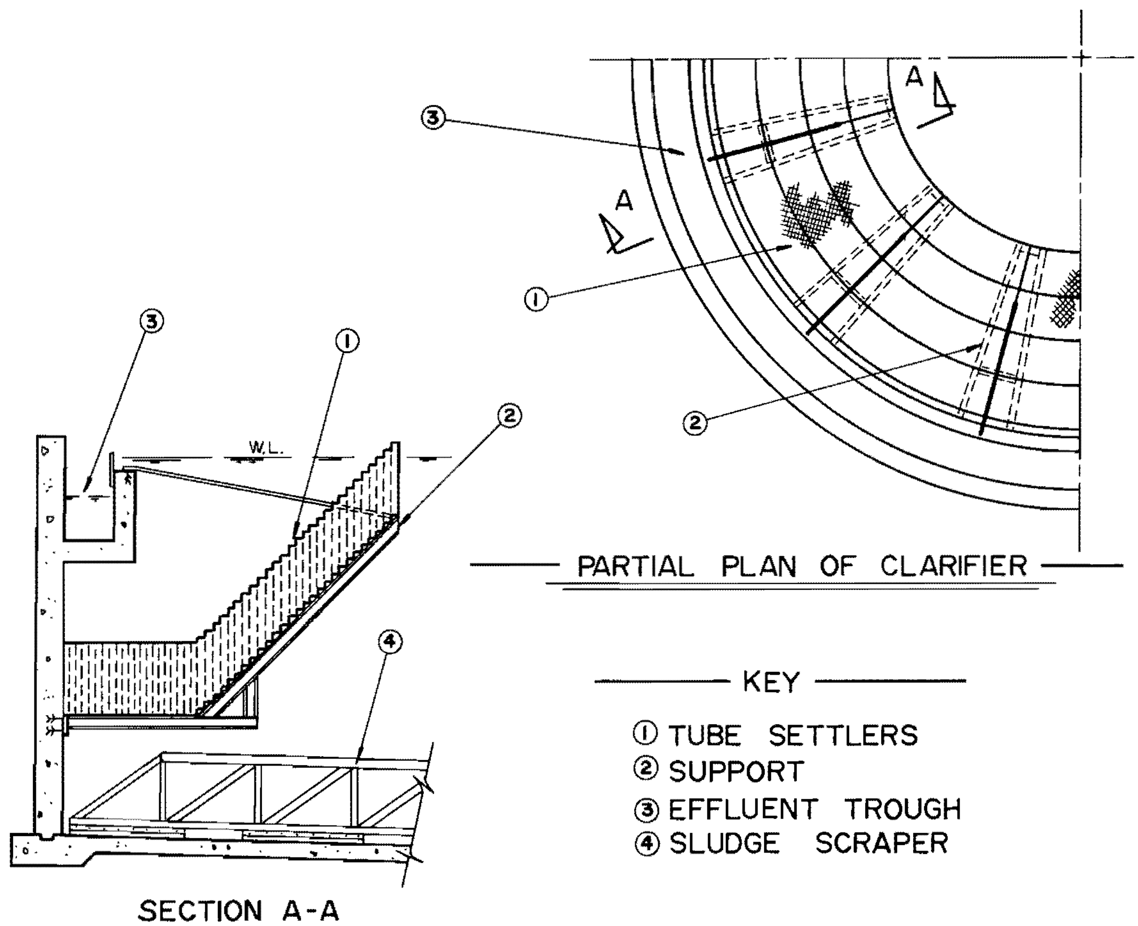

Figure 2. Complex curved tube arrangement.

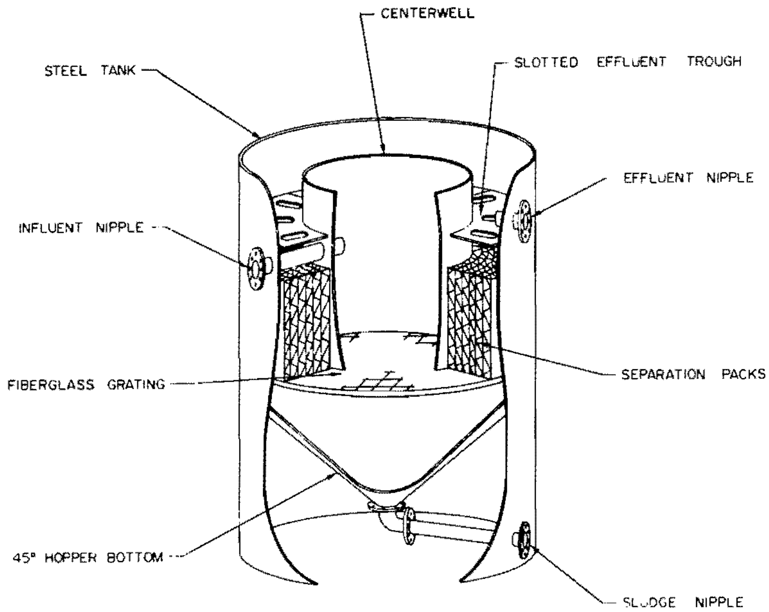

Figure 3. Small circular clarifier.

clarifier where the greater volume of sludge resides. The arrangement provides a more ideal hydraulic flow path with tapered velocity sections where they are appropriate. The need for baffles, additional launders, and the attendant complex support structure are eliminated. These circular tube modules can be molded to very small diameters (six feet outer diameter) to allow the practical adaptation of settlers to small existing clarifiers and even allow the possibility of conversion of abandoned filters and other tanks to miniature high rate clarifiers. This development is particularly important for industries attempting to meet EPA pretreatment standards, for small towns with overloaded systems, for control of rain runoff pollution control (EPA Clean Water Act) and a variety of other pollution control requirements. These small round modules have precisely the same tube profiles and operational characteristics as conventional rectilinear modules, but the tubes spiral around a centerwell allowing their use in tiny basins without the loss of capacity due to short tubes which terminate at the vessel walls (Figure 3).

The tapered, self-baffling tube module concept can be applied to rectangular basins as well as circular. There is, however, another tube arrangement that possesses a number of advantages over conventional tube configurations (Figure 4). This is the compound angle tube (patent pending). This device is particularly well suited to rectangular installations which utilize the popular traveling bridge clarifier underflow mechanisms. The tube modules are placed in the basins at a steep inclination with the relative tube to module angle adjusted to provide a 60° angle of the axis of the tubes to the horizontal. This arrangement possesses the same hydraulic, structural, and economic advantages of the configuration discussed above with the further benefit that up to 70 percent more tube settler media can be installed in a given basin area without increasing the flow velocity through the tube settlers themselves.

With the increasing versatility of tube settlers, the availability of simple pilot testing procedures, and the availability of operational data from an ever increasing number of installations, the use of tube settlers to upgrade overloaded clarifiers of all types is destined to become one of the major tools in the challenge of meeting future treatment demands.

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[1] Patent No. 1,020,013. March 12, 1912.

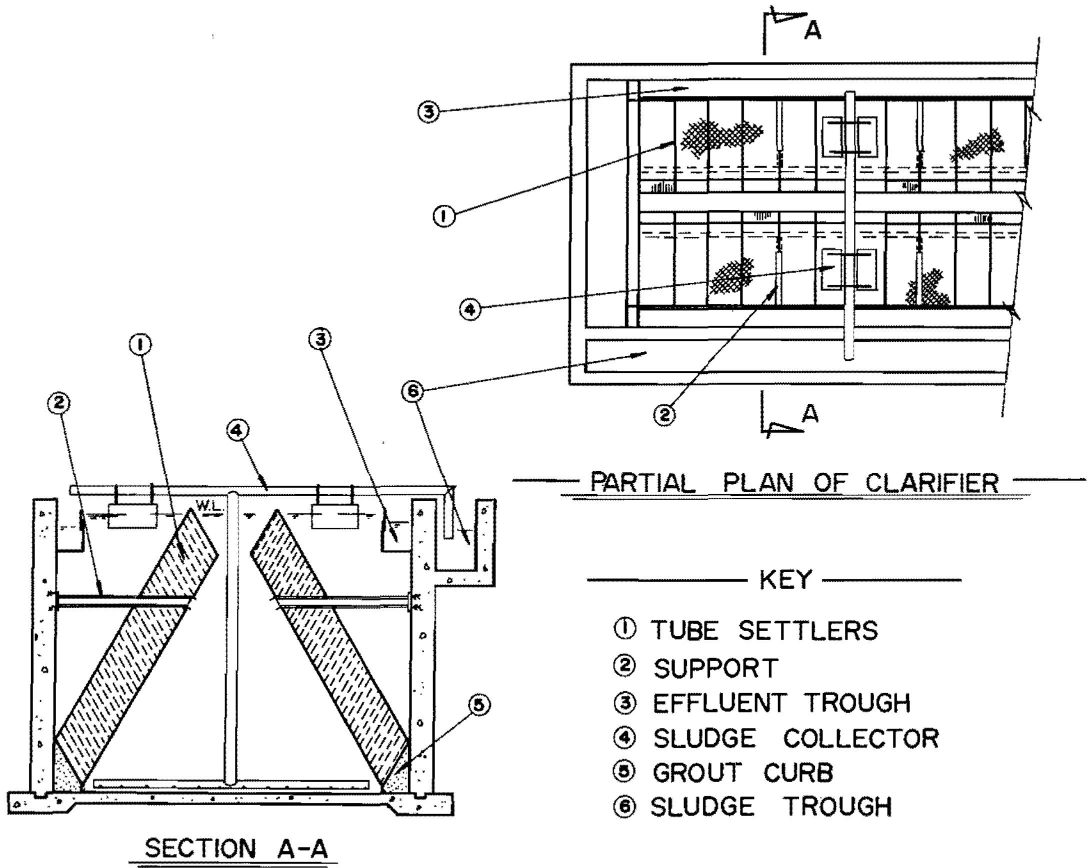

Figure 4. Compound angle tube module.

UPGRADING OF ANAEROBIC DIGESTION INSTALLATIONS

Lynn W. Cook

The only purpose for an anaerobic digestion facility is to provide sludge stabilization. To understand why an existing installation fails to perform this function adequately one must first understand the anaerobic digestion process.

The anaerobic digestion process takes place in two steps. In the first step, acid formers hydrolize and ferment the complex organic compounds found in the sludge substrate. These compounds are converted to simple organic acids. In the second step, methane bacteria ferment the organic acids to methane and carbon dioxide. The doubling time of the acid formers is only a matter of hours. The methane formers, however, have a doubling time of four days. Because of their longer doubling time, the methane formers are much more sensitive to digester upsets. Therefore, the digester environment must favor the methane formers. This is particularly critical since the methane formers are the organisms which accomplish sludge stabilization. Also, if the methane formers are destroyed, the digester will go "sour", since the production of the organic acids will continue.

By experience, the optimum environment for the methane formers has been fairly well established. The pH should be maintained between 6.6 and 7.4. A sufficient food supply must be maintained to insure the metabolism of the microorganisms. The tank must be kept completely mixed to insure adequate contact of the microorganisms with their food supply. The organisms must be protected from toxic levels of heavy metals, sulfides and free ammonia. The temperature of the sludge should be maintained near 35°C (95°F). These conditions should all exist in a properly functioning digester.

Anaerobic digesters can be designed for operation in one of two modes - standard rate or high rate (see Table 1). Standard rate digestion uses low volatile solids loading rates and long detention times. Mixing intensity is low [2.6 KW per 1000 m^3 (0.1 HP per 1000 ft^3)], is intermittent, and usually confined to the upper portion of the tank. The lack of complete mixing causes dead zones in the tank which typically occur at the bottom and around the periphery of the tank. Because of these dead zones, usually no more than 50 percent of the digester volume is utilized for active digestion.

High rate digestion uses high volitile solids loading rates and relatively short detention times. The main difference between a standard rate and a high rate digester is the mixing intensity [6.6 KW per 1000 m^3 (0.25 HP per 1000 ft^3)]. High rate digestion utilizes continuous and complete mixing. If mixed properly, all dead zones can be eliminated. This insures that 100 percent of the digester volume is utilized for active digestion. Proper mixing also insures intimate contact between the organisms and their food source. These two process improvements allow a digester to accommodate higher volatile solids loadings with shorter detention times. Biologically, there is no difference between the two modes. For any given raw sludge substrate at a given temperature, the biological reaction rates will be identical. The apparent process rate increases using high rate digestion because the organisms and the substrate are brought together more quickly and more often. Increased mixing intensity is the key element in high rate digestion.

Table 1. Comparison of standard rate and high rate digestion.

	Standard Rate	High Rate
Digester loading		
KG VS/day/cubic M	0.64 - 1.60	2.4 - 7.40
(LB VS/day/cubic ft)	(0.4 - 0.10)	(0.15 - 0.40)
Detention time		
Days	30 - 90	10 - 30
Type of mixing	Intermittent	Continuous
Mixing intensity		
KW/1000 cubic M	2.60	6.60
(HP/1000 cubic ft)	(0.10)	(0.25)
Digester temperature	35°C (95°F)	35°C (95°F)
Digester contents	Stratified	Homogeneous
VS reduction	30 - 40%	40 - 55%

With the above points understood, reasons for digestion inadequacies can now be discussed. A digestion system is considered to be inadequate when it fails to produce the required amount of volatile solids reduction (sludge stabilization). Poor volatile solids reduction occurs when the detention time of the sludge is too short for the mode in which the digester is operating. Shorter than required detention times occur for three reasons. The first reason is inadequate active volume within the digester. This usually occurs when the anticipated active volume has been decreased because of sediment in the tank bottom, dead zones caused by inadequate mixing, or excessive scum formation. The second reason is short circuiting. This occurs when a raw sludge inlet is located too close to a sludge withdrawal outlet and/or if the mixing is inadequate or improperly designed. Thin sludges require longer detention times because larger amounts of water are involved.

Digesters can also fail by going sour. This usually occurs as a result of sudden changes in the digester environment. Sudden localized temperature change, pH change, lack of food, or increased concentration of toxic compounds may cause localized upsets to occur. If left unchecked, these upsets

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will spread until the entire digester goes sour. A sour digester is the extreme result of improper digester operation. The proper design for detention time as outlined above, proper mixing, and proper maintenance procedures will greatly decrease the possibility of a digester going sour.

Probably the most common occurrence of digester inadequacy is when an older digester, originally designed for standard rate operation, has become overloaded and is being operated with a too short detention time. Several things can be done to correct this problem. One solution is to thicken the sludge. If the sludge concentration is increased the volume of sludge is decreased. This will increase the detention time of the sludge in the digester. It will also decrease the digester heating requirement and lighten the load on the downstream process facilities. Primary sludge is usually thickened by gravity thickeners but can also be thickened by dissolved air flotation. Waste activated sludge can only be thickened adequately by dissolved air flotation. Dissolved air flotation thickening offers several advantages over gravity thickening. It prevents anaerobic conditions from developing in the thickener. Since sediment does not float well, dissolved air flotation thickening greatly decreases the amount of sediment transferred to the digesters. Also, greater sludge concentrations are obtained when thickening waste activated sludge.

Another way to increase the sludge detention time is to increase the digester volume. This can be done directly by adding new digestion tanks or indirectly by increasing the active volume within an existing digester. To permanently increase the active volume in an existing digester, it must be converted to operate in the high rate mode. The complete mixing used in the high rate mode will discourage sediments from depositing in the tank bottom, and will eliminate dead zones. With these two problems controlled, the active volume within a digester can be as much as doubled.

It can be seen that upgrading of digester mixing is often all that is needed to restore a digester to satisfactory operation. Many types of mixing systems are presently available. For high rate digestion, a system must be selected which continuously mixes the entire digester volume. One such system is the external draft tube mechanical mixing system (see Figure 1). Mixing at the periphery of the digester is the most significant benefit of this system. This eliminates dead zones in the portion of the tank where the most volume is. On larger installations, a mixer can be provided in the center of the tank also. External draft tube mixers provide excellent scum control. The mixers may be retrofitted into any existing digester without altering the digester cover. If ever required, removal of the mixers is easy since they are located on the tank periphery. Also, if one mixer is removed, all other mixers may remain in operation since a water seal prevents escape of gas from the digester. External draft tube mixers may be fitted with a heat exchanger jacket if additional heating is required. Since the draft tube is external to the digester, hot water piping to and from the heat exchanger is easily maintained. Use of a heat exchanger jacket also eliminates the need for sludge recirculation pumps and piping. External draft tube mixers should be sized based on 6.6 KW per 1000 m^3 (0.25 HP per 1000 ft^3). At least two mixers should be provided per tank.

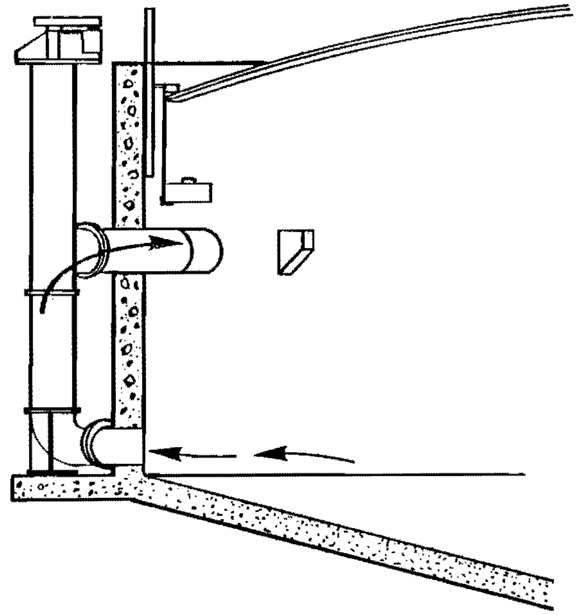


Figure 1. External draft tube mechanical mixer.

A continuous mixing gas mixing system can also be used on high rate digesters. The system should be designed to completely and continuously mix the entire tank contents. One such system is shown in Figure 2. In this system, gas is injected through several injectors simultaneously. The injector spacing is designed to insure that the entire tank volume is influenced and that surface energy is well dispersed for scum control. Also, the injectors reach to the bottom of the digester side wall to insure that the entire water column is mixed. The injector assembly is hardpiped and easily accessible for maintenance. The injectors can be individually removed without loss of digester gas. The compressor can be located at ground level for easy maintenance or on the digester cover. One added advantage of this system is that it can be retrofitted into any existing digester with very little modification to the digester cover.

Sometimes the loading conditions are such that improved mixing alone will not provide the necessary active volume in an existing digester. Sometimes even 100 percent active volume is not enough volume to obtain the required detention time. In such a case, new digesters will have to be built. The future success or failure of these digesters will be dependent on how much attention is given to the design parameters discussed in this paper, particularly with regards to digester mixing.

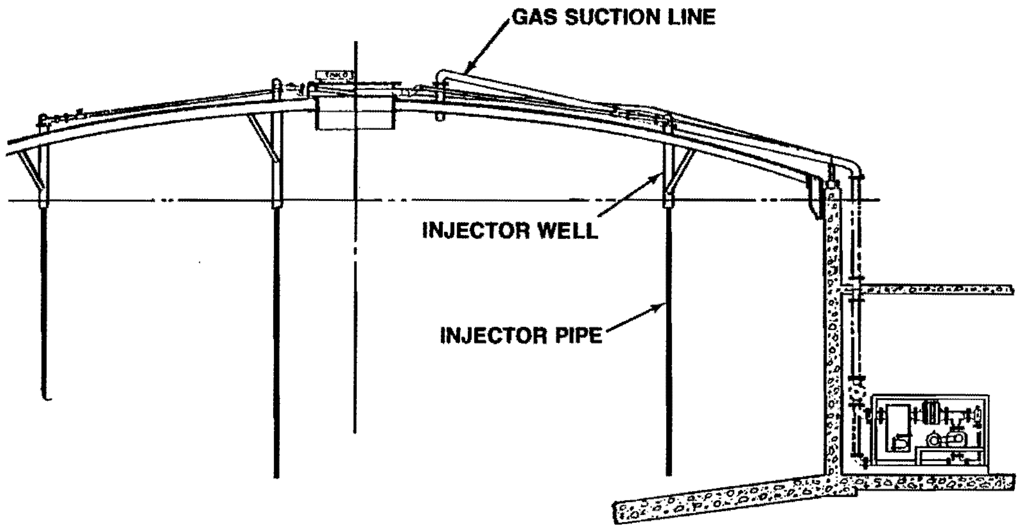
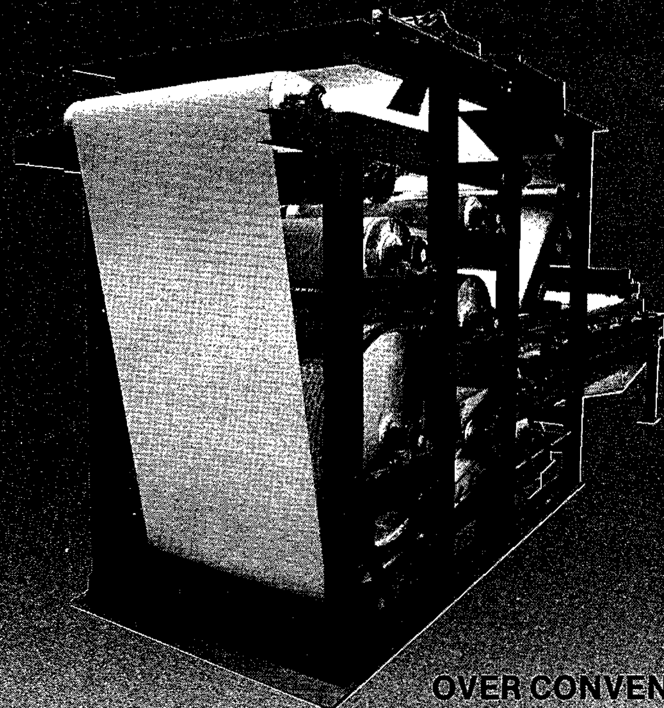


Figure 2. Continuous mixing gas mixing system.

EIMCO® EVT™ BELTPRESS



TYPICAL PERFORMANCE RATES WITH WASTEWATER SLUDGE		
MATERIAL	% DRY SOLIDS IN CAKE	POLYMER REQUIRED LBS/TON DRY SOLIDS
Primary Sludge	30 - 43	6 - 9
Mixed Primary plus Waste Activated	20 - 30	6 - 10
Waste Activated Sludge (W.A.S.)	17 - 23	7 - 12
Anaerobically Digested Primary Plus W.A.S.	25 - 40	5 - 9
Primary Plus Trickling Filter	27 - 38	6 - 10

PROVIDES IMPROVED FEATURES OVER CONVENTIONAL DEWATERING METHODS

The Eimco EVT Belt Press has been designed to work on all municipal and most industrial sludges. Prior to entering the Belt Press, sludge is flocculated with a polymer and conveyed on a gravity dewatering belt to the press roll belts. The decreasing diameter of the staggered rolls causes increasing pressure to the sludge cake, resulting in higher solids content when the belts separate and discharge the cake.

Versatility

Selection of the optional back-up high pressure belt roll is an exclusive feature offered on the Eimco EVT Belt Press. The standard model permits retrofitting the high pressure rolls if required at a future date.

Gravity Drainage Belt

Pre-dewatering of the sludge to a thin cake is achieved prior to the press roll section.

Slow Speed Belt

A thick cake is produced in the press section which discharges easier.

Decreasing Roll Diameters

Sludge cake is gradually and evenly compressed.

Roll Configuration

Efficient water drainage does not rewet the drier cake.

Cake Discharge

Strategic location of discharge permits conveyors, dumpsters, or other cake collection systems without building modification.

PROVIDES BENEFITS OVER CONVENTIONAL DEWATERING METHODS

Reduces power consumption **Reduces chemical costs** **Reduces maintenance**
Reduces operator attention **Produces drier cake**

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UPGRADING PERFORMANCE OF PRIMARY CLARIFIERS THROUGH CONVERSION TO CLARIFLOTATORS

*Terry Cassady**

INTRODUCTION

The majority of all wastewater treatment plants in the world employ some type of primary clarification as a first step in the removal of organic material. For some areas the primary clarifier is the only method of treatment. Figure 1 shows a flow sheet which is typical of those found throughout the State of Utah.

Aside from constructing a new primary clarifier or expanding other areas of the plant, there is presently very little an engineer can do to improve the performance of an overloaded primary clarifier.

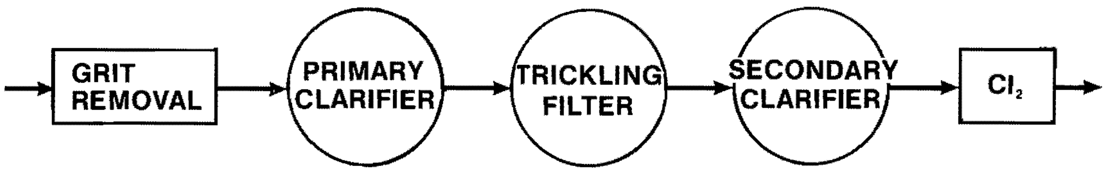

Figure 1. Typical secondary flowsheet.

BENEFITS

A modification can be made to any existing circular primary clarifier which can result in the following direct benefits:

a. Suspended solids removals of 70 percent or better.
b. BOD_5 removals of 50 percent or greater.
c. Floated solids concentrations as high as 6 percent which means smaller sludge handling equipment.
d. Continued use of entire surface area for clarification.
e. Ability to revert to simple primary clarifier operation during low flows or flotation equipment repair.
f. Increased ability to handle shock loads.
g. Fifteen percent removal of soluble BOD_5.
h. High mechanical rehability.
i. Relieves overloaded trickling filters and digesters.
j. Elimination of scum problem normally associated with primary clarifiers.

The modified clarifier would also have two indirect benefits:

a. The unit can be used as a thickener for both primary and secondary sludges thus reducing present loads on existing thickness or reducing hydraulic loads to digesters.
b. Increased removal of volatile solids and soluble BOD_5 will mean an increase in gas production in anaerobic digesters.

The equipment which can provide all of these benefits is a Clariflotator shown in Figure 2. As can be readily seen from the figure, any clarifier can be converted easily to a clariflotator.

DISSOLVED AIR FLOTATION (DAF) THEORY

The Eimco clariflotator is essentially a clarifier with a dissolved air flotation unit in the center. Due to this simple arrangement, existing clarifiers can be converted with little difficulty.

The dissolved air flotation section of the clariflotator is based on the same theory as is the conventional DAF unit. Dissolved air flotation is a method for separating and removing suspended solids from a waste water. The driving force for separation is accomplished by attaching micron sized air bubbles to the suspended solids particles, thus reducing the

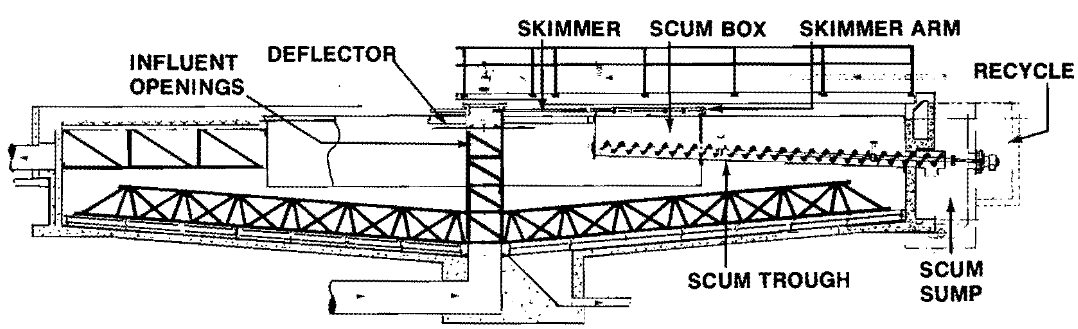

Figure 2. Eimco clariflotator.

specific gravity to less than that of water.

In a DAF unit air is dissolved under pressure in a clean liquid (usually recycled effluent from the DAF unit). This pressurized stream is sent to the DAF unit where the pressure is released and combined with a raw feed stream. The water becomes supersaturated as the pressure is released and the excess air comes out of solution in the form of micron sized bubbles.

These bubbles collide with the suspended solids and become attached. The net specific gravity of these agglomerates of particles and air is now less than that of water. They rise to the surface forming a float blanket which is easily removed by skimmers. The clear subnatant water is then withdrawn from below the float blanket for further treatment.

There are three types of dissolved air flotation pressurization systems in use today. The first is total flow pressurization in which the entire waste stream is pressurized and aerated. The second is partial pressurization where only a fraction of the waste stream is pressurized usually 30-50 percent. The third method is recycle flow pressurization which is favored when a waste stream is pretreated with coagulants and/or flocculants and the waste cannot be subjected to high shearing forces encountered in the pressurization pump and at the point of pressure release. In this system a portion of the clarified effluent is recycled to the pressurization system. This recycle flow becomes the carrier of the dissolved air later released for flotation. Recycle flow pressurization systems are favored when dissolved air flotation is used for thickening of biological sludges to minimize the possibility of fouling within the pressurization tank. It is favored in clarification applications because of the reduced power required. Recycle flow pressurization is used in all Envirotech dissolved air flotation and clariflotator units. A schematic of the recycle flow pressurization system is shown in Figure 3.

The air dissolution system is the most important component of the DAF unit. The design pressure has a direct effect on the size of the air bubbles generated when the pressure is released. At higher pressures more air can be dissolved into a given volume of water which means a lower recycle rate can be used to provide the same amount of dissolved air for flotation. Excessive recycle rates will increase the hydraulic loading and the turbulence, each having an effect on the amount of waste that can be treated for a given DAF tank size.

The two most important parts of the dissolution system are the pressurization tank and the pressure release valve. The dissolving efficiency of the pressurization tank should provide 80 percent or greater of theoretical saturation. Systems with lower saturation will require more recycle water to provide the same amount of dissolved air to the flotation compartment with the aforementioned problems of turbulence and artificial hydraulic loading.

Effective DAF operation requires micron sized bubbles to provide the proper air to solids bond. To provide these sized bubbles it is necessary to release the pressure of the recycle stream through a very thin opening. The Envirotech Haymore Valve

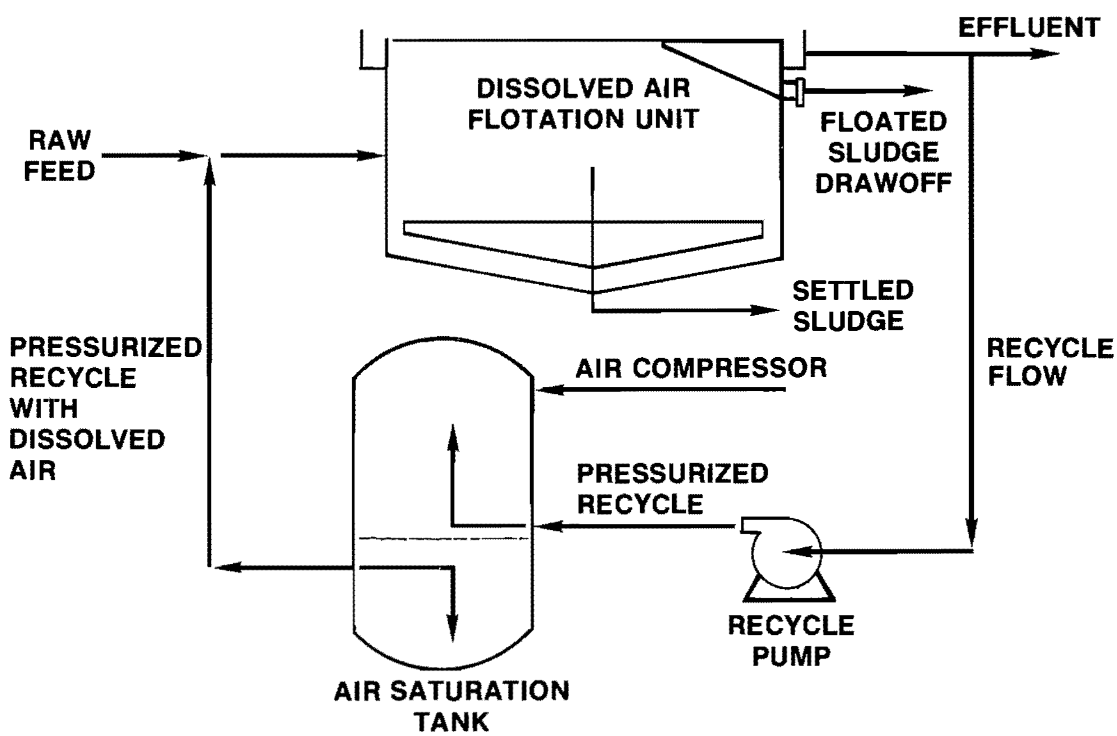

Figure 3. DAF recycle flow pressurization system.

provides this thin opening as the recycle stream shears through the annular space between the pipe wall and the stainless steel disc. Testing by Envirotech Research and Development revealed that the Haymore Valve released the dissolved air completely as micron sized bubbles producing no free air. These results were dramatic when compared to similar flow rates through a globe valve or a Saunders Valve both of which produce jet discharge with no effective shearing at the point of pressure release. The Envirotech Haymore Valve is also unique in that it releases pressure inside the DAF tank. This eliminates the possibility of air coalescence in the interconnecting piping since there are no horizontal runs after pressure release. Release of pressure inside the tank also provides excellent local turbulence and mixing with the waste stream to provide excellent air to solids bond. This allows very efficient use of the air released from solution forcing almost all the air to bond with solids. This bond insures excellent solids capture and precludes the possibility of coalescence or boiling near the inlet diffuser. Any debris that might build up at the valve disc will cause a pressure increase which will cause the valve to stroke, thereby dislodging the debris. The valve is shown in Figure 4.

CLARIFLOTATOR PRINCIPALS

The design parameters for the dissolved air flotation components of a clariflotator are essentially the same as for a DAF thickener unit except that the recycle rate is usually lower for clarification. These basic parameters are shown in Table 1.

Due to the low suspended solids concentrations found in most influents, the resulting solids loading rate is quite low. For most clariflotator applications the hydraulic surface loading rate is the governing parameter. It must be remembered that this includes the recycle flow rate.

Table 1. Flotation design and clarification zone parameters.

Flotation Design Parameters	
Recycle flowrate, percent of total flow	15-20
Hydraulic surface loading, m^3/day-m^2 (1.5-2.0 gpm/ft^2)	88.1-117.5
Solids loading, kg/m^2-hr (1.5-2.0 lb/ft^2-hr)	7.3-9.8
Clarification Zone Parameter	
Hydraulic surface loading, m^3/day-m^2 (800-1000 gpd/ft^2)	32.6-40.8

The entire surface area of the clariflotator is considered to be effective for gravity clarification. This approach is justified considering that light solids are removed in the flotation compartment that otherwise might not settle out before leaving the basin. The hydraulic surface loading design rate is based on only the raw influent flow since the recycle flow is withdrawn directly from the flotation zone.

The modification in some cases may require the complete removal of all existing steel depending on its structural integrity. If this is the case, a completely new clariflotator unit could be installed. If it were possible to reuse the existing raking mechanism and drive, then the modification would require only the addition of the flotation zone baffle, the skimmer arms, float withdrawal box, recycle inlet and outlet piping, and pressure release (Haymore Valve) valve. The pressurization pumps, compressor, and retention tank would be installed external to the basin.

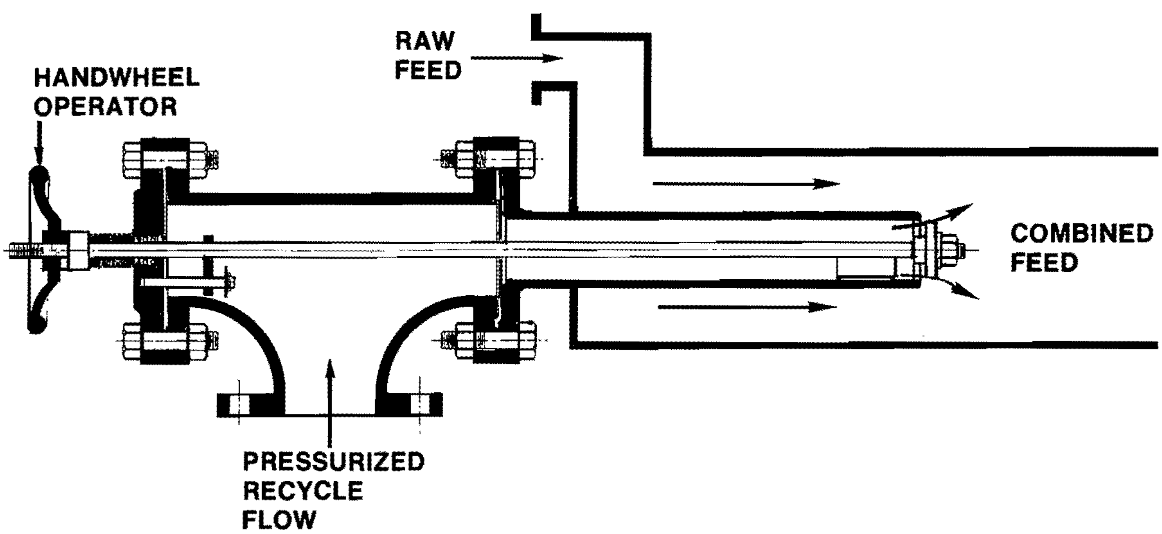

Figure 4. Pressure relief valve (Haymore Valve).

In most cases the existing drive would be of sufficient capacity to provide rotation for the rakes and skimmer arms. Since the amount of material that normally would settle now will be floated, the load on the rakes will be reduced leaving sufficient torque for the skimmer.

EXPECTED RESULTS

Eimco has not had the opportunity to retro-fit any large primary clarifier, and therefore, a direct comparison of performance before and after conversion cannot be made. Eimco does, however, have a number of clariflotator installations whose performance can be judged against the performance of a standard clarifier.

The largest clariflotator installation in the world is the Sand Island Plant in Honolulu, Hawaii. This plant has six 150 ft. diameter units with 80 ft. diameter flotation zones. The units provide the only treatment for the 71 MGD of wastewater before it is discharged to the ocean through an outfall. Table 2 presents a summary of the operating results for this plant.

Table 2. Clariflotator results, Sand Island, Hawaii.

	Infl.	Effl.	% Removal
Flotables (mg/ℓ)	11-58	0.1-0.8	98
Suspended Solids (mg/ℓ)	217	54	75
BOD_5 (mg/ℓ)	213	110	48
Float Concentration >	10 percent		

A clariflotator has operated at Milan, Illinois, for the past seven years achieving the results shown in Table 3. This unit has an overall diameter of 70 ft. with a 44 ft. flotation zone. The 1.7 MGD influent consists of domestic waste plus packing house waste. The clariflotator unit is followed by a dual biological system consisting of a trickling filter and activated sludge.

Table 3. Clariflotator results, Milan, Illinois.

	Infl.	Effl.	% Removal
Suspended Solids (mg/ℓ)	338	85	75
BOD_5 (mg/ℓ)	281	126	55
Float Concentration >	5.0 percent		

SUMMARY

Eimco believes that the Clariflotator can reduce the organic loading on secondary plants or on the receiving waters of a primary plant because it has the ability to:

a. Remove 70-75 percent of the suspended solids.
b. Remove 45-50 percent of the total BOD.
c. Remove 15 percent of the total BOD.
d. Provide floated solids concentrations of six percent.
e. Eliminate scum problems.
f. Provide thickening for secondary sludges prior to digestion.
g. Reduce hydraulic loads on anaerobic digestors.
h. Provide additional anaerobic gas production because of increased volatile solids and soluble BOD removals.

These benefits can be realized in existing plants through retrofitting existing primary clarifiers which normally obtain results far below those listed above. These benefits can also be built into a new plant, either municipal or industrial. Because of the equipment's versatility, it can be applied to a number of unusual industrial wastes.

Finally, clariflotators can bridge the gap where primary treatment is not enough but secondary is too much.

** Terry Cassady is an engineer with Eimco, Salt Lake City, Utah.*

POSITION STATEMENT: ELECTRICAL STANDBY

Clayton S. Hogstrom

I have been asked to give a formal policy statement for Utah Power & Light on the subject of co-generation. Co-generation can be a complex subject as there are many ways of looking at it. One group might say "Co-generation is much more efficient and it is in the public interest to operate efficiently, therefore we should co-generate." Another group might say "Although co-generation is a little more efficient, it is much more polluting and it is in the public interest not to pollute."

From the co-generators point of view the decision to co-generate must include considerations; such items as type of generation, fuel availability, present cost of fuel, anticipated cost of fuel, labor utilization to co-generate, maintenance of co-generation, dependability of co-generation, the reliability and necessary backup for such co-generation, the amount of space dedicated to co-generate, whether or not the various regulatory agencies will allow the co-generation because of pollution factors, etc. As you can see, the subject of co-generation is not an easy one to make general statements about.

I am sure that these and other aspects of co-generation will be considered today. Therefore, my policy statement on co-generation or self-generation will be limited to a rate making point of view.

I think it is safe to say that co-generation rarely produces a perfect match. If the system is designed to produce the required process heat, then the customer simultaneously produces too little or too much electricity. This requires that the customer either purchase the additional power needed or sells the excess power it generates.

Considering the first possibility, that is the customer needs to purchase additional power, our policy would be that we should not allow this customer to be a burden on our ratepayers. This customer should pay standby power rates and such rates should be structured to cover the costs of supplying him with his additional power. The actual rate design of such standby rates would reflect the cost of providing generating and delivery capacity to adequately protect the co-generator's operations.

Considering the second possibility, the rate we could pay for the excess capacity and energy would depend upon whether the power was firm power or not. If the power cannot be relied upon, that is, if it is available only upon a when-as-and-if basis, then the rate we could pay could not be more than the going rate for dump power at the time such power is available. The price at which a sale can occur is limited by two factors: The co-generators cost to produce and Utah Power & Light's cost to produce. The co-generator would not wish to sell power below his cost of production and Utah Power & Light cannot buy power at a cost higher than we could acquire it otherwise without burdening our other ratepayers. Hopefully there would be some point in between these two costs that would be mutually satisfactory.

If the power is firm power, that is, it can be relied upon to be available when contracted for, then we could of course, pay a higher price for that power than for power that is only available sporatically. The exact price and conditions of sale would be contracted for after negotiations.

Utah Power & Light recognizes and does acknowledge that it is our responsibility to see that our customers are charged rates that are equitable; in other words, we feel that one customer should not be allowed to be a burden upon our customers. Recognizing this fact, it is Utah Power & Light's policy concerning co-generation that we would be happy to participate in any co-generation project where a mutual benefit to both the co-generator and Utah Power & Light and our ratepayers would result.

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* *Clayton S. Hogstrom is with Utah Power & Light Company, Salt Lake City, Utah.*

NATURAL GAS AND COGENERATION

*Larry O. Murphy**

Mountain Fuel Supply has an adequate supply of gas for its customers and will continue to hookup customers. The maximum allowable usage per day for a new customer is 150,000 cubic feet. This volume of gas is generally sufficient for most applications.

If a customer needs a volume of gas greater than the 150,000 cubic feet per day, an application can be made to the Public Service Commission for a policy deviation. This deviation can be supported by Mountain Fuel Supply if the Company feels that the requested use has a high priority. A waste treatment plant that uses less than 150 MCF per day would be on a firm rate schedule, probably F-1 or F-2, depending on the volume of usage and the load factor. The load factor is the ratio of the average monthly usage to the peak monthly usage. A firm customer is not subject to curtailment of his natural gas supply.

Any customer such as a waste treatment plant using relatively small amounts (less than 150 MCF/day) of natural gas on a consistent basis throughout the year is good for our system. A non-temperature dependent load provides a high load factor usage which helps increase the efficiency of our distribution system. The peak load is nearly the same as the average load, thereby resulting in minimum demand fluctuations.

A waste treatment plant is a firm customer on a standard industrial schedule if natural gas is a primary source of energy, not a standby to an alternate source of energy. If an internal combustion natural gas engine is used in conjunction with an electrical generator to furnish the primary or total electrical requirements of a plant, natural gas will be used on a 60/40 basis or 50/50 minimum with the digester methane gas. This gas mix is required because the digester methane gas heat content (~450-600 BTU/ft^3) is not high enough to run the IC engine alone.

When natural gas (~950 BTU/ft^3) is mixed with the digester gas at a minimum ratio of 50/50, the BTU content of the mixture is approximately 740 BTU/ft^3. If 60 percent of the mixture is natural gas, then the mixture BTU content is approximately 780 BTU/ft^3. Either mixture has a high enough BTU content to allow an IC engine to run properly.

If the system is large enough, heat recovery from the exhaust and water jacket waste energy could be used to raise the system efficiency to 60-65 percent. Approximately 60 percent of the total energy input into the engine is rejected heat. Through the use of waste heat recovery boilers, the exhaust can be used to produce high or low pressure steam in much the same manner as a turbine. The jacket, however, is limited to the production of low pressure steam because of the temperature limitation of the engine and the working pressure of the engine jacket. These are usually 250°F and 15 psi, respectively.

Efficient use of natural gas is a means of slowing down our consumption of our natural resources. Mountain Fuel does have an adequate supply of natural gas for its firm customers because of an aggressive exploration program and contract gas purchases. Our natural gas comes from the following sources at the present time (Figure 1):

SOURCE	PERCENT OF TOTAL SUPPLY
Company Production	29%
Field Purchases	45%
Pipeline Purchases	
Canadian	11%
Domestic	15%

Additional natural gas exists in many underground formations accessible to Mountain Fuel pipelines. There exists enough proven gas that has already been discovered to last for 13-15 years at our present usage rate. It is the belief of Mountain Fuel Supply and many other companies that much more gas remains to be discovered. The *Wall Street Journal*, on April 27, 1977, editorialized that if gas prices were decontrolled, the gas and oil industries could find between 20,000 trillion and 50,000 trillion cubic feet of gas - enough to last 1,000 to 2,500 years at last year's rate of use! If the *Journal's* sources were in

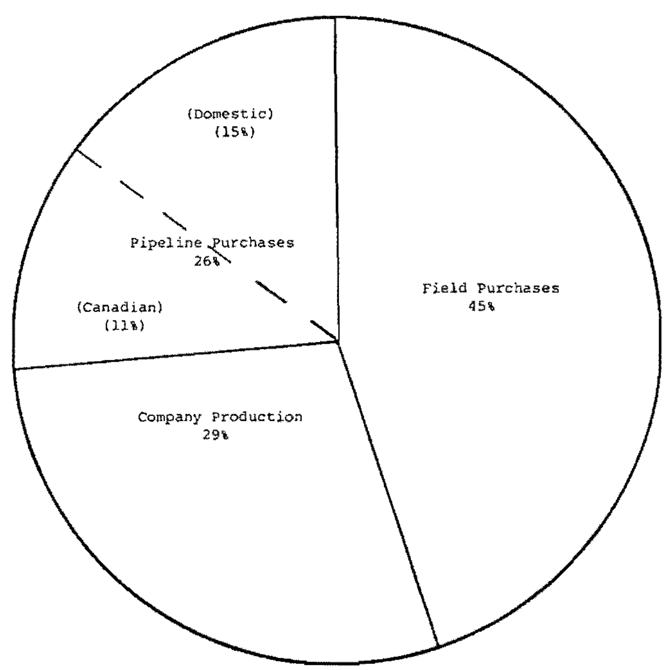

Figure 1. Gas supply by source.

error by over 90 percent, there would still be 100 to 250 years of reserves left.

In an excellent summary statement, the November 1978 issue of *National Geographic* discussed potentially massive reserves of natural gas from six different sources previously considered "unconventional": geopressured zones, deep basins, tight sand formations in our own Rocky Mountain area, coal seams, Devonian shale, and methane hydrates.

Here in the western United States the industry is beginning to develop substantial reserves in the overthrust belt area of Utah and Wyoming. Furthermore, major new finds are being reported almost daily from Mexico; and both Mexico and Canada, neither of which are members of the Arab dominated OPEC cartel, are re-evaluating former export restrictions on natural gas to this country and at prices far less than those which will probably be required for Alaska gas under present proposals.

The wide seasonal temperature swings in the Mountain Fuel Supply service area result in large demand fluctuations from summer to winter. The majority of the firm customers' usage is temperature dependent and Mountain Fuel Supply is, therefore, a "winter peaking" company (Figure 2). To minimize the huge capital expenditure in a larger capacity transmission and distribution system that would be required to serve all of the large industrial customers in addition to the residential and commercial customers, Mountain Fuel Supply serves the large industrial customers on an interruptible basis.

An interruptible customer (served under the "I" schedules) is provided natural gas service only on an "as available" basis and must have alternate fuel capacity at times they are not being served. That is, the "I" customers are only allowed on the system as gas and pipeline space are available. Mountain Fuel has long followed the practice of requiring its large industrial customers to be served only under the "I" schedules, thus allowing curtailment during periods when gas or pipeline capacity is needed to serve the "firm" customers. This practice has permitted Mountain Fuel to design its system so that it is required to serve only the firm customers on its peak winter days (over 92 percent of the firm customers are residential customers).

By placing its larger industrial customers on an interruptible schedule, Mountain Fuel has been able to reduce the investment required in its system which has, accordingly, resulted in lower rates for its customers. Mountain Fuel's interruptible customers are also important to its operations and rate design in that by serving such customers during those times of the year when firm customer demand is low (Figure 2), the interruptible customers are, through their rates, paying a portion of the costs of Mountain Fuel's system which would otherwise have to be paid by the firm customers.

By being able to serve interruptible customers during those times of the year when firm customer demand is low, Mountain Fuel has been able to acquire additional reserves of gas dedicated to its system, which gas will be available to serve additional residential and other firm customers in the future. At the present time, approximately 45 percent of Mountain Fuel's sales in Utah are to interruptible customers.

The Public Service Commission has found that such interruptible customers are an integral and necessary part of Mountain Fuel's rate design and that without such interruptible customers the firm customers, primarily the residential customers, would be required to pay higher rates, and additionally, that the gas reserves dedicated to Mountain Fuel's system would be reduced. The firm customers receive considerable benefit from having interruptible customers utilizing a portion of Mountain Fuel's system.

Mountain Fuel Supply presently has one waste treatment plant with cogeneration using natural gas/digester gas in an internal combustion engine/generator set. The Cottonwood Sanitary district system has been providing all of its own electrical power for nine years. Under the present state and federal regulations any treatment plant in our service area that wants to use cogeneration with natural gas will be able to receive that gas as a firm customer.

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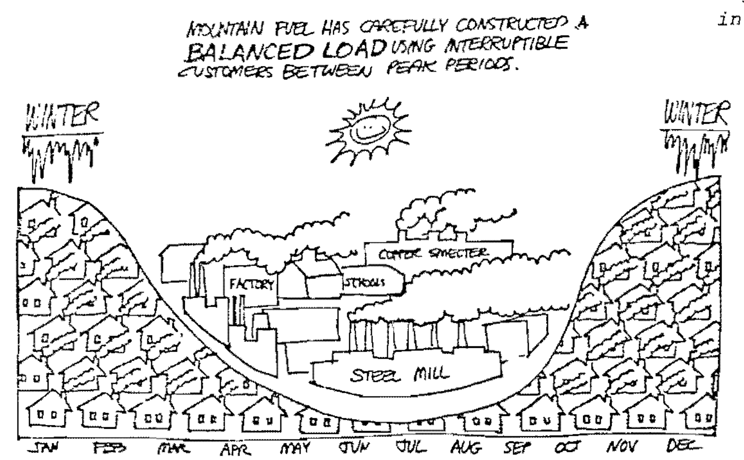
* *Larry O. Murphy is with Mountain Fuel Supply Co. in Salt Lake City, Utah.*



A graph representing natural gas sales to firm customers during a calendar year looks like a wide valley situated between two ranges of mountains. Those mountains represent the cold months of the year, when residential and small commercial customers have the greatest need for space heating. The valley between is the summer months when residential demand is lowest. That's a time when Mountain Fuel puts its excess gas into storage reservoirs for delivery when the weather turns coldest.

But there's another side to this concept. The summer months also provide an opportunity for Mountain Fuel to sell great quantities of natural gas to these interruptible industrial customers we have been talking about. These sales fill in this valley; it's a concept known as "load balancing" and it's one of the ways that Mountain Fuel has been able to achieve rates that are among the lowest in the nation for all its customers.

Figure 2. What industrial customers mean to Mountain Fuel's Service area.

ALTERNATE METHODS OF HEATING ANAEROBIC DIGESTERS

*Richard J. Eismin**

ANAEROBIC DIGESTION PRODUCES A DESIRABLE PRODUCT

The anaerobic digestion process has always been recognized to be extremely energy efficient. During most times of the year and in most parts of the country the energy from the gas produced is approximately equal to the energy required to heat the digester. It is obvious that employing an alternative method to heat the digester would liberate the digester gas to be used for other purposes.

As America's use of fossil fuels continues to grow faster than its supply, increasing attention is being turned toward alternative energy sources. The two alternative sources which hold the most promise for the State of Utah are Solar energy and Geothermal energy. These two energy sources are stationary sources and are therefore limited in their applications. Digester gas is desirable because it has a high energy value and it is portable.

DIGESTER GAS HAS MANY USES

Digester gas is approximately 70 percent methane with the remainder being carbon dioxide with a small amount of hydrogen sulfide. The heat value of digester gas is approximately 60 percent that of natural gas.

The digester gas can be stripped of the objectionable hydrogen sulfide by the iron sponge process. This will allow the gas to be burned in an internal combustion engine. Further refinement of the gas can be accomplished by removing the CO_2 with a wet scrubber or a tray tower. The resulting product can be used just like natural gas.

The digester gas can be used for many of the power needs around the plant. The most obvious are running pumps, blowers or aerators directly off of an internal combustion engine, or generating electric power.

The gas could also be sold to the local utility, a refinery, or other city departments. City vehicles could be run off of the sludge gas if properly equipped.

POPULATION OF 100,000 CAN PRODUCE 300 KILOWATTS (400 H.P.)

For comparison purposes we will calculate energy available on the basis of a 30 meter (90 ft) diameter digester with a 10 meter (30 ft) side wall. This digester would have an active volume of 5425 m^3 (190,000 ft^3) and could serve a population equivalent of approximately 100,000.

This digester would produce 3570 m^3 (125,000 ft^3) of digester gas per day. At 20 cents per therm (400 calories or 10^5 BTU's) this gas has a value of $54,750 a year, or would be equivalent to 400 shaft horsepower running 24 hours per day at 33 percent combustion efficiency.

With this much energy available in such a usable form from our digesters it would be very practical for us to heat our digesters in some alternative way.

GEOTHERMAL ENERGY MAY BE USED IN SOME AREAS

One possible alternate means of heating an anaerobic digester is with geothermal energy. Several conditions are required, however, for geothermal energy to be a viable energy source. First, the geothermal energy must be available close enough to the anaerobic digester for practical and economical considerations in transporting the fluid. Second, the brines must be sufficiently low in mineral content so that scaling is not a major factor. And last, some form of brine disposal must be available.

Although geothermal energy can probably be installed economically in very few areas in the entire country, we have one of these unique spots in Salt Lake City. A geothermal source is available less than a mile away from the Salt Lake City Water Reclamation Plant. If this heat source were used to heat a digester it could be diluted with plant effluent until the temperature was acceptable for use in a sludge heat exchanger, (see Figure 1). This could reduce the mineral concentration to a point where scaling was not a problem. The disposal of the spent brines which is usually a significant part of the cost of a geothermal system is eliminated here because of the Great Salt Lake's already high salt content.

Even for this ideal site, geothermal energy is not a panacea. The practicality of drilling the geothermal well, scaling problems in the brine collection

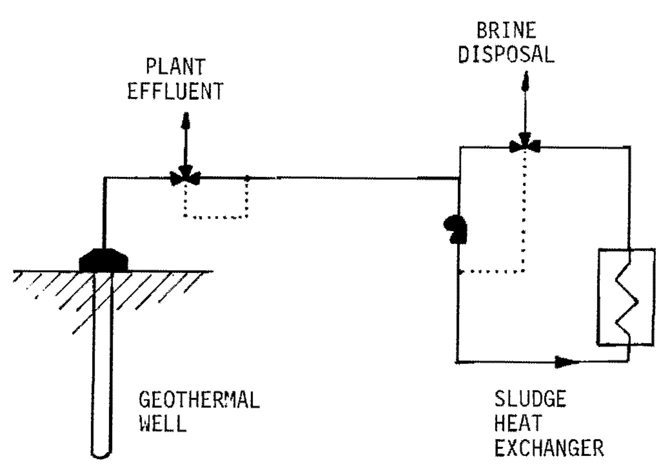


Figure 1. Geothermal energy can be used at some sites.

* *Richard J. Eismin is with EIMCO PMD Sedimentation Division, Salt Lake City, Utah.*

piping, and proper control of both temperature and mineral solubility, can be enormous operational nightmares. Close investigation is required before any geothermal project is undertaken.

AN IDEAL APPLICATION FOR SOLAR ENERGY

The most abundant energy source on earth is solar energy. The energy which falls on the surface of the earth in 40 minutes is equal to mankind's energy use for an entire year.

Utah with its generally sunny conditions is an ideal location for almost all solar energy applications, but applying solar energy to an anaerobic digester is one of the best possible applications for solar energy from an efficiency and economics standpoint. There are two major reasons for this. The first is that solar collection devices are most efficient at low temperatures. As shown in Figure 2, a typical solar panel's energy efficiency decreases rapidly as the operating temperature increases. Unlike other process heat requirements which require very high temperature heating, an anaerobic digester can be accomplished at very moderate temperatures.

The second advantage that heating a digester has over other forms of solar energy use is that solar heat storage does not add significant cost to the system. Quite typically the solar storage tank and controls are approximately 30 percent of the entire solar system cost. In many cases this solar storage can be eliminated by using digester gas as the backup heat source. Under conditions where it is not practical to use digester gas for this purpose, the digester cover can act as a solar energy reservoir without materially affecting the cost of the digester cover.

Solar panels could easily be blended into the architecture of a typical digester control building. For retrofit applications, the solar panels could be mounted on the digester roof, on the side of the digester tank or on the ground near the digester.

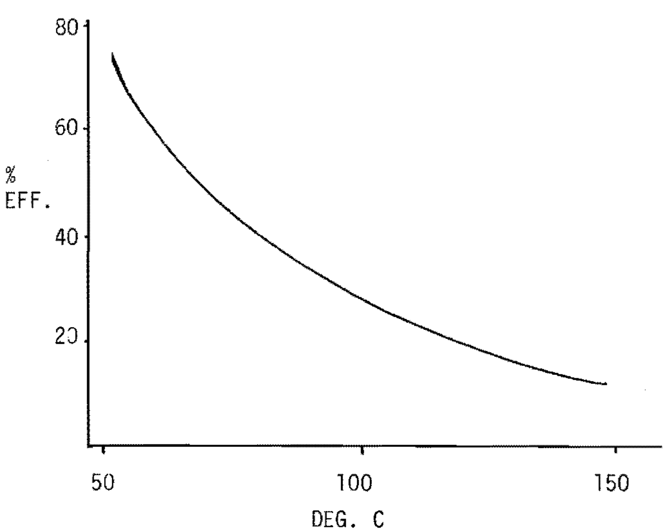


Figure 2. Solar collectors are most efficient at low temperatures.

LOW OPERATING TEMPERATURES MINIMIZE PANEL COSTS

The economics of a solar anaerobic digester are helped even more by the fact that "low technology" panels will do an excellent job. A typical panel shown in Figure 3 will consist of transparent cover plates, an insulated housing, and an absorbing surface through which the collection of fluid is pumped.

The temperature maintained in the solar collectors will be approximately 50° to 85°C (120° to 180°F). To obtain this temperature the panels need only single glazing, and moderate amounts of insulation. For higher temperature, much more sophisticated and costly panels are required causing the solar system to be must less cost effective than for this application.

THE SOLAR HEATING SYSTEM IS SIMPLE

The solar heating system shown in Figure 4 is a simple temperature control. The piping and controls are no more complicated than the controls on a

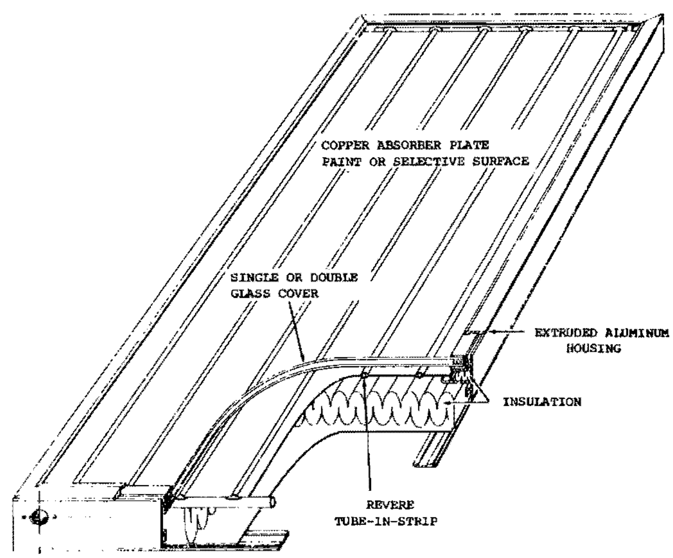


Figure 3. Flat plate collectors are economical.

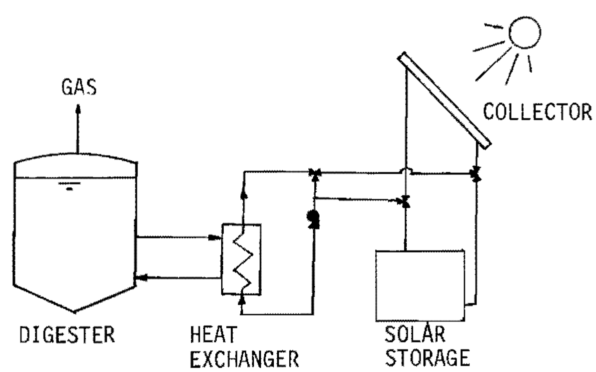


Figure 4. Solar heat exchange loop is simple.

conventional boiler and heat exchanger used for digester heating. The logic of the system which determines whether heat is taken from the solar panels, solar storage, or the alternate heating source is controlled by a small micro process circuit. This type of pre-programmed device will insure that the solar system uses solar energy whenever possible to maximize the solar fraction.

The department of energy estimates that approximately 23 percent of our energy needs will be supplied by solar by the year 2000. For these estimates to become a reality, solar energy will have to be applied wherever possible. Anaerobic digestion is one of the most economical and easily applied areas for solar energy and should be one of the first steps in our country's efforts to achieve solar energy goals.

DESIGN AND MAINTENANCE CONSIDERATION FOR USE OF DIGESTER GAS WITH A GAS ENGINE

*Ken Green**

The use of gas engines in sewer plants is not a new concept. The first major installation was made in 1928 in Charlotte, North Carolina, with the second sizeable installation in 1932 at Springfield, Illinois. These two plants showed that the use of sewer gas could be economically feasible. In 1934 there were nine plants in the United States using sewer gas as a prime power fuel. In 1940, 135 sewer plants were producing over 22,000 HP total rating, and growing to over approximately 193,000 HP operating on sewer gas by 1965, more than eight times the horse power produced in 1940. With the rising cost of diesel fuel, natural gas and utilities, it is becoming more economical to utilize digester gas to produce prime power than ever before.

Let's now take a look at where sewer gas engines could be utilized in a sewage treatment plant. The first that might come to mind would be power generation of on-site power. Here we would use the sewer gas to produce electricity to run and operate the remainder of the sewer plant. This could be used as either prime or standby power. The advantages of power generators are that you can centralize your engine generator room which will simplify operation and maintenance.

The second application could be power on a pumping station. This slide shows 5 - G342NA Engines driving five 15000 GPM vertical pumps in a typical lift station. Sewer gas engines could also be considered for direct driving blowers used in activated sludge processing. Shown is a Cat G398 direct driving a 12000 CFM Ingersol Rand blower. Both engines and blower operate at 1200 R.P.M.

Now that we have looked at where gas engines can be used in a sewer plant, let's look at some installation considerations. For this example let us look at a Gas Generator Set, the same consideration would apply to any gas engine application.

In order to make a complete system we will need an engine generator, a cooling system, an exhaust system and a fuel system. The engine generator portion will require selecting what R.P.M. you want the unit to operate at, either 1200 or 1800 R.P.M., the HP required to drive what you are driving and any other options you might want on the engine. You should be sure that you have oil pressure, water temperature and overspeed safety shutdowns.

The cooling system could consist of a unit mounted radiator with engine drive fan, a remote mounted radiator with electrically driven fan, a heat exchanger or an ebullient cooling system.

A unit mounted radiator is by far the simplest of all the systems. If will be easy to install but does require a large amount of air flow through the engine room and additional engine HP to drive the fan.

A remote mounted radiator will require more piping of a cooling water but is better for units that are mounted in a basement or any other area where air flow is a problem. These units can be roof mounted or any convenient location where air flow will not be a problem.

Heat exchanger cooling is also more complicated than either of the above two. This one is another good system where air flow is a problem. This system would also allow the recovery of the heat in the cooling system for boiler preheat or any other heating requirements close to the engine room. I would like to emphasize close to the engine room. If you design a complicated system it will only cause problems in years to come. You will need a good source of cooling water for this system.

The last cooling system is an ebullient heat recovery system. This is a very complex system and should be looked at and studied very closely before installation. Ebullient heat recovery systems are utilized to recover engine heat at higher temperatures. Minute steam bubbles form in the cooling mixture and it becomes less dense causing the steam and water mixture to rise in the system separator where the dry steam is produced. Additional steam is produced by the exhaust heat which is also routed through a heat recovery boiler and then to the steam separator. After the steam has given up its heat at the load (absorption air condition, heating, processing, etc.) the condensate is returned to the system to continue the cycle over again. There are several points of interest in the ebullient system that should be noted: no jacket water pump is used - circulation of the coolant is produced by thermal siphon action caused by the change in density. The optimum temperature for a total energy "ebullient" system is approximately 250°. A backpressure valve is always included in the system to insure relatively constant temperature of the engine coolant, regardless of steam demand. A separate cooling circuit must be provided for the oil cooler and aftercooler wherein the coolant temperature must not exceed approximately 90°F on gas engines.

The third system we will look at is the exhaust system. This system will consist of a flexible exhaust fitting to eliminate any vibration and a muffler. The muffler can be one used for silencing only or a heat exchanger used for heat recovery as well as silencing. Shown is a heat recovery muffler and secondary piping needed. Also you can see the flex fitting on the left hand side.

The last system is the fuel system (Figure 1). With the use of sewer gas this can become a complex system. The sewer gas comes from the digestion tank and through a scrubber. This scrubber will have to be sized and engineered to meet the requirements of the sewer gas. From the scrubber it would enter a large holding tank where it would be stored until needed. The holding tank also handles surges in supply and demand in your fuel system. You would then pipe the gas from the holding tank to a compressor to step the fuel pressure up to 40 - 50 PSI to insure a constant pressure on the regulator and

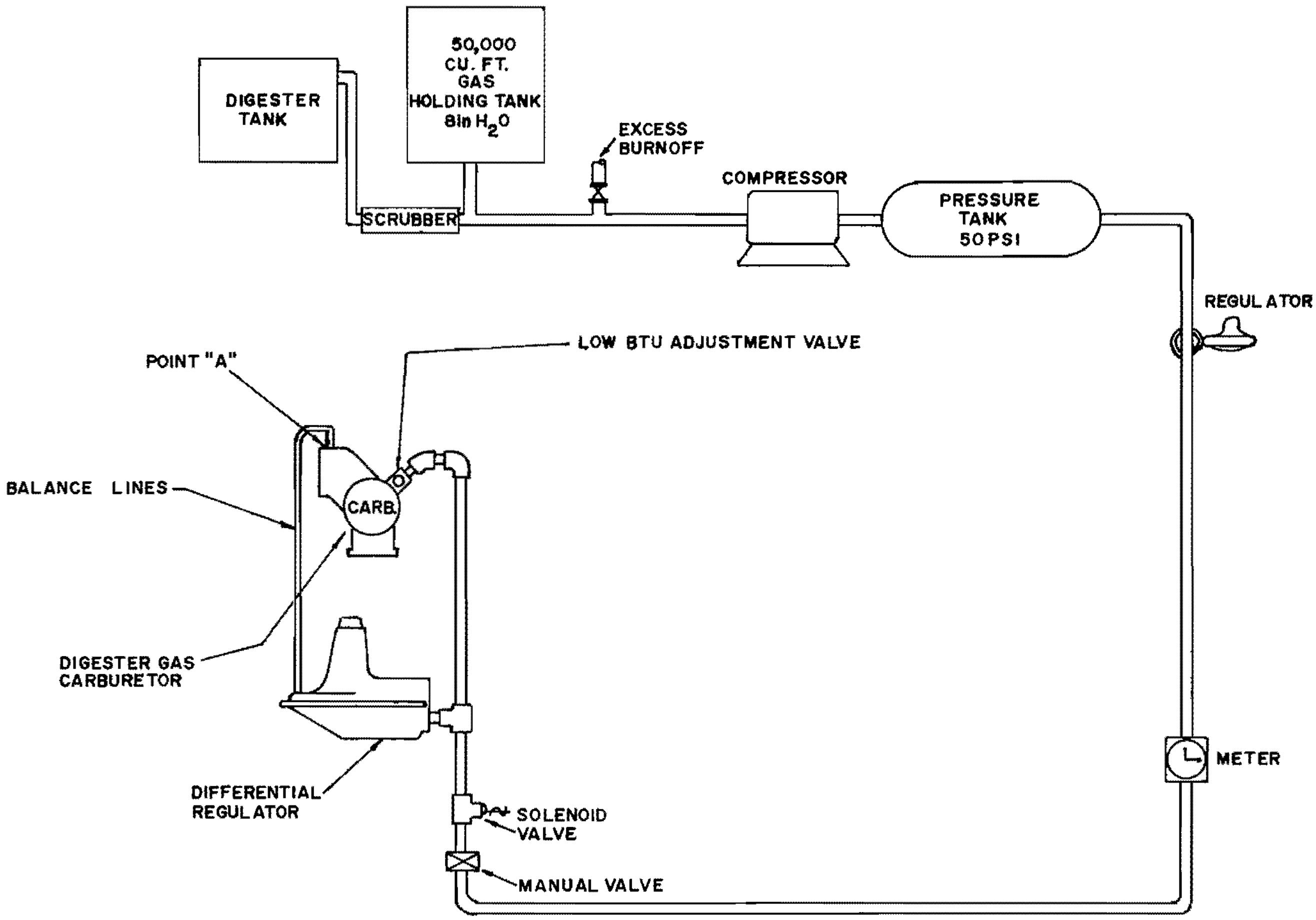
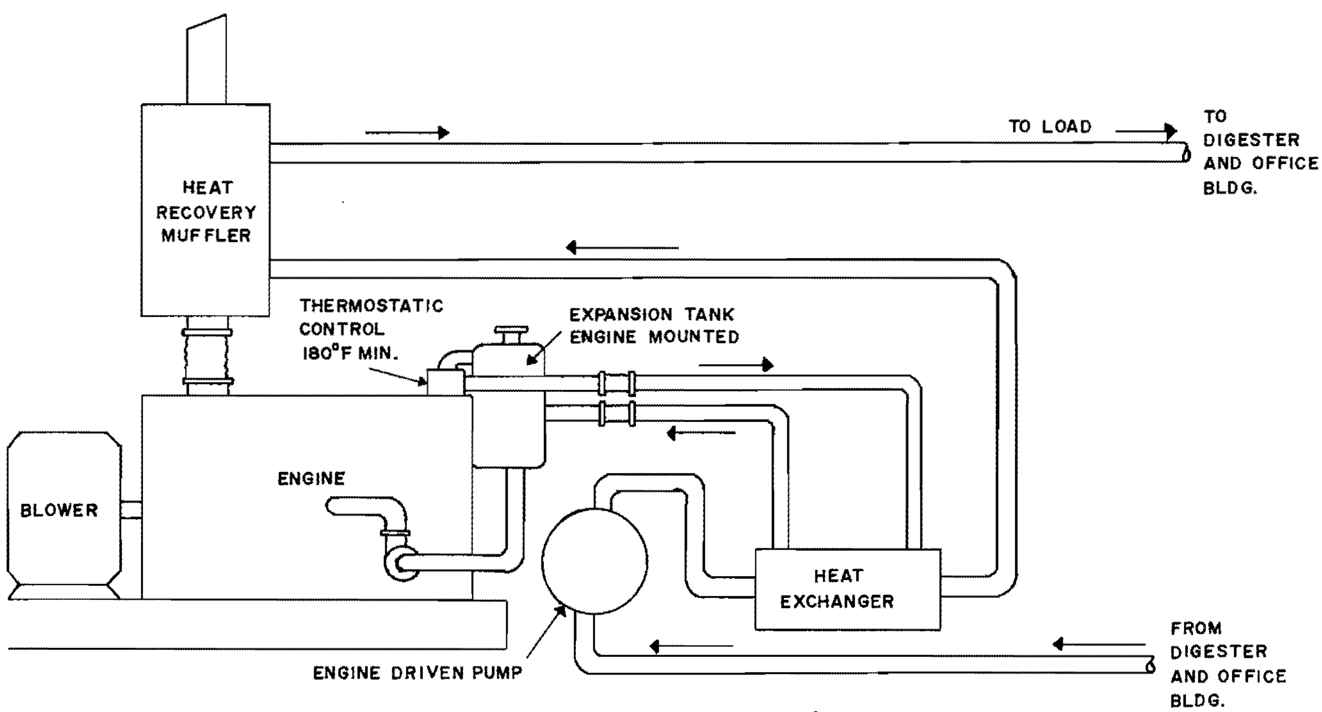


Figure 1. Fuel system.



Flow diagram, hot water system-normal temperature.

carburetor. In this line should be provisions to burn any surplus gas. A pressure regulator and flow meter are the next components in the system. This regulator reduces the fuel pressure to 20 PSI. The flow meter is used to mix the sewer gas with natural gas if this type of supplementary fuel is needed. After the flow meter we have a manual valve and solenoid valve. The manual valve is there for shut-off of fuel working on the engines. The solenoid valve is used when there is a safety shutdown to kill the gas supply to the engines. The last two components is a final pressure regulator to deliver a constant 12 PSI of gas to the digester carburetor. This is a quick overview of a typical system. Each one will be a little different.

One thing that should be taken into consideration is that a reciprocating engine will deliver approximately 30 percent of the fuel input energy to mechanical power. 30 percent of the input energy will go to the cooling system, 30 percent will go out the exhaust and 10 percent will go into the oil and radient heat. In a prime power application it would be favorable to recover all the available energy. This would be all the heat in the cooling system and half the heat in the exhaust system bringing the fuel efficiency rate to 75 percent of all fuel energy input.

The other important fact which needs to be considered is maintenance cost. The 1200 R.P.M. engines will run approximately 30,000 hours before factory recommended overhaul at a cost of about $1.63 per hour. These overhaul periods can only be realized if proper maintenance is done at recommended maintenance intervals and the sewer gas is scrubbed to meet the necessary requirements. If the gas is not cleaned properly you could experience premature engine failure.

This is a quick overview of a sewer gas installation and I would be glad to go over any portion with any of you at any time.

** Ken Green is the Engine Sales Representative for ICM Equipment Sales & Rentals, Salt Lake City, Utah.*

Carrousel...activated sludge treatment system 'can consistently meet all current and many future U.S. clean water requirements at capital and operating costs significantly lower than comparable plants...'

Engineering News Record, Jan. 5, 1978

Reporting on this significant development in wastewater treatment, ENR based its report on data obtained from 129 installations in the U.S. and Europe.

Betters State Requirements

Effluent from the 500,000 gpd system operating on a strong industrial waste in New Hampshire contains only 3-7 ppm BOD_5, well below state requirements. This is what Kim Barber, pollution control manager for the A.C. Lawrence Leather Co. tannery, told ENR.

Nitrate Removal

The system has the ability to break apart nitrates into nitrogen gas and oxygen without additional treatment, Barber explained. Removal of nitrates will be required under the EPA's clean water regulations for industry.

Some other key points about Carrousel installations as reported by ENR:

50% Reduction in Construction Costs

Construction costs for a 4.2 mgd Carrousel system for Campbellsville, Ky., will be about 50% lower than for a conventional activated sludge plant of the same capacity.

Construction costs are low because "it's concrete intensive as opposed to equipment intensive."

Reduces Energy Costs by 20%

Energy use will be 20% less than in conventional plants.

Want more information on this important cost-effective method of wastewater treatment? The complete story is contained in this 12-page technical brochure. Write Eimco Process Machinery Division, Envirotech Corporation, P.O. Box 300, Salt Lake City, Utah 84110, for bulletin No. PMD 5258.

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