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Proceedings of the U.S. Department of Energy Energy Optimization of Water and Wastewater Management for Municipal and Industrial Applications Conference

December 10-13, 1979

New Orleans, Louisiana

Volume 2

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FOREWORD

These proceedings document the presentations given at the Energy Optimization of Water and Wastewater Management for Municipal and Industrial Applications Conference, sponsored by the Department of Energy (DOE). The conference was organized and coordinated by Argonne National Laboratory.

The conference focused on energy use and conservation in water and wastewater. Each session was led by an authority in that field.

Conference sessions and workshops were attended by consultants, federal, state and local government employees, industrial representatives, researchers, engineers, scientists, educators and other professionals involved with water and wastewater treatment.

The first three talks of the General Session -- Introduction, Welcome and DOE Perspective -- spell out the objectives and set the mood for the presentations that follow.

The General Session also reflects DOE's commitment to the support and development of waste and wastewater systems that are environmentally acceptable.

The conference proceedings are divided into two volumes. Volume 1 contains the General Session and Sessions 1-5. Volume 2 covers Sessions 6-12.

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WATER SUPPLY

Chairpersons: Gary Yoshioka

Caroline Brooks

ENERGY CONSERVATION IN WATER TREATMENT

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In recent years, the conservation of energy has become a primary concern of the design engineer. In the design of a water treatment plant, the challenge to the engineer is to design a plant that will conserve energy and, at the same time, accomplish the necessary functions of a treatment facility. Many opportunities for both direct and indirect energy conservation are available to the designer.

Direct energy conservation includes savings in electricity, natural gas, fuel oil and similar types of energy. These savings can be achieved in many ways, but one of the most significant is by conducting a careful hydraulic analysis of the facility to keep pumping heads and, in turn, pumping costs to a minimum.

For example, the following calculation indicates that the energy needed each year to lift a flowrate of 1000 gpm one additional foot in height is enough to provide heat and lights for one month for the average electrically heated residence in the Mid-South area.

$$\text{KWH/Year} = \frac{Q \times H \times 0.746 \times 365 \times 24}{3960 \times \text{Pump eff.} \times \text{Motor eff.}}$$

$$= \frac{1000 \times 1 \times 0.746 \times 365 \times 24}{3960 \times 0.82 \times 0.90}$$

$$= 2236 \text{ Kwh}$$

Another opportunity for conserving direct energy is the careful selection of building materials. The use of effective insulation in building walls and roof structures, double glazed windows and skylights and other architectural features can easily be incorporated into the

design of a facility in order to reduce heating, cooling and lighting costs over the entire life of the facility. Specifying highly efficient motors, engines, pumps and other mechanical equipment offers still another opportunity for savings in direct energy.

An often overlooked direct energy savings technique is to make use of waste energy generated by the normal and necessary operation of the treatment facility, such as using the heat generated by large electric motors to heat other rooms in the building and reclaiming portions of the water used to backwash rapid sand filters.

Savings in indirect energy can be achieved by selection of building materials that are not energy intensive in their manufacture and by careful analysis in early planning to assure construction of necessary components only. Savings in future building materials can be realized by the careful layout and sizing of some components so that, if future expansion is needed to obtain additional capacity, slight modifications to these components will allow them to perform a different function, or the same function at an increased capacity, thus minimizing both new construction and renovation efforts.

Since energy is also used to manufacture and transport treatment chemicals, indirect energy savings can be achieved by reducing the quantity of chemicals needed in the water treatment processes. This can be accomplished by efficient use of the chemicals by thorough mixing and careful selection of feed points. The layout and relative location of feed equipment and storage areas can reduce spillage and waste of chemicals. Attention to future maintenance problems can reduce the quantities of chemicals wasted during these maintenance activities.

The primary function of a water treatment plant is to produce an acceptable quality of water and deliver it to the consumer, all at the most economical price possible. By its very nature, a water treatment plant, like many manufacturing facilities, is very energy intensive. Some of the components in a water treatment plant that require large amounts of energy are:

1. Raw water pumps: Raw or untreated water from a surface supply or an underground aquifer is delivered to the treatment plant by these pumps.

2. Intermediate pumps: As the water flows by gravity through the treatment process, intermediate lift pumps are sometimes required to reduce the below grade depth of tanks and structures, or, in the renovation of a treatment facility, to better utilize existing components.

3. Treated water pumps: Often referred to as high service pumps, these pumps deliver the treated water to the consumer and constitute one of the largest energy consuming items in a water treatment plant.

4. Building heating, cooling, ventilating and lighting systems: These items are required in a water treatment plant just as in any other building. Heat is needed for the comfort of operating personnel and to prevent exposed pipework from freezing. Air conditioning is essential, in some cases, to control the environment around sophisticated electronic equipment and is desirable in some areas of the plant for personnel comfort. Adequate lighting is always essential, especially around large machinery and equipment, both for maintenance activities and for the safety of the personnel. Condensation control equipment is often included in a design to reduce the quantity of water that condenses on the walls of water

containing pipes and structures. This condensation not only detracts from the overall appearance of the facility, but it can damage and deteriorate delicate equipment.

The North Water Treatment Plant in Jackson, Tennessee, which is currently under construction, was designed to incorporate the energy conserving ideas outlined above.

The plant was designed for a treatment capacity of 10 million gallons per day (MGD) with provision for future expansion to a capacity of 20 MGD. The water supply is from wells that are about 200 feet deep. The ground water contains high concentrations of carbon dioxide which, if not removed, causes corrosion in pipelines, pumps, and other equipment. The water also contains traces of dissolved iron which, if not removed, stains laundry and bathroom fixtures.

The well pumps were specified to be the most efficient practicable and the materials used in the construction of the pumps were carefully specified to maintain the high efficiency by reducing wear and corrosion. The high overall efficiency will insure minimum raw water pumping costs throughout the life of the facility. In constructing the wells, the water bearing sand was chemically treated to remove fine grained material. Removal of this fine grained material, which hinders the flow of water through the sand, reduces pumping costs.

The ground water is pumped to a coke tray aerator to reduce the carbon dioxide concentration to near ambient. The aerator has nine coke trays and an efficient two-stage tray distribution system, all of which are highly effective in the removal of carbon dioxide. By reducing the carbon dioxide concentration as much as possible, the quantity of chemicals, in this case, lime, needed to render the water non-corrosive is held to a minimum.

From the aerator, the water flows to a rapid mix basin, where a highly effective mechanical mixer efficiently disperses chemicals into the water.

From the rapid mix basin, the water flows through a detention basin to provide reaction time for the chemicals and then through mixed media, rapid sand filters. By carefully utilizing the grade of the site, the entire treatment process is accomplished by gravity flow, thus eliminating the need for intermediate pumping, even though the headloss through the plant after aeration is more than 10 feet.

In the rapid sand filters, special filter media and other equipment were specified to insure long intervals between backwashing and to insure that the quantity of backwash water used will be held to a minimum. This was done to reduce operating costs and conserve energy since every gallon of water pumped unnecessarily requires additional energy.

The detention basin was specifically sized and constructed to allow its conversion to rapid sand filters when needed in the future.

Treated water is pumped from an underground reservoir into the distribution system by four 200 horsepower high service pumps. Since these pumps constitute the largest energy consuming equipment in the plant, energy conservation with regard to these pumps is of primary importance.

The treated water pumps are automatically started and stopped by the water level in an elevated storage tank. Before a pump automatically starts, however, the plant operator receives a 60-second audible warning. If other pumps are already operating, the operator has the opportunity to override the pump start command if he knows from previous water consumption records that the high consumption period, which initiated the pump start command, will be of short duration and that the operating pumps can indeed

meet the water demand. By overriding the pump start command for one of these large horsepower motors, he has not only reduced power consumption but also reduced the wear on the pump, motor and auxilary equipment.

The four 200 horsepower high service pumps are located in the Pump Room along with a 100 horsepower backwash pump and a diesel powered auxilary engine. There is also space available for two future pumps which will be 250 to 300 horsepower each. A 200 horsepower electric motor will generate 50,000 BTU/hour in heat loss. With this in mind, the importance of proper ventilation in the Pump Room is obvious and the opportunity for recovering some of the waste heat becomes very attractive.

During the summer months, ventilation of the Pump Room is rather straightforward; outside air is simply directed toward the pump motor by two 9400 CFM roof mounted supply fans. The warm air is removed by two 9600 CFM exhaust fans. All of the fans are controlled by thermostats.

During the winter months, when heat is needed in the adjacent Maintenance Room, air handling equipment circulates the warm air from the Pump Room through the Maintenance Room to economically heat this part of the building. When heat is not needed in the Maintenance Room but ventilation is needed in the Pump Room, a fan jet system is used to furnish make-up air to the Pump Room. With this system, a high pressure fan inflates a polyethylene tube located near the ceiling of the Pump Room. Air exits the tube through a series of orifices at a high velocity. The high velocity causes the cold outside air to thoroughly mix with the hot inside air. The resulting warm air is then exhausted. Without the fan jet system, make-up air heaters would be required to reduce the chill of the incoming outside air in order to eliminate the cold zones which would cause discomfort to operating

personnel and, under very cold temperature conditions, freeze small water lines.

During the winter months, more than 184 million BTU of waste heat equivalent to 53,900 KWH of electrical energy, will be available to be transferred to other areas in the water treatment plant building. Approximately one-third of this waste heat will be used to heat the adjacent Maintenance Room.

Water used to backwash rapid sand filters has, in the past, been discharged to surface streams without treatment. Current stream pollution regulations have, however, rendered this practice illegal. The backwash water must now be treated to the degree required by regulatory agencies, either with treatment facilities on site or at another treatment facility, usually the municipal wastewater treatment plant. In either case, a holding structure must be provided to contain the backwash water because a very large volume of water is used in a relatively short period of time. The holding structure acts as a surge tank to contain the large volume of water until it can be treated. The backwash water is then treated at a lower flowrate over a longer period of time, thus reducing the size of the treatment facilities.

Experience has shown that most of the solids that are removed from the filters by backwashing can be removed from the backwash water by sedimentation. The clear water that remains can then be reclaimed for reprocessing as raw water.

At the Jackson plant, pumps have been provided to return the clarified portion of the backwash water to the treatment process. This provision could reclaim nearly 70 million gallons of water each year. In addition

to reclaiming the water, substantial savings in electrical energy are achieved since less power is needed to pump water from the holding structure than from the ground water supply. Savings in electrical energy could exceed 48,000 KWH annually. Additional energy savings are realized by reducing the quantity of water handled at the wastewater treatment facility.

In order to conserve energy used in heating and cooling the building and to reduce vandalism, windows were omitted in the design of this treatment plant. However, double dome skylights have been strategically located to provide light in hallways and other areas, eliminating the need for artificial light during daylight hours.

In conclusion, numerous areas for energy conservation are available to the design engineer. At the North Water Treatment Plant, Jackson, Tennessee, several of these areas were carefully considered, not only to conserve direct energy such as electricity, but also to conserve indirect energy in the form of building materials and chemicals. Systems to recover waste heat and backwash water, normally wasted, were also incorporated into the design of the facility.

In summary, it would be difficult to evaluate the indirect energy savings realized by an efficient layout and wise choice of construction materials. A less efficiently designed facility would be needed for comparison. Fortunately, however, most water treatment plants, designed by competent and conscientious design engineers, are efficiently designed.

However, the direct energy savings at the North Water Treatment Plant, Jackson, Tennessee can be quantified as follows:

1. Heat Recovery: 184 million BTU available per year equal to 53,900 KWH of electrical energy. Approximately 60 millions BTU (18,000 KWH per year) used initially (1).

2. Backwash Water Recovery: 70 million gallons of water per year
48,000 KWH per year.
8,000 lbs. of lime per year

(1) While all of the heat is not recovered from the high service pumps, the basic application used in this project is technically feasible and cost effective. Although more of the heat could have been recovered to heat the remaining parts of the building it was not cost effective at this time.

In future designs of water treatment plants, as well as other facilities, the designer should ask the question, "Is anything being wasted that can be economically reduced or recovered?" He must be reoriented to constantly try to apply some basic laws of physics and chemistry - that the movement and processing of matter requires energy. With this in mind, and as the cost of basic energy increases, many other applications will become apparent and can be applied.

ENERGY RECOVERY ON HIGH PRESSURE WATER CONDUITS

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INTRODUCTION

Since the turn of the century, the United States has experienced a growing dependence on oil as a source of energy. This dependence included both domestic and imported oil. The 1970's saw the formation of the Organization of Petroleum Exporting Countries and large increases in the prices of imported oil. The government controls on the price of oil has discouraged the development of alternative and more costly energy sources such as oil shale and coal gas. As a result the supply of oil has become limited and more costly.

Solutions to the oil crisis include the development of additional coal and oil deposits, more efficient use of oil and the development of other energy sources such as wood, solar, hydro and biomass. Because of the increased cost of energy in the United States, sources of energy which in the past were considered expensive are now becoming economically feasible. One such source of energy is small scale hydroelectric facility.

SMALL SCALE HYDRO

A small scale hydroelectric project is defined by Section 408 of the "Public Utility Regulatory Policies Act of 1978", PL-617, as any hydroelectric power project which is located at the site of any existing dam and which produces less than 15,000 kilowatts of installed capacity. In general, however, the potential for power development exists, not only at existing dam sites but also from high pressure water conduits and low pressure water irrigation canals.

The department of Energy studied the potential for small hydroelectric development in 1976. This study was further refined by the United States Army Corps of Engineering. These estimates show a potential for small hydroelectric development of approximately 44 billion kilowatt hours of energy annually. This energy production translates into a daily savings of about 200,000 barrels of oil per day.

These assessments are related to existing dam sites. There are a number of irrigation canals in the Western United States which are suited for small hydroelectric development. There are also a number of municipal water systems which have aqueducts and conduits delivering water under high pressure to the centers of consumption. These systems are also suitable for hydroelectric development where flow of water to the consumer is not interrupted and the quality or potability of the water is not affected.

Small scale hydroelectric development can be classified into two categories, (1) high flow-low head hydro and (2) low flow-high head hydro. The high flow (greater than 500 cfs) low head (less than 50 feet) hydroelectric development is normally seen on irrigation canals. The low flow (less than 500 cfs) and high head (greater than 50 feet) are customarily seen on high pressure domestic water systems. These classifications are illustrated in Figures 1 and 2.

In Southern California, several water supply agencies are evaluating the recovery of energy from high pressure water conduits. This paper contains a discussion of the evaluation factors which should be considered in the feasibility analysis for a small scale hydroelectric project. The factors discussed included technical feasibility, economics, regulation potential marketability and the environmental impacts.

The examples used in this paper are from the project being reviewed by the City of Santa Barbara, and the City of San Luis Obispo, both in the State of California. The Metropolitan Water District of Southern California is also actively involved in a program for installation of hydroelectric unit within its water system.

TECHNICAL

General

Figure 3 shows the project components for a typical high head-low flow hydro project. The components usually involved a main water supply, reservoir where runoff is collected and stored before treatment and delivery to the consumer. The delivery system may have a small reservoir ahead of the treatment plant to balance the flow difference between the rate of delivery of the treatment rate. The hydroelectric unit is usually

located at low point in the delivery system. Power transmission lines are needed to convey the electric power to the owner's facilities or to the power company grid.

In the evaluation of a small scale hydro project there are several technical aspects which should be considered. Some of these technical considerations include flow rate variations, available pressure, operating hours and selection of electromechanical equipment.

Flow Analysis

The first step in evaluating the power generation potential is to perform a statistical analysis of the past and projected water delivery flow rates. An example of this analysis is shown in Table I and Figure 4 for the City of Santa Barbara's Gibraltar Water Supply.

The annual deliveries from the Gibraltar system for 1950-51 to 1977-78 are summarized in Table I. These annual deliveries have ranged from a low of 2,497 AF in 1950-51 to over 11,000 AF in 1955-56, 1958-59, 1959-60, 1970-71, and 1971-72. These data have also been plotted on the probability graph shown in Figure 4. The graph shows that 50 percent of the time the deliveries from Gibraltar over the last 28 years were equal to or greater than 8,000 AFY. The annual delivery of 11,000 AFY was exceeded only 10 percent of the time.

TABLE I

ANNUAL DELIVERIES-GIBRALTAR RESERVOIR

Fiscal Year	Acre Feet	Fiscal Year	Acre Feet
1977-78	4,064	1963-64	8,527
1976-77	5,269	1962-63	9,410
1975-76	8,719	1961-62	6,467
1974-75	8,866	1960-61	4,301
1973-74	9,777	1959-60	11,669
1972-73	9,268	1958-59	11,851
1971-72	11,538	1957-58	7,918
1970-71	11,240	1956-57	7,091
1969-70	8,736	1955-56	11,287
1968-69	7,785	1954-55	4,818
1967-68	8,522	1953-54	6,600
1966-67	7,831	1952-53	4,972
1965-66	9,487	1951-52	2,803
1964-65	6,437	1950-51	2,497

The rate at which power can be generated on a pipeline may also depend upon the amount of on line storage. If a flow balancing reservoir exists between the water source and the point of delivery, this reservoir could function as an afterbay for the hydroelectric unit. This concept allows for the generation of power with some independence from the water supply demands.

The pipeline sizing will have an impact on the amount of available head for power generation and the economic benefits of the power generation station. Each foot of headloss results in a corresponding reduction in the amount of available head for power generation. This can be expressed in economic terms as the present worth of one foot of available head using the following equation:

$$PW/ft = (Q)\ (\gamma)\ \frac{(H)\ (eff)\ (Kw/hp)\ (\$/Kwhr)\ (operating\ hours)}{550}\ pwf$$

Q = 15 cfs (average)

γ = 62.4 lbs/cf

H = 1 foot

eff = 0.82

Kw/hp = 0.75

$/Kwhr = 0.05

hours = 6500

pwf = 15.762 at n = 50 yr, i = 6%

$$PW/ft = (15)\ \frac{(62.4)\ (1)\ (.82)\ (0.75)\ (0.05)\ (6500)}{550}\ (15.762)$$

PW/ft = $5400/foot at head

In the near future the average cost of power is estimated to be 50 mils per Kwhr. At this power cost, the present worth of one foot of available head is approximately $5,400 based upon 6,500 hour operation per year, a 50 year project life and 6 percent discount rate. The estimate is considered conservative because it does not reflect the escalation of the cost of power. The calculation shows that the pipeline should be sized to provide the optimum level of power generation consistent with hydraulic delivery capacity.

Under solely hydraulic delivery criteria, the basic pipeline size required to carry 15 cts is 18 inch diameter. An economic analysis should be conducted to determine the optimum size of the pipeline when it is considered as a penstock to the power station. These analyses of the present worth of the pressure head loss and the incremental construction cost of the pipeline greater than 18 inches in diameter can be compared for various pipe sizes. Table 2 shows an example of this calculation. The analysis was repeated for various operating times of 8,500 hrs/yr, 6,500 hrs/yr, and 4,500 hrs/yr. This comparison is shown in Figure 5. In this example, a 27 inch diameter penstock proves to be the most economical size.

TABLE 2

PENSTOCK ECONOMIC ANALYSIS[1]

Q = 15 cfs
Operation time = 6,500 hrs/yr

Penstock Diameter (inches) (1)	Friction Loss (for 15 cfs) (ft) (2)	P.W.[2] Value of Friction Loss (3)	Construction[3] Cost (4)	Construction Cost of 18 in. Line w/out Generation (5)	Net total[4] PW Cost for Generation (6)	PW Sum[5] of Costs h_f + Net Penstock (7)
20	58.5	315	258	250	8	323
24	22.7	122	318	250	68	190
27	10.0	54	360	250	110	164
30	6.9	37	402	250	152	189
33	4.1	22	450	250	200	222
36	2.6	14	504	250	254	268

[1] All costs in 1,000's dollars.

[2] P.W. = $5,400/foot of head.

[3] Penstock.

[4] Column 4 minus Column 5.

[5] Column 6 plus Column 3.

Assuming a typical hydro project as shown in Figure 3, and given that the 50 percentile annual delivery of 8,000 AF occurs over a nine month period at a continuous flow rate of 15 cfs and a residual pressure of 600 feet, the power generation potential can be determined as follows:

$$Kw = (Q)\ (\gamma)\ (H)\ (eff)\ (.75)$$

$$Kw = (15)\ \frac{(62.4)\ (600)\ (.82)}{550}\ (.75)$$

$$Kw = 630$$

If a balancing reservoir is available for use as an afterbay to the hydro-unit, the daily water deliveries can be made during a 12 hour period each day, thus doubling the amount of potential energy available during peak energy periods. In this operating mode, the reservoir would function as an afterbay to balance the flow fluctuations between delivery rates and demand rates. These available flow rates translate into a potential energy generation of 630 kw on a continuous basis and 1,260 kw on a peak use basis for annual yields which can occur 50 percent of the time.

Equipment

There are three basic types of turbines and each are characterized by their specific speeds. Specific speed is defined as:

$$n_s = \frac{N\sqrt{P}}{H^{5/4}}$$

where: n_s = specific speed
N = shaft speed (rpm)
P = rated power (hp)
H = rated head (ft)

The three types of turbines are propeller, reaction, and impulse turbines. Illustrations of these turbines are shown in Figure 6. The propeller turbine is characterized by high specific speeds of over 100 and is used in low head and high volume applications such as low dams and irrigation channels.

The reaction turbine, often called the Francis turbine, has a runner which is similar to a centrifugal pump impeller with the flow and rotational direction reversed. The water enters the runner radially from an enclosed spiral case and the flow is guided and regulated by vanes called wicket gates. Reaction turbines are designed to operate with the heads of 50 to 1,000 feet and in a specific speed range of 20 to 100. The reaction turbine must operate with a submerged runner to avoid cavitation. The efficiency of the reaction turbine is significantly reduced at low

load as shown in Figure 7. Maximum efficiency is achieved at 90 percent loading and reduces to 65 percent at 50 percent loading.

The impulse turbine has a runner with a series of buckets which are driven by a water jet regulated by one or more needle valves. The flow and power can be regulated by opening and closing this needle valve. Impulse turbines are designed to operate with heads up to 2,000 feet and have specific speed range of up to 20, depending upon the number of jets. The impulse turbine can operate with a free discharge thus allowing some aeration of the water. As shown in Figure 7, the impulse turbine can maintain good efficiencies over a wide range of flows.

An important consideration in the selection of a turbine is the runaway characteristics. If the generator is disconnected from the electrical system while power is being generated, the rotational speed of the turbine will increase until it reaches runaway speed which is usually 180 percent of the design speed. The turbine and generator are designed for this condition. However, the flow through the propeller and reaction turbines are effected by the speed of the runner. At low specific speeds, turbine can cause a flow reduction which may create upstream water hammer conditions. At high specific speeds, a turbine such as the propeller turbine, can cause high flows at runaway speed and water hammer conditions downstream. However, the flow through an impulse turbine is unaffected by the speed of the runner.

The type of turbine normally recommended for the high head-low flow application is the impulse type. The unit will provide high efficiencies over the range of flows, runaway speed and surge control is not a concern, and some aeration of the water may be provided at the free discharge. The impulse turbine is also the least costly because the reaction turbines are commonly not built for the low flow ranges and, at the present time, would require special fabrication by some domestic manufacturers.

There are two basic types of generators which can be used in a small scale hydro project. The generator types are the synchronous and induction. The selection of the proper generator type depends on the size of generator, site location, and power company electrical grid requirements.

The synchronous generator requires an exciter regulator to provide accurate voltage regulation over varying conditions of load, power factor frequency and load unbalance. Accurate control of generator voltage is achieved through the use of a closed-loop regulating system consisting of

valve or blade positioner, governor and exciter. Excitation power is normally obtained from the generator output voltage. The synchronous generator has the advantage of operating independently from the other power sources and is, therefore, applicable for isolated operation, end of the line installation and where the power factor is a concern.

The induction generator is driven at a speed slightly greater than the synchronous speed and obtains its excitation from the power system. As a result, this generator is usually less costly because it doesn't need the automatic speed control and accessories that are associated with the synchronous generator. The induction generator may be used for (1) operation in conjunction with a large power system, (2) low capacity and high generator speed and (3) applications where power factor correction is not a concern.

PROJECT COST

The single most important criteria for the development of a hydroelectric project is the economic feasibility. The economics of a project must consider basic construction and engineering costs, operating costs, the market value of the power generated and the estimated escalation of the cost components.

Table 3 shows a preliminary project cost estimate for a typical project. The estimate should be developed from a preliminary project design which has identified the various cost components. Depending on the level of the design effect, a contingency factor of 10 to 30 percent should be added to the project subtotal. The preliminary cost estimate for the hydroelectric project, including the incremental increase in penstock diameter and the power transmission line, is $728,000. Hydroelectric units of this type are operated without personnel attendance, have a long equipment life and low maintenance cost. Therefore, the annual operation and maintenance costs are very low; and in this case, it is estimated to be about $10,000/year in the first operating year. If the capital cost is amortized at 8 percent for 30 years, the initial annual cost of the hydroelectric station would be $74,000 per year. The station will produce power which has a present value of 30 to 40 mils per Kwhr. The value is expected to increase to 50 mils per Kwhr in the near future and to 100 mils per Kwhr within the next 15 years. Figure 8 shows a plot of the annual project cost and the variable value of the power generated as a function of the available annual yield from water source.

The breakeven points are the intersections of the estimated annual cost line with the estimate annual revenue value plotted for the unit energy cost of 40, 50 and 60 mils per Kwhr.

A cost-benefit analysis can also be performed. The example shown in Table 4 is based upon the following assumptions:

1. Available water - 4,000 AFY
2. Continuous flow - 15 cfs
3. Power generated - 750 Kw
4. Hours of operation - 3,200 hours
5. Energy value - 40 mils per Kwhr

The analysis is a comparison of the present worth of the annual project cost and the annual energy value. In this example, the unit realized by displacement of local power plus the sale of excess energy to the Power Company. The calculations were based upon a 30-year financing period and a 50-year equipment life. The annual operating costs and energy value were escalated at 6 percent per year and the present worth of the annual costs is based upon an 8 percent discount rate. The analysis, shown in Table 4, indicates that the initial benefit to cost ratio is in the range of 1.3 to 1.4 during the first few years of operation. As the value of energy escalates the ratio increased to 4.5 after 30 years. The benefits increase because the major portion of the total annual costs is the amortized capital cost which does not escalate. After the capital cost is repaid in the thirtieth year, the ratio significantly increases to 9.6. Over the 50-year project life the average benefit to cost ratio is estimated to be 5.9.

TABLE 3

POWER STATION
PRELIMINARY COST SUMMARY
(JANUARY 1979)

Power Station		
Earthwork	$ 3,700	
Structures	54,500	
Mechanical Equipment	275,600	
Electrical and Metering Equipment	32,500	
Piping and Valves	40,200	
Subtotal	$406,500	
Contingencies @ 10%	41,000	
	$447,500	
Transmission Power Line	7,500	
Incremental Penstock Size, 18 to 27 inch	110,000	
Estimated Project Construction Cost	565,000	
Engineering @ 15%	85,000	
Subtotal		$650,000
Escalation to January 1980 @ 12%		78,000
Total Estimated Project Cost		$728,000
Amortized Annual Capital Cost at n = 30 years i = 8%		$ 64,000
Estimated First Year O&M Cost		10,000
Estimated Annual Cost (First Year)		$ 74,000

TABLE 4

COST BENEFIT ANALYSIS

YEARS	ANNUAL CAPITAL	ANNUAL O&M	TOTAL ANNUAL COST	POWER REVENUE	CB RATIO
1	64000.00	10600.00	74600.00	101759.99	1.364
2	64000.00	11236.00	75236.00	107865.59	1.434
3	64000.00	11910.16	75910.16	114337.52	1.506
4	64000.00	12624.77	76624.77	121197.76	1.582
5	64000.00	13382.25	77382.25	128469.62	1.660
6	64000.00	14185.19	78185.19	136177.78	1.742
7	64000.00	15036.30	79036.30	144348.44	1.826
8	64000.00	15938.48	79938.48	153009.33	1.914
9	64000.00	16894.78	80894.78	162189.87	2.005
10	64000.00	17908.46	81908.47	171921.27	2.099
11	64000.00	18982.97	82982.98	182236.53	2.196
12	64000.00	20121.95	84121.95	193170.72	2.296
13	64000.00	21329.27	85329.27	204760.95	2.400
14	64000.00	22609.02	86609.02	217046.59	2.506
15	64000.00	23965.56	87965.56	230069.38	2.615
16	64000.00	25403.49	89403.49	243873.53	2.728
17	64000.00	26927.70	90927.70	258505.92	2.843
18	64000.00	28543.36	92543.36	274016.25	2.961
19	64000.00	30255.96	94255.96	290457.27	3.082
20	64000.00	32071.31	96071.31	307884.63	3.205
21	64000.00	33995.59	97995.59	326357.69	3.330
22	64000.00	36035.32	100035.33	345939.13	3.458
23	64000.00	38197.44	102197.45	366695.44	3.588
24	64000.00	40489.29	104489.29	388697.16	3.720
25	64000.00	42918.64	106918.64	412018.97	3.854
26	64000.00	45493.76	109493.76	436740.09	3.989
27	64000.00	48223.38	112223.38	462944.47	4.125
28	64000.00	51116.78	115116.78	490721.13	4.263
29	64000.00	54183.78	118183.78	520164.37	4.401
30	64000.00	57434.80	121434.80	551374.19	4.540
31	0.00	60880.89	60880.89	584456.63	9.600
32	0.00	64533.74	64533.74	619524.00	9.600
33	0.00	68405.77	68405.77	656695.37	9.600
34	0.00	72510.11	72510.11	696097.06	9.600
35	0.00	76860.71	76860.71	737862.87	9.600
36	0.00	81472.35	81472.35	782134.63	9.600
37	0.00	86360.69	86360.69	829062.69	9.600
38	0.00	91542.32	91542.32	878806.37	9.600
39	0.00	97034.85	97034.85	931534.69	9.600
40	0.00	102856.94	102856.94	987426.69	9.600
41	0.00	109028.35	109028.35	1046672.25	9.600
42	0.00	115570.05	115570.05	1109472.50	9.600
43	0.00	122504.24	122504.24	1176040.75	9.600
44	0.00	129854.49	129854.49	1246603.12	9.600
45	0.00	137645.75	137645.75	1321399.25	9.600
46	0.00	145904.48	145904.48	1400683.12	9.600
47	0.00	154658.75	154658.75	1484724.00	9.600
48	0.00	163938.27	163938.27	1573807.37	9.600
49	0.00	173774.55	173774.55	1668235.75	9.600
50	0.00	184201.02	184201.02	1768329.75	9.600

PWC = 462736.4.50 PWR = 27356036.00

CB RATIO = PWR/PWC = 5.912

POWER PLANT OPERATION

A small hydroelectric plant can be designed to operate in several modes depending on the water available, energy need, physical facilities and the basic economics of energy. In the case of the Santa Barbara project, four basic operating options were identified:

(1) Operate the water plant and pump stations from the power station on a 24-hour continuous basis and sell the excess energy to the utility company. This operation will incur a standby charge or a heavy demand charge.

(2) Purchase all power required for the water plant and pump stations and sell to the utility company all generated power on a 24-hour continuous basis. No standby charges will be incurred by this alternative.

(3) Operate the Water Plant and pump station from the power station on a 24-hour continuous basis and sell excess power to the utility company during "on peak" periods only. The alternative will require daily regulation of the flow and the utility company would require a standby charge for providing a second power source. A higher revenue may be received for the power sold during "on peak" periods.

(4) Purchase all power required for the Water Plant and pump station and sell to the utility company all power generated during the "on peak" period. No standby charge is incurred.

Whatever operating mode is selected the utility company will be involved either as a supplier of standby energy or actual energy and as a potential buyer of excess energy. The Southern California Edison Company is presently in the process of formulation policies for parallel generation. Discussions with the Edison Company have revealed the following policies which are under consideration.

Edison Company could provide a standby power service in parallel with the customer's hydropower generation mode. All the required electrical equipment including, but not limited to the primary switchgear; synchronization, protection and automatic metering equipment; pad-mounted transformer, etc., would be engineered, supplied and installed, operated and maintained by the Edison Company at a flat charge of 1-1/2 percent of the capital cost per month to the customer.

If the customer is able to meet his power requirements and has surplus hydropower available, Edison Company will purchase the energy at a price of 85 percent of the A.S.E.C. A.S.E.C. stands for the Average System's Energy Cost and is calculated on a daily basis. In early 1979 the A.S.E.C. was valued at approximately 15 mils/Kwhr. At the present time Edison is not offering higher rates for "on peak" power generation. Most of their co-generation customers do not have this ability so the policies do not provide this feature. To implement a co-generation program, where Edison would provide energy when needed and purchase energy when available, Edison would require a 10-year contract which defines the guidelines for the program.

Under a different scheme, Edison would purchase capacity in the power plant at approximately $30/Kw-year, if the capacity were available at least 70 percent of the entire year. With a firm commitment, Edison would purchase energy at a rate of 85 percent of the Oil Fired South Coast Basin System Cost which is presently 19 mils/Kwhr. For this type of a program Edison would require a 20-year contract.

Based upon the energy credit available from Edison, it is evident that the City should operate the hydroelectric station to first meet the energy requirements of City-owned facilities such as the Water Plant, the pump stations and any other facilities within reasonable transmission distance from the power station. The Edison Company would be agreeable to the rental of their poles to carry power lines to local City facilities which can utilize the energy. By displacing Edison's energy, the City can produce power which has a value to the City of 40 mils/Kwhr.

If the customer is not able to meet his power requirements and requires Edison power at any time during the year, he has the two billing choices, as follows:

(1) No Standby Service Charge
   a. Demand charge of approximately $2.10/Kw demand to be billed for the next consecutive 11 months.
   b. Energy charge of approximately 45 mils/Kwhr.

(2) Standby Service Charge of $1.50/Kw Demand
   a. Demand charge of $2.01/Kw demand to be billed during the next month.
   b. Energy charge of approximately 45 mils/Kwhr.

For the City of Santa Barbara, it appears that Atlernate No. 1 is the most cost-effective mode of operation based upon the currently anticipated power company policies. The method of operating the power station should be evaluated periodically to reflect changes to these policies. Whatever operating scheme is used, the system should be designed for parallel service. Figure 9 shows a proposed conceptual wiring diagram which provides parallel service with the power company and allows operation under any of the four possible operating schemes previously outlined.

One concept which was not mentioned above, is the concept of "wheeling". Wheeling power means the power is generated by one agency, distributed through another agencies grid system and retrieved by the original power generating agency at its point of use. The concept is used frequently by large power companies but is seldom applied to power company - customer related projects. The approach also involves a complex procedure for determining the wheeling charge for use of power company's grid system.

REGULATION

Water Rights

There are a variety of regulatory and legal problems which may face a small hydroelectric project. These problems may include water rights conflicts, licensing issue, environmental regulation and marketing disputes.

There are two principle bodies of law on water rights in the United States. In the eastern states riparian owners of land abutting a water way own rights to the thread of the non-navigable streams and the state owns the bed of all navigable streams. Before the state will lease the rights to the stream bed it must determine that the public's right of navigation will not be abused and that the use of the stream bed is in the public interest. Riparians, owners of land abutting the stream, have the right to the reasonable use on the natural flow of the water past their land. The generation of hydroelectric power is considered a reasonable use of flowing water.

In the western states, water rights are determined by the doctrine of prior appropriation. Under this doctrine the person who first uses the water has right to the water so long as the water is put to its highest and most beneficial use. A person may lose their rights if they are not perfected or used for the intended purpose within a reasonable time after acquisition.

Water rights doctrines may play a large role in the development of in-stream small hydroelectric projects. In the case of energy recovery in high pressure water conduits, the water purveyor has established his right to the use of the water for domestic or agricultural uses and the secondary beneficial of power generation is not usually disputed.

Licenses

Small hydroelectric project may require licensing from federal or local agencies. In many cases a federal license may be required by the Federal Energy Regulatory Commission, (FERC).

FERC issues three types of licenses (1) a minor project license (projects of less than 1.5 mw of capacity), (2) major project license for projects at existing sites in excess of 1.5 mw and (3) a major project license for projects at new sites. The licensing system for minor projects or major projects at existing sites have been simplified by new FERC regulations. In accordance with the National Energy Act, the FERC has established a short-form license application to encourage the use of small hydroelectric plants. The short-form application is limited to power plants with not more than 1.5 mw capacity. The application must be accompanied with an environmental report. These regulations are outlined in the FERC, Order No. 11, Simplified Procedures for Certain Water Power Licenses

FERC jurisdiction extends to four varieties of hydroelectric projects:

(1) Projects Located on Navigable Waterways

The FERC must license any hydroelectric project to be constructed on any water way which has ever been, is or may become navigable for purposes of commerce.

(2) Projects Affecting Interstate Commerce

The FERC must license hydroelectric projects located on non-navigable waterways if they affect interstate commerce. Commerce may be affected by the operation of the project in such a way as to affect the flow of water in a navigable waterway of which the non-navigable waterway is a tributary, or by the connection of the project to an interstate transmission grid.

(3) Projects Which Utilize Federal Land

The FERC must license hydroelectric projects which utilize public lands and reservations belonging to the federal government. These terms do not include all federal lands. Public

lands are those which may be devoted to private use under the public land laws. Such lands are generally administered by the Department of Interior. Reservations include national forests, Indian reservations and other federal lands withheld from private use. Neither term includes national parks or national monuments. The FERC has the power to reserve federal lands for hydroelectric development.

(4) Projects Which Utilize Surplus Water of Water Power from Government Dams

The property clause has been construed to apply to electricity generated at a government dam and to water made available at a government dam. Consequently, the FERC licenses the use of federal tangible property, just as it licenses the use of federal real property to be used for the generation of hydroelectric power.

The Federal Power Act permits a developer to request FERC to exempt a particular project from the federal licensing process. A developer may file a "Declaration of Intent" with the FERC and seek a waiver of the federal license requirement or, if the developer is installing a hydroelectric facility in a water conduit or water main which is not closely related to a dam, may ask for exemption from licensing. In this latter regard the Public Utility Regulatory Policies Act of 1978 has recently authorized the FERC to exempt "conduit" hydroelectric projects from the licensing requirements and, even more recently, the FERC promulgated regulations establishing a simple, quick procedure for obtaining the exemptions.

In the case of the private developer a permit may also be required from the Public Utility Commission (PUC). If the developer is a public agency such as a city or municipal water district, a PUC permit may not be required.

FUNDING

Under Title IV of the National Energy Act entitled Small Hydroelectric Power Project, the Secretary of the Department of Energy is directed to establish a program to encourage municipalities, electric cooperatives and industrial development agencies, non-profit organizations and other persons to undertake the development of small hydroelectric power projects in connection with existing dams which are not presently being used to generate electric power. This energy act can influence the project funding.

Under Title IV, Section 402, the Secretary after consultation with the Federal Energy Regulatory Commission (FERC) is authorized to make loans up to 90 percent of the cost of feasibility studies. This loan is forgiveable if the project is determined to be neither technologically nor economically feasible.

Project costs are also eligible for loans of up to 75 percent of the total cost if they meet the following requirements:

(1) The project will be constructed in connection with an existing dam or dams,

(2) All licenses and other required Federal, state and local approvals necessary for construction of the project have been issued,

(3) The project will have no significant adverse environmental effects including significant adverse effects on fish and wildlife, on recreational use of the water and on stream flow, and

(4) The project will not have a significant adverse effect on any other use of the water used by such project.

Loans made under Title IV are subject to an interest rate determined at the time the loan is made as set forth in Section 80 of the Water Resources Development Act of 1974. The term of each loan shall be determined by the Secretary, but shall not exceed 10 years for feasibility studies and 30 years for project costs. At the present time, the interest rate on Federal loans is in the range of 6-1/2 to 7 percent. Loans are also available for the preparation of FERC licenses.

Cities and other municipal agencies have the option of financing the projects through municipal bonds, such as general obligation bonds or revenue bonds. Depending on the agency's bond rating, municipal bonds can be sold at a discount rate of 6 to 7 percent with a 20 to 25 year repayment period. The federal loan programs will be particularly interesting to private developers who are faced with the high cost of private financing.

ENVIRONMENTAL CONSIDERATIONS

In the development of any small hydroelectric project the environmental impacts must be considered. The environmental review process is usually coordinated through federal, state or local agencies. Areas of concern are similar to other public works projects, such as water quality control,

protection of fish and wildlife and the preservation of historic places, archeological sites and natural areas. Environmental regulations which effect the development of small hydroelectric projects include the Clean Water Act, Federal Power Act, Anadromous Fish Conservation Act, the Fish Restoration and Management Act, and the Rivers and Harbors Act 1888. Projects effecting historic or archeological sites on federally owned or controlled lands must be permitted by the Secretary of Interior, through the National Park Service. The above regulations are of particular importance for projects which have federal funding or require an FERC license.

Experience, thus far, has shown little adverse environmental impacts for small hydroelectric projects located in high pressure conduits. The small hydro project creates little change in the water use, therefore, does not greatly impact the existing water quality conditions. The small hydro plant (less than 1.5 mw) requires little space and does not displace a significant amount of wildlife. The hydro plant is usually located near urban or rural development where the access is easy. The greater impacts may occur during the construction period, particularly if any new pipelines are being constructed.

CONCLUSION

As the cost of domestic and foreign energy sources increase, the small hydroelectric plant shows greater potential as an additional and economical energy source. The parts of the world such as Europe and Asia, where energy costs have been traditionally high, the small hydro electrical plant concept has been successfully used. In future years, this concept will receive greater application in the United States thus providing a new energy source to replace the foreign oil we have depended upon too much and to keep our economy growing while maintaining the American standard of living.

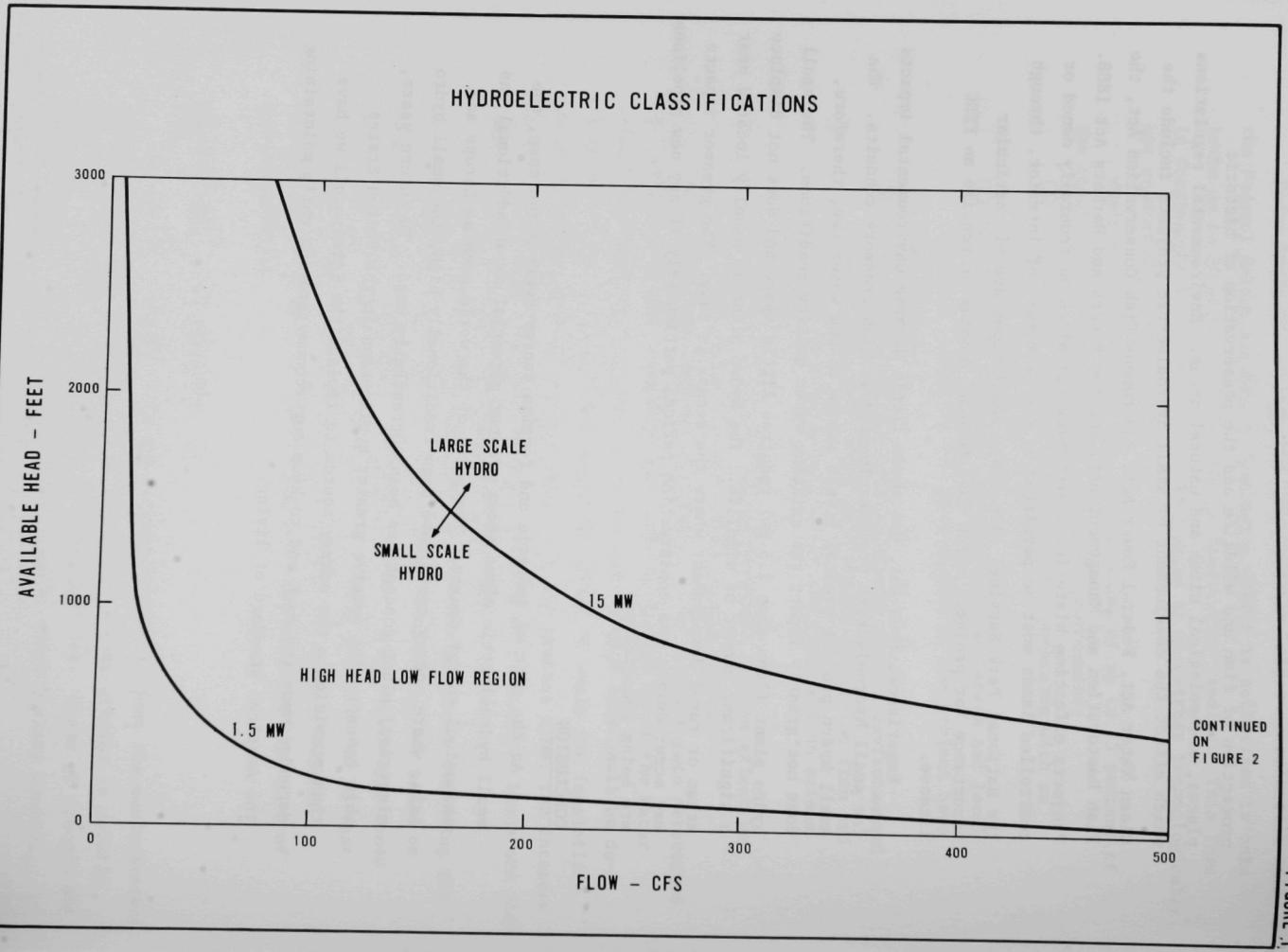

HYDROELECTRIC CLASSIFICATIONS
AVAILABLE HEAD - FEET
3000
2000
1000
0
0
100
200
300
400
500
FLOW - CFS
LARGE SCALE HYDRO
SMALL SCALE HYDRO
15 MW
1.5 MW
HIGH HEAD LOW FLOW REGION
CONTINUED ON FIGURE 2

FIGURE 1

HYDROELECTRIC CLASSIFICATIONS

(Continued from Figure 1)

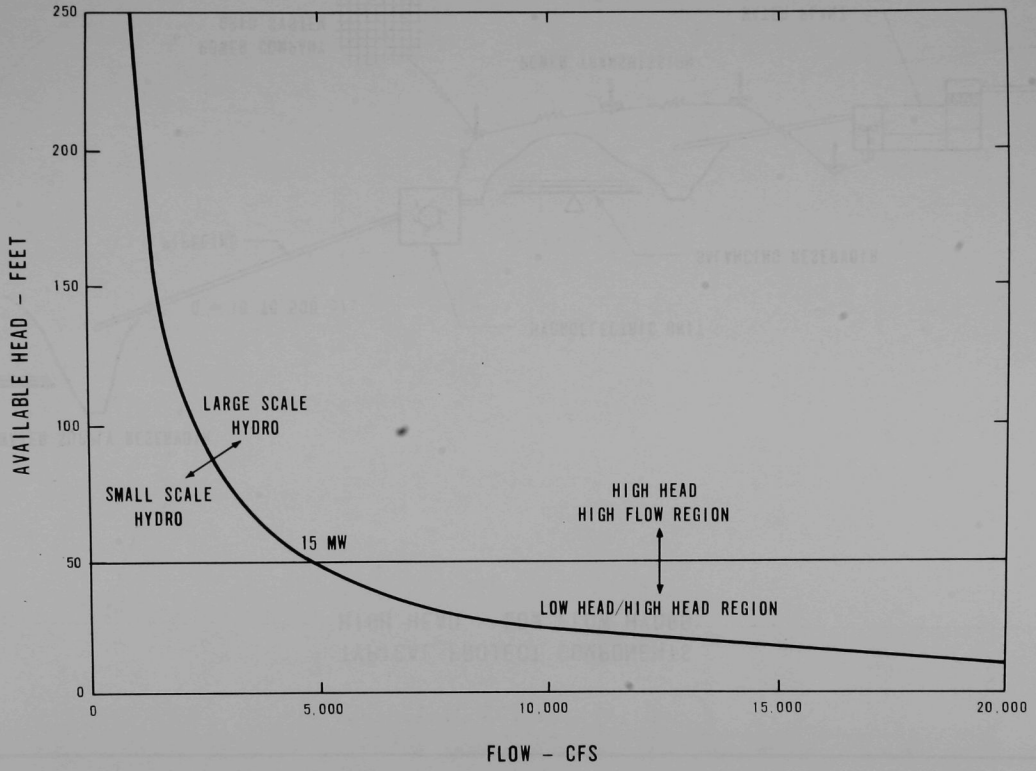


FIGURE 2

TYPICAL PROJECT COMPONENTS
HIGH HEAD – LOW FLOW HYDRO

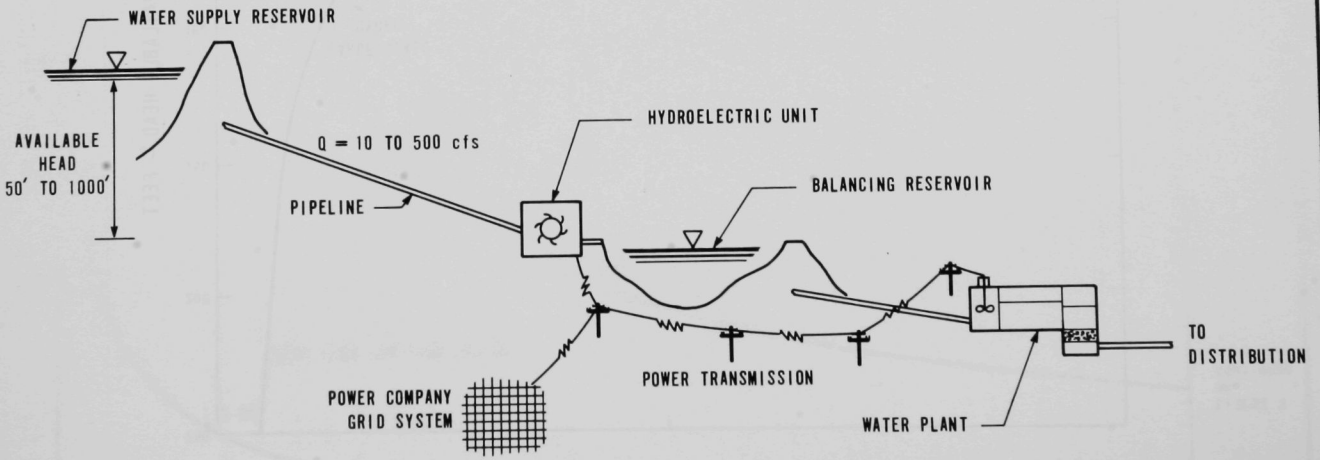


FIGURE 3

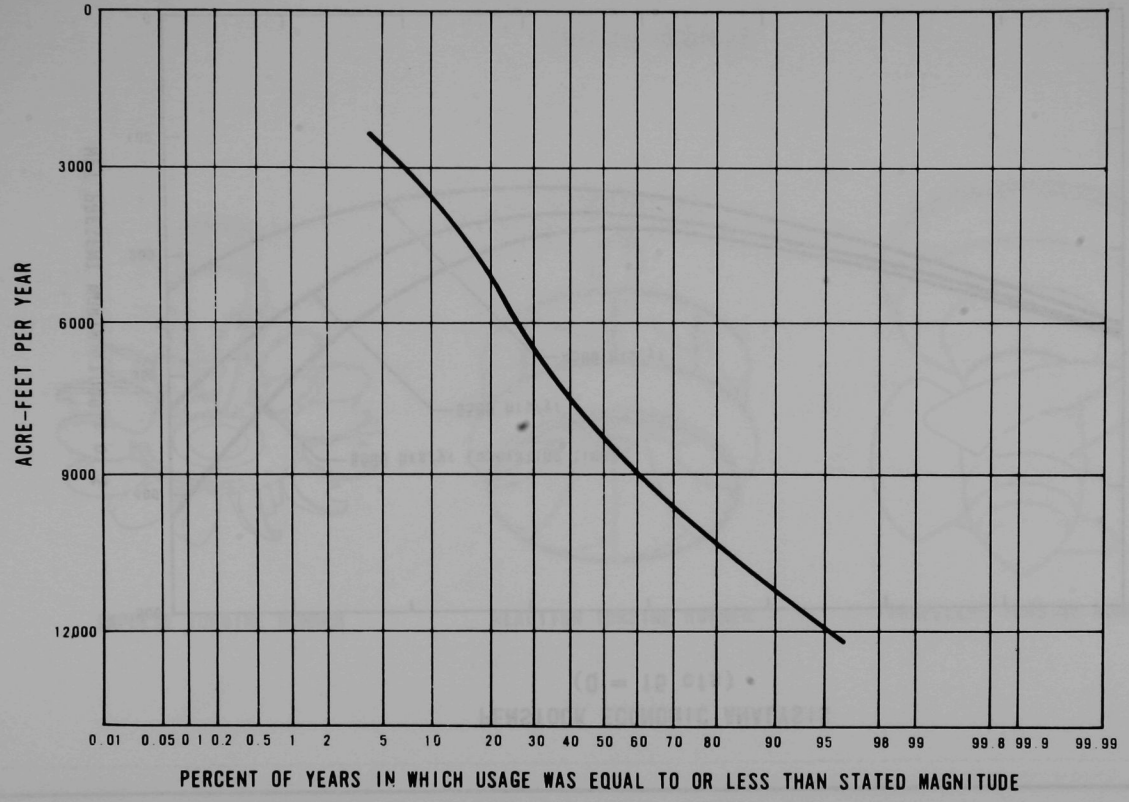

GIBRALTAR WATER DELIVERIES
1950–1978
ACRE-FEET PER YEAR
0
3000
6000
9000
12000
0.01 0.05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.8 99.9 99.99
PERCENT OF YEARS IN WHICH USAGE WAS EQUAL TO OR LESS THAN STATED MAGNITUDE

FIGURE 4

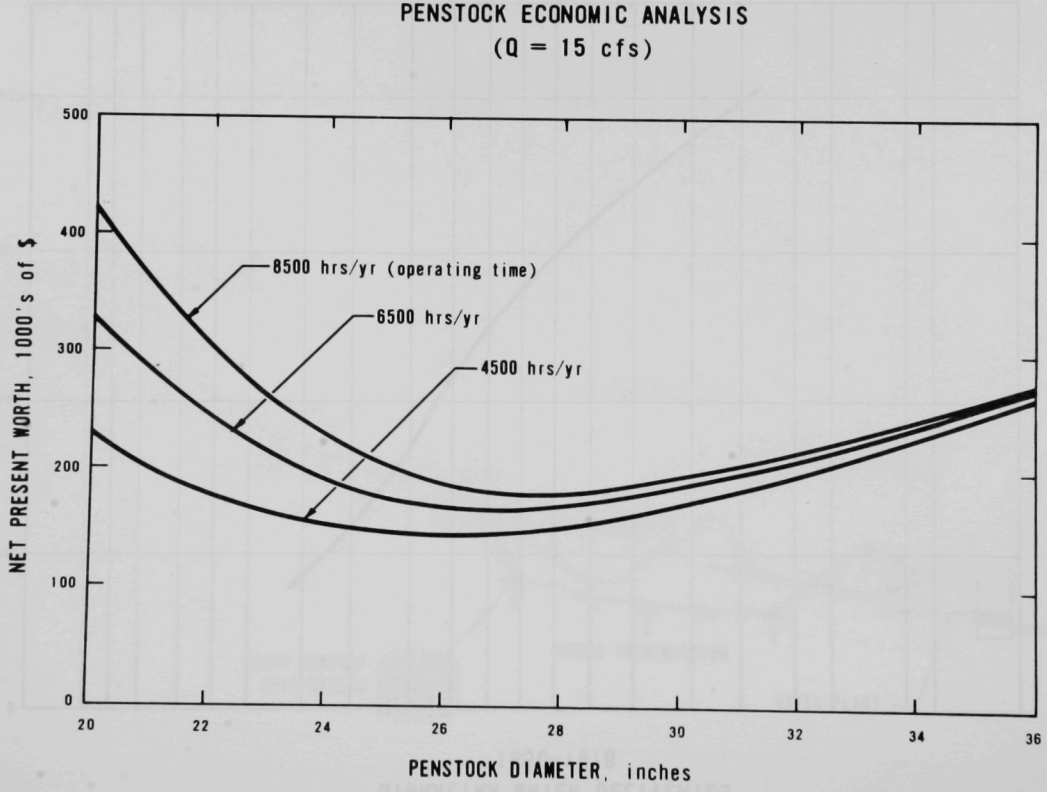

PENSTOCK ECONOMIC ANALYSIS
(Q = 15 cfs)
NET PRESENT WORTH, 1000's of $
500
400
300
200
100
0
8500 hrs/yr (operating time)
6500 hrs/yr
4500 hrs/yr
20
22
24
26
28
30
32
34
36
PENSTOCK DIAMETER, inches

FIGURE 5

TURBINE RUNNERS

IMPULSE TURBINE RUNNER

REACTION TURBINE RUNNER

PROPELLER TURBINE RUNNER

FIGURE 6

TURBINE EFFICIENCY

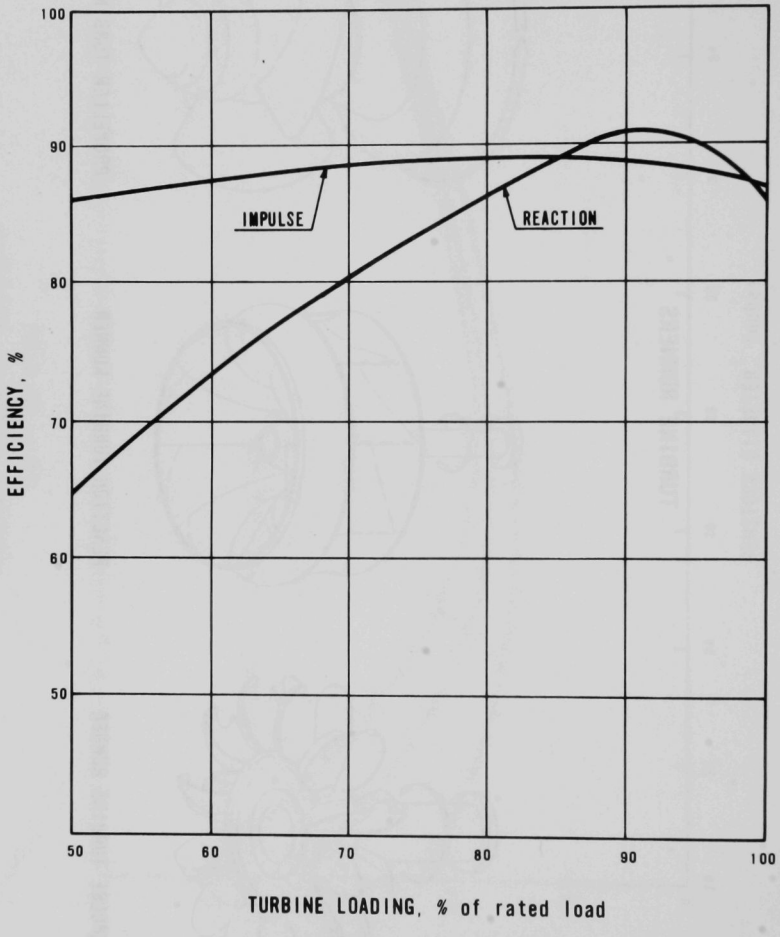
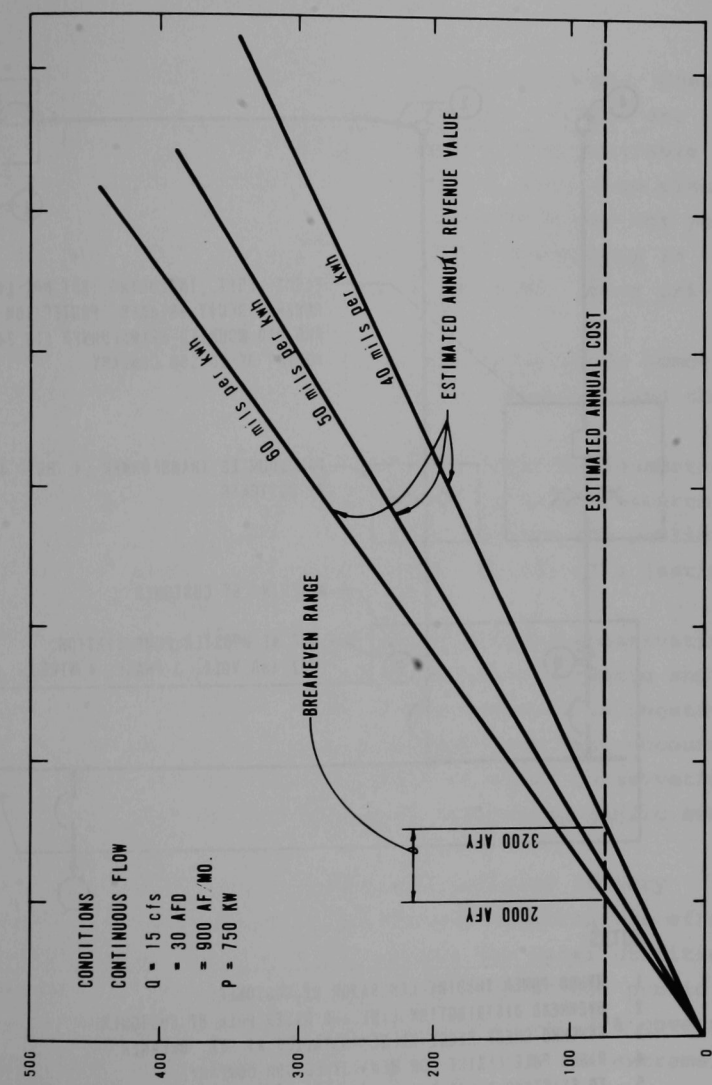
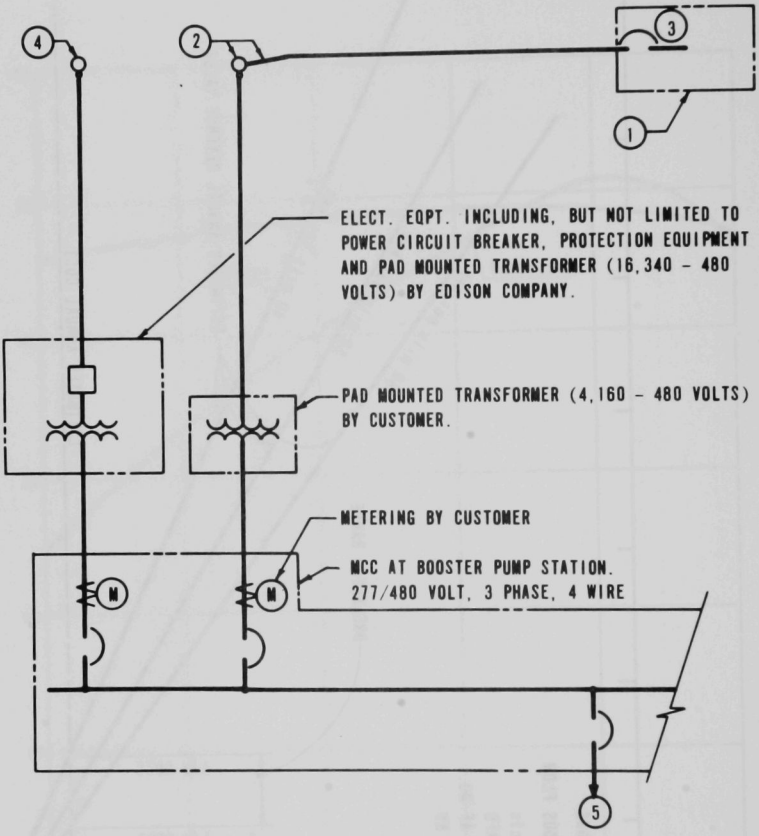


FIGURE 8


COST/REVENUE, $ x 1,000
ANNUAL WATER YIELD, acre feet x 1,000
ESTIMATED ANNUAL REVENUE VALUE
ESTIMATED ANNUAL COST
BREAKEVEN RANGE
2000 AFY
3200 AFY
60 mils per kWh
50 mils per kWh
40 mils per kWh
CONDITIONS
CONTINUOUS FLOW
Q = 15 cfs
= 30 AFD
= 900 AF/MO.
P = 750 KW
0
100
200
300
400
500
2
4
6
8
10
12
14

POWER STATION



NOTES:

1. HYDRO-POWER TURBINE-GENERATOR BY CUSTOMER.
2. OVERHEAD DISTRIBUTION LINE AND RISER POLE BY CUSTOMER.
3. SYNCHRO-CHECK RELAY TO BE INSTALLED AT GEN. BREAKER.
4. RISER POLE (EXIST. OR NEW) BY EDISON COMPANY.
5. TO EXISTING DISTRIBUTION SWITCHBOARD AT WATER TREATMENT PLANT. WILL REPLACE EXISTING U.G. POWER SERVICE.

REV. 12-5-78

PRIORITIES FOR INCREASING EFFICIENCY OF WATER USE

J. B. Gilbert
Vice President
Brown and Caldwell

We are seeing a basic change in public attitudes. Conservation has become a very popular topic indeed. People are beginning to perceive careful use of resources as desirable and beneficial. Support and enthusiasm, however, vary from place to place depending on local factors. Water users are not yet unanimous in their willingness to sacrifice; leadership in business and government is not yet all that clear about priorities for conservation.

But we all tend to think of it as basically good, something we ought to do that would be good for ourselves and the country.

This generally positive attitude will form the foundation for any significant program to husband any valuable resource--water, oil, gas, electricity.....energy. Without the public's understanding, acceptance and cooperation, little of a lasting nature can be accomplished.

The American Water Works Association defines conservation as, "Every effective means to prevent and minimize waste and to promote wise use." We don't think of conservation in negative terms, but rather in the positive "wise-use" way that encourages cooperation. We've distributed millions of water conservation booklets and have created award-winning television public service spots on this basic theme.

While this sort of public relations activity is very broadly based, the fact remains the maximum conservation effect can be achieved by working with relatively few water utilities.

The drinking water industry includes some 60,000 public water systems, of which 63% are investor owned and 37% government owned. The bulk of the investor utilities are extremely small, typically serving less than 25-hundred people. Although more than 80% of the country's water systems fit this less-than-

2500 category, they serve only about 8% of the public.

At the other end of the scale, the largest one per cent of our water systems, which serve at least 50,000 people, handle some 63% of the total population.

So you can see that concentrating conservation efforts among a relatively few systems promises benefits to a very large percentage of the population. Smaller systems shouldn't be ignored, of course, but the big ones are our best bet for rapid, meaningful results.

There are a number of basic programs in our industry that achieve conservation goals by simply emphasizing good management practices. For instance, metering the water provided to customers has several advantages. The most obvious is that customers tend to pay more attention to how much they use when the charge is based on that measured quantity instead of a flat rate. They conserve, because it's in their own best interests.

We estimate about 83% of the government-owned water systems and 73% of the investor-owned systems do use meters.

Accurate metering also helps a community determine how much water fails to reach customers; in other words, how much is being lost somewhere in the distribution system. We call it "unaccounted for" water. This can lead to development of effective leak detection programs that not only save on the cost of drinking water production, but support timely maintenance of the system.

It is not at all unusual for water systems to lose as much as 15% of their total production to leaks and other unaccounted-for losses. In some cities, where the pipelines are extremely old and where maintenance has had low priority, the leak losses may exceed 50%!

The utter waste of energy to treat and distribute water that never reaches its destination is obvious. But funds available to communities remain limited. And drinking water,

so long as it appears at the tap when a consumer wants it, remains a relatively low priority item in most community budgets.

The only time people really become conscious of water is during sustained periods of drought, such as much of the nation endured in 1976 and 77. But the public's willingness to cooperate wholeheartedly when it perceives a real emergency is little short of amazing. For instance, in Marin County just north of San Francisco, citizens cut home water usage by 40 to 60 per cent. They were encouraged by extremely high rate scales, but the accomplishment was largely a case of community self-discipline in the face of overwhelmingly obvious need.

In San Francisco itself, major use reductions were mandated, complete with severe punitive measures for failure to comply; measures that included flow restrictors in service pipelines, billing penalties and even termination of service. The public reaction, unfortunately, exceeded the city's goal of a 25% reduction, which led to a need for higher rates, which led to nationwide publicity on the Johnny Carson show, among other things, about having to pay more for the privilege of using less.

In Denver, where construction of new treatment facilities was delayed for several years, successful measures to reduce peak demand have included public education and moderately severe lawn-sprinkling restrictions. Washington Suburban Sanitary Commission near the nation's capitol has distributed free shower flow-reduction devices and toilet tank dams.

Increasingly, water rates are being modified to affect water use and demand. So-called inverted rate structures charge heavy users more than people who use less water. But naturally, without 100% metering, most creative rate making won't be entirely successful.

Two points should be appreciated. First, energy saving in the home is primarily a factor of having to heat less water.

Toilet tank dams and lawn sprinkling bans don't save energy at home because they don't save hot water. Second, at the production end, the major energy consumption points are treatment and distribution. Let's translate this into dollars and cents.

A few months ago, a survey of Los Angeles and some parts of Nevada indicated that for $100 expenditure in shower flow controls, faucet aerators and hot-water-line insulation, a homeowner could expect to save about $8 a year if he had gas water heating and $22 a year with electric water heating. The actual amount of hot water not used would be some 13 gallons a day per home or about 5,000 gallons a year.

For the water system itself, a projection of typical costs of production indicate about 4.4 cents per thousand gallons was a reasonable energy cost for treatment and distribution in 1976 -- about 15% of total system operating cost. In ten years, 1986, the projection saw energy-related expense rising to 27% of the total, or about 13.2 cents a thousand gallons. And if treatment has to be modified to include such energy-intensive techniques as granular activated carbon filtration, the energy cost could be 30% of the total.

In other words, the cost of water delivered to the customer can be expected to increase 2½ times by 1986, while the energy component of that cost may increase five fold.

What about immediate savings? The St. Louis Water Company estimated last summer that it might reasonably expect to cut its energy requirements through good management by some 20%. However, since the firm uses only about 3% of the energy in the St. Louis area, a 20% reduction would represent a saving of just 6/10ths of 1% of the area's total energy consumption.

Our association is developing a Water Conservation Handbook that will bring together in one place the activities, programs, techniques and practices of effective conservation by water utilities on both an emergency and a long-term basis.

We expect the book to be off the presses by late next year.

The reason for such diversity of approach in the book is the great diversity of water supply situations throughout the U.S. and Canada. For instance, in the arid West and Southwest water is always in short supply or facing the threat of short supply. But in the humid East, around the Great Lakes and along major rivers, water is readily available and can be used at virtually whatever rate the community desires.

It is simply a fact of life that what works in one location won't necessarily work in another. There is no single program or policy that will cover all conditions.

In addition to geographic differences, our new handbook will address conservation variations inherent in different types of water supplies. Water that comes mostly from high ground and can utilize gravity is far more energy efficient, for instance, than water that must be pumped several times from source to treatment to storage to customer. In another case, salt water intrusion that is due, say, to overpumping of a coastal aquifer may lead to energy-intensive desalination systems. The need for pure drinking water outweighs the need to conserve energy in a case like that.

Adding to the diversity of location and source is an immense diversity of customer demand. There was a study by Johns Hopkins University nearly 17 years ago that began to explore the reasons behind different levels of customer demand for water during different times of day, week or year. The goal, of course, was to find ways of matching peak and off-peak power requirements for pumping, treatment and storage with peak consumer demands for water service. Major potential savings in energy and the need for new facilities were clearly indicated.

The study was too limited. But it showed great promise as a significant management tool if its investigation could be extended throughout the nation. The American Water Works

Association Research Foundation has spearheaded an effort to revive and expand the Johns Hopkins Study, but unfortunately with little success. We had set aside $90,000 to support the effort but couldn't find enough other backers. It doesn't have high enough priority yet among water utilities and other organizations in the field, including the federal government.

An area where our Research Foundation has enjoyed good response is its monthly newsletter, "Municipal Wastewater Reuse News." This 24-page publication goes to 39-hundred subscribers who find its reports of activity in the reuse field an invaluable service.

Last July, we gave testimony in Washington in which we tried to make clear our dedication to the principle of water conservation, as well as our conviction that a pragmatic and flexible approach -- not a simplistic search for a cure-all -- was the only sensible way to proceed. We pointed out that measurable energy savings can only be achieved when a combination of practices is implemented; and that all conservation or energy-saving programs do not apply to all water systems. We especially emphasized (as we have here today) the great diversity of the 60,000 public water supply systems extant--their variations in size, age, source, treatment and energy requirements.

We recommended that the federal role, at the outset, be one of defining energy conservation and setting appropriate goals, with the assistance of representatives of the water and power industries. We wanted to avoid the familiar government tendency to launch massive and costly programs without first establishing a sensible target that reasonable people can agree with.

We know from experience that communities will conserve water when they're convinced there's a good sound reason for it. We know companies will practice conservation when they

recognize it's in their own best interests. And we know free-wheeling native ingenuity can often produce solutions where a straightjacket of regulations and requirements becomes counter-productive.

Current events have accomplished the most difficult task, capturing the public's attention and beginning to convince everyone that a real energy problem actually does exist. Close coordination and cooperation on local, state, regional and national levels is the only sensible road to travel now. It's a road people in the water supply industry are firmly committed to.

- 0 -

ENERGY CONSERVATION STUDIES FOR A MUNICIPAL WATER SYSTEM

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The cost for energy has assumed an increasing percentage of a water utility's operating expense in recent years, and it will likely continue to increase in the foreseeable future. Although always important, current trends require greater consideration of energy consumption in a water utility's operation.

In recognition of this situation, the City of Arlington, Texas has taken two steps. A study team was formed in 1978 which consisted of the City of Arlington, Texas A & M University, University of Texas at El Paso, Texas Electric Service Company and Freese and Nichols, Inc., Consulting Engineers. The primary objective of this study was to determine through field measurements and review of records how energy could be saved in the operation of a water utility system. The second step, which was under taken in 1979 by City of Arlington and Freese and Nichols, was to evaluate the potential energy conserving techniques as a part of the planning of water distribution system improvements for the 1980-2000 period. Each of these studies was funded by the State of Texas Governor's Office of Energy Resources under the Innovative Energy Conservation Grant Program and by the City of Arlington.

This energy conservation study concentrated on the Arlington 72-MGD Pierce-Burch Water Purification Plant and Pump Station. These facilities were constructed in four different phases, the earliest in 1957 and the latest in 1973. The plant currently provides service to an estimated population of 160,000. An expansion to 96 MGD is planned in 1980 to match Arlington's anticipated growth.

The evolution of the study on how energy could be saved in the operation of a water utility system suggested that there are a series of logical steps that should be followed to accomplish this objective in any water utility.

These steps are:

1. Determine the quantity of energy being consumed in the operation of the utility and at what cost.
2. Determine how or where the energy is being consumed.
3. Evaluate the physical characteristics of the water utility equipment that have the most significant impact on the energy requirements.
4. Evaluate operational procedures that involve this equipment with particular emphasis on establishing how efficiently the equipment is being utilized.
5. Establish revised operational procedures that are designed to make the most efficient use of the water utility equipment.

The determination of the quantity of energy being consumed in the operation of the water utility and at what cost can be made from a review of the previous utility bills. In the case of Arlington, a total of 15,290,980 KWH of electricity at cost of $274,698.10 were consumed during 1977. During 1978, 16,747,900 KWH of electricity (a 9.53% increase) were used at a cost of $364,077.80 (a 32.54% increase). A typical annual distribution of the electrical consumption and demand are illustrated in Figure 1. Slightly over one half of the total consumption occurs during the five month period June through October. This emphasizes the importance of efficient operation in both the high and low demand periods.

CITY OF ARLINGTON

WATER TREATMENT PLANT 1977 ELECTRICAL POWER CONSUMPTION

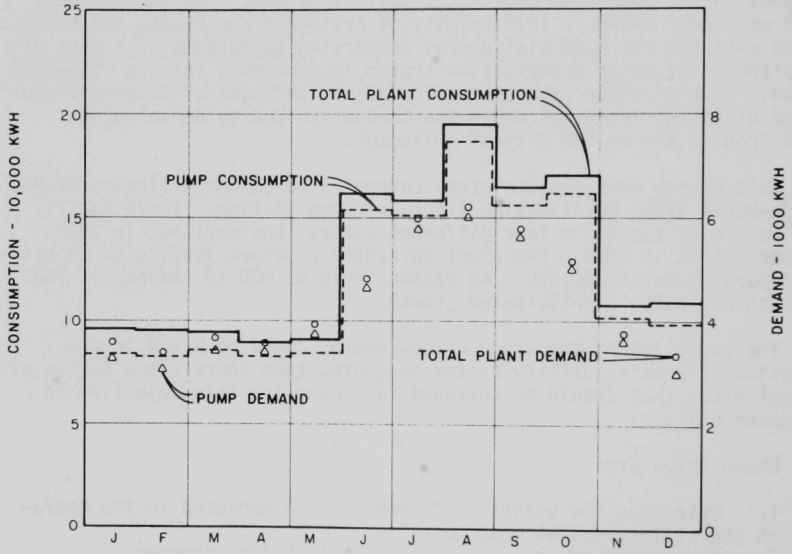

FIGURE 1

How or where the energy consumption is occuring can be answered by a detailed energy audit of the facilities. An energy audit is simply a tabulation of all items that utilize some form of energy with a listing of their significant energy consuming characteristics. The electric power consumed at the Arlington Water Purification Plant and Pump Station is recorded on ten Texas Electric Service Company meters. As the

energy audit was performed, the items were categorized with these meters which permitted a greater refinement of where or how the energy was being used than may be possible in many municipalities. The audit results indicated that, although a small amount of energy may be used for building heating and other secondary operations in the water system, pumping requirements are by far the largest consumer of energy. In Arlington, approximately 68.6% of the energy was used by the high service pumps and approximately 25.0% was used by the raw water pumps, or 93.6% of the total.

Since such a high percentage of the energy required is consumed by the pumping units, it is obvious that the recommended third step, to evaluate the physical characteristic of water utility equipment that have the most significant impact on the energy requirements, should be primarily towards the pumps. Although pump manufacturers often provide curves that describe the head-discharge-efficiency characteristics of a pump, they sometimes change with wear, and actual field operating conditions may not correspond to the design conditions. The Arlington pumps were tested in the field to determine whether the present head-discharge relationships remained in general agreement with the original manufacturer's curves when operating under actual field conditions.

Tests performed in the field are rarely as accurate as those that can be obtained under controlled laboratory conditions. A method of testing was devised which permitted the measurement of the operating head, the rate of flow, and the amount of power consumption with a reasonable degree of accuracy. The discharge operating head was measured with a pressure gage mounted on the discharge of each pump. The suction level was based on the water surface level in the clearwell. The operating head can be varied by manipulation of a valve downstream from the pump or by operating other pumps which discharge into the same header. It is important to vary the head over as wide a range as possible in order to obtain a reliable comparison with the original manufacturer's head-discharge-efficiency curves. The flow rate was determined by measuring the level change in a clearwell of known dimensions over a carefully noted time span during which the pump was operating against a reasonable constant discharge pressure. The elapsed time of the test should be sufficient to minimize the effect of any error in the determination of the change in volume. The electrical power consumed during the test can be measured using the standard power meter already installed by the electric service company. The rotating disc within the meter moves at a speed proportional to the power demand. The time required for the disc to complete an appropriate number of revolutions, usually five or ten, can be measured. This method of testing pumps is fairly inexpensive and requires no special equipment, yet the results obtained can give a valuable indication of each pump's performance. Although these methods used may not be possible in all systems, techniques can be developed to provide the necessary data in almost any system.

Using this method, field tests of head-discharge-energy requirements were performed on the Arlington eleven high service pumps and three raw water pumps. The results of the tests indicated that most of the pumps were still performing in accordance with the manufacturer's original pump curves. The tests did indicate that two of the older pumps were apparently no longer providing the flows indicated by the original manufacturer's curves. Of even more importance, the tests results indicated that the modern-day system curve did not allow the pumps to operate within their design head range during much of the year.

Operating conditions which differ significantly from the design criteria force pumping units to operate less efficiently than intended. The basic operational approach to the conservation of energy is through proper use of the pumping units. The fourth step in the determination of how energy can be saved is a thorough evaluation of the current pump operational procedures.

An evaluation of the water system operating procedures to determine modifications to the procedures or improvements to the physical system that will result in energy consumption reductions requires a detailed recording of the operational actions. A form was developed to record the daily operations. It contained entries for all critical items such as the raw water pumps, clearwells, high service pumps, high service discharge pressures, flow rates, elevated storage levels, and booster pump operations. The system operators were instructed to make an entry in every category each time a pump was started or stopped and on each hour. Although it does represent a considerable amount of effort to complete a daily form, the City of Arlington found it to be so beneficial to their operation that they continued the use of the form beyond the data collection period.

Data on the operating procedures should be collected over as wide a range of flow conditions as possible. In Arlington, data were collected for the months of April through July which included both low and high demand conditions. An in-depth evaluation of a system's operation can provide a very valuable understanding of a water supply system.

Before suggesting some general guidelines for energy efficient pumping techniques which can be used to evaluate the current operating procedures and as possible revisions for future operations, it is important to note that the prime responsibility of any water utility operator is to strive to provide adequate service to all customers. To satisfy this responsibility, often requires less than the most energy efficient operating procedures. This situation can result from equipment characteristics and system limitations; however, as opportunities arise, modifications can be made that will permit more energy efficient operating procedures while satisfying the prime responsibility of adequate service.

General guidelines for the most efficient pumping operations for energy conservation which are frequently suggested are:

1. Use the pump stations that operate against the lowest total heads, and deliver water to consumers by pumping the fewest times possible.
2. Anticipate the required daily pumpage and make efforts to meet demands with constant-rate pumpage combined with flow to and from storage.
3. Use the most efficient combination of pumps available at a given station to provide the required flow.
4. Ensure that all valves are completely open during pumping operations.
5. Avoid throttling or bleeding flow between pressure districts within the system.

The application of some of these general guidelines can be illustrated with examples from the City of Arlington evaluation. The first is the anticipation of required daily pumpage and efforts to meet demands with constant-rate pumpage combined with flow to and from storage. An average usage day (April 25, 1977) and a higher demand day (June 21, 1977) were selected to compare pumping rates with the hourly demands. These comparisons are shown graphically in Figures 2 and 3. In both cases, the pumping rates generally follow the demand pattern. On the average usage day, the pumping rates actually exceeded the demand during the higher use period. This action means that the elevated storage tanks were being filled rather than being utilized to level out the pumping. On the higher demand day, the elevated storage tanks were used during the highest demand period, but the full benefit of the storage was not realized and constant rate pumping was not achieved.

The second guideline that can be illustrated is the use of the most efficient combination of pumps available at a given station to provide the required flow. An evaluation of the pump operations for April 25, 1978, the average day, indicated that the selected pumps were operating in a fairly inefficient range during the early morning hours. The efficiencies attain a more desirable level in the evening. For the available equipment, the best selection of pumps may have been made.

As an indication of what might be achieved with the proper selection of pumps, consider this example. If all the pump efficiencies that were less than 80% for April 25, 1978, were increased to at least 80%, a 3.4% savings in energy could be obtained. If this degree of reduction had been possible during 1977, approximately a $9,300 (1977 dollars) savings would have been realized. At this rate, a more efficient pump for the lower demand rates would pay back in only a few years. Modification of existing equipment would pay back even sooner. The results of this study indicate that with a thorough understanding how energy is being consumed in a municipal water supply system, a savings of 3 to 5% should be possible with existing facilities through more effective operating procedures.

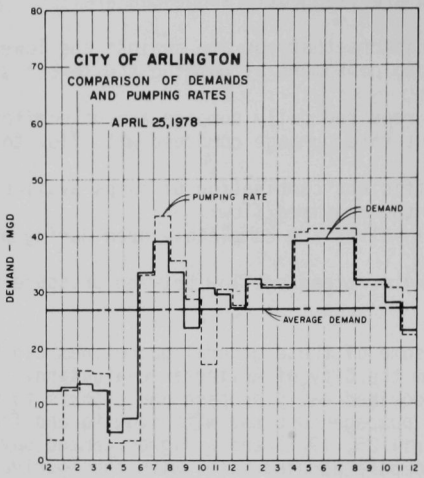

FIGURE 2

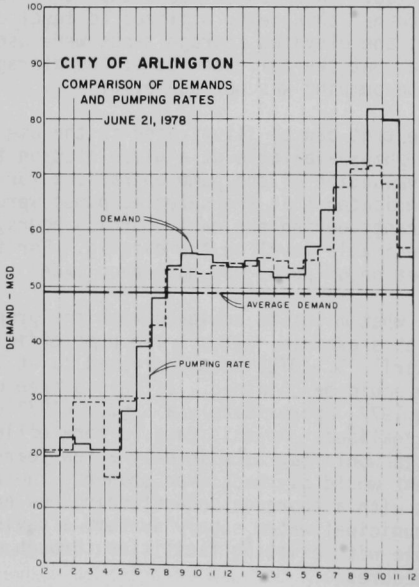

FIGURE 3

Arlington is expected to grow at a very high rate during the next twenty years. A population of 368,654 is projected for the year 2000. Numerous improvements to the water distribution system will be required to keep pace, and this situation provides an excellent opportunity to achieve even greater savings of energy through the selection of equipment that will provide energy efficient operations under both low and high demand conditions.

Studies were undertaken with the objective of determining the combination of facilities that would result in the minimal total cost through the year 2000. The facilities included in this evaluation were the pipelines, high service pumps and the elevated and ground storage tanks. The size or capacity of the required facilities were determined with a mathematical computer model of the water distribution system.

The size of each facility is related to some extent to the adopted operating rules. To include a range of operating procedures, three combinations of pumping and contribution from elevated storage to supply the maximum-hour demand were considered, 80% - 20%, 70% - 30%, and 60% - 40%. For each combination, maximum-hour, nighttime on peak-day, peak-day and average-day demand rates were evaluated to provide a range of flow conditions.

The topography of Arlington requires the separation of the water distribution system into two pressure planes. Identical procedures were employed for each pressure plane to provide some comparison of the impact the size of the system and the system configuration has on the total cost. The lower pressure plane maximum-hour demand is projected to increase from 110.81 MGD in 1980 to 178.94 MGD in 2000. During the identical time period, the upper pressure plane maximum-hour demand is projected to increase from 24.62 to 104.56 MGD. The larger, lower pressure plane has a single principal source of supply while the smaller, upper pressure plane has two sources of supply.

For each operating procedure, the system model was exercised and modified until an acceptable solution based on level of service criteria was obtained. The cost for energy and capital improvements were then computed. The cost for energy was based on a representative annual demand pattern and the derived water distribution system network system curve. The cost for capital improvements were based on current unit costs. All of the costs were expressed in present-day dollars by using a ten percent inflation rate and a 6.5% cost for money.

The results of these detailed evaluations indicated that the water system with 80% pumping and 20% contribution from elevated storage during the maximum hour had the least combined cost for both pressure planes. The 70% pumping - 30% elevated storage combination was only slightly higher while the 60% - 40% combination was significantly higher. The costs per 1000 gallons over the twenty-year period are illustrated in Figure 4. The small difference between the 80% - 20% and the 70% - 30% combination is considered important because it provides a great amount of flexibility in the operation of the system without a significant change in the combined costs.

ESTIMATED AVERAGE COST PER 1000 GALLONS
DURING 1980 - 2000 PERIOD FOR CAPITAL COST
AND ENERGY COST EXPRESSED IN 1980 DOLLARS

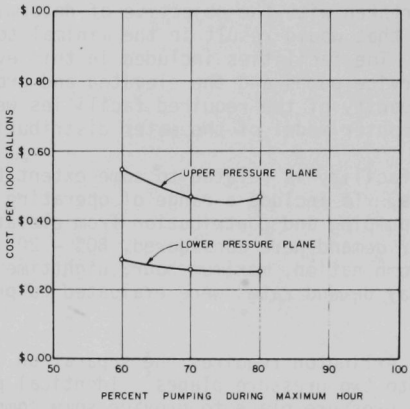

FIGURE 4

In Arlington over the next twenty years, the cost for energy will represent approximately 12.9% of the combined cost of the lower pressure plane and approximately 6.1% of the combined cost of the upper pressure plane. The lower percentage for the upper pressure plane results from a lower pumping head and from the tremendous capital expenditures required to statisfy a developing area. In this case even though energy costs are projected to increase at a significant rate, capital improvements to the water system will represent the preponderance of the water utilities expenditures. This relationship does not diminish the importance of minimizing the total cost through effective energy conservation.

The results of this study lead to the following conclusions:

1. A thorough understanding of a water utility system's energy consuming the equipment and its use are essential to energy conservation.
2. Savings of 3 to 5% in the energy requirements through more effective operating procedures with the existing facilities can be achieved.
3. Designing a water distribution system to serve a developing community with from 80% pumping - 20% contribution from elevated storage during maximum-hour to 70% pumping - 30% contribution from elevated storage during maximum-hour will result in the least combined construction and energy cost over the next twenty years.

4. Energy cost during the next twenty years for a developing system such as Arlington will represent approximately 5 to 15% of the combined cost, but this relationship does not diminish the importance of minimizing the total cost through effective energy conservation.

ENERGY CONSERVATION WITH WATER FLOW CONTROLS

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Institute for Research on Land and Water Resources
The Pennsylvania State University

Flow controls are part of a growing effort to reduce water and energy use. The potential of flow controls to reduce residential energy use is outlined in a paper by Muller (1976) and the recent congressional testimony of Hartsell (1979). Hartsell points out that a significant contribution to energy self-sufficiency can be achieved through the use of these devices.

However, there has been a considerable amount of disagreement over the amount of water and energy that can be saved in this manner. Cohen and Wallman (1974), Baker (1976), and Bishop (1975) have all reported little or no savings as a result of using shower flow restrictors. In fact, Bishop reported water-use increases for an apartment building in a study conducted by the Washington Suburban Sanitary Commission.

Many are as yet unconvinced that shower flow controls are effective in reducing shower-water use. Detractors argue that restrictions in shower flow rates will be compensated for by showers of longer duration and that average flow rates are presently so low as to allow for little savings. The aforementioned studies tend to support their point of view. Unfortunately, all of these studies are open to question because either the sample tested was small or the study was not adequately controlled. More recent studies have shown shower flow controls to be effective water and energy conservation tools. Schatzberg et al. (1975) found that shower duration was little affected by a change from a 4.2 gpm conventional shower to a 0.5 gpm air assisted shower. In Schatzberg's study, a mean shower time of 2.8 minutes for the conventional shower was increased to 3.2 minutes for the air assisted shower, an increase of only 14 percent. Sharpe (1978) reporting on a carefully controlled study in two dormitories at The Pennsylvania State University showed significant water use reduction as a consequence of installing shower flow controls. A summary of Sharpe's data appears in Table 1. As the data presented in Table 1 indicate, substantial reductions in water use accompanied the installation of shower flow controls.

A recently completed study at Penn State (Sharpe, 1979) confirms these earlier findings. In this study six showers in a recreational camping facility were alternately equipped with 2 gpm and conventional showerheads on a weekly basis for nine months. Cold water use for these periods with and without the 2 gpm flow limited showerheads is given in Table 2.

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Table 1. Summary of Cold Water Use Data

Location	Average Daily Water Use		
	Period 1	Period 2*	Period 3
	gal	gal	gal
Snyder-2nd floor	109.4	248.2	95.7
Snyder-3rd floor	155.8	250.4	158.5
Stuart-2nd floor*	282.1	246.9	260.9

* No flow controls.

Table 2. Cold Water Use Data for Men's Showers

Use Period	Total Use
	(gal)
Conventional	
3/27 - 3/31	163.5
4/10 - 4/14	376.4
4/26 - 4/28	282.0
5/8 - 5/12	568.5
5/23 - 5/26	392.0
6/5 - 6/9	66.7
7/10 - 7/14	1417.9
7/24 - 7/28	1882.4
10/30 - 11/3	319.8
11/6 - 11/9	367.0
Mean	583.6
Flow Limited	
4/3 - 4/7	93.0
4/17 - 4/21	130.3
5/1 - 5/5	120.9
5/15 - 5/19	70.3
5/30 - 6/22	572.3
6/27 - 6/30	232.2
7/3 - 7/7	538.1
7/17 - 7/21	1059.8
9/18 - 9/22	396.7
10/2 - 10/6	140.9
10/16 - 10/20	382.2
10/23 - 10/27	174.1
Mean	322.1

Mean weekly water use with conventional showerheads was 583.6 gallons, while that for the 2 gpm flow limited showerheads was 322.1 gallons. On average, the conventional showerheads used 81 percent more water than the flow limited showerheads. Camp occupancy varied from week to week but the mean occupancy was essentially equal for both test conditions.

Shower duration was measured indirectly by means of a thermocouple attached to each showerhead and connected to a strip-chart recorder. The temperature change at the showerhead caused by turning the shower on and off was recorded. By measuring the elapsed time between temperature changes an estimate of shower duration was obtained. Shower water temperature was also recorded. Although the measurement method was admittedly crude, relative differences between the two types of showerheads under evaluation are valid. Table 3 contains a summary of shower duration and temperature data.

Table 3. Summary of Shower Duration and Temperature Data in Penn State Environmental Learning Center Study.

	n	Length (+ 0.6 min.)	n	Temperature[1/] (F°)
Conventional				
Mean		6.4		99.5°F
Range	203	(2-23)	207	(90°-110°F)
Flow Limited				
Mean		6.6		102.2°F
Range	510	(2-33)	304	(90°-110°F)

[1/] Events with recorded temperatures <90°F and >110°F not included.

As the data in Table 3 indicate, the difference in mean shower duration for the two types of showerheads is insignificant. Average shower water temperature was slightly higher for the flow limited showerhead but the difference was also not significant. The results of these studies do not support the hypotheses that shower duration will increase when shower flow rates are limited to a maximum of only 2 gpm. Further, these and other studies plus information gained from the experiences of an increasingly large number of users indicate that flow controls can effectively reduce the amounts of water and energy used for showering.

There is no doubt that in institutional and commercial settings proper use of shower flow controls will result in substantially reduced water and energy use. The same can be said for single family residences where showers are the principal form of bathing.

Reliable estimates of the reduction in water use for flow limited showers can be made using the formula:

$$Q_c = Q_m(C_c) \quad (1)$$

where: Q_c = water use with flow controls

Q_m = water use without flow controls

C_c = conservation coefficient

C_c can be obtained by:

$$C_c = \frac{F_m - F_c}{F_m} \quad (2)$$

where: F_m = flow rate without controls

F_c = flow rate with the control in place

Estimates obtained using equations 1 and 2 will be valid as long as the control remains in place. If the control enjoys user acceptance, this condition will be met. Our research and the rapidly growing experiences of the many users of these devices indicate that showerheads limited to maximum flow rates of 2 gpm can meet these conditions.

Water conservation programs involving the retrofit of shower flow controls and restrictors in existing dwellings have been conducted at numerous locations around the country. In almost all cases the sponsoring utility has purchased the devices and distributed them at no cost to their customers. Because of the large aggregate cost of the devices, most utilities have selected shower flow restrictors of low unit cost for mass distribution. One notable exception is Hamilton Township, New Jersey, (Horn 1978). Most of the restrictors used have been designed to limit flows to a maximum of 3.0 gpm (gallons per minute) at a water pressure of approximately 50 psi (pounds per square inch).

Limited physical testing of shower flow devices has been undertaken by several agencies with extensive recent tests by the California Department of Water Resources (1978). Additional test results involving subjective evaluations by a selected group of shower users has also appeared in Consumer Reports (1978).

Most of these testing programs have placed heavy emphasis on the flow rate characteristics of the shower device at either a single water pressure or at several pressures simulating the range of service pressures experienced in water supply systems. Additional criteria such as ease of cleaning, type of construction, appearance of the device and cost have also been applied to the shower device selection process. In some cases a subjective

evaluation of the quality of the shower has also been made by soliciting the opinions of users.

Flow rate comparisons are of limited value in judging the suitability of shower devices because they bear little relation to the quality of shower received by users. In general the device that most closely approximates the selected flow rate (usually 3 gpm) for the water pressures used in the test is judged best. However, if the shower is as satisfactory at 1.5 gpm and 30 psi as it is at 2.5 gpm and 80 psi the difference in flow rate matters little to the user. In fact such a difference may be desirable in that it does allow for some variation in water flow to accommodate the preferences of different users. Such variation may be obtained by the user by opening cold and hot water valves to allow for greater water flow.

Another criterion that has been used to judge shower devices is that of spray adjustment. Some evaluators have felt that the showerhead should be adjustable from needle to gentle spray with the device in place and that both of these sprays should be satisfactory. Whether or not an adjustable spray is really important to the average user is a matter of conjecture.

Cost is a criterion used to judge shower devices that has often been misapplied. Certainly, devices that meet all other established criteria should be differentiated on the basis of cost, but all too often cost is weighted too heavily in comparison with other criteria. The result has been selection of cheaper devices that involve greater risk of poor user acceptance, or devices that are not effective conservers of water. Device costs are only one factor in the cost decision. Distribution costs, costs of discarded or unused devices, and the cost savings resulting from the use of the device must also be considered. In the long run it may be more economical to select a device with a much higher initial cost that will be more acceptable to the user.

Determining the maximum flow rate of the shower devices to be selected is also an area of potential difficulty. National codes have suggested maximums of 3.0 gpm. The American National Standards Institute standard is 2.75 $\pm$ .25 gpm which is essentially a 3.0 gpm standard. Most of the really inexpensive devices are engineered to limit flows to 3 gpm at a pressure of around 50 psi. With most cheap restrictors, maximum flow rates below 3 gpm would cause a higher risk of producing an unacceptable shower, especially at lower water pressures. Unfortunately, maximum savings of water cannot be achieved at flow rates around 3 gpm and savings resulting from the use of such devices may be disappointingly modest. Showerheads are available that give an acceptable shower at 2 gpm with a much greater actual savings of water; consequently, the 2 gpm showerhead should be given much stronger consideration than it has in the past, irrespective of its higher cost.

The most important criteria for selecting a shower device in order of importance are user acceptability, compatibility with existing installation, durability, amount of water saved, and lastly, cost. The 2 gpm, all metal showerhead with the necessary ball joint adapters meets these criteria. Follow-up surveys of user acceptance of mass distributed

shower devices show a user installation rate of 40 percent for a less costly plastic showerhead with built-in restrictor, Rodgers (1976) and 65 percent for a 2 gpm showerhead, Horn (1978).

Until more information becomes available on user acceptance of 2.0 gpm devices, code requirements for water saving showerheads in new construction should specify a maximum flow rate of 2.75 gpm as per ANSI A112.18.1-1978.

Flow controls have also been applied to faucets with impressive results. Important work on water conservation in lavatory sinks was conducted in the United Kingdom during the 1950's, Crisp and Sobolev (1957). This work resulted in the development of a single spray tap that delivered water of a preselected temperature to the user. Tests of this faucet showed that it was actually preferred by users. Amazingly, the use of this faucet reduced hot water use in an office lavatory by 86 percent.

In the 1979 Penn State study, lavatory sink faucet flows were limited to a maximum of 1 gpm. Comparative data for shower hot water use combined with total lavatory sink use are given in Table 4.

As the data in Table 4 indicate a substantial reduction in water use was obtained by limiting faucet flows to 1 gpm in conjunction with the 2 gpm flow limited showerheads. Uncontrolled faucets and showers used 106 percent more water. Water use reductions resulting from the use of faucet-flow controls in residences will probably be less spectacular, but nevertheless highly worthwhile.

Most mass retrofit programs have ignored faucet water use because of the relatively low potential savings. Flow rates from faucets other than the tub filler should be limited to a maximum of 1 gpm. Most recent code revisions stipulate maximums of 2.5 gpm. If shutoff valves are provided for the hot and cold water service lines to the fixture, they may be partially closed to allow a flow of 0.5 gpm from the hot and cold water faucets. Adjustment can be made by timing the filling rate of a vessel of known volume. If a single, center-set faucet is in use, the cold and hot water lines should be set to deliver a maximum flow of 0.5 gpm. However, in the kitchen sink, such a low maximum flow rate may be undesirable.

Various inexpensive and easily installed faucet aerator-flow control devices are available to fit threaded faucet spouts. Care should be exercised in the purchase of such devices because faucet thread sizes and diameters vary considerably. These devices are easily installed.

Based on the information presented, the following recommendations appear to be justified.

Retrofit Programs

For programs involving the mass distribution of water conservation devices the following are recommended:

1) Showerheads producing a flow rate of approximately 2.0 gpm at or

Table 4. Hot water use for showers and total lavatory sink use in 1979 Penn State Study.

Use period	Shower and sink use (gal.)	
	(Total for Period)	(Daily Average)
Conventional		
3/27 - 3/31	878	220
4/10 - 4/14	1,576	394
4/26 - 4/28	1,085	542
5/8 - 5/12	2,247	562
5/23 - 5/26	1,761	587
6/5 - 6/9	117	29
7/10 - 7/14	2,222	556
7/24 - 4/28	5,311	1,328
10/30 - 11/3	745	186
11/6 - 11/9	945	315
Mean	1,689	472
Flow limited		
4/3 - 4/7	1,346	337
4/17 - 4/21	816	204
5/1 - 5/5	746	186
5/15 - 5/19	545	136
5/30 - 6/2	1,282	427
6/27 - 6/30	634	211
7/3 - 7/7	1,126	282
7/17 - 7/21	916	229
9/18 - 9/22	835	209
10/2 - 10/6	461	115
10/16 - 10/20	685	171
10/23 - 10/27	439	110
Mean	819	218

near 50 psi service pressure with the required ball joint adaptors if necessary.

2) Flow control faucet aerators limiting flows from bathroom and lavatory sinks to a maximum of 0.5 gpm.

Newfit Programs

For programs involving the regulation of new or replacement construction the following is recommended:

1) An ordinance or code change requiring water saving toilets (3.5 gallons per flush) at a specified pressure in all new or replacement construction where tank type toilets are to be used.

2) Water saving showerheads designed to operate with a maximum flow rate of 2.75 $\pm$ .25 gpm (ANSI A112.18.1-1978).

3) Bathroom and utility sinks (other than kitchen) with faucet aerator-flow controls limiting the flow to a maximum of 0.5 gpm.

4) Institutional (gas stations, bars, restaurants, hospitals, dormitories, etc.) lavatories should be equipped with flow limited (as per item 3), spray type faucets or taps.

5) Pressure reducing valves where normal water service pressure will exceed 60 psi.

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ENERGY OPTIMIZATION IN GROUNDWATER DEVELOPMENT AND PRODUCTION

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INTRODUCTION

It has been estimated that ninety seven percent (97%) of the fluid fresh water on our planet Earth - on the order of 9.9×10^9 cubic hectometres (8×10^{12} acre feet) - is underground.[1/] The total groundwater withdrawals in the United States are estimated to exceed 95×10^3 cubic hectometres (77×10^6 acre feet) annually. This is a relative contribution to total water use in the U.S. of on the order of 17%.[2/] A large percentage of groundwater withdrawals must be lifted to ground surface or higher for distribution and application. This requires large amounts of energy.

Groundwater can be found in a variety of geologic formations or aquifers, such as limestone, volcanic rocks, glacial, alluvial and eolian deposits, and is extracted from wells, springs and horizontal infiltration galleries. Unconsolidated deposits of sand and gravel are considered to provide more than 90% of all groundwater pumped in the conterminous United States.[3/] This paper considers energy optimization in the development and production of groundwater from wells in sand and gravel aquifers.

Energy optimization in groundwater development and production is concerned with minimizing pumping lifts and increasing pumping efficiencies. The potential for energy savings is significant. For example, if annual groundwater withdrawals in the U.S. were lifted an average of 300 mm (1 foot) less, the annual energy savings would be on the order of 129×10^6 kilowatt hours (KWH). This assumes an average pumping efficiency of 60%. Further, if overall pumping efficiencies were increased 1%, approximately 2.1×10^5 KWH could be saved annually for each 300 mm the water is lifted.

Minimization of drawdown during pumping begins with proper well design and construction. Little can be done to influence the piezometric elevation at which groundwater is encountered or the aquifer characteristics that determine its ability to yield water. However, competent design, proper drilling, construction and development of wells can reduce the pumping lift re-

quired to withdraw groundwater from the well.

Properly designed and maintained well pumping equipment complete the energy optimization equation. Since almost limitless opportunities exist in selection of pumps and prime movers for groundwater production, it is essential they be matched to the well characteristics to provide an efficient operating unit. In addition to a proper maintenance program, the performance of the well pumping equipment should be monitored to detect changes which indicate reduced efficiency.

A more detailed discussion of factors to consider in energy optimization in groundwater development and production follows.

DRAWDOWN IN WELLS

The drawdown in a well producing water from an unconsolidated sand and gravel aquifer is the result of viscous friction forces as water flows through the aquifer toward the well and enters the well through the openings in the well casing. Two flow regimes are generally encountered in wells; laminar flow in the aquifer surrounding the well and turbulent flow through the openings in the well casing or well screen and the filter or aquifer material adjacent to the screen. The transition point from laminar to turbulent flow will vary with well construction, water temperature and velocity.

Drawdown in a well is frequently characterized by the formula:

$$S = C_1 Q + C_2 Q^n$$

where S = drawdown Q = production rate

C_1 and C_2 are constants known as the formation and well constant, respectively. The exponent n usually falls in the range of 2 to 3.

Though this formula is not universally accepted, it is useful in discussing the components of drawdown in pumped wells. For example, the drawdown in the aquifer varies linearly with well production whereas that in the well entry zone varies exponentially with production. As production is increased from a given well, the well loss, C_2Q^n, becomes the dominant component of drawdown. Nonetheless, both components are important and merit an effort to minimize through proper design and construction of water wells.

FORMATION LOSS

The expression C_1Q, the formation loss, is the first area in well design

and construction which is subject to management. The formation constant, C_1, is affected by several factors, such as:

1. Hydrogeologic characteristics of the aquifer.
2. Depth of well penetration in the aquifer.
3. Well diameter.
4. Development of the well.
5. Location of well relative to other wells.
6. Groundwater basin hydrology.

The transmissivity and storage coefficient of the aquifer; whether the aquifer is confined, semi-confined, or unconfined are established hydrogeologic characteristics that are relatively unchangeable. Within the limits of these characteristics, it is possible to lessen drawdown associated with the formation by fully penetrating the aquifer with the well. Plate I illustrates the relationship of partial well penetration and attainable specific capacity for wells in homogeneous confined aquifers.[1/]

If the general hydrogeological characteristics of the area are known before the well is drilled, the additional cost of energy to overcome the greater drawdown due to partial penetration can be estimated and compared in a present worth evaluation with the cost of greater depth of the well. This will enable the designer to estimate the most economical depth for the well.

Well diameter also affects drawdown. The following table shows the relation between well diameter and yield for an unconfined aquifer.[1/]

Well Diameter vs Yield Ratio, in %

6"	12"	18"	24"	30"	36"	48"
100	110	117	122	127	131	137
-	100	106	111	116	119	125
-	-	100	104	108	112	117
-	-	-	100	104	107	112
-	-	-	-	100	103	108
-	-	-	-	-	100	105

It can be seen the specific yield of a 12-inch well can be increased by 11% by doubling its size to 24-inch. In the case of a 12-inch well producing 500 gallons per minute with a drawdown of 10 feet, the drawdown would be 9 feet in a 24-inch well. The economic feasibility of increasing well diame-

ter to save energy requires high energy costs.

Well development is probably the most important factor to minimize drawdown related to formation loss. When a well is drilled, the earth materials that are penetrated by the bore hole are disturbed. Fines generated during drilling or drilling mud may invade the aquifer due to the fluid pressure in the bore hole exceeding that in the aquifer in order to control caving of the hole, and carry drill cuttings out of the hole. There are greater hazards in conventional mud rotary drilling than reverse rotary or cable tool drilling; however, a well engineered mud program will minimize mud penetration in most aquifers. Additionally, some aquifers have an element of fines that create high velocities in the aquifer surrounding the well with attendant drawdown. One of the aims of well development is to open up the aquifer by drawing this fine material into the well where it can be removed. Three beneficial results are brought about during well development:

1. Correction of damage or clogging of the aquifer which occurs during drilling.
2. Increases in porosity and permeability of the aquifer in the vicinity of the well.
3. Stabilization of the sand/gravel formation around a screened well forming a filter to keep sand from entering the well.

Methods of well development will be discussed below, however it is emphasized that complete development of a new well may take several days, even weeks. If proper techniques are not used, the well may never be completely developed. Development is usually considered complete when the specific yield of the well (gallons per minute per foot of drawdown or similar unit) does not change with continued development or pumping.

The location of the well relative to other wells must be considered because of the possibility of interference among wells increasing drawdown. This occurs when the cone of water level depression around a pumped well spreads out to intersect a nearby well. Interference can be predicted from aquifer transmissivity values determined during well tests or by plotting simultaneous drawdown measurements in observation wells against distance of the observation wells from the pumped well. This plot is made on semi-log paper with distance plotted on the log scale. The zero drawdown intercept of the resulting sloping line approximates the limit of the cone of depres-

sion. When estimating well interference in this manner, the pumped well should be operated at anticipated production rate for a sufficient length of time such that plots are reasonably straight lines and parallel when plotted for successive time intervals. Allowance should be made for further spread of the cone of depression as pumping continues over a long term.

The following equation develops the optimum well spacing of two production wells pumping at the same rate from a thick, areally extensive aquifer.[4/]

$$r_s = \frac{2.4 \times 10^8 \ Cp \ Q^2}{KT}$$

r_s = optimum production well spacing, in feet.
Cp = Cost to raise one gallon of water one foot. (Power, operation, maintenance and cost of capital.)
K = Capitalized cost of operation, maintenance and capital for the transmission pipeline connecting the wells.
Q = Pumping rate of each well, in gallons/minute.
T = Coefficient of transmissivity, in gallons/day-ft.

Groundwater basin hydrology affects drawdown in wells to the extent that there is sufficient recharge to overcome the effects of long-term pumping or not. This factor can sometimes be altered by artificial recharge and groundwater replenishment programs. Wells should be located close as practicable to the source of recharge and far as possible from underground barriers to the flow of groundwater.

WELL LOSS

The expression for well loss, C_2Q^n, is influenced largely by the construction of the well and its development. Narrow or relatively impermeable aquifers may also contribute to this loss where groundwater migrating to the well reaches turbulent velocities.

Well construction practice varies throughout the world. In many cases, local practice is the result of years of experience with regional groundwater formations. All wells have certain elements in common, however. In sand and gravel aquifers, the most important is the opening through which the water enters the well. The water inlet may consist of rough slots pierced in the well casing after it is in place in the well; mill slotted casing; pre-formed louvered or bridge slot casing or wire wound screen.

There are several important factors to consider in the selection of a water inlet in order to minimize well loss and the attendant drawdown. They are:

1. Corrosion resistance and strength of material.
2. Ability to control the sand/gravel around the well to form an effective, permeable filter.
3. Maximum open area per unit length of inlet.

Resistance to corrosion is important because the buildup of corrosion products can reduce the open area of the inlet and increase the drawdown in the well. Inlets should have sufficient strength to resist deformation during handling, well development and production. The ability of an inlet to control the sand/gravel around the well is a function of its design and slot size as related to the natural formation or gravel pack, if used. In coarse gravel aquifers, particle sizes are often large enough to bridge over most slots; however, if fine sands are encountered it may be necessary to gravel pack the well to create an artificial filter. In this instance, the ratio of the median particle size in the gravel pack to that of the formation should be about 5 to 6 in order to maintain a high permeability in the gravel pack, yet effectively filter out the formation sand. Screen slot size is selected to retain 90% of the gravel pack.

In any screened well, it follows that some portion of the screen openings are occluded by formation or gravel pack particles. This can be mitigated by selecting well rounded gravel pack material. However, the greater the area occluded, the greater the drawdown in the well at a given flow. The use of well screen with maximum open area per unit length, coupled with low water velocity through screen openings, should minimize this condition. Maximum design velocity through the well screen openings is recommended not to exceed 0.1 feet per second.

Typical screen types with open areas are shown below:

TYPICAL SCREEN OPEN AREAS

Screen	Percent Open Area
Bridge Slot	2%-13%
Louvered	2%- 9%
Wire Wound Screen	28%-57%

Note: Based on 12-inch I.D. screen and average slot opening, one tenth inch.

Another important consideration in selecting well screen is the mode of development intended. If the well is in a formation little affected by the drilling method and requiring minimal development, the screen plays a lesser role. Well development can be accomplished by pumping the well at various rates, stopping the pump frequently and allowing the water in the pump column to surge back through the screen.

When formations have been penetrated by drilling mud or contain fines which must be removed to provide optimum well yield, more force must be exerted during development. Methods such as surging and pumping with an air-lift, surging with a surge plunger, and high velocity jetting may be used. In unusually difficult circumstances, chemical aids, such as detergents or acids may be employed. Whatever the method, it can be postulated that screens with the largest percentage of open area will allow the greatest amount of development energy to be transmitted into the formation and gravel pack around the well.

PUMPING EFFICIENCY

Vertical turbines are one of the most common type of pump encountered in groundwater production, therefore are the sole type discussed. Various elements of the following discussion apply to all types of pumps, however.

There is a truly large variety of vertical turbines from which to select. This also applies to prime movers, thus the possible combinations are legion. In addition to flow desired and pumping lift, the selection of a pump requires a knowledge of hydrogeology of the aquifers exploited, well construction and efficiency to the extent these factors will effect drawdown. Ideally, flow and total pumping lift should be constants allowing the pump to be designed for its most efficient point of operation. This rarely occurs however, due to changes in drawdown or in pump discharge pressure.

Plate II shows a typical family of head-capacity curves for a vertical turbine. It can be seen that pump output increases as total pumping head decreases and vice-versa. There is, however, only one most efficient point of operation and departures from this point carry penalties in energy consumption. When designing the pump and its prime mover, the factors which will cause changes in the head-capacity relationship must be considered and a selection made that will yield the greatest overall efficiency over the anticipated operating range. Accomplishing this is more direct for a con-

stant speed electric motor than for variable speed prime movers such as engines or variable speed electric motors where additional load related factors affecting efficiency must be considered.

Once the pump and prime mover are selected and placed in operation, it can be anticipated that the pump will show a drop in performance over a period of time due to wear or changes in pumping conditions. In a deep well turbine, major performance deterioration due to wear can result from sand in the water. This causes increased clearances at seal rings and bearings as well as changes in hydraulic passages in the pump bowls and impellers. To detect deteriorating pump performance, it is necessary to secure sufficient data at regular intervals to determine the rate of change in performance. The data necessary consists of:

--Flow rate
--Total pumping head
--Specific yield of the well
--Horsepower input or amperage
--Hours of operation
--Visual observation of vibration, shaft run out, excessive shaft seal or packing leakage and occurrence of sand or gases in the water.

If possible, each data measurement should be taken at the same total pumping head to establish reference. Pump shutoff head should also be measured as it is indicative of wear. Most vertical turbine pumps have sealing surfaces between the impellers and pump bowls whose clearance can be reduced by lowering the impellers, and this adjustment should be made if performance drops off. Continued drop in pump performance over a long period can be extrapolated to indicate when pump removal and repair will be economically justified. Extraordinary drops in performance should be investigated when first discovered inasmuch as serious mechanical trouble may be indicated.

If well specific yield declines, it may become necessary to rehabilitate the well by redevelopment or chemical treatment of the well.

A recent article presented performance standards for how an average new pumping plant should perform in terms of water horsepower hours per unit of fuel.[5/] These standards are reproduced below.

NEBRASKA PERFORMANCE STANDARD FOR
DEEP-WELL TURBINE PUMPING PLANTS

Fuel	whp-hrs per unit of fuel*
Diesel	10.94 per gallon
Gasoline	8.66 per gallon
Propane	6.89 per gallon
Natural Gas	0.0667 per cubic foot
Electric	0.885 per Kwh

*Whp-hrs refers to the actual work being accomplished by the total system (pump and engine).

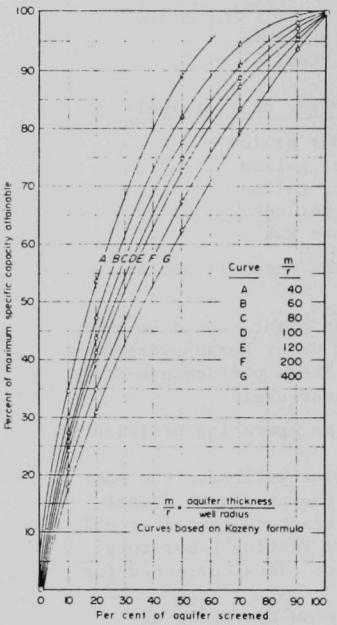
An average, new deep-well turbine pumping plant should be able to perform at or above the values given for water horsepower-hours per unit of fuel used in the above table. These performance standards are based on several assumptions. These are:

-- The average, new turbine pump will have an operating efficiency of 75%.
-- The average, new internal combustion engine will have the same fuel use efficiency (for the respective fuel types of diesel, gasoline and propane) as the average of tractors tested at 85% of maximum PTO hp at the Nebraska Tractor Testing Laboratory. (Natural gas values are from manufacturers' data corrected for 5% drive loss.)
-- The average, new electric motor will have an operating efficiency of 88% (although they may vary from 85% to 92%, depending on horsepower) and has a direct connection with the pump shaft.

Because the standards are based on how an average new pumping plant should perform, some pumping plants will exceed the standards.

CONCLUSION

The impact of energy optimization measures in groundwater production and development can be illustrated by a hypothetical well producing 114 cubic meters/hr (502 gal/min) with a total pumping lift of 30 meters (98 ft), a power cost of 3¢/Kwh and pumping efficiency of 60%. Power for this well costs 46.5¢/hr. If, through the energy optimization measures discussed herein, a pumping lift of 25 meters (82 ft) and efficiency of 70% could have been attained, power costs would be reduced to 33.2¢/hr. If the well were operated 300 days each year, the present worth of the power saving would exceed $8,000 for a twenty year well life. Energy savings of this nature multiplied by the thousands of wells throughout the nation can contribute significantly to the alleviation of our shortage and therefore must be pursued.



Relationship of partial penetration and attainable specific capacity for wells in homogeneous artesian aquifers.

PLATE I

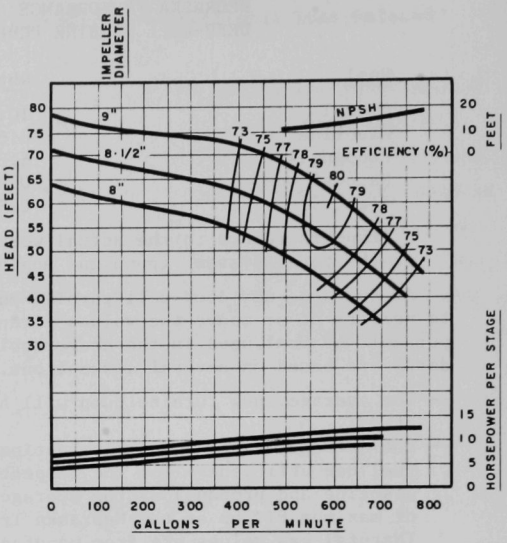


PLATE II

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PROJECTING THE IMPACT OF TIME-OF-USE ELECTRIC RATES ON MUNICIPAL WATER UTILITIES

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About three percent of all electrical energy consumed in the United States is used to pump water for agricultural and municipal purposes, and over ninety percent of the pumping energy used for irrigation in the country is electrical.[1] Furthermore, because in the U.S. both water and electric utilities experience peak consumption during the hottest summer days (with the exception of the Pacific Northwest, where electrical heating dominates), a similar percentage of electrical generating capacity is dedicated to the production of electricity for pumping water during the peak demand period. In recent years it has become difficult for electrical utilities to increase their generating capacity. Environmental concerns, rising construction costs, and other factors have all played a part. As a result, utilities have begun to turn away from rates schedules that measure and charge on the basis of the quantity of energy consumed toward rates that measure and charge on the basis of peak demand as well as the quantity of energy consumed. This paper presents a technique for projecting the financial impact of new electric rate schedules on water purveyors and operators of large water systems. Of special interest are Time-of-Use (TOU) Rates, and the way that this kind of rate structure may alter the operation of municipal water utilities.

INTRODUCTION

Electric utility customers who pump water have long been large and conspicuous consumers of energy. In many areas of the country, pumping and agricultural electricity customers operate under separate and often favorable rate schedules. Typical electric rate schedules for pumping have been of the decreasing block type, but recent difficulties in constructing new generating capacity have caused a trend toward rates based upon peak power demand and the time of use of the power, as well as total energy consumed.

To understand the relationship between electricity demand and water demand, let us first examine the diurnal consumption pattern of the two utilities as shown in Figures 1 and 2. Water and electricity producers have very similar problems in meeting the demands of their respective consumers. Both water and electric utilities experience broad temporal fluctuation in the demand for their services, on both a seasonal and a daily basis. Both are generally required by law to supply as much of their product as their customers require, more or less irrespective of the consumption rate, pattern of use, or other characteristic of the demand. Utilities have created rate structures that attempt to recover the average cost of service to a particular class of customer, generally categorized on the basis of type of user and the capacity of the user to place a peak load on the system. A lighting service, for example, has a different kind of electric demand than the electric service that is used to pump water. Thus, the two services would typically operate under different rate schedules. Until recently, however, more precise knowledge of the characteristics of the demand was not considered necessary for the utility to fully recover the cost of service.

FIGURE 1

WATER DEMAND CURVE FOR DAYS OF HIGH DEMAND

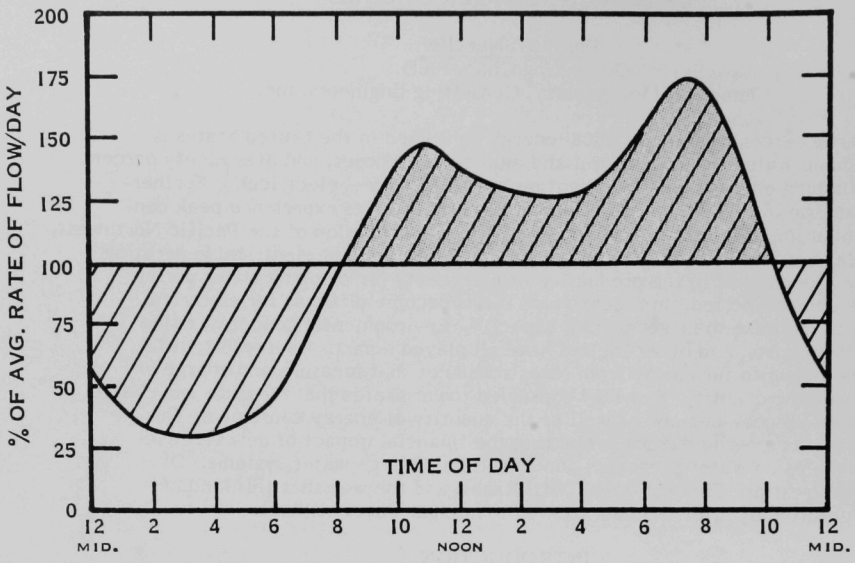
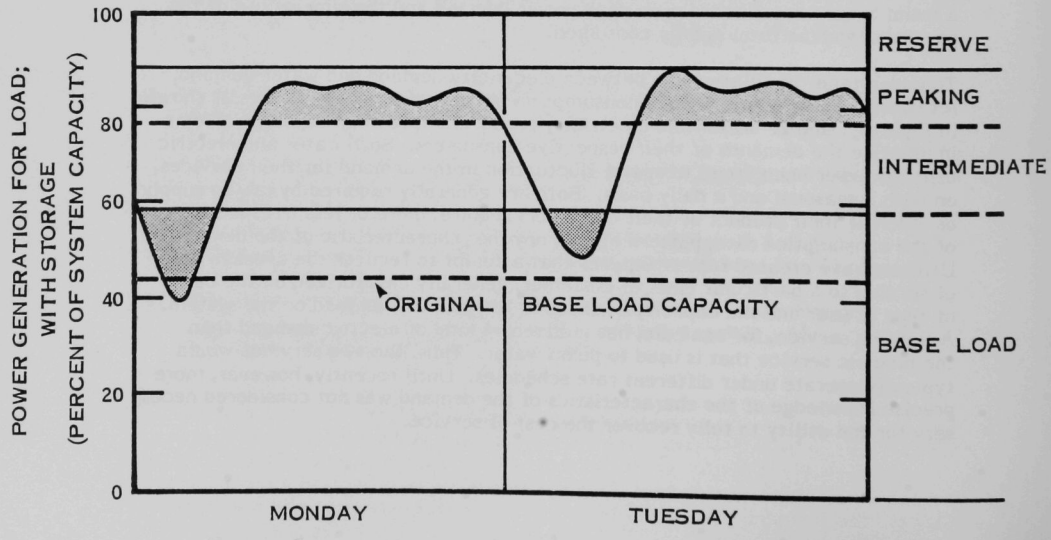


FIGURE 2

ELECTRICITY DEMAND CURVE FOR DAYS OF HIGH DEMAND



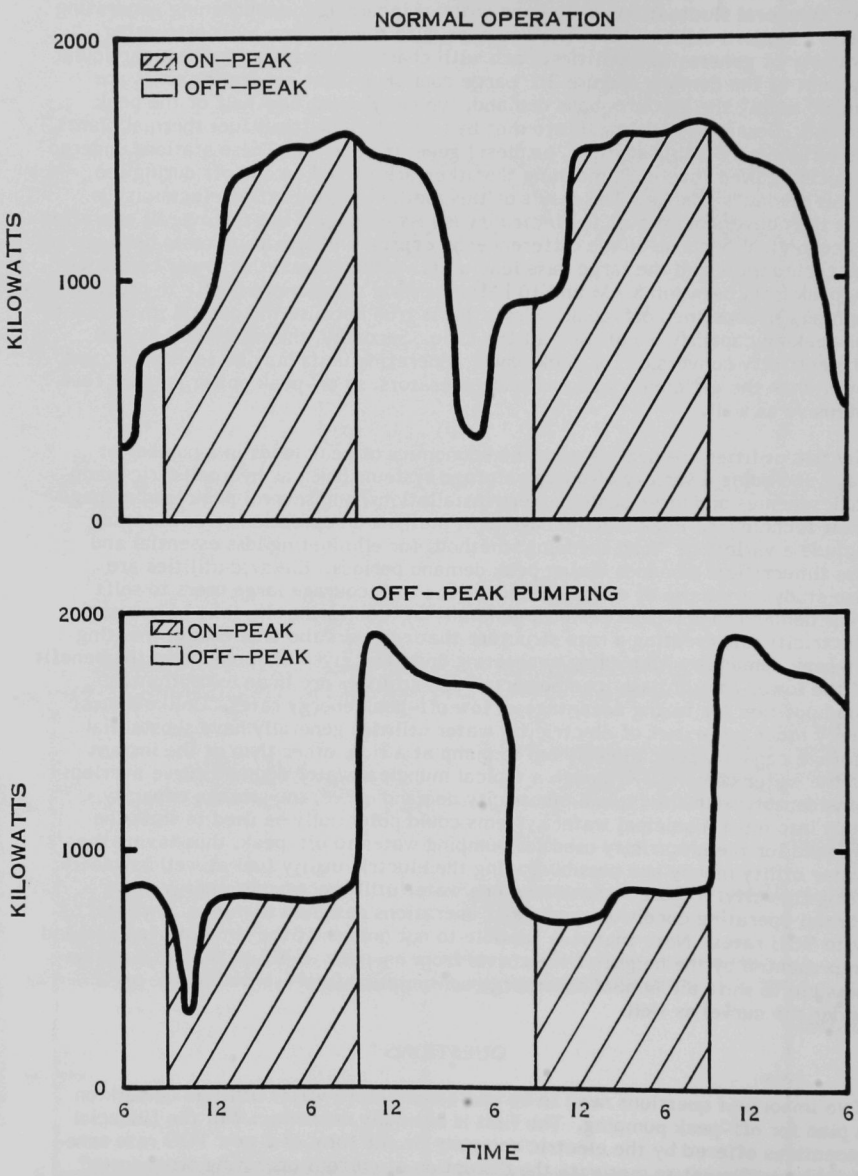
From their inception, electric utilities have had to deal with the problem of meeting wide temporal fluctuations in demand without incurring overburdening generating costs.[2] Modern utilities have tried to cope with the problem by constructing a variety of generating facilities, each with characterisitics suited to a particular segment of the demand (Figure 2). Large coal or nuclear generating units are used to supply the system's base demand, typically about one-half of the peak demand. Less steady demands are met by less efficient fossil fuel thermal plants, gas turbine generating stations, or diesel generating units. These stations undergo a process called "cycling", meaning that they are turned on and off during the day as demands change. The result of this method of generating electricity is that the conversion of fuel to electricity is less efficient, but the overall operation is economical because of the differences in capital and operating costs between the cycled units and the large base load units. Nonetheless, the power consumed on-peak (say, between 8 AM and 10 PM) is several times more costly to produce than power consumed off-peak. First, this is true because the capital investment for peaking capacity is not used all the time. Secondly, the efficiency of fuel to electricity conversion for peak power generating units may be several percent lower than the efficiency of base load generators, so on-peak power is more fuel-intensive as well.

Electric utilities are dealing with the economics of peak loads in a number of ways, including a variety of energy storage systems (such as hydroelectric pump-back schemes and large scale battery installations) and several peak load management techniques. Peak load management methods in operation or under study include a variety of "load shedding" methods for eliminating less essential and less timecritical demands during peak demand periods. Electric utilities are also studying the use of economic incentives to encourage large users to shift their demand to off-peak periods, generally by monitoring the time of use of electricity and creating a rate structure that recovers the true cost of meeting on-peak demands with cycling generating units and gives off-peak users the benefit of the lower cost of base-load power. Water utilities are in an exceptionally good position for taking advantage of low off-peak energy rates. Unlike almost every other large user of electricity, water utilities generally have substantial storage capacity that allows them to pump at a time other than at the instant of the water demand. Although a typical muncipal water demand curve overlaps considerably with the typical electricity demand curve, the storage capacity built into most municipal water systems could potentially be used to shift the demand for the electricity used for pumping water to off-peak, thus saving the water utility money and possibly saving the electric utility fuel as well as generating capacity. Figure 3 shows how one water utility consumes energy under normal operating conditions and under operations designed to minimize costs with TOU rates. Note that it is possible to not only shift the time of peak demand (represented by the height of the curve) from on-peak to off-peak, but it is also possible to shift the bulk of the energy consumption (represented by the area under the curve) as well.

QUESTIONS

Two important questions need to be addressed before water utilities embark on a plan for off-peak pumping. The first is basically economic: Will the financial incentives offered by the electric company (in the form of a new TOU rate schedule) be sufficient to motivate the disruption of current operating procedures? Unless the financial gains are substantial, few water purveyors will risk disruption

FIGURE 3
ENERGY DEMAND



of current pumping schedules to save a few dollars. Second, is the proposed operation scheme a practicable and feasible one that will not decrease the level of service or reliability currently enjoyed by the water consumers? If the level of service would be impaired, are there simple and relatively inexpensive measures that can be instituted to recover the previous standards of service? For example, would the money saved in electricity bills be sufficient to amortize the cost of new pumping or storage facilities? One of the authors (Chao) has described before a conceptual analysis which provides some clues on these issues.[3] He was able to show that a defineable minimum cost differential must exist between off-peak and on-peak power before it will be advantageous for the water purveyor to shift to off-peak pumping. That analysis is based on the differences in head loss in pipes under the two pumping schedules, and is therefore accurate only for a simple pump/pipe/reservoir arrangement, and does not consider the effects of a hydraulic network. Furthermore, it provides no mechanism for estimating deleterious effects of off-peak pumping schedules, such as changes in service pressure and excessive velocity in pipelines.

The resolution of these questions is not a simple matter. The relationship between electricity consumption and water demand is highly complicated and includes consideration of the water system's storage capacity, the location and other characteristics of the water demand, pumping capacity, pipeline capacity, and other factors. Because of the non-linear relationship between water demand and head loss in pipes, the mathematical equations describing the operation of a water system are difficult to solve.[4] However, in recent years a number of techniques for the solution of these equations have been devised, making possible the writing of a computer program to simulate the operation of a water system. The program allows engineers to model the financial impact of time-of-use rates, decreasing block rates, or any other electric rate structure, and to simulate the impact of modifications to the operations and facilities of the water utility. By operating the model under a variety of real or imaginary conditions, the engineer is able to deduce the combination of facilities and operational methods that result in minimal cost of operation. Furthermore, it is possible to project the changes in energy consumption that may be caused by new operation schemes and to measure the impact of those schemes on the generating and fuel requirements of the electric utility.

To fully understand the potential for savings in electric service costs to a water utility, it is first necessary to understand something about the many different rate structures that are in use throughout the country. Most agricultural and municipal pumping plants operate under a decreasing block rate schedule, in which the cost of each unit of energy decreases in steps or blocks as the total amount of energy consumed increases (Table 1). The theory behind a decreasing block rate is that the incremental or marginal cost of service decreases as the user consumes more energy because a large portion of the cost of service is fixed (capital cost of generating facilities, cost of accounting, etc.). One rate schedule that may well replace the decreasing block rate is the time-of-use (TOU) rate, in which the impact of the temporal distribution of electric demand is explicity considered in determining the cost of electric service. Table 1 also shows a typical TOU rate schedule.

TABLE 1
EXAMPLE RATE SCHEDULES

Decreasing Block Rates

Demand Charge	
First 75 KW	$ 281.75/month
All Excess KW	$ 3.75/month
Energy Charge	
First 150 KWh per KW	$ 0.04057/KWh
Next 150 KWh per KW	$ 0.03857/KWh
Excess KWh	$ 0.03657/KWh

Time-of-Use Rates

Demand Charge	
Based on Standard Transformer Size	
Minimum of 12 months	$ 1.50/KW/month
Energy Charge	
On-peak (8 AM to 10 PM)	$ 0.05397/KWh
Off-peak (10 PM to 8 AM)	$ 0.03497/KWh

EXAMPLE ANALYSIS

The example we will discuss here examines the potential for economic benefit for a moderate sized water utility in Southern California in the conversion to a TOU rate from a decreasing block rate. One pressure zone (Figure 4), encompassing about twenty-five percent of the water system's total demand of 100 million gallons per day, was studied to determine the feasibility of converting to a TOU rate and to estimate the impact of the conversion on system performance and reliability of water service. The city's water master plan had pointed out the need for consideration of off-peak pumping in the system's numerous elevated pressure zones. Work is currently under way to increase the storage capacity in many areas of the city, and it is likely that there will initially be some excess storage in certain areas as demands grow toward the expected future level. Thus, it appears that the city could utilize the new storage and pipeline capacity to pump during the off-peak period and thus reduce electric service costs.

A computer program was used to simulate the operation of the water system during a forty-eight hour period representing typical high demand summer days. The program, called TIMER, is based upon a methodology for static analysis of hydraulic networks first described by Wood and Charles in 1971.[3] The program linearizes the non-linear equations describing the head loss in pipes as a function of flow and solves the equations iteratively by matrix methods. This technique was selected for the analysis because it is especially efficient for repetitive analysis, such as that done in simulating the daily cycle of a municipal water system. Since most water distribution systems cannot be supplied by gravity flow alone, they rely on a combination of pumping stations and elevated storage reservoirs.

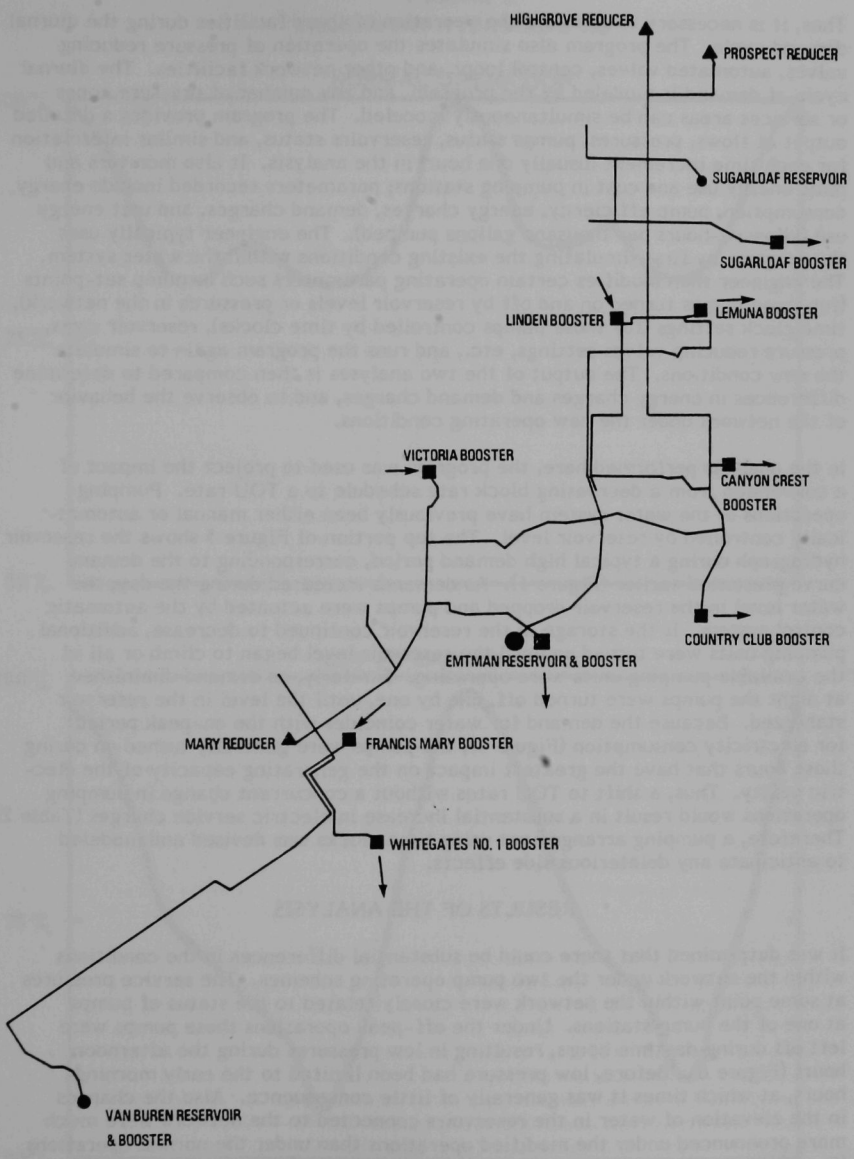


FIGURE 4
EXAMPLE HYDRAULIC NETWORK

Thus, it is necessary to simulate the operation of these facilities during the diurnal demand cycle. The program also simulates the operation of pressure reducing valves, automated valves, control loops, and other network facilities. The diurnal cycle of demand is modeled by the program, and any number of pressure zones or services areas can be simultaneously modeled. The program provides a detailed output of flows, pressures, pumps status, reservoirs status, and similar information for each time increment (usually one hour) in the analysis. It also monitors and plots energy use and cost in pumping stations; parameters recorded include energy consumption, pump efficiency, energy charges, demand charges, and unit energy use (kilowatt-hours per thousand gallons pumped). The engineer typically uses the program by first simulating the existing conditions within the water system. The engineer then modifies certain operating parameters such as pump set-points (for those pumps turned on and off by reservoir levels or pressures in the network), time clock settings (for those pumps controlled by time clocks), reservoir sizes, pressure reducing valves settings, etc., and runs the program again to simulate the new conditions. The output of the two analyses is then compared to determine differences in energy charges and demand charges, and to observe the behavior of the network under the new operating conditions.

In the analysis performed here, the program was used to project the impact of a conversion from a decreasing block rate schedule to a TOU rate. Pumping operations in the water system have previously been either manual or automatically controlled by reservoir level. The top portion of Figure 5 shows the reservoir hydrograph during a typical high demand period, corresponding to the demand curve presented earlier (Figure 1). As demands increased during the day, the water level in the reservoir dropped and pumps were actuated by the automatic control system. If the storage in the reservoir continued to decrease, additional pumping units were turned on until the reservoir level began to climb or all of the available pumping units were operating. Similarly, as demand diminished at night the pumps were turned off, one by one, until the level in the reservoir stabilized. Because the demand for water coincides with the on-peak period for electricity consumption (Figure 3), the pumps were generally turned on during those hours that have the greatest impact on the generating capacity of the electric utility. Thus, a shift to TOU rates without a concurrent change in pumping operations would result in a substantial increase in electric service charges (Table 2). Therefore, a pumping arrangement using time clocks was devised and modeled to anticipate any deleterious side effects.

RESULTS OF THE ANALYSIS

It was determined that there could be substantial differences in the conditions within the network under the two pump operating schemes. The service pressures at some point within the network were closely related to the status of pumps at one of the pump stations. Under the off-peak operations those pumps were left off during daytime hours, resulting in low pressures during the afternoon hours (Figure 6). Before, low pressure had been limited to the early morning hours, at which times it was generally of little consequence. Also the changes in the elevation of water in the reservoirs connected to the network were much more pronounced under the modified operations than under the normal operations (Figure 5). Until additional reservoir capacity comes on line (to meet expected future requirements), it may not be possible to allow the reservoir to be drawn down to such a level. (It should be noted here that, like many water storage reservoirs, the reservoirs in this system are used for three purposes: operational or equalization storage, fire protection storage, and emergency outage protection.

FIGURE 5
RESERVOIR HYDROGRAPHS

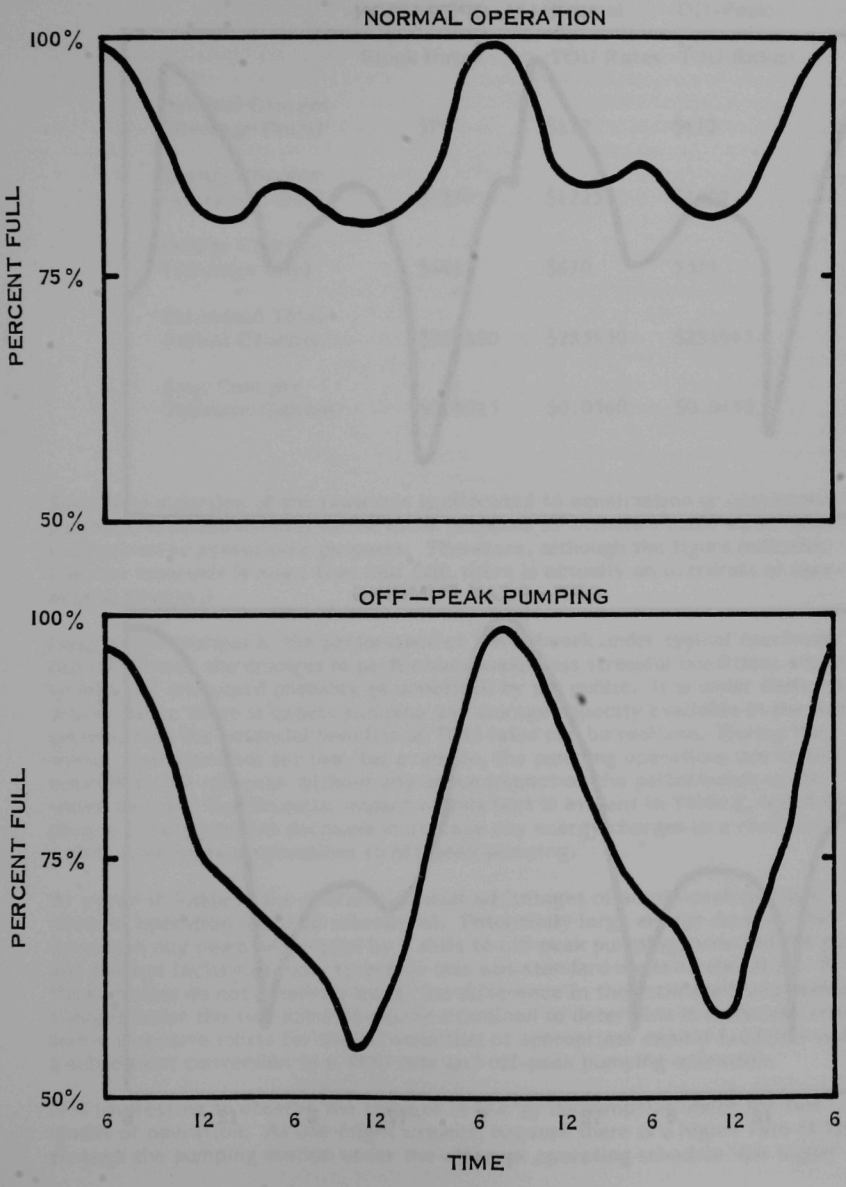


FIGURE 6
TYPICAL SERVICE PRESSURE

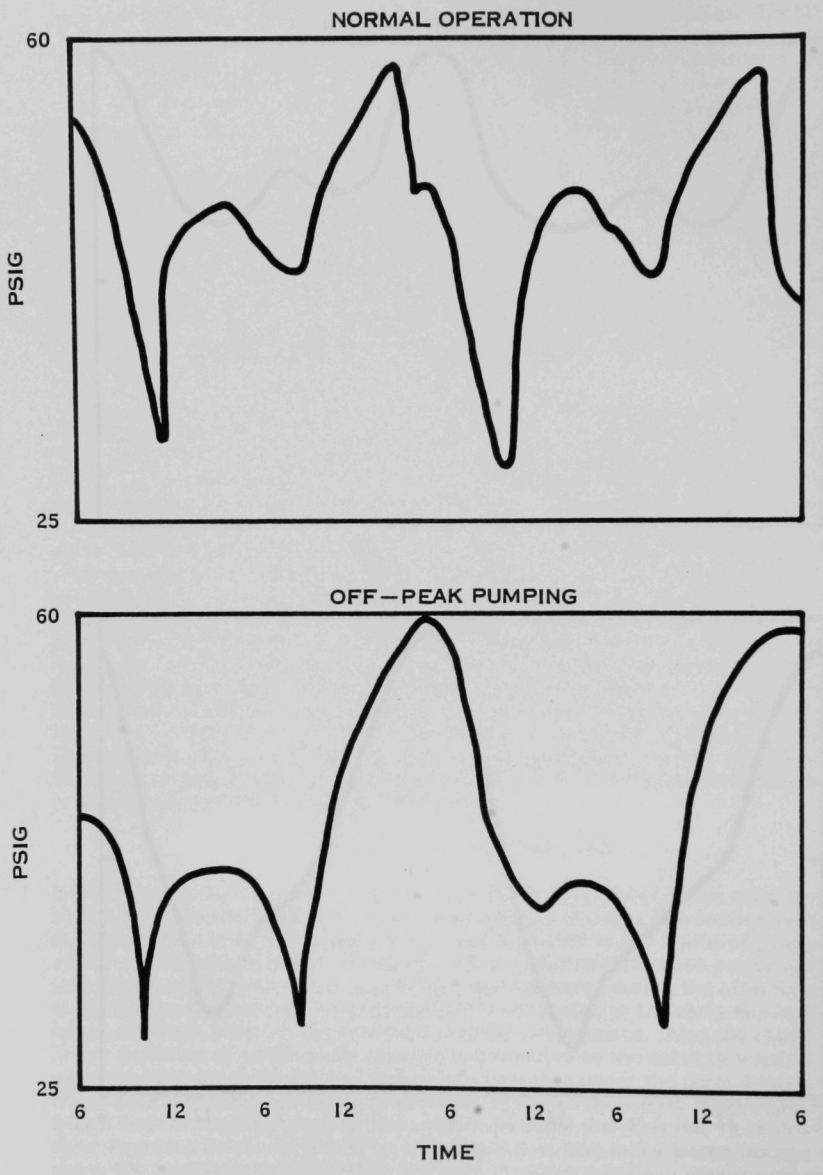


TABLE 2
TYPICAL ENERGY SERVICE CHARGES

	Normal Operation Block Rates	Normal Operation TOU Rates	Off-Peak Operation TOU Rates
Demand Charges (Average Daily)	$79	$112	$112
Energy Charges (Maximum Day)	$1239	$1225	$1207
Energy Charges (Average Day)	$641	$670	$521
Estimated Total Annual Charges	$262800	$285430	$231045
Avg. Cost per Thousand Gallons	$0.0515	$0.0560	$0.0453

Since only a portion of the reservoir is allocated to equalization or operational purposes, it is detrimental to the other purposes to withdraw more water than is allocated to operational purposes. Therefore, although the figure indicates that the reservoir is more than half full, there is actually an overdraft of operational storage.)

Despite the changes in the performace of the network under typical maximum day conditions, the changes in performace under less stressful conditions would be minimal and would probably go unnoticed by the public. It is under these conditions, when there is excess pumping and storage capacity available in the water system, that the potential benefits of TOU rates can be realized. During the winter when demands are low, for example, the pumping operations can be shifted entirely to the off-peak without any undue impact on the performance of the water system. The financial impact of this fact is evident in Table 2, which displays a twenty percent decrease in average day energy charges as a result of a shift from normal operations to off-peak pumping.

As shown in Table 2, the overall financial advantages of an off-peak pumping mode of operation could be substantial. Potentially large energy costs on the maximum day could be avoided by a shift to off-peak pumping, provided the pumping and storage facitlities exist to enable this non-standard mode of operation. If the facilities do not presently exist, the difference in the estimated total annual charges under the two schemes can be examined to determine if sufficient economic incentive exists for the construction of appropriate capital facilities and a subsequent conversion to a TOU rate and off-peak pumping operation.

It is interesting to observe the changes in energy consumption under the two modes of operation. As one might suspect, because there is a higher rate of flow through the pumping station under the off-peak operating schedule (the higher

peaks in Figure 3) there are greater energy losses to friction. Through the use of the network analysis and simulation model described above, it was possible to quantify these losses for the particular system under consideration in this example. Off-peak pumping would require approximately three to five percent more energy than the normal pump operating schedule. However, because of the nature of the system analysed here it appears that a slightly higher range, say five to seven percent, would be expected for a typical network. The loss in overall (global) system efficiency, i.e., the efficiency computed by dividing the energy value of the fuel used at the power plant into the potential energy added to the water that was pumped, would probably be somewhat less than two percent. This is due to the higher efficiency of the base-load generating stations when compared with the cycling generating units.

SUMMARY

In the future, there is likely to be ever more careful control of out very limited energy resources and continuing difficulties with increasing electric generating capacity. As time-of-use electric rates become more prevalent, the use of off-peak pumping in municipal water systems to provide a kind of pumped storage of energy will no doubt be exploited. It has been shown that off-peak pumping can result in substantial savings to water utilities currently operating under TOU or decreasing block rates, provided that the differential between on-peak and off-peak energy rates is non-trivial. A technique for projecting the potential financial impact of TOU rates has been presented, utilizing a dynamic computer model of the hydraulic network. Now that this methodology has been developed, it remains only for the institutional framework to built on the basis of common gain for the utilities involved.

ACKNOWLEDGEMENTS

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EXTENSIVE POWER SAVINGS THROUGH MAXIMIZATION OF SAGINAW-MIDLAND WATER SUPPLY SYSTEM PUMPING EFFICIENCY - A CASE STUDY

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In 1946, the Michigan cities of Midland and Saginaw, located in the Saginaw Bay area of Lake Huron, established the Saginaw-Midland Water Supply System (SMWSS) to provide for their combined water supply needs. The system, with a current average demand of about 50 million gallons daily (mgd) used over 30×10^6 Kwhr of electricity annually.

At average energy costs of $0.035 per Kwhr, the Saginaw-Midland system spends over one million dollars annually for power use alone. In view of a total operating budget of $2.5 million, the amount spent for power is a significant operating cost of the system. In response to rising energy costs in the early 1970s, the SMWSS took steps to monitor power usage more closely. The environmental consulting firm of Camp Dresser & McKee Inc. (CDM) was retained to conduct system studies in 1973, and again in 1974, in an effort to develop an effective energy monitoring approach.

SYSTEM DESCRIPTION

Since 1946, the Saginaw-Midland Water Supply System has grown to include three pumping stations which supply two major municipal treatment facilities for a total design capacity of about 80 million gallons daily (mgd), see Figure 1. Raw water is obtained from outer Saginaw Bay - at a point approximately 6 5 miles from the two cities to ensure good water quality. The capacities, pumping configurations and available storage volumes of each pumping station are summarized in Table 1.

At one end of the system is the Whitestone pumping station with five constant-speed pumps; three two-stage 20 mgd units; one 30-mgd unit; and one 40-mgd unit. The 20-mgd pumps can be uncoupled for single-stage operation, Presently, Whitestone can deliver about 41 mgd directly to the treatment plants. If total system demand exceeds this volume, Whitestone pumps directly to Pinconning, the next station downstream rather than bypassing it.

Pinconning has five pumps: three two-stage, 20-mgd, constant-speed units; one two-stage, 20-mgd, variable-speed unit; and one single-stage, 50-mgd, constant-speed unit. The four 20-mgd pumps can be uncoupled for single-stage operation (although they have never been so operated in the past). Pinconning reservoirs have a usable volume of 2.1 million gallons. As presently operated, Pinconning can deliver about 64 mgd directly to the treatment plants. If total system demand exceeds this volume, Pinconning pumps directly to the Junction station, rather than bypassing it.

Junction has four constant-speed pumps: three two-stage 20-mgd units and one single-stage 40-mgd unit. The 20-mgd pumps cannot be uncoupled, but could be modified by valving and piping changes to allow single-stage operation. All units can pump directly from either the 48-in. Pinconning line or from a 5-mg reservoir (usable volume = 3.38 mg). This last of the three pumping stations can pump a total system demand of about 80 mgd to the treatment plant.

The system hydraulics usually require throttling when flow is pumped from the Junction Pumping Station to the treatment plants. Pressure in the 36-in. Midland transmission line is usually maintained below 200-ft of water on account of past problems when this head was exceeded. It is possible through valve operations to use two of Junction's 20-mgd pumps to pump flow separately to the Midland treatment plant while the Whitestone or Pinconning station is pumping directly to the Saginaw facility.

Midland currently has a 125-mg raw water reservoir (overflow elevation - 651.5 ft, normal operation range = upper 3 ft), with a usable volume of 21 mg. Water is ordinarily supplied to the Midland East Water Treatment Plant at a hydraulic gradeline elevation of about 685 ft. By SMWSS agreement, Midland is allocated about 47 percent of the total system capacity. In 1973, Midland's average usage was 22.94 mgd, of which about 14 mgd were delivered to Dow Chemical (hydraulic gradeline elevation 663 ft), and 9 mgd to the East plant. Maximum day usage was 35.59 mgd. Nominal capacity of the East plant is 8 mgd, although it has been operated at 12 mgd. Raw water from the reservoir can be delivered to the treatment plant by two 15-mgd pumps.

Saginaw has a 50-mg raw water reservoir, of which approximately 27 mg are hydraulically usable. Overflow elevation of the reservoir is 587 ft; permissible drawdown is 11.2 ft. At present, there are five raw water pumps with capacities ranging from 7 to 14.7 mgd (total capacity 47.3 to 55.7 mgd). However, significant taste and odor problems in the Saginaw raw water reservoir have resulted in consumer complaints whenever it has been used.

In 1973, Saginaw's average usage was 28.52 mga; maximum usage was 46.08 mgd. The required hydraulic gradeline elevation at the Saginaw treatment plant influent splitter box is 607 to 609 ft, depending on flow rate.

Under current demand conditions, the Whitestone station operates 52 weeks per year, Pinconning 48-50 weeks per year, (e.g., whenever demand exceeds 41 mgd), and Junction about 2 weeks per year, (e.g., when demand exceeds 64 mgd).

EFFECTIVE ANALYSIS

Two general routes were taken to develop efficient operating procedures: (1) SMWSS in-house analysis (comparison of operating records with equipment specifications and actual operation under varied demand situations), and (2) consultant analysis by CDM engineers experienced in systems analysis, hydraulics, pumps, and motors.

In-house analysis. A number of different operating procedures were tried by the SMWSS over the past few years. Daily log sheets were maintained, and overall efficiencies of the various techniques were compared to develop optimum pumping configurations. Operating procedures based on the above data were then analyzed by CDM. Significantly, the maintenance of daily records, and the fact that the SMWSS had operated the pumping stations using different pumping configurations for the same flows, were factors greatly facilitating the evaluation.

Consultant analysis. To evaluate potential energy savings, pump characteristic curves and pumping station log sheets maintained by the SMWSS were evaluated (log sheets contain daily information on power use per pump, average TDH per pump, and total system demand), and pump performance curves were plotted and compared with manufacturers' curves. The actual efficiency, flow and head characteristics of each pump were then established. System curves were developed between pumping stations and between each pumping station and treatment plant (three stations, two plants). Pump characteristic curves were plotted on each system curve, and the most efficient pumping arrangements for various system demands were determined.

Next, the existing reservoirs at Pinconning, Junction, and the treatment plants were examined to determine the feasibility of dampening demand fluctuations from storage (thereby allowing each pumping station to deliver water at constant- and most efficient-rates). Since the raw water reservoir at the Saginaw plant is seldom used, and the required hydraulic gradeline elevation is about 76 to 78 ft less than at the Midland plant for nearly equal flows, conditions along the Midland route usually dictate pressure characteristics at the SMWSS pumping stations.

Although the normal elevation in the Midland plant raw water reservoir is about 651 ft (34 ft lower than the plant requirement), there is a high elevation of 661 ft about one mile upstream to be overcome before flow can be delivered to the reservoir. Dow Chemical is at elevation 663 ft downstream of the treatment plant. Thus, even if the SMWSS pumped directly to Dow Chemical and the Midland reservoir, conditions at Midland would still determine system pressures. Maximum efficiency, therefore, could be attained if Saginaw's demand were allowed to fluctuate and total system demands were dampened from storage at Midland. (In this way, the stations could still pump at a constant rate.)

ALTERNATIVES STUDIED

Three alternative operating techniques were investigated: (A) pumping to both treatment plants at relatively constant overall rates and repumping short-term deficiencies from the Midland raw water reservoir; (B) pumping at constant rates to Dow Chemical, Midland raw water reservoir, and Saginaw treatment plant, with Midland repumping all flow to its treatment plant, and (C) pumping to the Saginaw treatment plant from Whitestone or Pinconning (bypassing Junction), drawing the remaining total system flow through Junction to Dow Chemical and the Midland treatment plant. The latter alternative was evaluated with two-stage or single-stage operation.

The first alternative would allow the SMWSS to pump at constant rates, and would eliminate throttling. The second alternative would lower head requirements slightly, and thus possibly increase the capacity of the pumping station. The third alternative would lower head requirements at Whitestone and Pinconning by 76-78 ft, and thus allow the pumps at these stations to deliver greater flows at constant rates. Demand fluctuations would be met by the Junction storage reservoir or directly by the two 20-mgd Junction pumps.

To evaluate these alternative pumping modes, an energy efficiency index (EEI) which related power consumed (Kwhr) to pumpage (mgd), was developed. The EEI under 1973 SMWSS conditions was plotted, and the effect of each alternative operating technique was then superimposed on the reference curve, Figure 2. A combination of the alternative techniques was found to yield the most efficient power consumption.

Alternative A was recommended for system demands up to 41 mgd, and for demands between 72 and 80 mgd. Alternative B was recommended for flows between 47 and 64 mgd. Alternative C was recommended for flows between 41 and 47 mgd, and 64 and 72 mgd. In addition, it was recommended that single-stage pumping be utilized over certain flow ranges.

STORAGE CONSIDERATIONS

In order to avoid frequent pumping changes when operating in the above modes, it is essential to utilize the Pinconning and Junction storage facilities most efficiently. Figure 3 illustrates the time required to empty or fill the usable portions of Pinconning and Junction reservoirs for given differences between inflow and outflow. Consider the following example:

Average total system demand = 74 mgd

Pinconning reservoir = initially full, Junction reservoir = initially empty

Whitestone station = initially pumping at 74 mgd, Pinconning at 80 mgd, and Junction at 70 mgd.

Assuming a normal flow split, 34.4 mgd would be required for Midland and 39.6 for Saginaw. At Junction, the reservoir inflow would be 80 mgd, the outflow 70 mgd. An average flow of 39.6 mgd would be delivered to Saginaw and 30.4 mgd to Midland; the remaining 4 mgd would be repumped from Midland's raw water reservoir to the Midland East treatment plant. Table 2 summarizes typical operation for the above example. Based on Figure 3, Junction would fill in 8.1 hours, after which this station would pump 80 mgd to the treatment plants (39.6 mgd to Saginaw, 40.4 mgd to Midland, 6 mgd to the Midland raw water reservoir). After 10.1 hours, the Pinconning reservoir would be empty, and the Pinconning station would pump 64 mgd with the 50-mgd pump for 4.6 hours. Over a 23.3 hour period, nine pumping changes would be made (three at Whitestone, three at Pinconning, and three at Junction) - an average of one change per station about every 8 hours.

It was also determined that use of Midland's raw water storage reservoir could delay transmission system improvements by increasing present system

capacity to 89 mgd. During high-demand periods, flow could be pumped at a constant rate (80 mgd) to the cities, with water entering the reservoir at night and on weekends and pumped from the reservoir on weekdays.

SUMMARY

Through in-house field experimentation and selective engineering consultation, the SMWSS was able to realize substantial power savings. For example, in 1975 the first full operating year in which CDM's recommendations would show their effects - at an average usage of 49.91 mgd, approximately 31,200,000 Kwhr were consumed at a cost of $624,000. Without the recommended changes to past operating procedures, an estimated 32,800,000 Kwhr would have been used at a cost of $656,000, for a savings of about 1,600,000 Kwhr, or $32,000. In addition, an increase in overall system capacity from 80 to 89 mgd was found to be feasible through more effective utilization of available storage. System managers should take note! Power costs are increasing rapidly, and usually are a major portion of water-utility expenditures. A well-managed system, with selective engineering input, could reduce power usage and thus costs. Figure 4 is a plot of the EEI since 1949. In 1972 the last major pipeline construction was completed, the EEI figures for 1972 through 1978 most accurately reflect the results of the energy efficiency analysis.

Many water works systems that operate pumping stations, reservoirs, or treatment plants could save power by more efficient operating techniques. A basic goal in saving power in pumping station operations is to pump at relatively constant rates without throttling. To evaluate potential power savings, the water-utility manager should determine:

1. how much storage can be used safely;
2. the operating characteristics of each pump;
3. the required discharge pressure at particular demand conditions (i.e., system curves).

In determining the amount of storage that can be utilized safely, it is important to consider total storage volume, storage volume required for fire protection (this was not applicable to the SMWSS, because fire protection is provided after treatment plants), and storage volume required for emergencies (such as power outages, if no standby power source is available).

Daily pumping records should be maintained, showing hours run, average suction and discharge pressures, average flow, and total power use in kilowatt-hours. This will enable the water-utility manager to determine actual pump characteristics and required discharge pressure for specific flows. Once the operator has determined these parameters, engineering analyses and system experimentation can be used to maximize efficiency and minimize power use.

The techniques described in this article could also be applied to private industry, sewer or drainage systems, and other systems that utilize pumping stations. Potential savings in power usage can be achieved by many utilities throughout the country. Dollars saved by improved operations can then be applied to system improvements and/or reduced consumer billings.

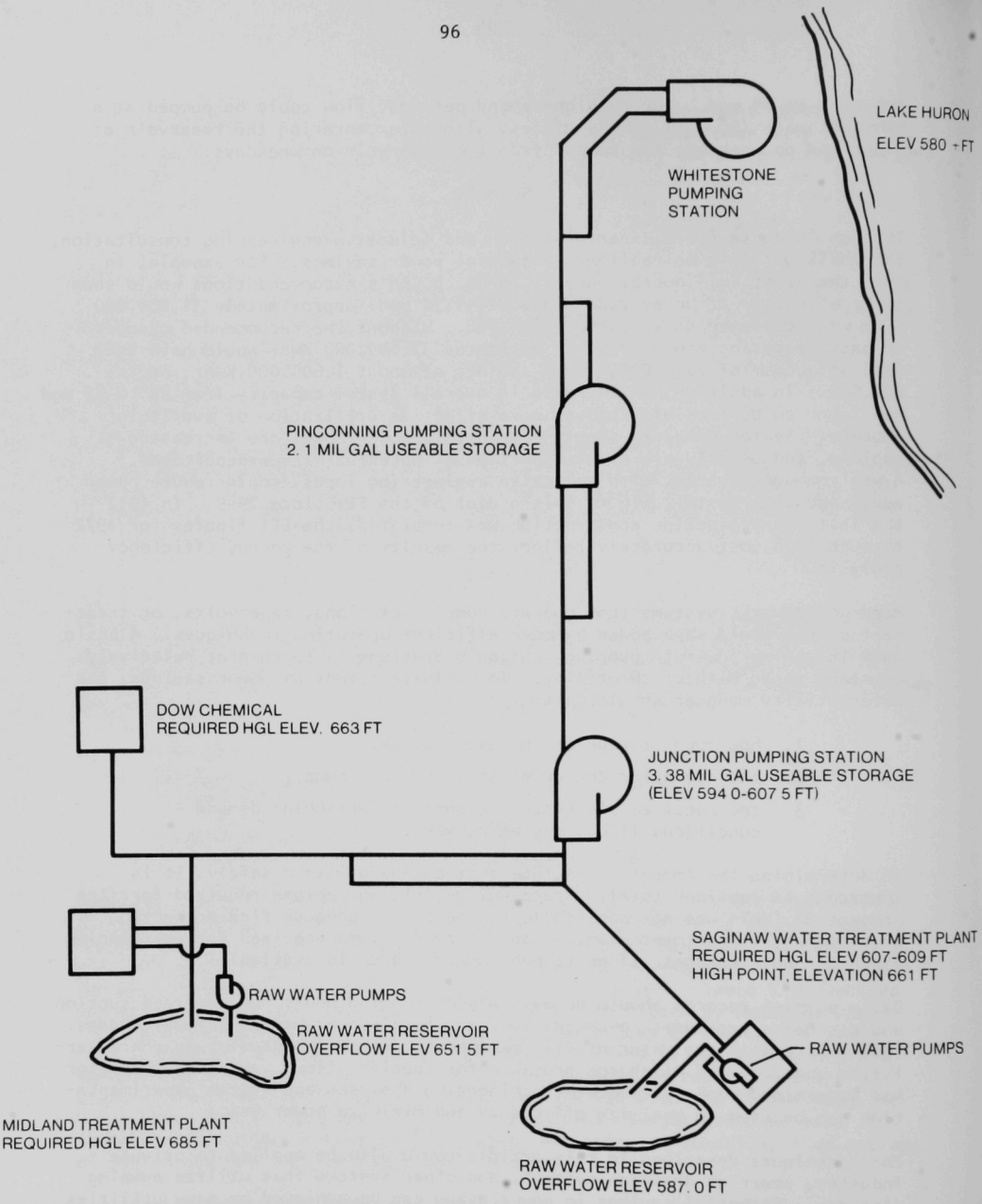

FIG. 1 SCHEMATIC DIAGRAM OF TRANSMISSION SYSTEM

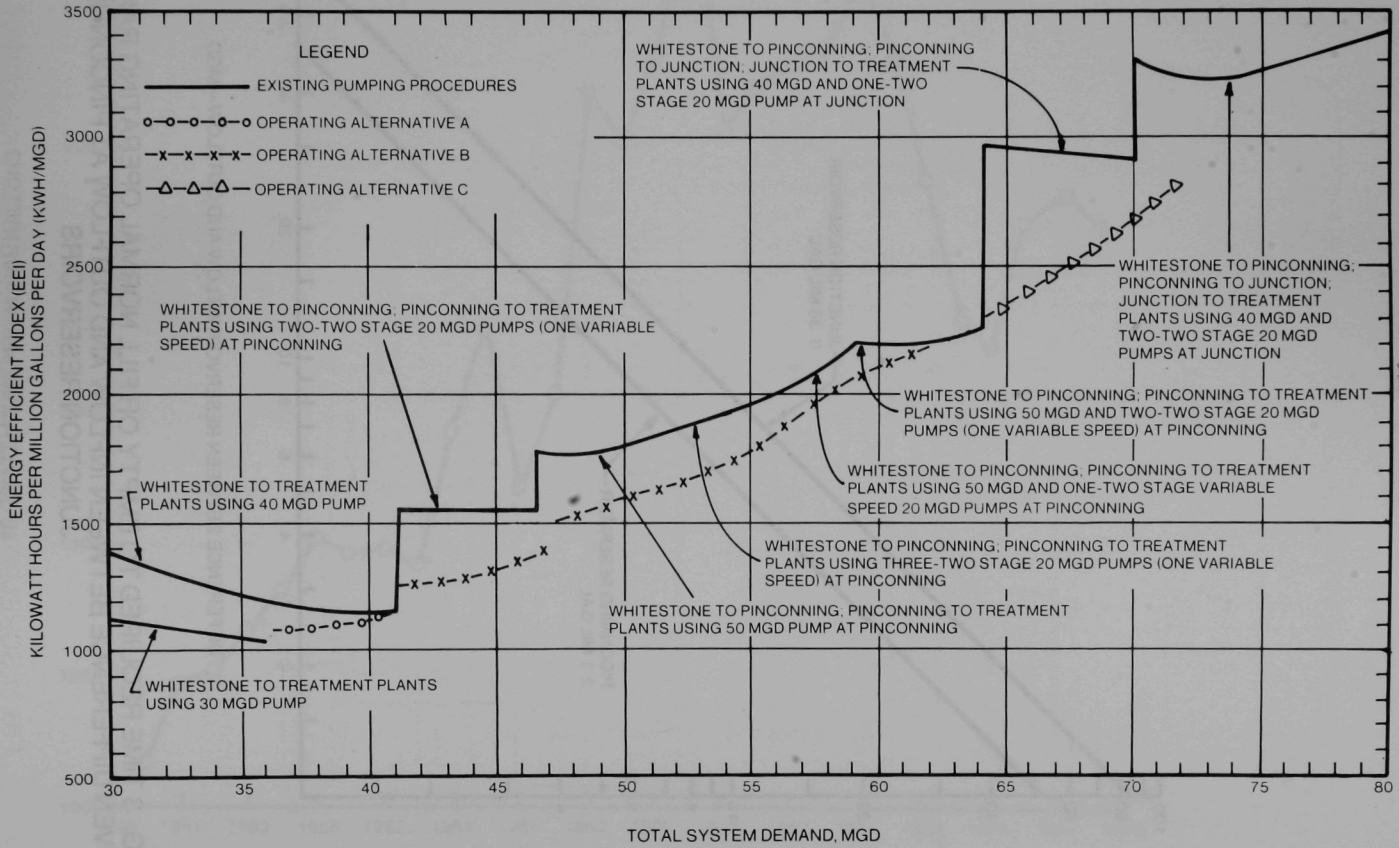

FIG. 2 UNIT POWER USAGE PER MILLION GALLONS PUMPED FOR EXISTING OPERATING PROCEDURES AND OPERATING TECHNIQUES A, B, AND C.

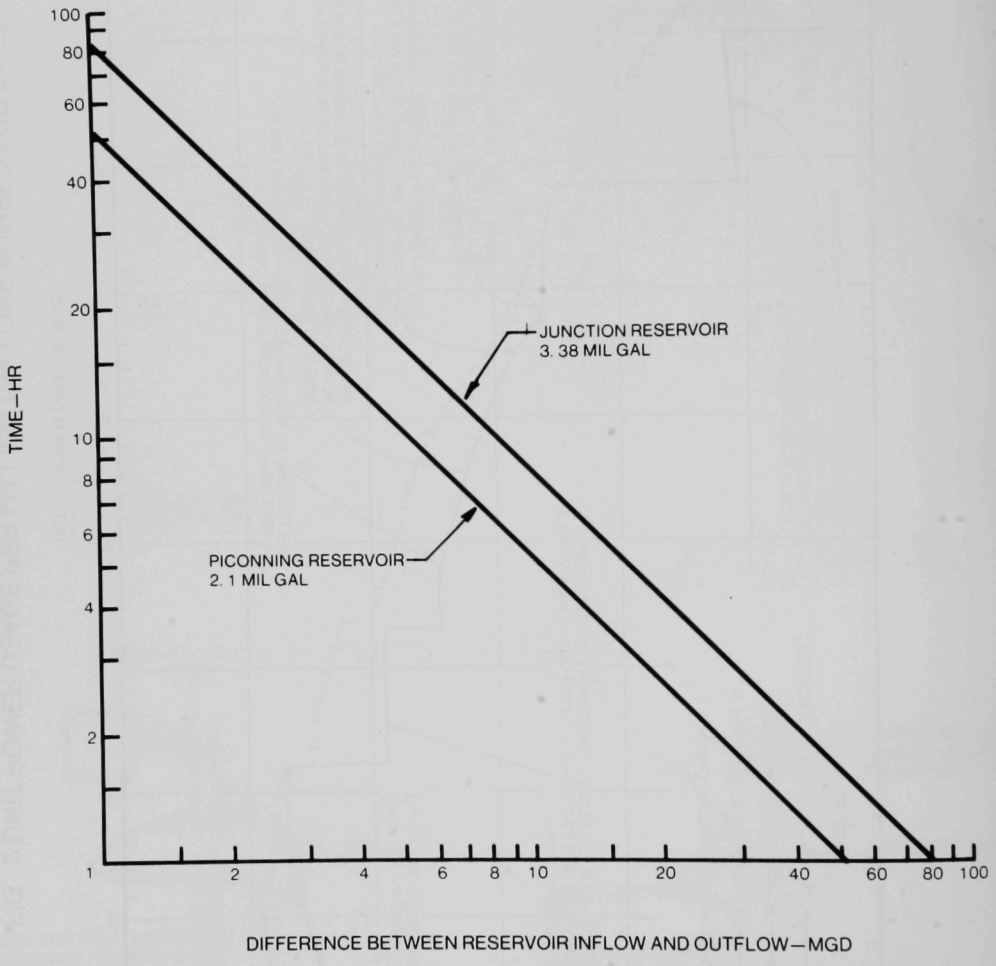

FIG. 3 TIME REQUIRED TO EMPTY OR FILL NORMAL OPERATING RANGE FOR GIVEN DIFFERENCE BETWEEN INFLOW AND OUTFLOW AT PINCONNING AND JUNCTION RESERVOIRS

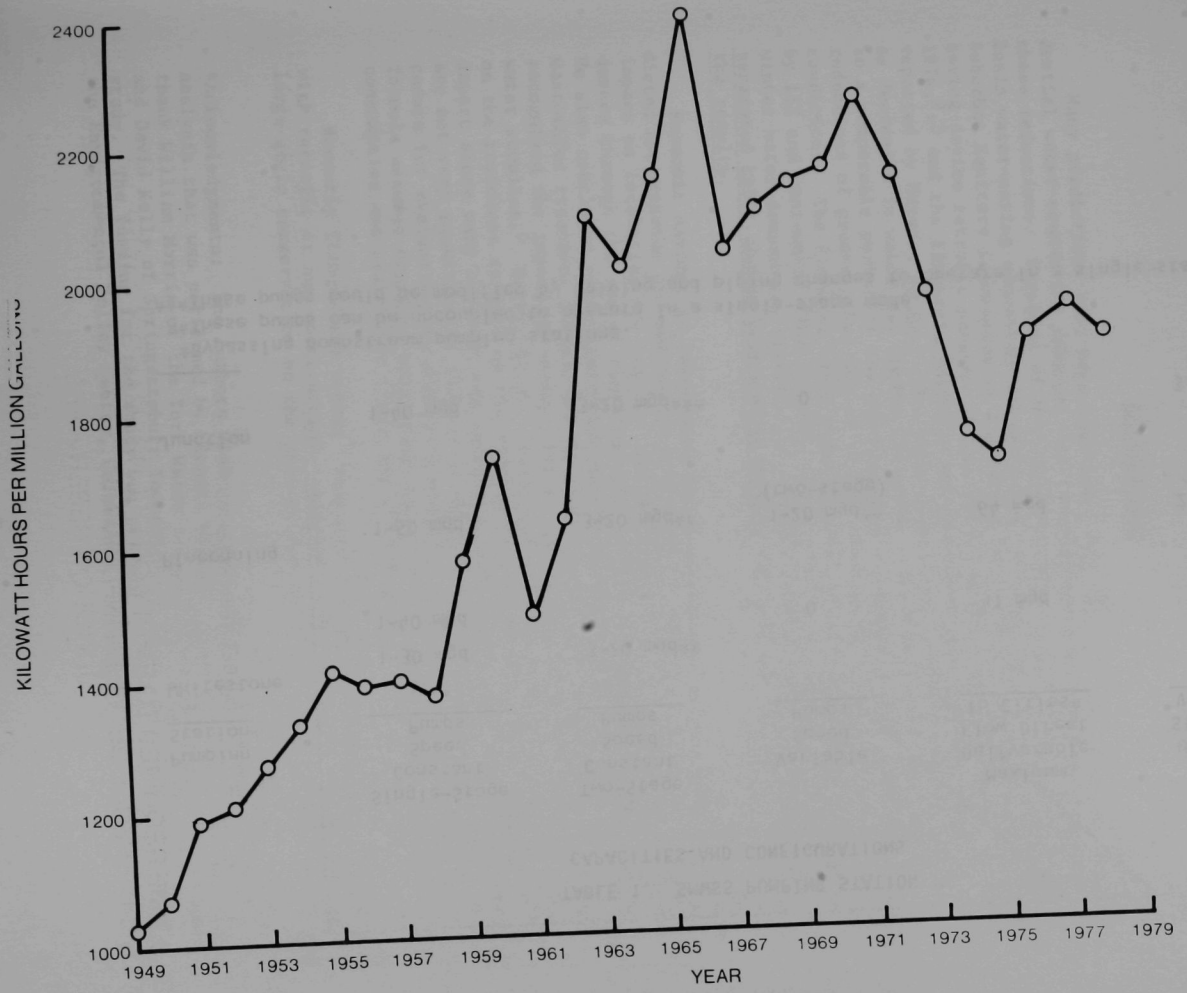

FIG. 4 ENERGY EFFICIENCY INDEX

TABLE 1. SMWSS PUMPING STATION
CAPACITIES AND CONFIGURATIONS

Pumping Station	Single-Stage Constant Speed Pumps	Two-Stage Constant Speed Pumps	Variable Speed Pumps	Maximum Deliverable Flow Direct To Cities*	Usable Storage Volume
Whitestone	1-30 mgd 1-40 mgd	3-20 mgd**	0	41 mgd	
Pinconning	1-50 mgd	3-20 mgd**	1-20 mgd** (two-stage)	64 mgd	2.1 mgd
Junction	1-40 mgd	3-20 mgd***	0		3.38 mgd

*Bypassing downstream pumping stations.
**These pumps can be uncoupled to operate in a single-stage mode.
***These pumps could be modified by valving and piping changes to operate in a single-stage mode.

Economic Analysis of Potential Impact of Watersaving Toilets*

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BACKGROUND

Many predictions have been made for water savings through use of residential water-conserving devices, but few authors have been able to verify these reductions. Examples of such studies include the Cabin John Drainage Basin water-saving education and appliance test program of the Washington Suburban Sanitary Commission in 1973,[1] the Marin County, California, water-saving device retrofit program of the Marin Municipal Water District in 1976,[2,3] and the Elmhurst, Illinois and Ann Arbor, Michigan programs as reported by Meyers, et al.[4] The Cabin John study found increases as well as decreases in water use during the water conservation program as compared to a comparable period of the previous year. The Marin County program found reductions of greater than 60% of normal consumption under drought emergency conditions. The Elmhurst study reported that water consumption was reduced by 15% and wastewater flows by 10%. In the Ann Arbor program the average winter water demand was reported to decrease by 3-4%. None of these studies separated habit change from device change, and none statistically verified the results.

Economic savings to homeowners and utilities have similarly been predicted for areawide and nationwide conservation, but studies of the economic inpact on localities are scarce. The potential for energy savings for residences through the use of flow-reducing devices was estimated by Muller.[5] He also concluded that significant energy savings could be found at water and wastewater treatment plants, but no investigation was conducted. Johnson recognized the impending energy crisis, and spoke of its relationship to water supplies.[6] Baker, et al.[7] evaluated the effect of water conservation on the residence as well as the treatment plant. However, he exaggerated the impact since many costs at the wastewater plant including energy requirements, are not very sensitive to changes in water flow. Konen[8] also developed a procedure for evaluating the energy consumption for a utility. However, his formula assumed constant unit costs and a uniform relationship between energy consumption and wastewater flows, which is in error.

Recently Tiemens and Graham[9] evaluated the money savings to a household with retrofit or new construction. However, they too ignored the effect of large-scale conservation on the unit prices for water and wastewater treatment.

*Acknowledgments: The authors wish to acknowledge the data collection and analysis that was performed by Michael Moulds, student assistant. They also thank William Morris of the York Water Company and Thomas Neel, James Crooks, and David Kelly of Springettsbury Township for supplying the data for the study. The funding for the study was provided by the Ford Foundation through the Environmental Policy Center, University Park, Pennsylvania.

They predicted that a retrofitted household would save $54/year, and a new household using water-conserving devices would save $96/year. Approximately one-half of this estimate was energy savings which would not be affected even if the unit cost of water should go up drastically. The potential nationwide savings from a total residential conservation program between 1978 and 1990 is to amount to $836 million on capital costs for plants and $1,045 million on pipe costs. The energy savings for the same period was estimated at $19,500 million. The sum total results in a staggering $23.7 billion savings, and it only includes the estimated fraction of new construction that would be affected by the flow reduction.

Lounsburg, et al.[10] estimated future energy requirements of municipal water supplies, sewage treatment, and irrigation for six major cities and three regions of the Southwest. They projected that regional differences in water supply were very large, significant increases were likely for the energy required to treat sewage, and the Southwest would face an extremely difficult choice in balancing energy, water, and agricultural land resources after the year 2000. At least as early as 1974, persons forecast the mammoth demands of the national energy requirements for wastewater treatment. Shortages were predicted for the future, but they are here now.[11] The cost of energy today is three times that of 1974.

Conservation is an approach for alleviating these problems. The literature reveals several areas of water conservation policy and technology that need study. This article addresses the potential flow reductions and economic benefits of energy and chemical savings in water conservation. A companion study addresses the social ramifications of the devices in homes and communities.

STUDY PLAN AND OBJECTIVES

Erb and Fabian[12] summarized the limited data on water conservation in three Pennsylvania areas: Elizabethtown Borough, the city of Gettysburg, and Springettsbury Township. Two other locations in Pennsylvania - Mt. Joy Borough and Seven Springs Borough - have also utilized water conservation fixtures. Mt. Joy had six demonstration homes which were investigated. Of these groups only Springettsbury Township has carried-out a water conservation program to a degree sufficient to warrant evaluation. Springettsbury Township encouraged people to install water-saving toilets and water flow restrictors in shower heads.

The existence of communities already involved in water conservation practices provides a strategic research area in which to assess reactions to the use of flow-restriction devices, water conservation attitudes and behaviors, and chemical, energy, and dollar savings. A study of the savings in chemicals and energy, and their impact on water bills was the major objective of this study. Actual reductions in water consumption were investigated using a matched set of residences, those that had watersaving toilets, and those that did not. The energy and chemical usage reductions were then projected based upon an assumed level of adoption of the water-saving toilets.

RESULTS

Water Savings -

Watersaving Toilets

The bimonthly results of a matched set of houses having watersaving toilets (3.5 gal/flush) versus non-watersaving toilets are shown in Table I. For each group the number of people and gallons consumed in the period were used to calculate a gallons per capita per day (gpcd) for each period. The watersaving toilet group consistently used less water then the non-watersaving toilet group up to 21% reduction with the exception of the November-December period. Analysis of the individual houses with watersavers indicated that only a few houses caused the abrupt increase in the November-December period. However, conversations with owners did not pinpoint a specific reason for the increase. No guests or additional family were home for the holidays. Figure 1 possibly better shows the impact of the watersaving toilets.

Statistical analysis of the water consumption data for the 14 month period beginning January 1978 verified that there was indeed a significant reduction in water consumption caused by the watersaving toilets. Figure 2 is a histogram of the per capita consumption for each household for each billing period. The result for each set of data is somewhat skewed toward the low end. However, it was felt that statistical probabilities based upon a normal distribution would still be applicable. Table II summarizes the results.

When a normal distribution is assumed, a confidence level may be determined using the Students T-test. Use of this technique indicated that at the 92% confidence level the two matched groups are distinctly separate populations. The difference between the means was 4.9 gpcd.

Why was this reduction, 7%, so small? A reduction of 15% was expected. It has been assumed in the past that the amount of water consumed per flush by a standard toilet was 5 to 6 gallons. This number has been questioned by several of the plumbing equipment manufacturers, who believe it to be more nearly 4.5 gal/flush. In addition many of the new "3.5 gal/flush" watersaving toilets actually consume 3.6-3.7 gal/flush. The toilet usage has been found to be 40%[13] of total domestic water usage. This 40% reduction applied to the reduced toilet usage flow, 4.5 minus 3.7 gallons, results in a reduction of 7.11% not 15%. Therefore the actual measured reduction for new homes built in the past two years containing watersaving instead of non-watersaving toilets is reasonable.

Retrofit

Water conservation by use of retrofit devices such as flow-restricting shower heads and inserts, and toilet-daming devices was investigated. Within Springettsbury Township Pennsylvania are several apartment complexes. Several of these have taken watersaving measures, but one had an extensive program. Toilet dam devices were installed in a complex of some 300 units two years previous to our survey in 1978. A sample of 20 of the units indicated that only 2 were yet functional (10%). The others had either been removed, or were lying in the bottom of the toilet tank. If this sample is considered

Table II, Statistical Analysis of Water Savings

Parameter	Water Saving Toilets	Non Water Saving Toilets
sample size, n	200	189
mean, $\bar{y}$,gpcd	65.8	70.7
standard deviation, σ_x	27.2	25.9
standard deviation of the mean, $\sigma_{\bar{x}}$	1.92	1.89
variance of the mean $\sigma_{\bar{x}}^2$	3.69	3.57

the deviation, D = 70.7-65.8 = 4.9 gpcd or a 6.9% reduction

representative of the retrofit program, it could be considered a failure!

Mt. Joy Borough in Lancaster County, Pennsylvania, instituted a modest watersaving retrofit demonstration in 1976. Six residences with varying property values were selected. Toilet dams, shower head inserts and faucet aerators were installed. During the study one family moved. Water usage for only five families was used to determine the gpcd for two years previous to installation and two years after installation. Table III summarizes the the results and indicates that there was no reduction in water usage. In fact an increase occurred even though the devices remained in use and no changes were made in water using appliances or family number. Comparison of water use by month and season also did not show any reduction. It was concluded that these devices on this small sample produced no verifiable water savings and the increase in usage with time was due to changing life styles.

Economic Analysis -

Water Supply: Short Term Impact

The majority of Springettsbury Township, York County, Pennsylvania is supplied by the York Water Company. An analysis of the potential impact of watersaving was made for 1978. It was assumed that all the residential users of water had reduced their consumption by 7%. Operational costs for this surface water system that could be related linearly to water usage were electric power (primarly pumping), chlorine, alum, lime, ammonia, permanganate and carbon. The total cost of these items for the year was $368,057. A 7% saving on the domestic water usage would result in $13,603 saving. (52.8% of the water usage is domestic). The total cost for the 7,002,273 thousand gallons water processed in 1978 was $.5172/thousand gallons or $3,621,575. Other operation and maintenance (O & M) costs and amortization overshadow the small savings. The savings possible to the system if 7% less residential water was used are only 0.375% of total costs.

Further investigation was necessary to determine if the 123,555 residents would actually benefit from any of these savings. The York Water Company has

Table III, Water Conservation Retrofit Demonstration, Mt. Joy Borough, Penna.

Family	Devices Installed	Average Consumption, gpcd	
		Condition prior to Water Conservation (2 years)	During Water Conservation (2 years)
A	Flow-limiting shower head Faucet aerator Toilet tank dam	40.5	42.5
B	Shower head insert Faucet aerator Toilet tank dam	59.8	64.4
C	Flow-limiting shower head Faucet aerator Water saving toilet float assembly	55.7	63.0
D	Shower head insert Faucet aerator Toilet tank dam	39.0	41.1
E	Shower head insert Faucet aerator Toilet tank dam (one dam removed) (one still in place)	59.1	60.4

a decreasing block rate structure subject to minimum charges that are based on 2,300 gal. per month. In addition there was a surcharge that raised the minimum bill to $5.90 per month for 1979 (Table IV). Assuming that there were 3.5 persons per residence and a consumption of 66.3 gpcd[14]* the consumption per household with no water conserving fixtures would be 7,058 gal. per two month billing period. The water bill for this residence would be $14.38 excluding the surcharge. If 7% of the water usage were eliminated through waterconserving devices such as the toilet, the bill would be $13.75. In this situation water conservation would save less than $1. Thus it is not cost-effective for a resident to replace two toilets at a probable installed cost of $200 to save a maximum of $7.56/year. If one assumes that the usable life of a toilet is twenty years, a uniform savings of $7.56 per year may be compared with the value of the toilet by present worth of (PW) analysis. The PW of $7.56 per year (P/A, 6%, 20) is $86.71. This compares with a PW of the toilets of $200 and makes retrofit with new water saving toilets very unfavorable at present rates. However up to $86.71 could be spent today for water conservation yielding 7% reduction and still be favorable to the homeowner. If the cost of water increases rapidly as may well happen, the situation is

*Consumption is calculated by dividing the total number of persons into the sum of the residential metered, flat rate and unaccounted water usage.

Table IV, York Water Company Output Charges

Volume	Rate per 1,000 gallons Gravity	Repumped*
First 5,000 gal. per month	$1.784	$2.347
Next 45,000	.779	1.284
Next 150,000	.504	.773
Next 800,000	.479	.642
All over 1,000,000	.426	.551

more favorable for the replacement with water-conserving toilets. However, it will be difficult even in the best situation to convince homeowners to remove functional conventional toilets and replace them with new water-conserving ones.

If widespread water conservation were to occur the rate charge must go up to account for the fixed costs that are relatively inelastic with respect to volume. It is difficult to estimate this impact with a decreasing block rate structure, but an estimate may be made by investigating the effects on the average cost of water per thousand gallons. The overall average cost of water in 1978 was $.5172. per thousand gallons based on the 7,002,273 thousand gallons that were consumed. If residential water users (52.8% of total consumption) used 7% less water the cost of treatment would drop by $13,603, but the water volume would drop to 6,743,469 thousand gallons. The result would be an increased unit cost of $.535 per thousand gallons. Applying these rates to a residence having 3.5 persons using 66.3 gpcd, the cost per year is $43.80 before conservation and only $42.14 after conservation.** This savings of $1.66 is insignificant. If only a few residences would utilize devices in reducing their annual water usage by 7% the rate would probably not change and the charge would be $40.74 with a greater savings. Alternatively if a residence did not conserve water or already had been operating at a minimum when mass water conservation was instituted its costs would increase to $45.31 because of the higher unit rate. (They would also be very unhappy and puzzled about the impact of water conservation)!

Water Supply: Long Term Impact

Water conservation has a long-term as well as short term impact, in fact the long-term impact will be much greater. York Water Company has adequate supply and treatment facilities for the next ten years. However, they eventually will develop new supply and will enlarge treatment facilities.

*Residential metered water fits in this category. The monthly residential minimum is 2,300 gal. at a cost of $5.90, including a surcharge.

**These results are neither typical of residential annual costs nor would they be expected to be typical.

The Company provides water to Springettsbury Township, as well as the city of York and other surrounding communities. Present and future supply is from surface waters and will require coagulation and filtration.

An engineering firm for the York Water Company has made detailed studies of future water demand to the year 2020. This demand curve is shown in Figure 3 together with other curves that depict the effects of water conservation. The maximum-day curve is 1.45 times the average day. Each set of average-day and maximum-day curves represent three situations where: (1) no water conservation was assumed, (2) water conserving toilets were to be installed in all new residential construction after January 1980. This action would result in 7% new water savings and a smaller rate of increase after 1980. (3) water conserving toilets were to be installed in all new residential construction and retrofitted into all existing residential construction. This action would result in an abrupt 7% reduction on existing residential water usage and a smaller rate of increase after 1980.

The reduction assumed was only 7%, a decrease observed in the study on Springettsbury Township. (Larger reductions are possible if water-conserving shower heads and clothes washers and an education program are used in addition to watersaving toilets).

A present worth analysis was made for the two alternative water conservation proposals with the status quo of no water conservation. This analysis takes into account three levels of expenditures and annual savings. Residential water-conserving toilets are able to lower the projected future demand for water, and postpone the date for needed construction of new supply and treatment facilities, as listed in Table V. Annual operating costs are less because of the reduced chemical and energy consumption. Figure 4 summarizes the operating cost results for water conservation on all residences and on only new residences.

Table V, Projected Construction Needs for York Water Company*

Facility*,[15]	Cost*,[15] 1980$	Date Construction Required		
		No*,[15] Conservation	7% Conservation on New Residences after January 1980	7% Conservation all Residency after January 1980
Bascule Gates on Lake Redman	1,254,000	1991	1991	1994
Filter Plant Expansion	1,254,000	1995	1996	1998
Reservoir No. 3	7,777,280	1996	1996	1999

*Adapted from data in "Source of Supply and Facility Study, The York Water Company, York, PA - 1975" Alvord, Burdick and Howson, Engineers - Chicago

Present worth calculations were made for the alternatives shown in Table VI. An interest rate of 10% was used since it was the typical value that was used by utilities. By comparing the PW of each alternative it appeared that up to $45,008 could be spent on a water conservation program in 1980 for new residential construction, and still a net savings to the area served by York Water Company would occur. Up to $795,166 could be spent for a water conservation program for retrofit and new construction and a net savings still could be realized. Inflation however, clouds the issue of PW. Postponement of construction may actually gain nothing since inflation may consume PW benefits. On the other hand cost of energy and chemicals will be increasing at least as much as the cost of construction.

All things considered, use of watersaving devices on new or replacement construction still makes good sense. The PW of the savings in operating costs alone are attractive. For new residences it shows that a maximum of $17,662 could be spent to institute the program. For combined retrofit and new residences a maximum of $190,395 could be spent to institute the program. Even though these savings are calculated for a special site, comparable savings are predictable for similar communities.

Table VI, Comparison of Present Worth of Water Saving*

Investment $	Situation		
	No Conservation	7% Conservation on New Residences after January 1980	7% Conservation on all Residences after January 1980
1,254,400	$439,667 (P/F,10%11)	$439,667 (P/F,10%,11)	$330,284 (P/F,10%.14)
1,254,400	300,303 (P/F,10%,15)	272,957 (P/F,10%,16)	225,666 (P/F,10%,18)
7,777,280	1,692,336 (P/F,10%,16)	1,692,336 (P/F,10%,16)	1,271,585 (P/F,10%,19)
Annual** Operating Cost Saving	0	-17,662 (P/F,10%,0-20)	-190,395 (P/F,10%,0-20)
Total	2,432,306	2,387,298	1,637,140

SUMMARY AND CONCLUSIONS

Water and energy shortages make conservation an attractive alternative for our future water policies. Devices will play a very important role in residential conservation since most appear cost-effective, and they do not generally require change in consumer habits. A matched set of residences in Pennsylvania studied over one and one-half years statistically verified at a 92% confidence level that a 6.9% reduction in water usage resulted from use of watersaving toilets. No conscious effort was made by one group over

*20 year period, 1980 $ **From Figure 4 for annual savings

the other to consererve water so consumer attitudes and habits did not account for the differences.

Two toilet retrofit installations were studied. The devices in a apartment complex were found to be functioning in only 10% of the units. In six demonstration homes no reductions in water usage were found.

A projection of 7% residential water savings to all residential housing served by York Water Company resulted in an annual saving in chemical and electrical usage of only $13,603 in 1978 or .375% of the total cost of the water consumed by 123,555 persons. If a limited number of customers adopted watersaving toilets or other devices so that they reduced their usage by 7% they would save $7.56 on their annual water bill. It is not cost-effective to replace their toilets at a cost of $200 to save $7.56. If wide-spread water conservation were to occur the unit cost of water probably would have risen and the savings would have been negligible. If a residence did not participate in the watersaving or already was operating at a minimum when the wide-spread conservation occurred its charge would increase and the user would be very unhappy and puzzled about the impact of water conservation.

Water conservation has, however, a much larger impact if it is considered over a longer period of time. Capital expenditures for expanding the York Water Company's treatment and supply system can be postponed varying lengths of time, based on the degree of conservation. Seven percent conservation for all new residences after January 1980 results in only a slight deferment of construction but 7% conservation by retrofit as well as on new residential construction results in a deferment of construction by approximately three years. The present worths (PW) were compared for the three alternates: no conservation, new residences and new plus retrofitted residences. Based upon the PW analysis, up to $45,008 for conservation in new residences and up to $795,166 for conservation in new plus retrofitted residences could be spent for water conservation and still there would be a savings to society.

It appears that for Springettsbury Township, water conserving toilets in new residences are playing an important role in reducing long-term water usage. Toilet device inserts used for retrofit could not be evaluated because of the poor maintenance and performance. Overall a policy requiring use of waterconserving toilets on new construction appears valid.

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Table 1. Water Consumption for Matched Set of Households

Period Ending	Water-Savers			Non-Water-Savers			Per Cent Reduction
	Number of Residences (people)	Gallons Used	Gal. Per Cap.-Day (gpcd)	Number of Residences (people)	Gallons Used	Gal. Per Cap.-Day (gpcd)	
2/78	16 (52)	174,300	56.8	25 (100)	423,100	71.7	21
4/78	26 (85)	272,200	52.5	26 (103)	359,500	57.2	8
6/78	28 (43)	361,300	63.7	26 (103)	432,100	68.8	7
8/18	31 (107)	534,700	80.6	28 (110)	564,800	82.8	3
10/78	33 (116)	394,200	55.7	28 (110)	400,800	59.7	7
12/78	33 (116)	515,800	72.9	28 (110)	476,000	70.9	-3
2/79	33 (116)	432,400	63.2	28 (110)	444,700	68.5	8
4/79	33 (116)	395,800	55.9	28 (110)	396,900	59.2	6
6/79	33 (116)	430,400	60.8	28 (110)	447,900	66.8	9

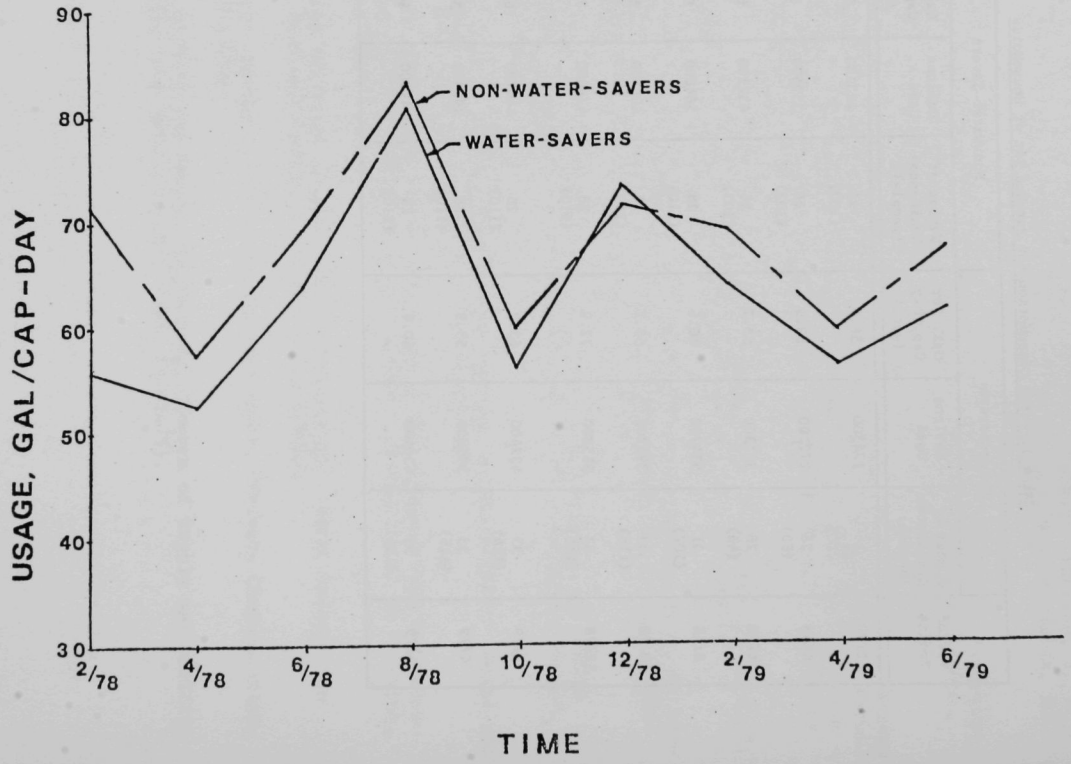

Figure 1, Water Consumption for Matched Set of Households

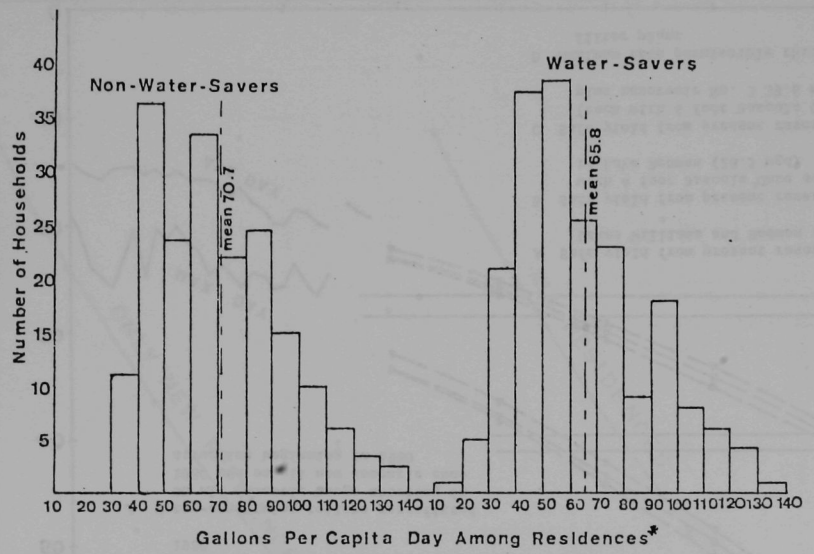

Figure 2, Water Consumption Among Residences*

*1978 values

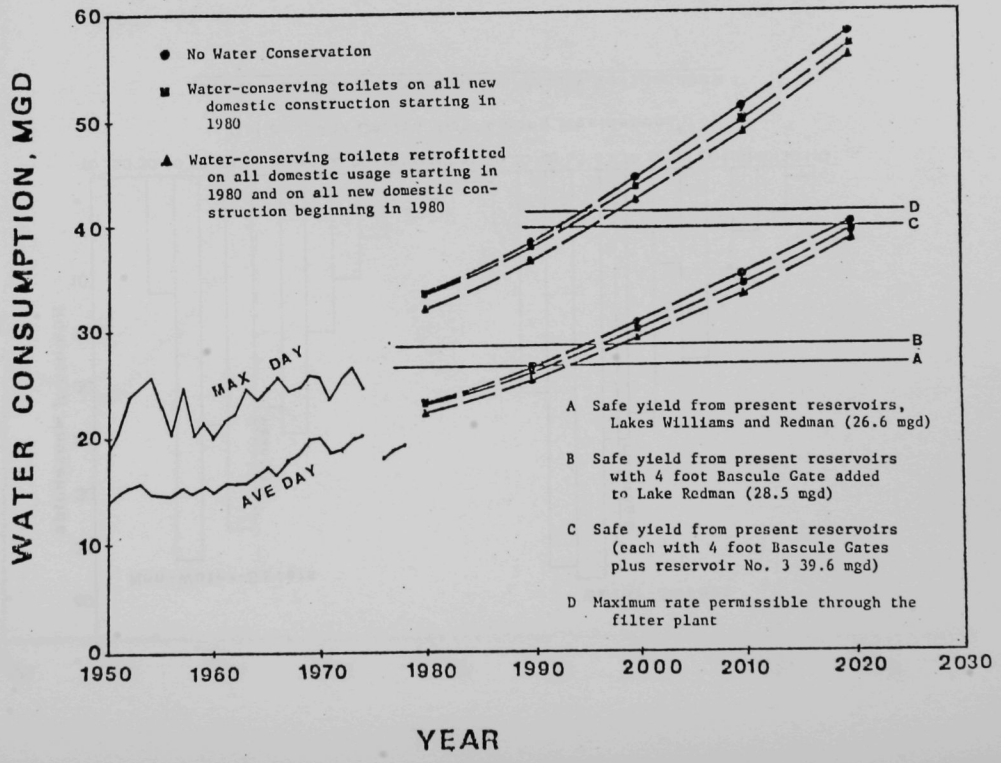

Figure 3, Projection of Water Consumption for Area Supplied by York Water Company*

*Reference No. 15

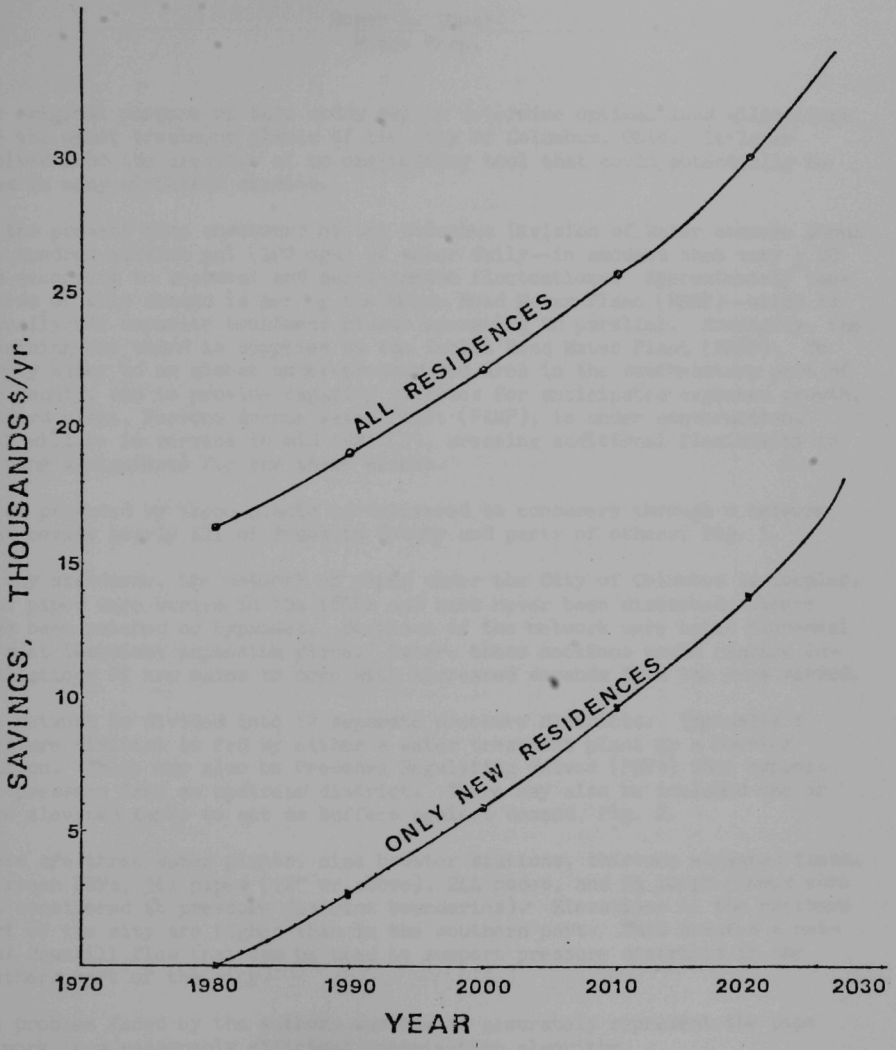

Figure 4, Savings for Operational Costs for New and Retrofit plus New Water Conservation Beginning 1980

OPTIMIZATION OF WATER DEMAND FROM THREE PLANTS

Samuel E. Barnes
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Roger L. Dumas
Mitre Corp.

The original purpose of this study was to determine optimal load allocations for the water treatment plants of the City of Columbus, Ohio. It later evolved into the creation of an engineering tool that could potentially be used in many different studies.

At the present time customers of the Columbus Division of Water consume about one hundred million gal (100 mgd) of water daily--in amounts that vary ± 20 mgd according to seasonal and quasi-random fluctuations. Approximately two-thirds of this demand is met by the Morse Road Water Plant (MRWP)--which is actually two separate treatment plants operating in parallel. Nominally, the remaining one third is supplied by the Dublin Road Water Plant (DRWP). To supply water to an almost under-pressurized area in the southeastern part of the county, and to provide capacity reserves for anticipated expected growth, a third plant, Parsons Avenue Water Plant (PAWP), is under construction. PAWP will be in service in mid 1981 (1), creating additional flexibility in pumping assignments for the three plants.

Water produced by these plants is delivered to consumers through a network that covers nearly all of Franklin County and parts of others, Fig. 1.

By any standards, the network of pipes under the City of Columbus is complex. Some pipes were buried in the 1800s and have never been disturbed; others have been patched or bypassed. Sections of the network were built piecemeal to meet immediate expansion plans. Later, these sections would require installations of new mains to cope with increased demands from the area served.

The network is divided into 12 separate pressure districts. Typically a pressure district is fed by either a water treatment plant or a booster station. There may also be Pressure Regulating Valves (PRVs) that support the pressure from an upstream district. There may also be included one or more elevated tanks to act as buffers against demand, Fig. 2.

There are three water plants, nine booster stations, thirteen elevated tanks, thirteen PRVs, 361 pipes (12" or above), 244 nodes, and 94 loops (loops were not considered at pressure district boundaries). Elevations in the northern part of the city are higher than in the southern part. This creates a natural downhill flow that can be used to support pressure districts in the southern part of the city.

The problem faced by the authors was how to accurately represent the pipe network in a reasonably efficient optimization algorithm.

The primary restrictions imposed on those who would tamper with (even via a model) a municipal water system are in assuring that all the demands are met and adequate pressure maintained. This can only be accomplished by accurate-

ly modelling the entire distribution system. Once this model has been built a system of constraints must be satisfied to produce a feasible solution.

$$\sum_{\text{ie inflow}} Q_i - \sum_{\text{ie outflow}} Q_i + Q_{N_k} = 0, \text{ for all nodes } k$$

$$H_k \geq H_{min}, \text{ for all nodes } k$$

where Q_{N_k} = demand at node k

$Q_i = 135.406 \cdot C_{HW_i} \cdot D_i^{2.63} \cdot \Delta H_i^{0.54} / L_i^{0.54}$ = flow through pipe i

H_k = pressure at node k

H_{min} = minimum allowable pressure

ΔH_i = pressure loss in pipe i

L_i = length of pipe i

C_{HW_i} = Hazen-Williams coefficient for pipe i

D_i = diameter of pipe i

PREVIOUS WORK

Problems involving water distribution systems have evolved in two directions. First of all, quite a bit of work has been done in the area of system design. These problems, however, have been limited in size and complexity. They are all concerned with designing systems that are almost entirely new. Their approaches would be extremely inefficient for handling large existing systems. They are discussed briefly to provide a background for someone who might be interested in water distribution system design.

Watanatada (2) used an efficient Davidson-Fletcher-Powell Method to solve the nonlinear programming problem by incorporating the previously mentioned constraint set into the objective function by means of a penalty function. He reports that his method has been applied to a system with 54 nodes, 52 new pipes and 22 existing pipes. Jacoby (3) presented a cumbersome numerical gradient technique that was applied to a system of seven pipes with two loops and five branches. Tong, et al. (4) used a method of equivalent pipe lengths that was later discredited by Khanna, et al. (5). Deb, et al. (6) used a method of equivalent diameters. All of these methods solve design problems with theoretical pipe diameters that are later rounded to commercially available diameters. In a complex system, however, there could be many combinations of commercially available diameters that would have to be checked if the true optimal solution is to be found.

Attempts to solve the design problem assuring commercially available diameters have been made by several authors. Liang, et al. (7) used dynamic programming on a system with no loops. Lam (8) used discrete gradient techniques while Artina (9) tried linear programming. Cenedese, et al. (10) presented a design algorithm that starts with an open network and adds connections until the system is complete. The computer times are described as being acceptable even for large systems but the pipe network presented is still relatively simple. It is based on the Cross Method of solving the flow

and energy equations for a pipe network. This method is not known to be rapidly convergent on large systems.

The other approach is that of the practitioner. Using commercially available programs an engineer can set up a pipe network as he believes it should be and solve the system equations to determine the flows and the pressures. The advantages of this approach are obvious. The user can solve large systems efficiently. He can interface with existing pipe networks easily. The "canned" routine will have special features that have been perfected with years of experience. Finally, the user doesn't need advanced coursework in optimization techniques. Although lacking in elegance, this approach is effective in solving problems where total enumeration is feasible.

BUILDING THE MODEL

For our problem, the decision was made to first develop a model that accurately represented the distribution system. This model could then be used in an organized search procedure to identify the optimal load allocations for the treatment plants. If accurate enough, the model could be used as an engineering tool for system studies. Also, by using an organized search, an optimal or near optimal solution may be identified. The word "optimal" looses some of its meaning in a system of this size because of all the inherent inaccuracies in estimating parameters. A good solution may be identified by the procedure but the feasibility must be established by empirical testing.

Before the model was put together some preliminary analyses had to be performed. The frequencies of tank elevation perturbations had to be studied to determine the average values and ranges of the elevations. It was also necessary to estimate the percent of time that the tank was being filled or emptied. The Latin Square design in Reference 11 was used to sample tank elevations over a 5-week period. With this design we were able to evaluate the time-of-day and day-of-the-week effects on the tank levels. It was found that the tank levels in the pressure districts fed by booster stations did not show a time-of-the-day effect or a day-of-the-week effect, whereas the tanks that were fed directly by water treatment plants did show a time-of-the-day effect but not a day-of-the-week effect. This information was useful in determining the blocks of time for the study days and establishing ranges for sensitivity analysis.

Since there was no permanent record of PRV settings available, tank charts were used to approximate some PRV settings. The tank levels in the smaller pressure districts seemed to stabalize in the early morning hours. With no pumps on, it had to be assumed that these levels were being supported by PRVs.

Booster stations in the system (with a few exceptions) have a number of similar pumps in parallel. They are centrifugal pumps and can vary in number from two to seven. The number of pumps operating at a particular time depends upon the level of the tank in the district. Therefore, if given the operating policy or the booster station one can determine the number of pumps operating by specifying the tank level and whether the tank is rising or falling. Knowing the number of pumps operating can then be used to determine the pump characteristic curve. This is used in the computer program to calculate the flow through the booster station against the system pressure. A

typical pump characteristic curve is presented in Figure 3.

A detailed analysis of pump characteristic curves was not made. Performance Test Curves were used when available and curves from similar pumps were used when the Test data was not available.

The pipe lengths were scaled from the Division of Water map showing all 12" or larger pipes (12). In areas where the pipe configuration was observed a more detailed map put together by Pitometer and Assoc. was consulted. The elevations of the pipe junctions were interpolated from the USGS Contour Map for Franklin County. The map showed 10 ft contours.

A blanket value of 110 was used for the Hazen-Williams coefficient except where the values had been determined by other studies (13). More accurate values would require a detailed analysis of every pipe.

To incorporate this information into a model of the system a computer program for the analysis of pressure and flow in pipe distribution systems by Don J. Wood (14) was modified. The program is extremely efficient even with large systems and included special features that made the modelling much easier.

The pressure districts were set up as individual systems. New routines were added to model the booster station operating policies and to adjust the demands upstream of the booster stations and PRVs. Routines were also added to prohibit flow reversals in the treatment plants and PRVs. The demands at the junctions were also determined in a separate routine as will be described later.

The general operation of the computer model, Fig. 4, uses sparse-matrix subroutines. The districts were ordered such that a district would be analyzed before any districts upstream of it would be analyzed. The pressure districts are connected by booster stations and PRVs. Once the flows through the booster stations and PRVs were determined they were added to the demands at the nodes directly upstream in the next pressure districts. Thus, the program was altered to solve the pipe systems sequentially from downstream to upstream while preserving the relations between the districts. This created other problems that will be discussed later.

A good model of the physical system means very little if the demands at the junctions are not approximated well. The method used to distribute the demands was first seen in system design study performed by Alden E. Stilson and Assoc. (13) on a smaller part of the city. The underlying assumption is that the demands will be distributed around the city as the population and industry is distributed. Traffic-zone forecasts for Franklin County for 1974 and 2000 were compiled by the Mid Ohio Regional Planning Commission (MORPC)(15). The forecasts included projections of residential population, average income, employment breakdowns by type of industry, as well as parameters related to automobiles. These were recorded for each of the 723 traffic zones that cover Franklin County. The total projected population for the year 2000 was 1,025,000 people. This information was placed in a file and used to estimate demands in the traffic zones.

The traffic zones were assigned to nodes in the model. Some information on the breakdown in water consumption by type of customer was available from the

Division of Water Annual Report for 1978. Unfortunately, Div. of Water annual reports give data on booster station pumpage only, and this data does not always correlate with the traffic-zone analysis done by MORPC. A more formal study could be done to relate customer usage to the MORPC parameters. This may provide more accurate demand distributions.

In our model any desired scenario can be determined by inputting the total city demand rate in millions of gallons per day and the percentage of the demand that is to be attributed to each of the MORPC parameters.

After the model was developed and before an optimization procedure could be established some type of a cost model had to be formulated. Since the booster stations are operated independently of the rest of the system, booster station costs were not included. No construction costs are being considered for this study so the only costs are those that vary directly with flows from the water plants. These include the chemical treatment costs and the electrical costs to pump the water (labor costs were not included initially but may be considered if the possibility of closing a water plant for one or more shifts becomes feasible).

Chemical costs can be considered linear with flow for a given day. The chemical costs actually vary with pumpage and with water quality which, in turn, can vary seasonally. Data on the water quality and treatment costs for all of the raw water sources over a three year period was available from a Division of Water, Water Research Lab report (16). These costs were scaled to present day costs.

Electric costs are much more difficult to represent. As mentioned earlier, Performance Test Curves are available for the pumps in the water plants, Fig. 3. Among these are curves of input horsepower vs. flow. These curves are usually slightly concave at their peaks but nearly linear elsewhere. If a linear approximation is used then the overall electric cost model would be piecewise linear. If all of the pumps in the plant were identical then the cost model would be exactly linear. Unfortunately, not all of the pumps are similar. Some have different capacities and some are variable-speed pumps. It is also possible that different numbers of pumps could be putting out the same flow with drastically different power consumption. Another complication is that this model does not represent the variable in-plant consumption (e.g. lighting, air conditioning) or the low service consumption (raw-water pumpage).

Fortunately, there is continuous metering of electric consumption at the Morse Road Water Plant and it will soon be available at the other plants. With continuous metering we are able to study the electric consumption as the meter sees it.

What the electric meter sees is a continuous series of pulses. This series is integrated every half hour to determine the average consumption rate over that period, and it is recorded on computer tape. At the end of the billing period the computer tape is used to print hard copies of the recorded half-hour demands. These demands are sorted by computer to determine the maximum (this helps to calculate the Capacity Demand Charge) and they are added up to determine the total energy consumption for the billing period.

These printouts were used along with finished-water pump charts and Pump Room Logs to statistically analyze the electric consumption with regard to flow-related parameters. Some of the parameters considered were finished-water pumping, raw-water pumping, number of pumps, pressure, time-of-day, day-of-the week. The only statistically significant parameters were finished-water pumping and raw-water pumping. The number of pumps operating was close to being statistically significant but did not add much to the consumption model.

A Latin Square design was used to gather the data to assure accurate estimation of the time-of-day and day-of-the-week effects. The analysis also provided us with an estimate of the variability in the electric consumption. This is particularly important when trying to estimate the Maximum Capacity Demand during peak demand hours.

SOLUTION APPROACH

As mentioned earlier, there are several difficulties with the computer model that was developed. Since the system is solved from downstream to upstream it may happen that the upstream pressures may not be able to support the PRV settings in the downstream district. Similarly the pressure settings upstream of the booster station have to be approximated before the model is run and may not be correct. In either case the pressures have to be reset and the model has to be run again.

A trade-off must be made between the accuracy of the pressure settings and the time and computer costs required to re-run the model.

Every functional evaluation of the simulation is more like an experiment. It takes time and resources to get the results and inaccuracies still exist (although they can be eliminated by enough runs of the program). With this in mind it was decided that Response Surface Methodology (17) could be used to identify an optimal (or near optimal) solution.

Response Surface Methodology is a statistical technique used to analyze problems where a dependent variable (cost) is influenced by several independent variables (water plant pressure settings). Basically it fits a series of first-order linear models to experimental data to get an approximation of the cost surface. A gradient is determined and an inaccurate line search follows. This is continued until the linear model is no longer appropriate. This is determined by studying the analysis of variance for the fitted model. The next step is to fit a second-order model to take advantage of any curvature in the cost surface near the optimal solution. This is continued until the optimal solution is found.

SUMMARY

A model of the system has been built, an approach to the solution been defined and the process has begun. It had been hoped that results would be available at this writing, but the process requires a lot of user interaction and is very time consuming. Nevertheless, a model of the system exists and can be used in system design studies as well as load allocation studies.

It is hoped that this model can become part of a continuing user-interactive

algorithm to help the distribution network evolve in the future into a more controlled and efficient system.

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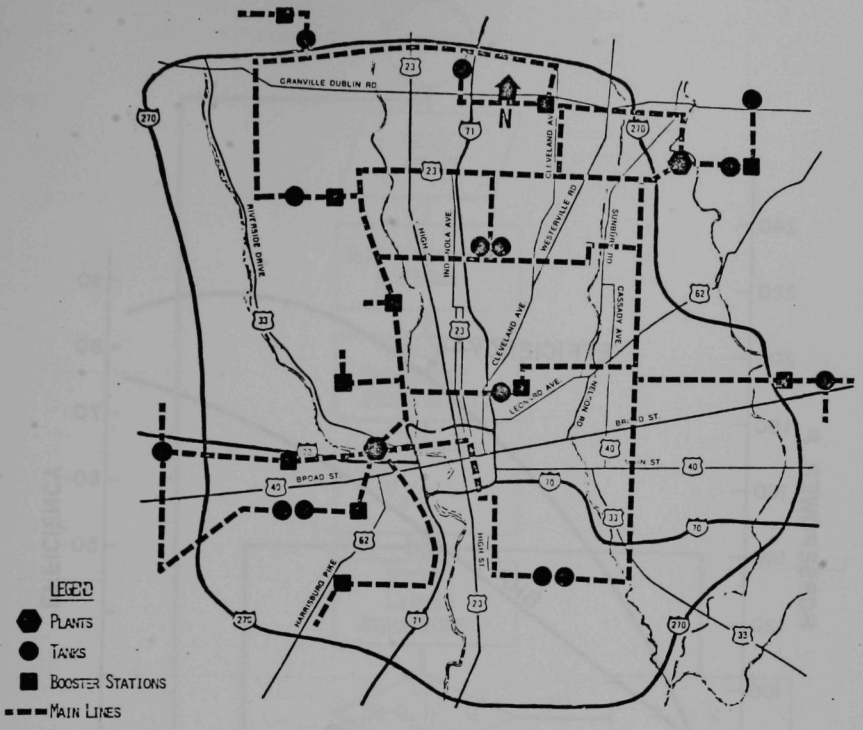


Fig. 1 — Rough schematic of Columbus, Ohio, distribution system. Of the 2200 miles of pipeline, about 500 miles (22%) are 12-in. diameter or larger. Dashed lines shown are for 24-in. and larger.

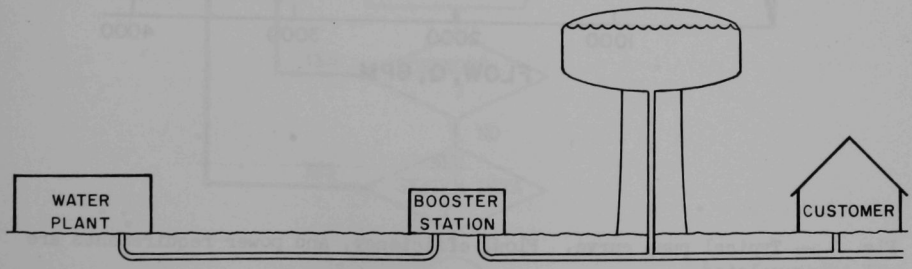


Fig. 2 --Schematic of typical distrubition system elements.

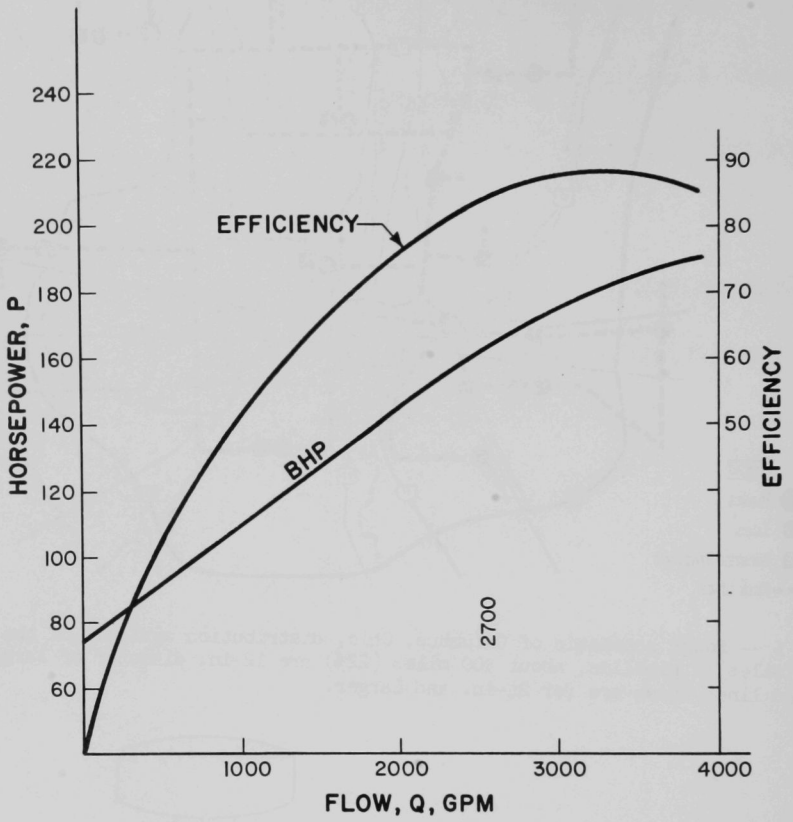


Fig. 3 -- Typical pump curve. Flow, efficiency, and power requirements are all inter-related.

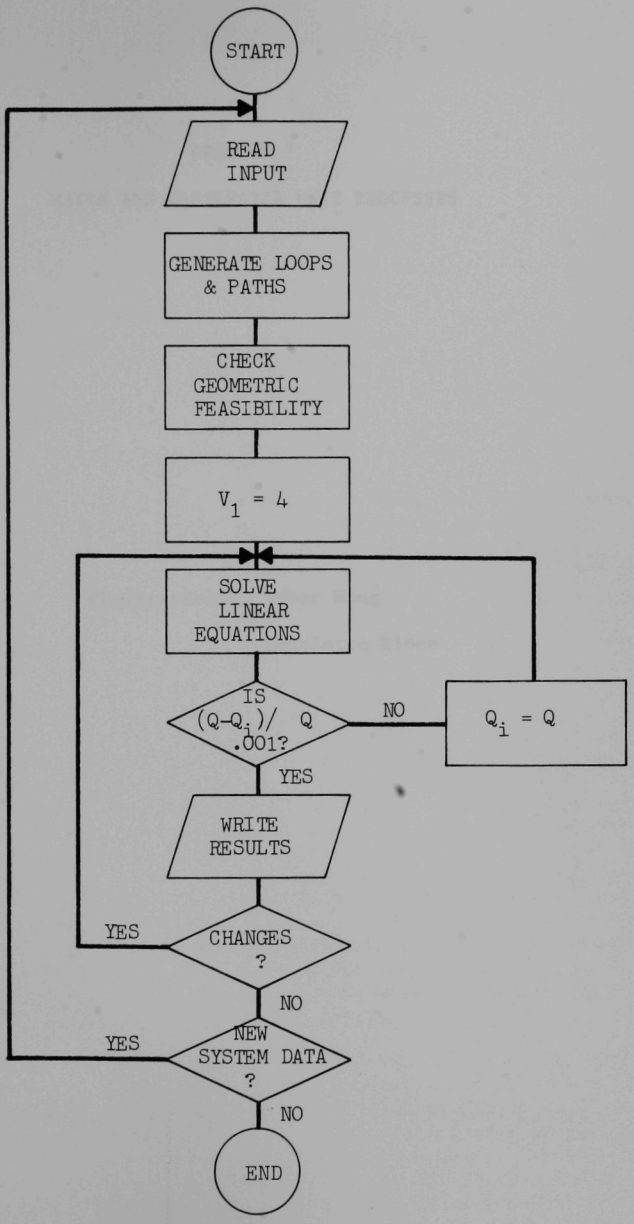
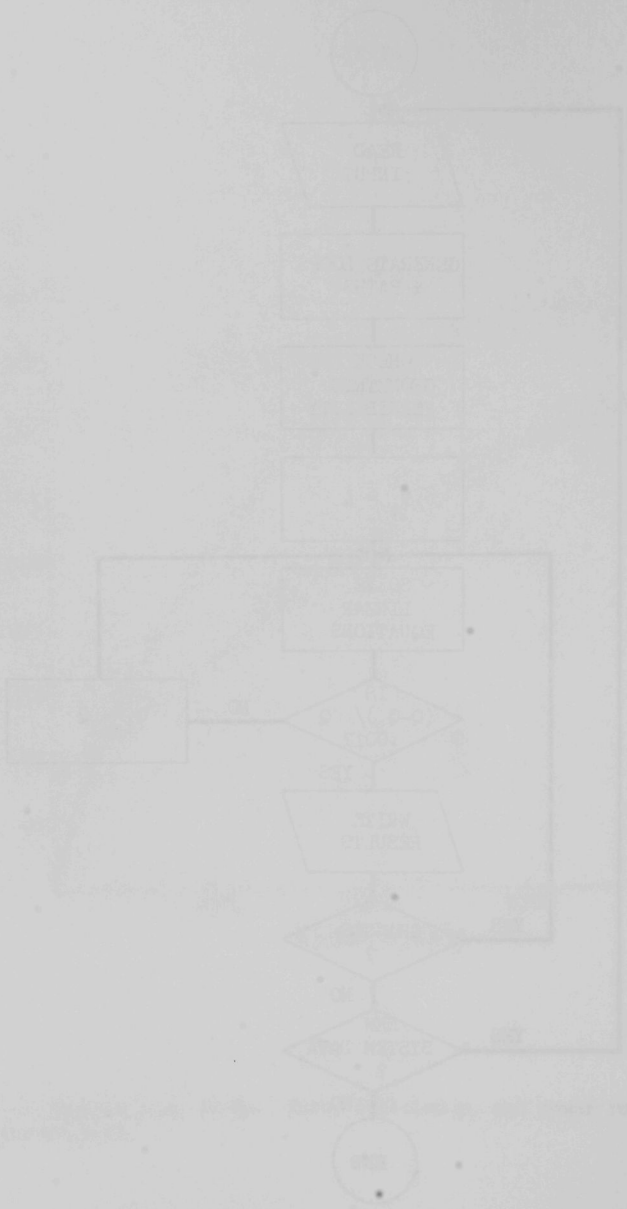


Fig. 4 -- Flow diagram of the Kentucky analysis program. Input data is parameter values for each pipe with pressure and flow specifications. Geometric check is to ensure that system is feasible (not disconnected).



SESSION 7

WATER AND WASTEWATER UNIT PROCESSES

Chairpersons: Amber Wong

Charlotte Rines

ENERGY CONSUMPTION BY THE ROTATING BIOLOGICAL CONTACTOR WITH COMPARISONS TO ALTERNATIVE TREATMENT TECHNOLOGY

Ronald L. Antonie
Autotrol Corporation

ROTATING BIOLOGICAL CONTACTOR (RBC) PROCESS

The rotating biological contactor is a fixed-film biological treatment process using corrugated plastic media which is slowly rotated while half-immersed in wastewater. See Figure 1. Alternate contact of the attached biomass with wastewater and exposure to air promotes aerobic removal of organic pollutants. Although RBC technology has been in commercial use in the United States for only ten years, there are now over 300 installations for treating many industrial wastes and a wide range of domestic waste applications including plant sizes ranging from small package plants up to 40-50 mgd municipal installations.

ENERGY CONSUMPTION BY THE RBC PROCESS

One of the major reasons for the rapid growth in use of the RBC process is its low energy consumption. Low energy consumption by the RBC process results from reduced requirements for oxygen transfer and for mixing. This can be seen in the process flow diagram in Figure 2. Wastewater from primary treatment flows through a multi-stage RBC system where it undergoes a progressively increasing degree of treatment. The treated wastewater and stripped, excess biological solids flow into a secondary clarifier where they are separated. The effluent is directly discharged and the settled solids are sent directly to solids disposal. The RBC system operates on a "once-through" flow basis with no effluent or sludge recycle. The absence of sludge recycle has a significant benefit for the RBC process when compared to alternative treatment systems with sludge recycle. The principle mechanism of substrate removal in the RBC process is diffusion of soluble substrate into the bio-film where it is metabolized. Suspended organic matter is bio-flocculated in the RBC process and settled in the final clarifier. It is then sent directly to disposal without exerting a significant oxygen demand in the treatment process. Thus, the RBC process requires oxygen only for treatment of the soluble BOD in the wastewater. For systems with sludge recycle, suspended organic matter is also flocculated and captured in the final clarifier. However, it is also recycled with the balance of the settled sludge and remains in the treatment process for an extended period of time (three days or more) where it is hydrolized and metabolized and thus exerts most of its oxygen demand in the treatment process.

This difference in process operation would indicate that the volatile solids content of waste sludge from the RBC process would be greater than that from a process with sludge recycle. This characteristic is an advantage when the waste solids are sent to an anaerobic digester because there is a greater potential for gas generation and energy recovery. When using aerobic digestion there is no disadvantage for the RBC process because the long solids retention times required of 15-20 days usually result in total energy requirements for mixing which equal or exceed those for oxygen transfer.

Because the RBC process maintains a large inventory of fixed biological solids on the media, it is able to achieve high degrees of treatment for relatively short wastewater retention times. Typical retention times for BOD removal on domestic waste range from 60-90 minutes, compared to a range of 4-6 hours for conventional activated sludge systems and 24 hours for extended aeration systems such as the oxidation ditch. In addition to significantly reduced reactor volumes, the RBC process contains only approximately 100 mg/l of suspended solids compared to 2,000-3,000 mb/l for activated sludge systems. These two factors significantly reduce mixing energy requirements for the RBC process.

During the past two years, power consumption measurements have been made on full scale operating RBC plants using polyphase wattmeters. These independently certified power consumption measurements show that at typical operating conditions for secondary treatment of domestic wastewater, RBC power consumption is 2kw per 100,000 sq. ft. of media surface area. Because RBC power consumption is principally a function of the drag characteristics of the media design, different media configurations will have different energy consumptions and this must be carefully evaluated when making energy consumption comparisons.

Comparison of energy consumption of the RBC with alternative technology will be done utilizing indicated energy consumption for other processes taken from EPA Publication MCD-32. In this publication a hypothetical municipal waste application is used with wastewater characteristics indicated in Table 1. Because the RBC process is designed on the basis of soluble BOD removal, the appropriate soluble BOD values are also indicated. Design loading rates to achieve three different levels of treatment are listed in Table 2. These loading rates have been developed on the basis of operating data from full scale RBC plants which have been in operation since 1974 using the same media design as the plants where field power measurements were taken. Table 2 indicates the power consumption for the RBC process to achieve the three different levels of treatment.

COMPARISON WITH ALTERNATIVE TREATMENT TECHNOLOGY

Using EPA Publication MCD-32, power consumption for alternative biological treatment technology is shown in Tables 3 and 4. The indicated power consumption for rock trickling filters is approximately the same as that for the RBC process however, construction costs for rock trickling filter systems is quite high and they are also subject to significant losses of treatment efficiency during cold weather operation. These factors, together with odor problems and filter fly problems have resulted in significantly decreased use of the rock trickling filter. Plastic media trickling filters have reduced some of the problems associated with odors, flies, and heat loss, but because of the greater height of the filter media the power consumption is increased significantly. Also, increased recirculation requirements add to the power consumption so that the requirement indicated in Table 3 for the plastic media trickling filters is two to three times that of the RBC process for equivalent effluent BOD values. In general, trickling filters are not capable of producing high effluent qualities for acceptable capital cost expenditures and they have not shown to be cost effective for today's more stringent treatment standards. As an attempt to improve the

overall treatment efficiency of the trickling filter, the combination of a redwood media filter with a subsequent activated sludge system has been developed. It is able to produce the high quality effluents but power consumption is approximately two to three times that of the RBC process.

Power consumption for the activated sludge process utilizing various aeration techniques is shown in Table 4. Again, the power values are taken from EPA Publication MCD-32. The differences in energy requirements for the different methods of aeration are due principally to the efficiency of oxygen transfer. Using the power consumption figures in Table 4, it can be said that in general the power consumption by activated sludge to produce equivalent effluent is between two and three times the amount of energy as the RBC process. An important factor in power cost comparison between the RBC process and the activated sludge process is that the power consumption figures in Table 4 for the activated sludge process would decrease by less than 10% if the design effluent requirement were raised from 20 to 30 mg/l which is a more typical value for secondary treatment design. By comparison, power consumption for the RBC process shown in Table 2 would decrease by more than 30% by changing the effluent design value from 20 to 30 mg/l. For the 30 mg/l design condition it can be said in general that the activated sludge process consumes between three and four times as much energy as the RBC process.

Energy consumption for the activated sludge process shown in Table 4 includes only the energy required in the aeration tanks. In addition, there are energy requirements for sludge recirculation which would typically add an additional 10% to the total energy consumption. Waste solids from an activated sludge plant which are removed from the secondary clarifier underflow are typically 1% solids concentration. The RBC process does not require sludge recirculation and also imposes a very low solids loading on the secondary clarifier. Because of this, settled solids can be allowed to remain in the hopper of the clarifier for a period of one to two hours before removal. Under this condition they will thicken to approximately 3-4% concentration and if removed with a controlled volume sludge removal pump, will produce an underflow concentration of 2.5-3.0% solids. Therefore, for the activated sludge process to produce a waste sludge flow equivalent to that of an RBC process, air flotation thickening should also be provided. This will result in approximately an additional 10% energy consumption for the activated sludge process. For a typical secondary effluent design condition of 30 mg/l BOD, this would raise the typical power consumption for an activated sludge process from approximately 30 hp/mgd to 40 hp/mgd.

For activated sludge plants less than 10 mgd capacity another factor which effects the overall energy consumption is the efficiency of the blowers providing the compressed air. Table 5 lists several commercially available blowers as a function of their CFM capacity and CFM produced per hp of energy consumed. Blowers whose capacity is significantly below 5,000 CFM produce smaller amounts of air per unit of energy consumed. This means that small activated sludge plants which use the smaller blowers will require more energy per unit flow. The effect of energy consumption as a function of plant size is shown in Figure 3. For treatment plants above 10 mgd, total power consumption by the activated sludge system is approximately 40 hp/mgd

capacity. Below 10 mgd, the power consumption increases gradually, until for very small plants it approaches 70 hp/mgd capacity. Because of the modular nature of RBC process construction, the power consumption per unit flow is independent of plant size. This means that the difference in energy consumption between the RBC process and the activated sludge process will be even greater on smaller plants than on the 10 mgd or larger sizes listed in Table 4.

The energy consumption shown for the oxidation ditch process in Table 4 would be approximately the same as for other types of extended aeration processes. In an extended aeration process, there is some stabilization of waste biological solids because of the long aeration period. To compare RBC power consumption to this figure requires that additional energy be considered for a separate digestion step. When the separate digestion step in an RBC treatment plant is an anerobic digestor, this additional energy consumption can be quite small if methane gas generation can be used to offset the digestor energy requirements. In the case where an aerobic digestor is used in the RBC plant, the amount and concentration of waste sludge produced would require approximately 25 hp/mgd capacity. Adding this energy requirement to that shown in Table 2 for the RBC process, still results in a total energy consumption approximately 1/2 that for the oxidation ditch or other types of extended aeration systems.

PRESENT WORTH ANALYSIS

When evaluating secondary treatment systems, total plant costs, both capital and operating must be considered. A present worth analysis takes a flow of future annual operating costs or cost savings and discounts them back to the present time at a specific rate of interest to determine their present worth. The present worth analysis evaluates capital and operating costs on an equivalent basis so that the treatment alternative with the lowest total present worth is the most cost-effective.

To determine the present worth of a single horse power of energy saved at a 6% interest rate and for a 20-year period, the annual savings would be multiplied by the present worth factor of 11.47. For a typical power cost of 3¢ per kwh, each horsepower of energy saved in a treatment plant then has a present worth of $2,250. For a medium sized plant of 5 mgd, such as the plant shown in Figure 1, the RBC process could be expected to save approximately 30 hp/mgd compared to activated sludge as shown in Figure 3. The present worth of this energy savings is about $338,000 or about $86,000 per mgd capacity. With a wastewater flow equivalent of 100 gallons per day per person, the savings are equivalent to about $7.00 per person.

ENERGY COST ESCALATION

In order to make an accurate and meaningful determination of the present worth of operating costs for a treatment plant, it is important to consider the effect of cost escalations. In the past, the general attitude toward cost escalations is that they should not be considered because costs will increase at the same rate as the general rate of inflation. While this may be true for some operating costs such as chemicals, manpower, etc., it is not true for electrical energy.

Figure 4 shows the escalation of energy costs that has been experienced during recent years. Since 1973, electrical energy costs have increased at an annual rate of over 14%. The concensus among many experts in the field is that this rate of escalation will continue for the next 20 years. This means that with a 20-year annual average rate of inflation of 6-7%, there is an escalation in the real cost of electrical energy of 7-8% per year. To provide an accurate present worth analysis, it is important that the real increase in energy costs be properly evaluated. This can be done using Figure 5. With a general rate of inflation of 6% and an escalation of energy cost of 14%, the net escalation in real power cost is 8%. In Figure 5, at an escalation rate of 8% and a present power cost of 3¢ per kwh, the present worth of each horsepower saved is $5,000. Using the medium size 5 mgd treatment plant example discussed previously, the present worth of energy savings is now 150 hp x $5,000/hp = $750,000. This is equivalent to $150,000 per mgd plant capacity or $15.00 per person. These examples indicate the significance of proper energy cost escalation and their impact on the end user. When these savings are compared to the treatment plant construction costs, it can be seen that they represent a significant portion of a total cost of treatment.

At the present time, the EPA is enforcing a policy which in spite of the trend in power costs forbids proper escalation of energy costs in cost-effectiveness studies. This attitude by EPA administrators is in direct conflict with the goals of national energy conservation and with the basic intent of the construction grants program.

The "Grants Regulations and Procedures", Revision of Part 40 CFR 30.420-6" (Federal Register, May 8, 1975) provides that:

> "Grantees must participate in the national energy conservation program, by fostering, promoting and achieving energy conservation in their grant programs. Grantees must utilize to the maximum practical extent, the most energy-efficient equipment, materials, and construction and operating procedures available."

The key words in the above statement are "maximum practical extent" which imply the need for some professional judgement on the part of the grantee and his consulting engineer. Professional judgement would include a rational evaluation of present and future energy costs to the grantee. By arbitrarily setting a policy which forbids consideration of future energy cost increases, the EPA is substituting its judgement for that of the consulting engineer and is usurping his right and obligation to act in the best interest of his client.

Ignoring future increases in energy costs will cause a severe imbalance in the relative importance of capital and operating costs in cost-effectiveness studies. A policy which favors treatment technology that has lower construction costs but high operating costs could increase energy consumption by new treatment plants by 200% or more. This is directly contrary to the national goal of energy conservation.

One of the original reasons for establishing the construction grants program was to make wastewater treatment economically feasible by relieving

the burden on the local community. In this way the nation as a whole would benefit from the improved water resources. A policy which ignores future power cost increases and favors technology with high operating costs could increase the local cost burden by 200% or more because all increases in operating costs are passed on totally to the local community. This policy could make local participation in the grants program economically or politically unfeasible and would be directly contrary to the original intent of the program. It could also lead to shutdown of part or all of treatment plant operations by local communities in an effort to reduce operating costs and worsen the existing plant operating problems and non-compliance with discharge standards

It is suggested that a formal liaison be established between the EPA and the Department of Energy to establish a rational procedure for the proper escalation of power costs in cost-effectiveness analyses and that judgement in the use of these procedures be returned to the consulting engineer. In this way the success of the efforts toward energy conservation and of the construction grants program will be best realized.

UPGRADING TREATMENT PLANTS

The RBC process has a high degree of flexibility in its use because of its modular construction, requirement for relatively shallow excavation, and low hydraulic head loss. These features, along with the absence of effluent or sludge recycle requirements allow it to be incorporated into existing secondary treatment plants for upgrading without additional energy requirements for pumping. For upgrading a trickling filter plant, RBC units can be placed immediately following the trickling filter. Partially treated wastewater leaving the trickling filter is introduced directly into RBC units for the additional required treatment. After the RBC process, the treated wastewater flows directly to existing secondary clarifiers without the need for additional pumping.

For upgrading existing activated sludge plants the RBC process has a unique application called the Surfact Process. In the Surfact Process, RBC units are mounted in the existing aeration tanks. The specially adapted RBC units utilize a portion of the diffused air being delivered to the aeration tanks to assist in rotation. The addition of the fixed biological culture on the RBC units to the culture maintained in suspension in the activated sludge process increases the capacity of the activated sludge plant from 50-100%. The increase in treatment capacity is achieved with significantly lower energy requirements than would be necessary for an expansion of the activated sludge plant in a conventional manner.

CONCLUSION

Through six years of full-scale field experience, the RBC process has demonstrated that it is the lowest energy consuming biological treatment process now available to meet current treatment standards. Energy cost savings with the RBC process represent a significant portion of the total cost of treatment, especially when proper consideration is given to future increases in energy costs.

TABLE 1

Wastewater Characteristics

Parameter	Concentration mg/l
Raw Sewage	
BOD	210
Suspended Solids	230
Primary Effluent	
BOD	136
Suspended Solids	80
BOD (soluble)(1)	88
Secondary Effluent	
BOD	20
Suspended Solids	20
BOD (soluble)(2)	10

(1) Based on BOD (soluble) = BOD - 0.6 x Suspended Solids

(2) Based on BOD (soluble) = BOD - 0.5 x Suspended Solids

TABLE 2

RBC Power Consumption at
2 KW per 100,000 ft^2 of Media

Loading Hydraulic GPD/ft^2	Soluble BOD lb/D/1000ft^2	Effluent BOD mg/l	HP/MGD	lbBOD/HP-HR
2.0	1.5	20	13	3.1
2.7	2.0	30	9.8	4.1
3.0	2.25	40	8.7	4.6

TABLE 3

Trickling Filter Power Consumption
From EPA Publication MCD-32* for 10 MGD Plant

Process	Effluent BOD mg/l	HP/MGD
Rock-High Rate	45	8.6
Rock-Low Rate	30	12.5
Plastic-High Rate	35-45	23
Plastic-Super High Rate	82	30
Redwood plus activated sludge	20	30.6

TABLE 4

Activated Sludge Power Consumption
From EPA Publication MCD-32* for 10 MGD Plant
Effluent BOD = 20mg/l

Aeration Method	HP/MGD
Coarse Bubble	44
Fine Bubble	32
Mechanical	26
Turbine	32
Static	37
Jet	25
O_2-PSA	38
O_2-Cryogenic	26
O_2-Micro Bubble	30
Oxidation Ditch	67

TABLE 5

Centrifugal Blower Power Consumption
CFM/HP at Full Capacity and
8.0 psig Discharge Pressure

Capacity, CFM	Hoffman	Roots/Dresser
100	9.1	17.5
500	13.2	-
1,000	16.7	-
2,000	18.2	-
5,000	20.4	24.3
8,000	21.0	24.2
10,000	20.8	24.3
12,000	-	25.0
15,000	-	24.6

*Energy Conservation in Municipal Wastewater Treatment
EPA 430/9-77-011, March, 1978

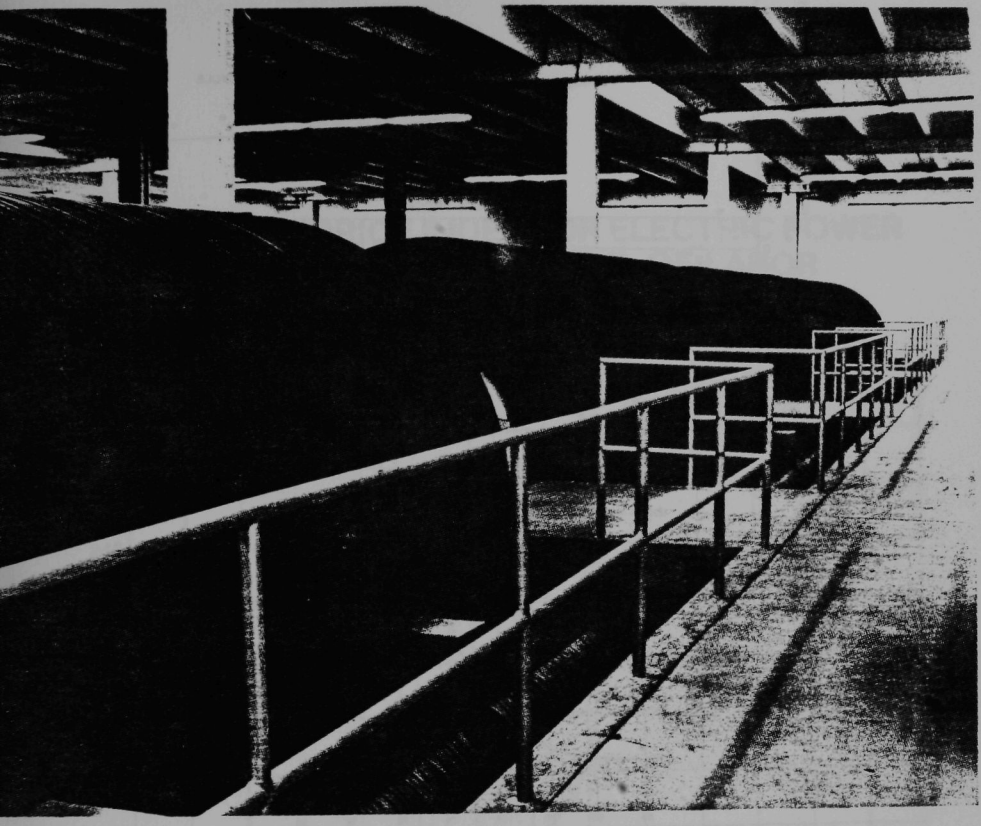


Fig. 1. 5MGD RBC Installation at Kirksville, Mo.

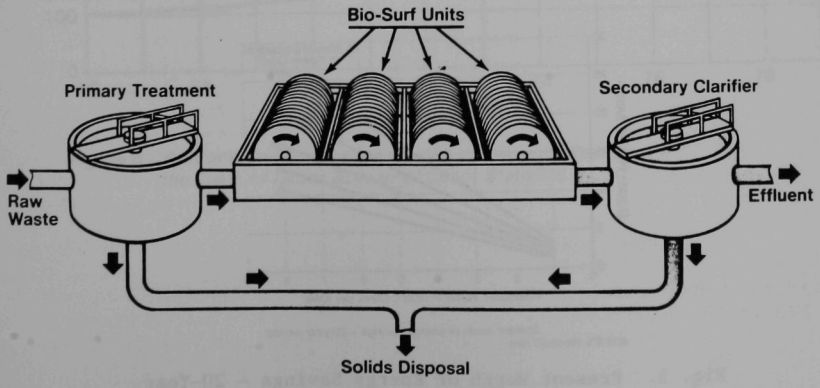


Fig. 2. Process Flow Diagram

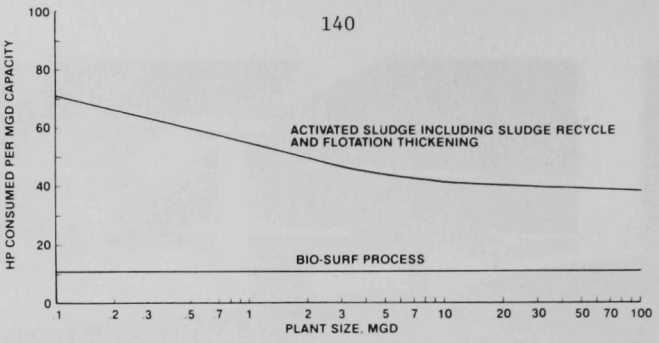


Fig. 3. Comparative Power Consumption

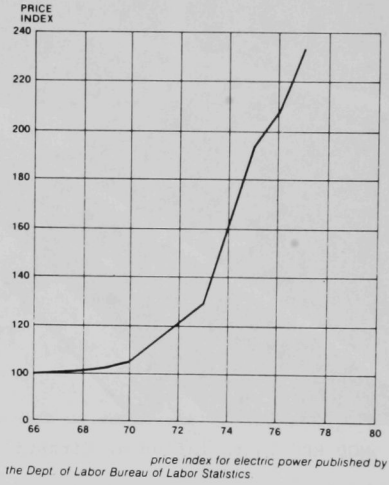


Fig. 4. Price Index for Electric Power Published by the Dept. of Labor Bureau of Labor Statistics

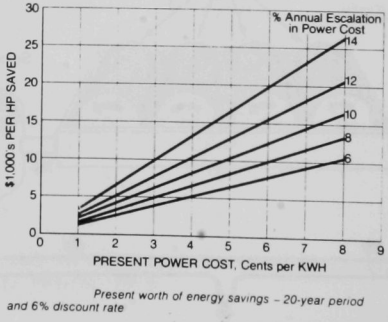


Fig. 5. Present Worth of Energy Savings - 20-Year Period and 6% Discount Rate

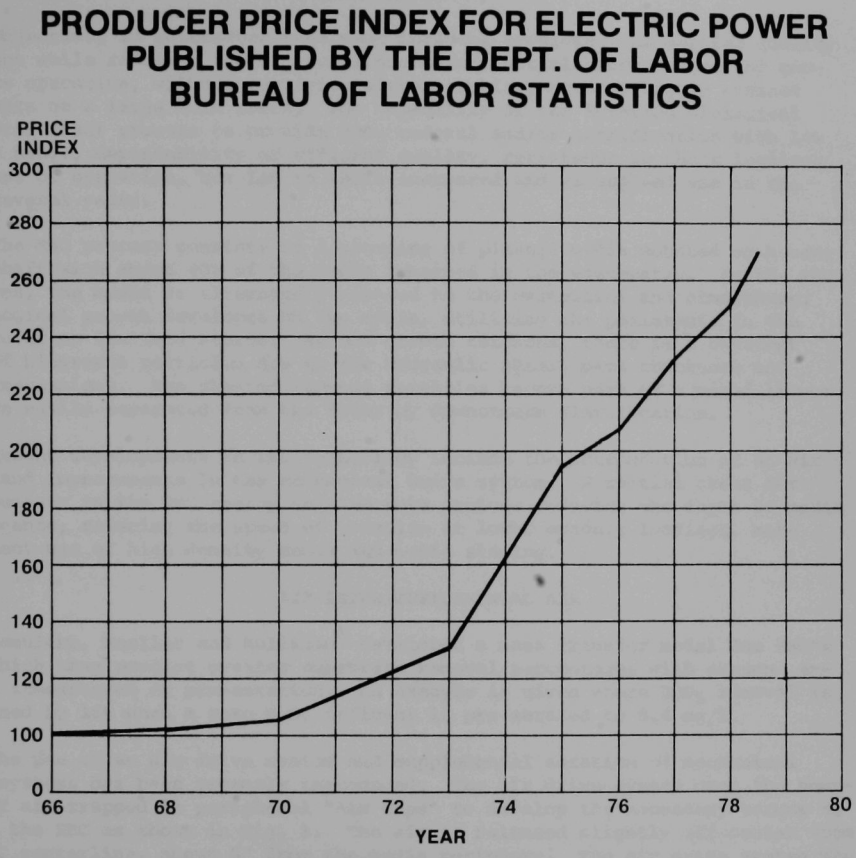


Figure 6 *Producer price index for electric power published by the Dept. of Labor Bureau of Labor Statistics.*

ENERGY REDUCTION CONSIDERATION FOR THE RBC PROCESS

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INTRODUCTION

Objectives in wastewater treatment include; providing the desired quality effluent while reducing the financial needs for initial construction and continuous operation, without requiring either sophisticated or highly trained operators or a large work force. The capability of the Rotating Biological Contactor (RBC) process to provide BOD_5 removal and/or nitrification with low energy usage, dependability of effluent quality, resistence to shock loadings and ease of operation, has led to their increased and widespread use in the past several years.

The RBC process consists of a grouping of plastic media mounted on a central shaft with about 40% of the media immersed in the wastewater. As the RBC revolves, the media is alternately exposed to the wastewater and atmosphere. A biological growth developes on the media, utilizing the pollutants in the wastewater as its food source. As the growth thickens, there is a constant loss of biogrowth particles due to the hydraulic shear, mass thickness and other mechanisms. The sheared biomass particles become part of a mixed liquor and are easily separated from the water by downstream clarification.

Recent developments in RBC technology include the introduction of an air drive and improvements in the mechanical drive system. Potential areas for improvements in the RBC energy requirements include reducing the depth of media submergence, reducing the speed of rotation at lower organic loadings, more efficient use of high density media and media staging.

AIR DRIVE/SUPPLEMENTAL AIR

Famularo, Mueller and Mulligan[1] developed a mass transfer model for RBC's from which they predict greater substrate removal percentages with oxygen enriched atmospheres or pre-aeration. An example is given where BOD_5 removal is increased by 16% when a zero D.O. influent is pre-aerated to 6.4 mg/l.

The use of an air drive system and supplemental aeration of mechanical drive systems has been recently introduced. The air drive system uses the bouyancy of air trapped in peripheral "Air Cups" to develop the necessary torque to rotate the RBC as shown in Fig. 1. The air is released slightly off center from the RBC centerline, about 6" from the media periphery. The air drive system may require as much as 200CFM per shaft[2,3], with only 20%[2,3,4] of the air supply available to rise up through the media, the other 80% being trapped in the air cups to develop the necessary bouyancy for torque.

Hynek and Chou[4] of Autotrol Corp. refer to an unpublished pilot study where a slight increase in BOD removal in an air drive RBC as compared to a mechanical drive RBC was determined when treating synthetic wastewater. They also report improved substrate removals during the Autotrol Corp. South Shore Test Program when comparing their air drive media and their mechanical drive media.

At Alexandria, VA a pilot program was performed to determine the effect of adding supplemental air to an existing Autotrol mechanical drive system. The average of five months of testing[2] when 200 ± CFM per shaft was added to the mechanical drive RBC is shown in Table 1. This data includes all testing phases with and without the addition of FeCl3 and shows 2 mg/l difference in total BOD_5, 7 mg/l difference in soluble BOD_5 and 2 mg/l difference in the SS.

Table 2 also shows the summary of data obtained by Greeley and Hansen Engineers[5] when only 1 stage and all 4 stages were subjected to supplemental air. Improvements in soluble BOD_5 were observed, with and without chemical addition, at the high supplemental aeration rates.

The Alexandria, VA program shows that soluble BOD_5 removal is increased when supplemental air was added to the RBC's at a rate of 200 ± CFM per shaft. A comparison of the additional capital and operating costs and energy use between the addition of 200 CFM/RBC and adding additional RBC Units has not been done.

COLD SPRING PILOT STUDY

Each manufacturer of RBC's utilizes its own media configuration. Since the reported advantages of supplemental air or air drive systems use Autotrol Corp. media that utilizes radial flow passages to permit movement of wastewater and air to the inner media, a test program was established at the Village of Cold Spring, NY Wastewater Treatment Facility to determine the effects of supplemental air when Clow Envirodisc's open media is used.

The pilot program at Cold Spring is depicted in Fig. 2. Wastewaters were pumped from the headworks, measured and split into two equal flow paths. The pilot plant is a Clow Envirodisc Model A-10 providing 11,000SF of 12' diameter media. The tankage was built so that wastewater could not pass through the baffles, only through the external piping. In this manner, each flow path consists of two separated stages in series. Samples were obtained from the influent and from the effluent of each stage.

Since 80%[2,3,4] of the air in an air drive system is used for motive power, 200CFM applied to a 100,000SF RBC leaves 40CFM of air for passage through the media. Since the Alexandria, VA project provided 200 ± CFM of air for movement through the media, it was desired to determine the effect of the lower air flows that are available for movement through the media as in an air drive system. The air flow at Cold Spring was adjusted to provide 0.6 CFM/1000SF, 50% more than calculated as being non-motive air for an air drive.

Seven hour composite samples were taken by plant personnel from June through November, 1979 with BOD_5 and SS analyses performed at a New York State approved laboratory. Dissolved oxygen, pH, and temperature analyses were performed at the treatment plant laboratory by plant personnel.

Table 2 shows the June through November summary of analyses from the project. During this period the hydraulic loadings varied from 10,000 to 95,000 GPD (0.91 to 8.64 GPD/SF); the organic loadings varied from 0.68 to 14.04 lbs. $SBOD_5$/1000SF and removal rates of up to 6.18 lbs. $SBOD_5$/1000SF were recorded.

The data obtained from this study shows virtually no difference in total or soluble BOD_5 and a small increase in TSS as a result of the supplemental aeration, regardless of the hydraulic or organic loadings.

Although the results appear to contradict the results reported at Alexandria, VA, there are major differences between the two studies. The hydraulic and organic loadings at Alexandria, VA, were fairly constant (3-5 lbs. SBOD/1000SF) with testing done at high supplemental aeration rates (2± CFM/1000SF) on relatively closed media. At Cold Spring, loadings varied from 0.7 to 14 lbs. SBOD/1000SF at low supplemental aeration rates approximating the available non-motive aeration in an air drive system, using a relatively open media.

RBC FEATURES THAT GENERATE ENERGY NEEDS

The Features of an operating RBC that generate energy needs are: 1. The Drive Motor; 2. The Speed Reducer; 3. The Bearings; 4. The Hydraulic Drag as the media travels through the wastewater; and 5. The Biomass/water weight differential about the centerline of the mainshaft.

The use of high quality tapered roller bearings provides the most efficient (least energy consuming) choice of bearings with a B-10 life in excess of 50 years available.

Similarly, use of high quality triple reduction speed reducers leaves little room for improving the efficiency of the reducer. However, Clow Envirodisc introduced the use of shaft mounted speed reducers in 1978. This speed reducer, specially designed by Dodge Reliance for driving the RBC, eliminates the conventional chain and sprockets normally used. The removal of the chain and sprockets has eliminated the misalignment and chain tension problems, periodic maintenance, chain oil baths and guards and increased the drive system efficiency by a fraction of a percent. The shaft mounted reducer facilitates initial installation and reduces maintenance by requiring only an annual oil change for most operating environments.

The remianing areas where energy needs in the RBC are generated are currently being studied to determine where energy use can be reduced without sacrificing substrate removal capabilities.

ELECTRIC MOTOR SELECTION

Typically 100,000SF or 150,000SF RBC's use a 7½HP motor, although the actual operating requirement is usually 2½ to 4½HP at 1.5 to 1.6RPM. The actual HP requirement at constant speed of rotation is determined primarily only by the nature of the biological growth. When a RBC is heavily loaded, the thickness of the biogrowth and the quantity of liquid carried out of the wastewater increase. As the weight differential between opposite sides of the RBC mainshaft increases, the torque and HP demand increases. When a RBC is first started up, particularly after having been "Down" for a period with wastewater still in-basin, up to 150% of the normal HP demand may occur for several minutes until some imbalance and excess biomass is reduced. Thus, in a train of 6 RBC's in series, the 1st and 2nd units may receive a high organic loading and have heavy biogrowth. The 3rd and 4th RBC's would have light to moderate loadings

and growths and if the 5th and 6th RBC's provide nitrification, they would have a relatively light biogrowth. With this type of situation, the actual HP demands may be 4½, 4, 3½, 3, 2½ and 2½HP. Until recently, each of these units would use a 7½HP standard electric motor.

When a RBC is only lightly-moderately loaded, a 5HP motor could be used to operate the RBC. If a 1.15 service factor is specified (5.75HP total) ample excess capacity is available to restart a "Down" RBC. When the KW used is plotted against the HP demand for standard 5HP and 7½HP motors as in Fig. 3, the average difference in KW within a power requirement range of 2½ to 4½HP can be determined as 0.05KW, or 438KW-h annually. At $.05/KW-h, this represents an annual savings of $21.90 per year. The total present worth savings assuming a 20 year average rate of $.07/KW-h, a 7% interest rate and a 20 year replacement is calculated at $232. Since the 5HP standard motor costs $197 less than the 7½HP standard motor, a total present worht savings of at least $429 is possible.

When a 5HP high efficiency (HE) motor is considered in lieu of a 5HP standard (S) motor, as on Fig. 4, the average KW difference is 0.18KW, resulting in a savings of 1577 KW-h and $78.85 annually. With an increase in cost of $222 to purchase the HE motor, payback occurs in 2.8 years. The total present worth savings by using the HE 5HP is $613.

Since a 7½HP S motor has been the norm, the difference between a 7½HP S motor and 5HP HE motor is shown in Fig. 5. By using 5HP HE motors in lieu of the norm of 7½HP S motors where loading permits, 0.275KW are saved with an annual savings of 2409 KW-h and $120.45. Since the 5HP HE motor costs $63 more than the 7½HP S motor, payback occurs in ½ years. The total 20 year present worth savings becomes $1213.

Where organic loadings still require a 7½HP motor, a HE motor should be used. From Fig. 6, an average reduction of 0.185KW as determined, resulting in an annual savings of 1621 KW-h and $81.05. With the HE motor costing $260 more than the S motor, payback occurs in 3.2 years. The total 20 year present worth savings by substituting the HE for the S motor (7½HP) becomes $599.

Invariably, there will be designers and specifiers who will not select a 5HP motor where tradition has called for a 7½HP motor. However, in order to take some advantage of the concepts included herein, a 7½HP HE motor may be selected. When the selected 7½HP HE motor is compared with the possible 5HP HE motor as shown in Fig. 7, it is shown that the selection results in increases of 0.1KW, 876KW-h/year, $43.80 operating cost/year, $419 purchasing cost and a 20 year present worth operating cost of $782. Instead of being conservative in selection but still attempting to reduce energy use and costs, the failure to select the 5HP HE motor increases the total present worth cost by $1201 and wastes 17520KW-h over the 20 year period.

EFFECT OF MEDIA SUBMERGENCE

Ellis and Banaga[6] report some increases in substrate removal when the media submergence was reduced to 26%. Since the fluid drag contributes a major portion of the energy demand and the fluid drag decreases as the media submergence decreases, a reduction in the media submergence would decrease tankage costs and energy requirements. Fig. 8 shows the results of torque testing

on a 22,000SF, 12' diameter Clow Envirodisc RBC. As would be expected, the torque required varied linearly with the percent of media submerged. If future tests confirm that little or no loss in substrate removal occurs with decreased media submergence, the potential for construction cost and operational cost reduction may be realized.

However, the potential savings may not be so easily realized. The media and biomass bouyancy decrease and the net mainshaft loadings increase as the media submergence decreases. Furthermore, less biomass is exposed to the shearing action as the media passes through the wastewater, increasing the thickness of the biomass and producing a second source of net loading increase to the RBC mainshaft. This decrease in bouyancy and increase in biomass weight would necessitate improved structural properties for the RBC.

If studies can confirm satisfactory treatment capabilities at lower submergence the potential construction and operation savings must be evaluated with respect to the cost of increasing the RBC structural capabilities.

SPEED REDUCTION

Friedman, Robbins & Woods[7] studied the effect of varying the rotational speed of RBC's and concluded that at higher organic loadings, mass substrate removals increased, with little change in removals at low organic loadings. It is thus suggested that at low organic loadings, the RBC RPM's could be reduced to save energy while still maintaining the desired removals.

A series of tests to determine the effect on energy at varying combinations of media area and RPM's was conducted at Baldwin Place, NY on a Clow Envirodisc Model B-20. Fig. 9 shows the results of these tests with a nearly cubic relationship found between the RPM and energy. Thus, it does appear that reducing the speed of "Downstream" RBC's to save energy is feasible. However, further study at full sized operating installations is recommended before full confidence can be obtained for this procedure.

Where a RBC is operating at less than design capacity, an attempt could be made to reduce the RBC speed to save energy. Similarly, where effluent limits do not reflect seasonal change and the RBC is designed to meet the limits at colder temperatures or higher flows, an attempt to save energy by reducing speed could be made during periods of less severe inflow conditions.

Since the reduction in speed will also reduce the biomass shearings and increase the total load imposed on the RBC, careful observation and/or shaft weighing should be taken when conducting the procedure.

CONCLUSIONS

1. Supplemental aeration of Clow open media RBC at rates approximating nonmotive air in an air drive RBC did not effect the RBC removal rates.
2. Supplemental aeration at high rates increases removal rates of a closed media RBC.
3. A properly sized high efficiency motor can result in energy savings.
4. Reduction in media submergence will reduce energy needs; further evaluation as to the effect on the RBC structure and removals is necessary.
5. Reduction of RBC speeds is a means of saving energy; evaluation at full sized operating facilities is recommended to determine actual design factors.

TABLE 1
Summary of Alexandria, VA Project

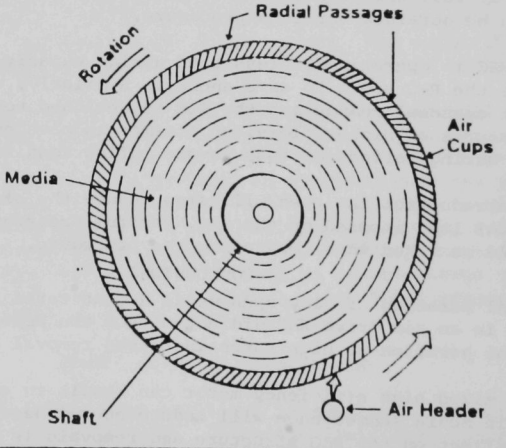
	Mode & Stages	Total BOD_5	Sol. BOD_5	SS.
5 mo. test avg.	M	43	15	48
	MS	41	8	50
W/O Chem Addition	M-1 stg.		19	
	MS-1 stg.		16	
	M-4 stgs.		28	
	MS-4 stgs.		11	
W/Chem Addition	M-4 stgs.		13	
	MS-4 stgs.		6	

NOTE:
1. All results in mg/l
2. M = mechanical RBC
3. MS = mechanical RBC W/ 200± CFM/ shaft supplemental air

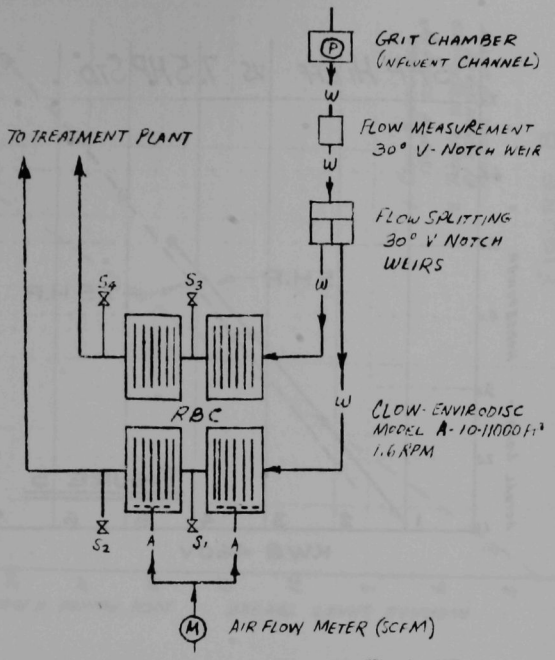
TABLE 2
Summary of Cold Spring, NY Project-mg/l

	Mode	Inf.	Stg. 1 Eff.	Stg. 2 Eff.
Sol. BOD_5	M	82.9	46.4	33.4
	MS	82.9	47.2	32.4
Tot. BOD_5	M	153.0	54.5	36.2
	MS	153.0	55.1	37.1
D.O.	M	4.1	3.5	4.6
	MS	4.1	3.7	4.7
Tot. SS	M	133.2	121.7	98.9
	MS	133.2	115.1	104.2

NOTE:
1. All results in mg/l
2. M = mechanical drive
3. MS = mechanical drive with 0.6 CFM/1000SF supplemental air



AIR DRIVEN RBC SCHEMATIC
Fig. 1



COLD SPRING, N.Y. PILOT PLANT

FIGURE 2

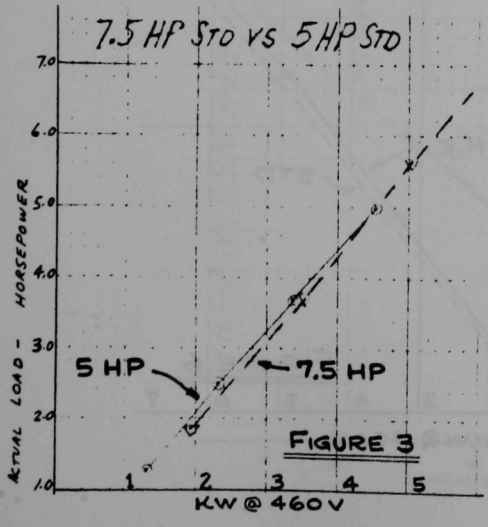


FIGURE 3

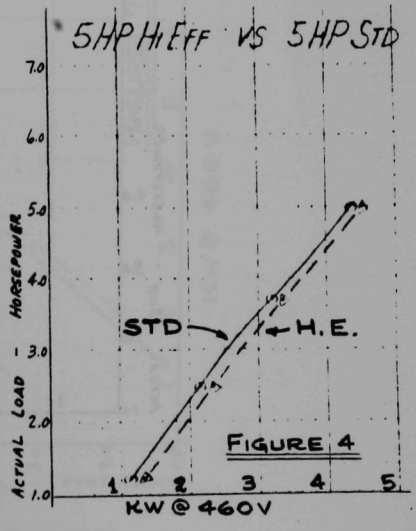
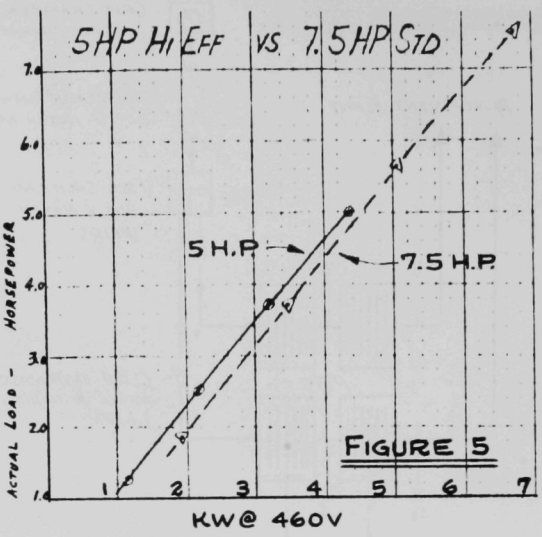
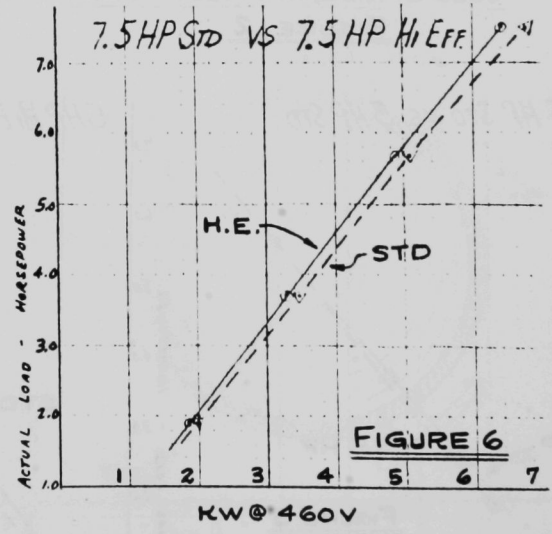


FIGURE 4


5HP Hi Eff vs 7.5HP Std
5 H.P.
7.5 H.P.
FIGURE 5
ACTUAL LOAD - HORSEPOWER
KW@ 460V


7.5HP Std vs 7.5 HP Hi Eff.
H.E.
STD
FIGURE 6
ACTUAL LOAD - HORSEPOWER
KW@ 460V

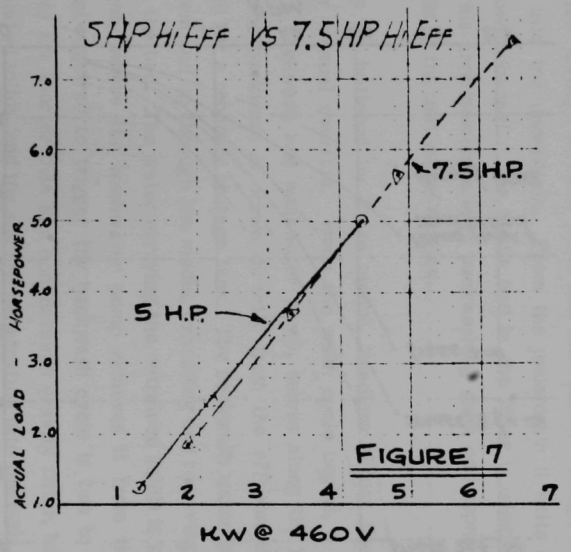


FIGURE 7

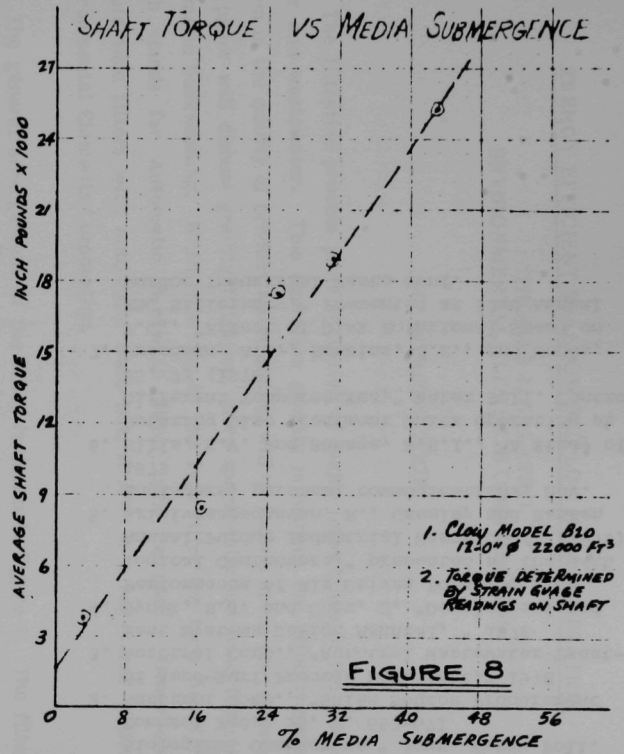


FIGURE 8

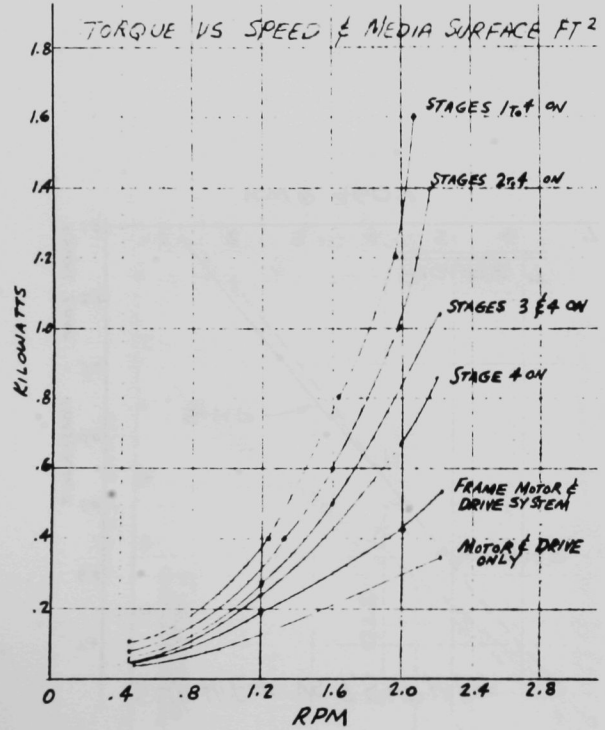


Fig. 9

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ENERGY EFFICIENT SHALLOW BED LOW HEAD FILTRATION

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I. INTRODUCTION

The filtration process is an operation of major importance in the treatment of water and wastewater. This operation greatly influences the effectiveness of the system in the quality of product water and, as will be shown, in the cost of production. This paper will discuss gravity filtration, more specifically low head filtration with automatic backwashing. When referring to this type filter the term A.B.W.® Filter, which stands for Automatic Backwash Filter, will be used. Other companies manufacture filters with automatic wash but, A.B.W. is a registered trademark of Environmental Elements Corporation.

The general arrangement of the filter is shown in Figure 1. The filter media consists of an eleven inch deep sand bed supported by porous plates. During filtration, water enters the filter through influent ports and then passes through the media and support plates and into the underdrain. From the underdrain it exits thru effluent ports into the effluent channel. The media bed in the filter is comprised of many individual cells. This arrangement allows backwashing each cell separately without taking the other cells of the unit out of service.

Backwash can be initiated by three means, headloss across the filter, time between washes, or manual override. When the wash cycle begins, a motor driven carriage, carrying a backwash and washwater pump, moves along the length of the filter. Water for the backwash is drawn directly from the effluent channel which eliminates the need for a separate storage tank. The backwash water is pumped into the underdrain system and up through the media, expanding and removing accumulated solids deposited on the bed. The water carrying the entrained solids is then collected in the hood assembly where the washwater pump removes it from the unit. The pressure differential required to trigger the backwash cycle is two to six inches of water. Filter cycles are shorter than in a conventional gravity filter, but the smaller quantity of accumulated solids, and the shallow penetration into the bed, allow the use

of less total backwash water than in conventional filters. Also, the area of the single cell being backwashed is small by comparison and requires drastically less horsepower for the wash pumps.

One of the many installation and operational cost advantages of the low head filter, is the lack of pipe galleries and valves. All flow is controlled by the use of channels and weirs. All pumps and controls are readily accessible on the carriage.

The total depth of the filter is 6'–4" which, with a total head loss across the filter of less than 12" under normal operating conditions, will greatly reduce any pumping requirements. There is no need for washwater storage, the washwater flow can be returned to the preceeding operation without disturbing the process.

II. PERFORMANCE EXPERIENCE

The low head filter in water and wastewater applications typically operates at hydraulic loading rates of 2 gpm/ft.^{2}, with backwash water requirements of 2%, or less, of the flow through the unit. The backwash flow rates range from 15 to 30 gpm/ft.^{2} depending on the application. This translates to a pump capacity of 150 to 300 gpm in a filter 16' wide with 8" cells.

Low head filter performance is demonstrated by the following operating data. A potable water plant in New York, for the year 1974, averaged 1.21% backwash water usage with raw water turbidity of 1.87 Formazin Turbidity Units (F.T.U.) and effluent turbidity of 0.30 F.T.U. The average loading rate during this period was 1.8 gpm/ft.^{2}, design capacity, 2.0 gpm/ft.^{2}. A 20 day test in December 1977 on the filter at the Los Alisos water district showed an average of 0.28 Jackson Turbidity Units (JTU) effluent turbidity with an influent of 4.49 JTU. The backwash water used during the test averaged 1.28% with a hydraulic loading of 0.9 gpm/ft.^{2}. Other plants report similar results, all with excellent effluent water quality, (Table 1).

A large amount of operating information for the ABW Filter has also been collected from its use in wastewater filtration. For example, a high rate loading test of the filter was conducted at the Hanover Park, Illinois, wastewater facilities from

September of 1972 through May of 1974. Two units were tested at hydraulic loading rates from 2.5 gpm/ft.^{2} to 4.5 gpm/ft.^{2} with influent suspended solids in excess of 30 mg/l.

HIGH RATE LOADING TEST

Loading Rate (gpm/ft.^{2})	T.S.S. Influent (Mg/l)	T.S.S. in Effluent (Mg/l)	Backwash (% total Flow)
2.5 – 3.5	76.7	4.4	1.41
3.5 – 4.5	36.6	4.8	1.57

The results show that the filter effluent has less than 5 mg/l T.S.S. for extended periods of time with both high hydraulic and solids loading rates, and with less than 2% backwash water used. Other examples of A.B.W. Filter operation, at more conventional loading rates of 1.5 to 2.5 gpm/ft.^{2}, show that with average influent T.S.S. of 10 to 20 mg/l, effluent T.S.S. are 1 to 4 mg/l (Table 2). The BOD reduction, which of course is associated with the solids removed, averaged 80%. These data demonstrate that the low head filter operates with excellent results using small percentages of throughput for backwashing. There are presently over 400 installations of the ABW Filter.

III. ENERGY USAGE

When comparing the energy consumption of a low head filter with that of a conventional unit, the most significant advantage is the small headloss across the unit. As stated in the introduction, the headloss across the low head filter is 12 inches or less, which usually means gravity flow is possible without pumping. A conventional filter is considered to have a sandbed depth of 20" or more with a headloss across the filter of 8 to 10 feet. For example, the City of Houston's 69th Street Wastewater Treatment Complex uses ABW Filters. Based on the design flow of 200 MGD, a total of 330 horsepower would have been required for pumping using a conventional filter. No pumps are required for the ABW Filter installation, this provides a savings of

approximately 590 KW/day.

Use of the low head filter also results in reduced energy consumption for backwashing. Operating data for the year 1978 from a water plant using conventional filters (Table #3), shows an average backwash water requirement of 3.67%. The filters at this plant operated at approximately 2.0 gpm/ft.^{2}. Backwash was conducted on an average of once a day at a rate of 21 gpm/ft.^{2}. The pumps for this flow require 150 connected horsepower and 30 KW-hrs./day/filter. The operating data for the year 1974 from a water plant with an ABW Filter (Table #1), at loading rates averaging 1.8 gpm/ft.^{2}, used 1.2% backwash water. The wash rate was approximately 15 gpm/ft.^{2}, with a total connected horsepower of 3.75. The total energy usage for backwash was 6.5 KW-hrs./day/filter. If only power usage was compared, the plant savings for the low head filter is 23.5 KW-hrs/day/filter. Although these figures are derived from data of two different plants operating under similar hydraulic and solids loadings, they emphasize the low power consumption of the low head filter.

Some further examples of power consumption in low head filtration are taken from the data in Tables 1 and 2. At the Hanover Park, Illinois, Waste Treatment Plant, two ABW Filters were loaded at an average rate of 2.77 gpm/ft.^{2} with a backwash water consumption of 1.3%. Design load of the filters is 2.22 gpm/ft.^{2} with a wash rate of 25 gpm/ft.^{2}. The connected horsepower for each of the filters is 3.75, which calculates to 7.5 KW–hrs/day/filter or a total of 15 KW–hrs./day.

The Newburg, N.Y., water plant operated at an average loading rate of 0.84 gpm/ft.^{2} for the year 1975. Design of the filter calls for 2.0 gpm/ft.^{2}. The wash rate for the filter is 15 gpm/ft.^{2} with a connected horsepower of 3.75. These numbers result in a energy usage of 5.4 KW–hrs./day/filter.

As stated previously, the efficiency of the filter will vary depending on the application and operating conditions. In the Middletown, N.Y. plant, consumption was 6.5 KW–hrs./day/filter at design conditions, whereas the Newburg plant showed 5.4 KW–hrs./day/filter when run below design capacity and the Hanover Waste Treatment plant ran at 7.5 KW–hrs./day/filter when overloaded.

IV. SUMMARY

When using the low head filter, the largest power saving results from the low headloss across the filter which permits gravity flow without pumping. In addition, the smaller backwash water requirements permit lower connected horsepower for pumping. The total operating power consumption is therefore greatly reduced as compared to conventional filters.

Another area in which there are energy savings, although intangible, is in the initial installation of the low head filter as compared to a conventional system. The amounts of labor and material, with their resulting energy consumption, would be smaller since less concrete, piping, and accessory equipment is needed to install a low head filtration system. When all savings are added up, low head filtration is an energy efficient alternative in the treatment of water and wastewater.

TABLE 1

ABW® Filters - Potable Water

Location	Raw Turbidity yearly avg.	Filtered Turbidity yearly avg.	% Backwash
Middletown, N.Y.	1.87 FTU	0.3 FTU	1.21
Newburgh, N.Y.	2.10 FTU	0.4 FTU	2.00
Anaheim, Calif.	1.00 JTU	0.4 JTU	----
Los Alisos, Calif.	4.49 JTU	0.3 JTU	1.28

TABLE 2

ABW® Filters - Municipal Wastewater

Location	Year Avg. TSS (mg/l)		Year Avg. BOD		%Backwash
	IN	OUT	IN	OUT	
Hanover Park, Ill.	17	4	14	4	1.3%
Nassau Bay, Texas					
1972	---	3.4	---	4.1	---
1973	---	2.2	---	3.6	---
1974	---	4.2	---	2.9	---

TABLE 3

Conventional Filters - Potable Water

Location	Raw Turbidity	Filtered Turbidity	% Backwash
Baltimore MD. #1	1.8 T.U.	0.3 T.U.	3.67%
Baltimore MD. #2	1.2 T.U.	0.4 T.U.	4.78%

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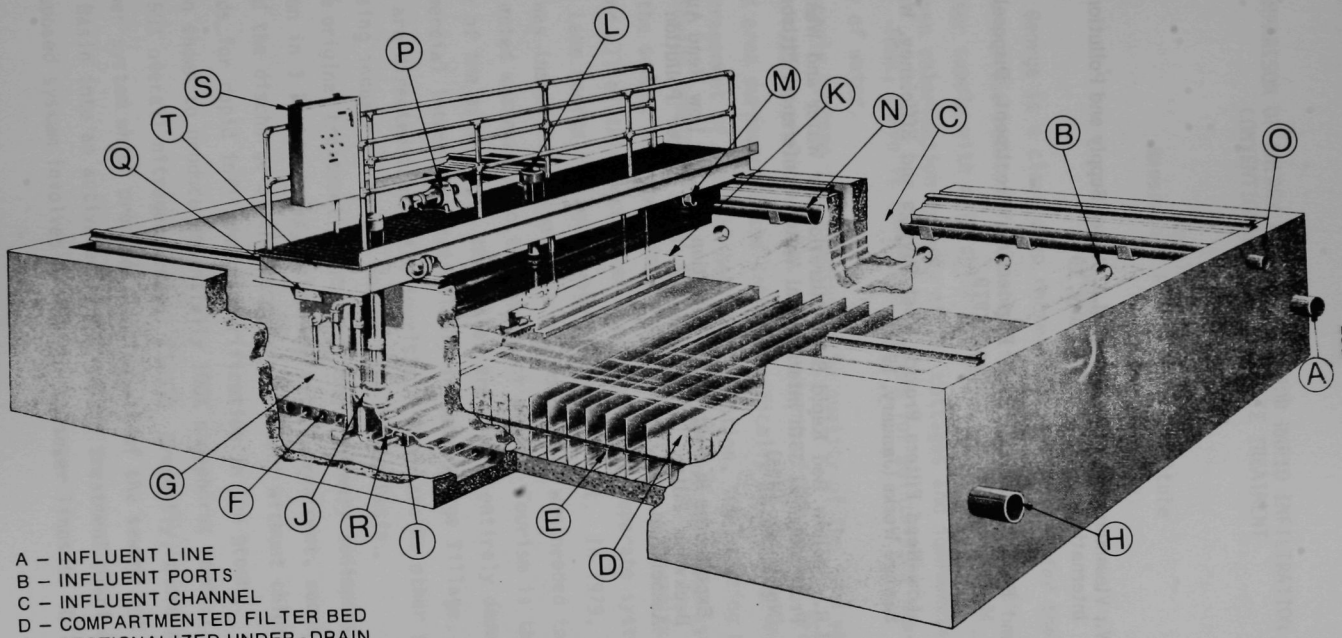
AUTOMATIC BACKWASH FILTER



A – INFLUENT LINE
B – INFLUENT PORTS
C – INFLUENT CHANNEL
D – COMPARTMENTED FILTER BED
E – SECTIONALIZED UNDER-DRAIN
F – EFFLUENT AND BACKWASH PORTS
G – EFFLUENT CHANNEL
H – EFFLUENT DISCHARGE LINE
I – BACKWASH VALVE
J – BACKWASH PUMP ASSEMBLY
K – WASHWATER HOOD
L – WASHWATER PUMP ASSEMBLY
M – WASHWATER DISCHARGE PIPE
N – WASHWATER TROUGH
O – WASHWATER DISCHARGE

P – MECHANISM DRIVE MOTOR
Q – BACKWASH SUPPORT RETAINING SPRINGS
R – PRESSURE CONTROL SPRINGS
S – CONTROL INSTRUMENTATION
T – TRAVELING BACKWASH MECHANISM

ENVIRONMENTAL ELEMENTS CORPORATION
Subsidiary of Koppers Company, Inc.

figure 1

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COMPARISON OF ENERGY REQUIREMENTS FOR RAPID INFILTRATION AND CONVENTIONAL DOMESTIC WASTE TREATMENT

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INTRODUCTION

Lake George is a clear deep recreational lake noted for the clarity of its waters and the beauty of its shoreline. The lake water is used for a public water supply with only chlorination as treatment and is used directly without even chlorination by many of the cottages and homeowners surrounding the lake. Thus, there is continued concern for maintaining the purity of this body of water.

The early concern for preserving the quality of the lake was shown by an effort in 1936 to provide sewers and treatment of the wastes from the most populated area surrounding the lake, specifically Lake George Village. With the encouragement of the Lake George Association, the existing Lake George Village Sewage Treatment Plant was constructed and began operation in 1939. In 1965 the sewers were extended to portions of the surrounding Town of Lake George and all of this wastewater was pumped by a separate system to the existing Lake George Village Sewage Treatment Plant. In 1979, a new pumping station was installed so that only one force main was needed to convey all the collected wastewater to the treatment plant. Tourism is the principal industry of the area, thus, the wastes are almost entirely domestic. There are commercial interests, particularly in Lake George Village, but these in general are directed toward the tourists. There are no other significant water using industries in the entire Lake George Basin.

The original treatment plant provided primary sedimentation with sludge digestion in 3 Imhoff tanks, trickling filter treatment, secondary sedimentation and the discharge of the secondary effluent without chlorination onto sand beds for rapid infiltration purification in the ground. This system has been shown to produce essentially 100% phosphorus removal and approximately 50% overall nitrogen removal (1-3). Presently, there are plans for a new sewer system which proposes pumping all of the sewage out of the Lake George Basin into an activated sludge sewage treatment plant at Glens Falls. The proposed system involves extending the sewer lines to Bolton Landing on

the west shore and to Pilot Knob on the east shore. This would accommodate all of the concentrated population areas in the south end of the lake. There are several other populated centers in the north end of the lake but these are essentially insignificant. Pumping the wastewater out of the basin was proposed as an ultra-conservative point of view in order to prevent any existing or potential contribution of nutrients to Lake George. This proposed system has raised some questions regarding the quality of the existing effluent vs. the lesser degree of treatment which would be accomplished at the new Glens Falls Sewage Treatment Plant. There are other questions concerning the impact of diverting 5.5 mgd (20800 m^3/day) out of the Lake George Basin into the Hudson River Basin. Another concern which has not received much attention is the additional nutrient loading which would be imposed on the Hudson River Basin.

A prime concern at this time of rising energy costs is the relative cost of the proposed activated sludge system compared with the existing rapid infiltration system at Lake George. It is difficult to make an equitable balanced evaluation since the existing system provides the equivalent of tertiary treatment, whereas the proposed system will provide only secondary treatment. Regardless of this difference, an effort was made to determine the energy consumed at the existing Lake George Village Sewage Treatment Plant and to compare this with the anticipated energy consumption in treating the same amount of wastewater with the proposed system.

DESCRIPTION OF THE EXISTING TREATMENT PLANT

The wastewaters from both the Village of Lake George and the immediately surrounding areas of the Town of Lake George are collected by gravity to a central location in a park adjacent to the lake. Here a new pumping station forces the sewage approximately 55 m (180 ft) through a 1.6 km (1 mi) force main to the treatment plant. The design flow of the existing plant is 1.75 mgd (6600 m^3/day) with peak maximum summer flows in the range of 1.25 mgd (4700 m^3/day). Originally the summer flows were three times the winter winter flows, but recently the summer flows have been only double the winter flows, primarily due to the greater year-round population, regional ski areas, and the winter ice carnival held during the month of February.

The existing treatment plant is shown in Figure 1. The sewage enters

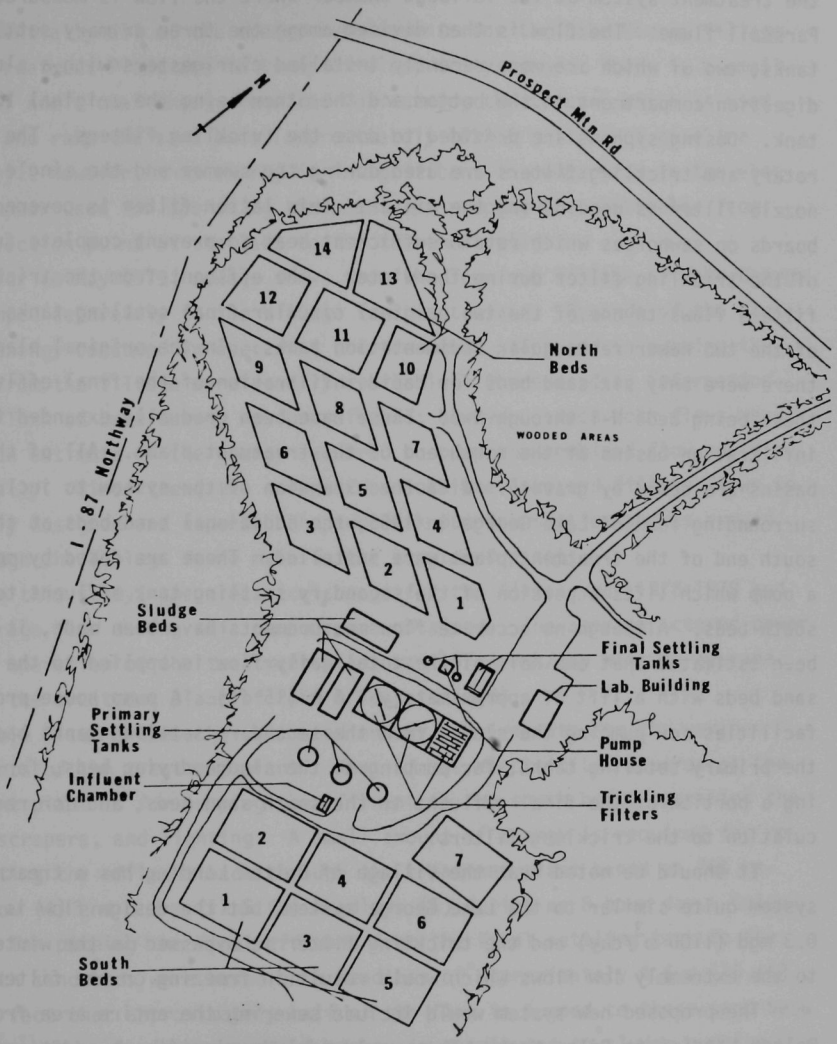

FIGURE 1
PLAN OF THE LAKE GEORGE SEWAGE TREATMENT PLANT

the treatment system at the influent chamber where the flow is measured in a Parshall flume. The flow is then divided among the three primary settling tanks, two of which are more recently installed clarigesters with a sludge digestion compartment in the bottom and the other being the original Imhoff tank. Dosing syphons are provided to dose the trickling filters. The two rotary arm trickling filters are used during the summer and the single fixed nozzle filter is used during the winter. This latter filter is covered by boards on sawhorses which retain sufficient heat to prevent complete freezing of the trickling filter during the winter. The effluent from the trickling filters flows to one of the two original circular final settling tanks or one of the two newer rectangular sedimentation tanks. In the original plant there were only six sand beds for rapid infiltration of the final effluent, these being beds N-1 through N-6. These have been gradually expanded to 14 infiltration basins at the north end of the treatment plant. All of these basins are dosed by gravity. With the expansion of the system to include the surrounding Town of Lake George in 1965, the additional sand beds at the south end of the treatment plant were installed. These are dosed by means of a pump which lifts a portion of the secondary settling tank effluent to the south beds. Although no accurate flow measurements have been made, it has been estimated that one half of the total daily flow is applied to the south sand beds with a lift of approximately 4.6 m (15 ft). A pump house provides facilities for pumping the sludge from the secondary settling tanks back to the primary settling tanks, for pumping to the sludge drying beds, for pumping a portion of the final effluent to the south sand beds, and for recirculation to the trickling filters.

It should be noted that the Village of Bolton Landing has a treatment system quite similar to the Lake George system, but the design flow is 0.3 mgd (1100 m^3/day) and the trickling filter is bypassed in the winter due to the extremely low flows which would result in freezing on the filter.

The proposed new system would include sewering the entire area from Bolton Landing to Pilot Knob. This would include significant additional sewers and the use of 23 pumping stations through the Lake George Basin. The combined sewage would be pumped out of the Lake George Basin into the Hudson River Basin through a force main having a lift of approximately 73 m (240 ft).

ENERGY USAGE AT THE EXISTING LAKE GEORGE VILLAGE SEWAGE TREATMENT PLANT

Essentially all of the energy requirements for the Lake George Village Sewage Treatment Plant are in terms of electricity usage. There is a small amount of additional energy required for running the tractor which is used to disc and rake the sand beds (approximately 1623.4 gallons of gasoline annually). However, this energy is insignificant in comparison to the electrical power used for operating the plant and pumping the wastewater from the collection point at the lake to the treatment plant.

Prior to May 1979, sewage flows from the Town and Village of Lake George were pumped separately to the Village treatment plant. Records of the electrical energy consumed in operating the treatment plant and in pumping the sewage from the Village were readily available. Corresponding information for the Town of Lake George was unavailable; however, since both flows were known, an estimate of the energy required to pump the combined flow could be made easily. Since then, the sewage from both the Town and Village flows to the newly designed and expanded Shepard Park pumping station and is pumped on through the Village force main to the treatment plant.

For this study, the electrical usage for the fiscal year 1978-1979 and the average flow during this period were obtained. The average metered power for the operation of the treatment plant was 197.4 kwh/day with an average flow at the treatment plant of 2650 m^3/day (.7 mgd).

Table 1 summarizes the estimated energy requirements for the various components of the treatment plant. Most of the electricity required at the treatment plant is for pumping the effluent to the south beds, operating the sludge scrapers, and lighting. A small amount of energy is required for recirculating the effluent back through the treatment plant based on 20% return flow (4) and 5.3 m (17.5 ft) headloss (5) during an 8 hour day. Calculations for pumping the final effluent from the final settling tank to the new south sand beds were based on a headloss of approximately 4.6 m (15 ft) and the fact that approximately half of the flow is pumped to these sand beds during any day. A minor amount of energy is attributed to sludge pumping.

Power ratings for the sludge scraping equipment were not available. Estimates were therefore based on the design flow and are the same as those for the proposed system at Glens Falls. Since these estimates were based on rectangular settlers, the power required to drive the scrapers is related

to the length of settler which in turn depends on design flow. Lighting and miscellaneous power (operation of hand tools, office and laboratory equipment, etc.) make up the bulk of the remaining estimated energy usage.

TABLE 1. ELECTRIC POWER CONSUMPTION AT THE LAKE GEORGE VILLAGE SEWAGE TREATMENT PLANT (kwh/day)

	2650 m^3/day (.7 mgd) Average Flow	6600 m^3/day (1.75 mgd) Design Flow	20800 m^3/day (5.5 mgd) Projected Flow
Influent Pumping (55 m [180 ft] elevation head)	644	1523	4787
Treatment Plant			
Recirculation (20% flow) (4)	4.17	10.4	32.8
Effluent Pumping[1] (4.6 m [15 ft] headloss)	26.8	63.5	199.5
Sludge Pumping[2]	.03	.08	.24
Sludge Removal[3]	77.6	77.6	154
Lighting & Misc. Power	75	75	135
Treatment Plant Sub-total	183.6	226.58	521.54
TOTAL	827.6	1749.58	5308.54
Metered Treatment Plant Power (June 1978 - May 1979)	197.4		

[1] South beds with 50% flow

[2] .0052 mgd sludge/mgd assumed; 5 ft headloss; 8 hour operation

[3] Primary and secondary settlers (combined)

Finally, similar estimates were made with the anticipated flow for the year 2000 from the proposed sewered area around Lake George. For a flow of 20800 m^3/day (5.5 mgd), the power consumed in operating the plant was estimated to be 521.24 kwh/day.

Pumping the sewage from the lake to the treatment plant requires more energy than does plant operation. From June 1978 to May 1979, an average of 326 kwh/day was used to pump the Village flow (.46 mgd average) to the treatment plant. For the average (total) combined Village and Town flow, an estimated 644 kwh/day would be used. At the design flow, this increases to 1523 kwh/day and at the projected 20800 m^3/day (5.5 mgd), this would require 4787 kwh/day.

ENERGY UTILIZATION IN THE PROPOSED WARREN COUNTY SEWER SYSTEM TO REPLACE THE LAKE GEORGE VILLAGE SEWAGE TREATMENT PLANT

The proposed Warren County sewer system involves the collection of all wastewaters from Bolton Landing to Pilot Knob. This would be collected at numerous points utilizing 23 pumping stations throughout the Lake George Basin. Finally, all of the wastewater from the Lake George Basin would be pumped over the hill out of the basin and into the Hudson River Basin. At Glens Falls a treatment plant would be built to treat this wastewater along with the wastewaters from the City of Glens Falls, numerous surrounding suburbs and several large industries in the area. The total design flow for that treatment plant is 60,000 m^3/day (16 mgd). Although it is realized that there is economy of size, it was considered fair to calculate the energy requirements based on the existing design flow and the projected flow for the year 2000 from the Lake George area only.

The estimated energy consumption (in terms of kilowatt-hours per day) for the various unit processes of a conventional activated sludge treatment plant at the above flows is summarized in Table 2. The largest single expenditure of energy in this system is for pumping the wastewater out of the basin. From the facilities plan (4) the best estimate of the lift to pump the effluent out of the basin was estimated to be 240 ft (73 m). This does not include the friction headloss in the pipe which would be longer than the 1.6 km (1 mi) force main presently in use. The additional pumping required for the other 22 stations is not considered. If the present sewer system is expanded, it is assumed the pumping stations would be essentially the same regardless of the final discharge of the effluent. Adjustments in power consumption at an operational average flow of 2650 m^3/day (.7 mgd) are also noted for those unit processes which depend on flow.

Mathematical models for estimating electrical energy consumption for the various unit processes involved in municipal waste treatment were developed by Wang and Wang (6) based largely on studies made by Smith (5). Except for influent pumping and chlorination, the values reported for the various unit processes are based on these models. In determining the power required for influent pumping, the Wang and Wang model applies only to a 30 ft dynamic head. As previously mentioned, the elevation head alone to pump the sewage out of the basin is 240 ft. The power requirements for chlorinating the effluent are negligible if chlorine gas is used. Smith (5) reports only

.72 kwh/day is required in both a 1 and 10 mgd plant (corresponding to 400 and 2000 lb chlorine per day). The Wang and Wang model predicts 7.8 and 41.6 kwh/day for a 1.75 and 5.5 mgd plant. These values would be reasonable if liquid chlorine were used; the additional energy is required to evaporate the chlorine.

TABLE 2. ELECTRICAL ENERGY CONSUMPTION (KWH/DAY) FOR A CONVENTIONAL ACTIVATED SLUDGE WASTEWATER TREATMENT PLANT (6)

	2650 m^3/day (.7 mgd) Average Flow	6600 m^3/day (1.75 mgd) Design Flow	20800 m^3/day (5.5 mgd) Projected Flow
Influent Pumping (73 m [240 ft] elevation head) (4)	858.6	2030.8	6328.7
Screening & Comminution	5.1	5.1	30.6
Grit Removal	1.8	1.8	2.5
Primary Sedimentation	38.8	38.8	77
Secondary Sedimentation	38.8	38.8	77
Plant Lighting & Maintenance	70.5	70.5	134.3
Chlorination (5)	.72	.72	.72
Diffused Air Flotation	372	931	2926
Return Activated Sludge (50% flow)	31	79	242
Sludge Pumping	1.9	4.7	14.7
Sub-Total	1419.2	3201.2	9887.5
Sludge Handling			
Gravity Thickening	9.0	11.8	16.7
Anaerobic Digestion	102.5	168	321
Vacuum Filtration	47.4	81	201.3
Incineration	47.5	70	150
Sub-Total	206.4	330.8	689
TOTAL	1625.6	3532	10,576.5

The proposed system for Glens Falls employs diffused air activated sludge with anaerobic digestion, mechanical dewatering (centrifuges) and landfill (4). This would correspond to the sludge handling scheme II defined by Smith (5). The total electric power required at the design flow was estimated to be 3532 kwh/day and 10576.5 kwh/day at the projected year 2000 flow.

Table 3 summarizes the energy consumption for the existing treatment plant and that required to treat the same flows with a conventional activated sludge system. Values are given for the average and design flows at the existing treatment plant and the projected flow for the year 2000. Since pumping the raw sewage to the treatment plant is a very large portion of the energy required, these values are listed separately from those for the treatment plant itself. Whereas the overall energy utilization in the proposed Warren County system would be approximately twice that of treating the same waste flow in the Lake George Village Sewage Treatment Plant, the energy requirements for treatment alone are almost 10 times those for the existing system at both the design and projected year 2000 flow.

TABLE 3. TOTAL ELECTRICAL POWER CONSUMPTION (KWH/DAY)

	2650 m^3/day (.7 mgd)	6600 m^3/day (1.75 mgd)	20800 m^3/day (5.5 mgd)
Lake George Village (Rapid Infiltration)			
Influent Pumping (55 m [180 ft] elevation head)	644	1523	4787
Treatment Plant	183.6	226.58	521.54
TOTAL	827.6	1749.58	5308.54
Proposed System (conventional activated sludge)			
Influent Pumping (73 m [240 ft] elevation head)	858.6	2030.8	6382.7
Treatment Plant	767.0	1501.2	4193.8
TOTAL	1625.6	3532	10576.5

SUMMARY

It is difficult to make a direct comparison between the energy measured for the existing treatment plant and the estimated utilizatin of energy in the proposed treatment system. Furthermore, for such a comparison, the costs of pumping the sewage from around the basin to the major force main at Shepard Park at Lake George Village should be included. One could argue that for either scheme this pumping cost would be the same; however, the alternative would be to continue using septic tanks in the presently

non-sewered areas. This would eliminate the need for the 22 pumping stations surrounding the lake. Thus, to continue the existing program for wastewater treatment in the Lake George area (the existing system plus septic tanks) would represent the least energy consuming alternative. Further study is underway to evaluate the energy requirements for the existing septic tanks and sewer system. Once this is accomplished, a better comparison can be made.

Some estimates can be made of the differences in the cost of operation between the two systems. At the design flow of 6600 m^3/day (1.75 mgd), the difference in energy consumption between the proposed system and the existing system is approximately 1782 kwh/day. At $0.0467 per kwh this amounts to a difference of $30,375 per year. The same calculation for the year 2000 flow shows a difference in energy usage of 5268 kwh/day, at an estimated future cost of 8¢ per kwh (4), almost $154,000 per year additional.

It is very obvious that the proposed activated sludge treatment system for the proposed Warren County Sewer System accomplishing only secondary treatment would be considerably more energy intensive and expensive to operate than the existing Lake George Village Sewage Treatment Plant which presently accomplishes the equivalent of tertiary treatment of the wastewaters. Upgrading the proposed system to achieve equivalent tertiary treatment would impose an even greater energy demand.

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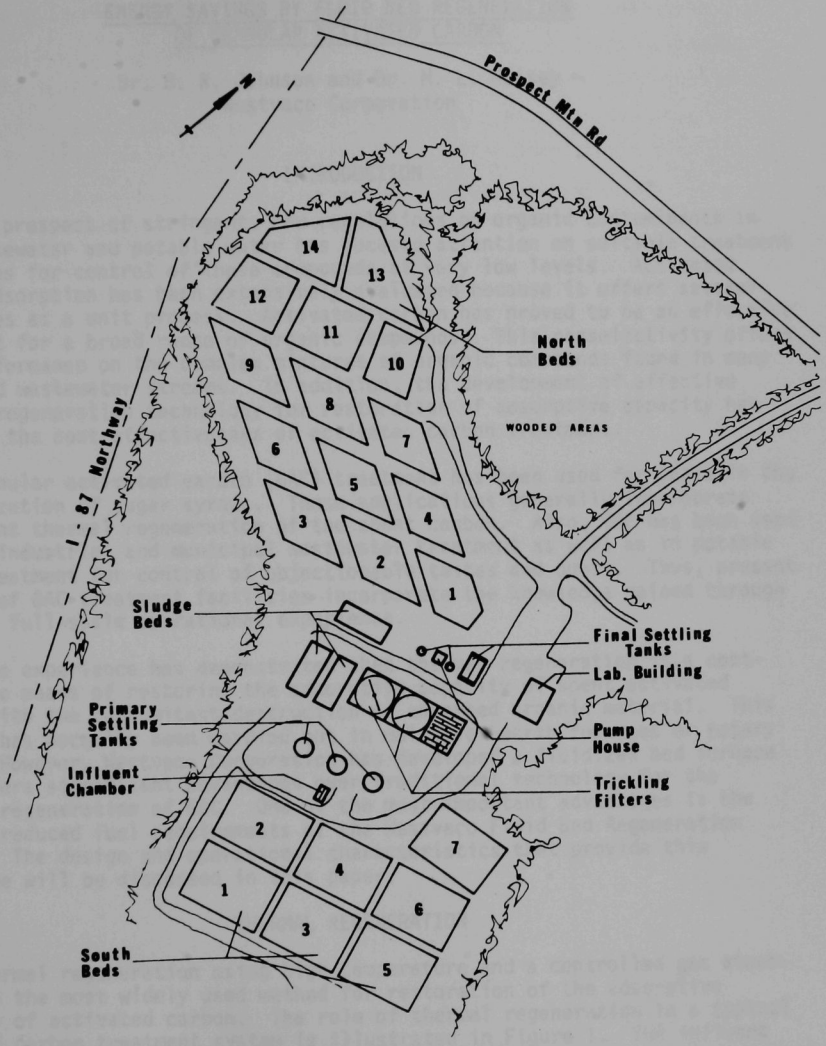

FIG 1.- Plan of the Lake George Village Sewage Treatment Plant

ENERGY SAVINGS BY FLUID BED REGENERATION OF GRANULAR ACTIVATED CARBON

Dr. H. R. Johnson and Dr. M. L. Massey
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INTRODUCTION

The prospect of stringent, new regulations on organic contaminants in both wastewater and potable water has focused attention on suitable treatment techniques for control of these compounds at very low levels. Activated carbon adsorption has been extensively evaluated because it offers several advantages as a unit process. Activated carbon has proved to be an effective adsorbent for a broad range of organic compounds. This nonselectivity offers good performance on the complex mixtures of organic compounds found in many water and wastewater streams. In addition, the development of effective thermal regeneration technology for restoration of adsorptive capacity has improved the cost-effectiveness of activated carbon treatment.

Granular activated carbon (GAC) treatment has been used for years in the decolorization of sugar syrups. These applications generally incorporate subsequent thermal regeneration of the spent carbon. Also, GAC has been used in both industrial and municipal wastewater treatment as well as in potable water treatment for control of objectionable tastes and odors. Thus, present designs of GAC treatment facilities incorporate the knowledge gained through years of full-scale operational experience.

This experience has demonstrated that thermal regeneration is a cost-effective means of restoring the adsorptive capacity of spent activated carbon with the concomitant destruction of adsorbed organic material. This process has normally been carried out in multiple-hearth furnaces or rotary kilns. However, Westvaco Corporation has developed a fluidized bed furnace that offers significant advantages over traditional technology for the thermal regeneration of GAC. One of the most important advantages is the greatly reduced fuel requirements of the Westvaco Fluid Bed Regeneration System. The design and operational characteristics that provide this advantage will be discussed in this paper.

THERMAL REGENERATION

Thermal regeneration using high temperature and a controlled gas atmosphere is the most widely used method for restoration of the adsorptive capacity of activated carbon. The role of thermal regeneration in a typical granular carbon treatment system is illustrated in Figure 1. The influent flow is treated by filtration through a bed of granular carbon where dissolved organic contaminants are removed by adsorption. Once the capacity of the carbon for organic compounds is exhausted, the spent carbon is removed from the adsorption column and transferred to a storage vessel for regeneration. This transfer is normally carried out by hydraulically conveying the carbon through a pipeline in a water slurry that may typically contain about 1 pound of GAC per gallon of water.

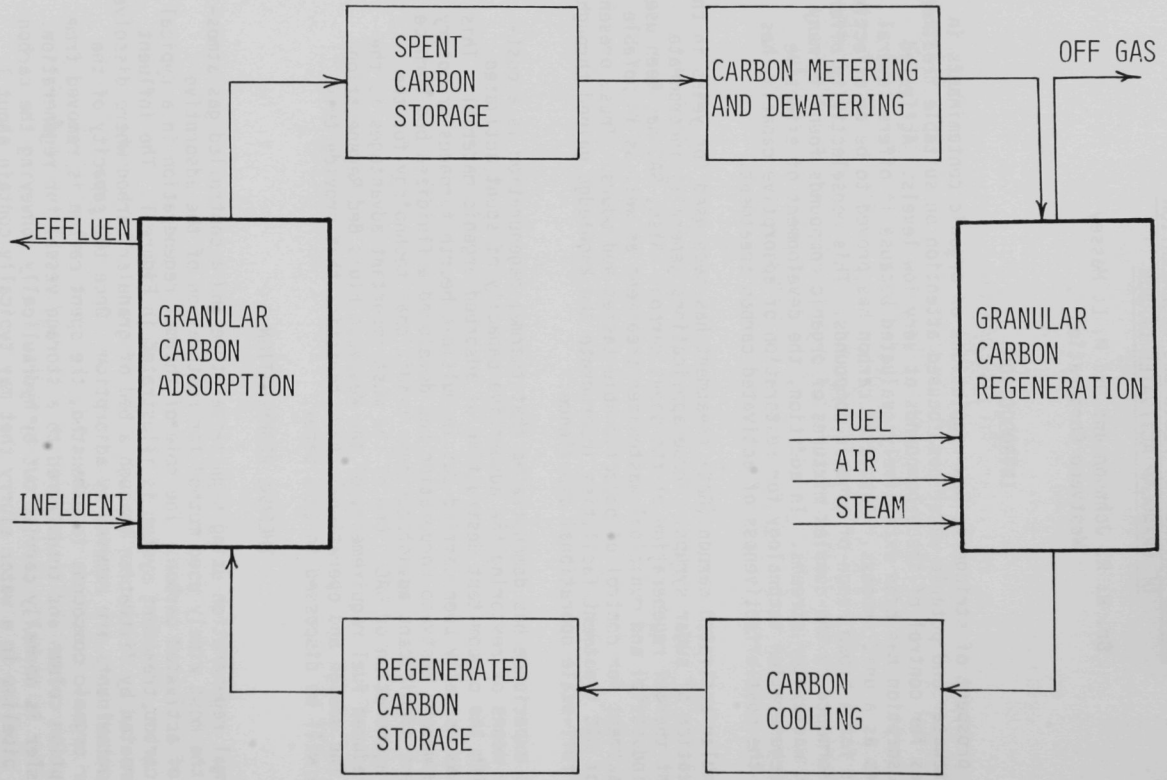

Figure 1

TYPICAL GRANULAR CARBON TREATMENT SYSTEM

The spent carbon is then removed from the storage vessel, again by water slurry to a metering and dewatering device. Typically, this is an inclined screw conveyor where the slurry water is drained from the carbon, reducing the moisture content of the spent carbon to about 35-50% by weight. Wet, spent carbon is then fed into the regeneration furnace where the carbon is first dried, and then regenerated. Fuel, air and steam are added to the regeneration furnace to supply the heat requirements as well as the oxidation gas for the final regeneration reaction. The regenerated carbon is discharged from the furnace, cooled by water quench, and transferred to a storage vessel or recharged directly to the adsorption column.

Table 1 illustrates the reactions occurring within the regeneration furnace itself. The granular carbon containing the adsorbate and 35-50% moisture is heated to about 250-300°F to dry the carbon. Most of the adsorbate normally remains within the carbon pores during the drying step, although some of the more volatile organic compounds may be vaporized off the carbon. In the second step, the temperature of the carbon is increased to about 900-1400°F, where a majority of the organic compounds are removed by volatilization during the pyrolysis step. Depending on the characteristics of the adsorbed material, 50 to well over 90% by weight of the adsorbate is removed during this step. That part of the adsorbate which is not removed by volatilization remains as a char residue in the pores of the activated carbon. This char residue is removed in the last and most important step of the regeneration process. The char residue is selectively removed by reaction with water vapor at temperatures of 1500-1800°F to form CO and H_2.

TABLE 1

PROCESS STEPS IN REGENERATION OF GRANULAR CARBON

Drying

$$\text{Carbon} \bullet \text{Adsorbate} + H_2O \xrightarrow[300°F]{\text{Heat}} \text{Carbon} \bullet \text{Adsorbate} + H_2O$$

Pryolysis

$$\text{Carbon} \bullet \text{Adsorbate} \xrightarrow[900\text{-}1400°F]{\text{Heat}} \text{Carbon} \bullet C^* + \text{Volatilized Adsorbate}$$

Regeneration

$$\text{Carbon} \bullet C^* + H_2O \xrightarrow[1500\text{-}1800°F]{\text{Heat}} \text{Carbon} + C^*O + H_2$$

Control of the regenerator gas composition during the pyrolysis and regeneration stages is an essential part of controlling the process. The elimination of excess oxygen from the combustion gases is required to minimize carbon burn-off losses during regeneration. Granular carbon losses during the adsorption/regeneration cycle have been observed to range generally from 3-10% in properly controlled systems utilizing coal-based carbons, with 5-7% the most common range.

REGENERATION EQUIPMENT

Multiple-hearth furnaces and rotary kilns have traditionally been used for regeneration of granular carbon, with the multiple-hearth furnace being far more common. This type of furnace consists of several self-supporting refractory hearths located one above the other contained within a cylindrical, refractory-lined shell. The furnace also has a central, rotating shaft that drives rabble arms to move the carbon from hearth to hearth. The central shaft and rabble arms are hollow to allow air to flow through these parts for cooling. Wet, spent feed is introduced onto the top hearth where the carbon is dried. The carbon is then moved down to the middle hearths where the pyrolysis step is carried out. Regeneration of the carbon is performed on the bottom hearths. The heat requirements for the regeneration process are met by burning fuel in two or three burners per hearth on the bottom three hearths. Steam for reaction with the char residue is added through nozzles, generally on the bottom three hearths. The off-gas stream from the multiple-hearth furnace normally passes through an afterburner for incineration of the volatilized organics followed by a wet scrubber.

Reported energy requirements for multiple-hearth furnaces are about 4000 BTU's/pound of regenerated carbon for regeneration and 4000 BTU's/pound for afterburner requirements (1). These values will vary depending upon the application and nature of the adsorbate.

WESTVACO FLUID BED REGENERATION SYSTEM

The Westvaco Fluid Bed Regeneration System is illustrated in Figure 2. This system is a two-stage fluidized bed process with the drying step accomplished in the upper stage and the pyrolysis and regeneration steps carried out in the lower stage. The furnace consists essentially of a refractory-lined steel shell with staging provided by two stationary plates which serve to distribute the fluidizing gas and support the fluidized beds. The wet, spent carbon (35-50% water) is fed from a dewatering screw into the upper drying stage. In this stage, the spent carbon is completely dried (the discharge material containing less than 1% water) by operating the fluidized bed at 250-300°F. The amount of heat entering the drying bed is controlled at a level above that needed to dry the wet, spent feed. To soak up the excess heat and to provide close control of the drying stage temperature, additional water is injected into the bed. This system also functions to maintain the bed temperature even when the wet, spent feed is completely cut off.

The dried, spent carbon flows by gravity down into the lower fluidized bed where the pyrolysis step is first carried out at temperatures of 900-1400°F followed by the regeneration step at temperatures of 1500-1800°F. Heat for the process is supplied by burning fuel in either one or two burners which fire into a combustion chamber which is an integral part of the furnace. Steam is also injected into this combustion chamber to serve two purposes: to provide the steam needed to achieve the relatively high regeneration reaction rates and to lower the combustion gas temperature. The regenerated carbon discharges into a water filled quench tank for cooling, and it is then transported into either a storage tank or back into the adsorption system.

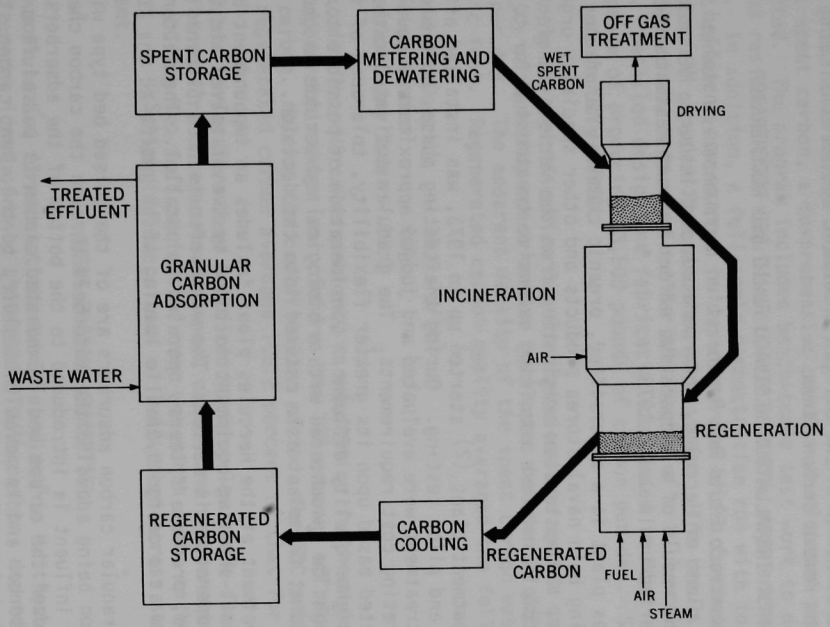

Figure 2

WESTVACO FLUID BED REGENERATOR IN A TYPICAL ADSORPTION SYSTEM

A unique feature of the Westvaco system is the injection of air into the space above the lower fluidized bed. This air serves to burn the organic compounds volatilized from the carbon during pyrolysis in addition to the H_2 and CO formed by the steam oxidation reaction. Part of the heat derived in this zone is radiated down to the regeneration fluidized bed with the remainder serving to provide heat for carbon drying. Incineration in this manner can serve in many applications to eliminate the requirement for an external afterburner with the off-gas treatment system consisting only of a wet scrubber to remove carbon fines.

FIRST INSTALLATION-WESTVACO FLUID BED REGENERATOR

The first Westvaco Fluid Bed Regeneration Furnace was installed at the waste treatment plant of Hercules, Incorporated, Hattiesburg, Mississippi. This unit was the result of a cooperative effort between Westvaco and Hercules and started up in July, 1978.

The Hercules plant is a diversified, organic chemical manufacturing facility producing both naval stores products and other specialty organic chemicals. Thus, the wastewater originating from the various manufacturing processes is a complex mixture requiring extensive treatment prior to discharge.

The GAC treatment plant,[2] started up in 1973, was installed after full laboratory and pilot testing. During the testing phase, both biological and carbon treatment were evaluated and judged approximately equal in capital and operating cost requirements. The granular activated carbon system was selected based upon its greater flexibility, tolerance of feed variations and higher quality effluent. Consideration of possible toxicity problems that might be encountered with a biological system due to the products produced at the plant aslo entered into the decision.

Primary treatment at the Hercules plant includes an impoundment basin with associated oil-skimming equipment followed by two dissolved air flotation clarifiers operating in series. The wastewater is then filtered by three mixed media, pressure filters, operated in parallel. The filters are operated downflow at a design hydraulic loading of 13 gpm/ft^2.

The three granular carbon adsorbers are of the pulsed bed type with regenerated carbon being added to the adsorbers through the carbon charge tanks. Filtered influent is introduced to the bottom of the adsorbers and flows upward through the carbon bed. Exhuasted carbon is pulsed from the bottom of the adsorber and is moved in a slurry to the spent carbon storage tank. The adsorbers were originally designed for packed bed operation but have recently been modified to expanded bed types.

The original carbon regeneration furnace was a five-hearth furnace, 12 feet in diameter with a rated capacity of 33,600 pounds of spent carbon per day. The furnace was fired with natural gas and had provisions for steam supply. The system also included an afterburner to insure complete combustion of organic compounds in the furnace off-gas and a wet scrubber for particulate control.

Recurring maintenance problems with the multiple-hearth furnace occurred during the operation of the plant. These maintenance problems resulted in long downtimes of over a week's duration for furnace cooling, maintenance and restarting. This was unacceptable and prompted Hercules personnel to investigate regeneration furnace alternatives. One of these alternatives was a Westvaco Fluid Bed Regeneration Furnace.

In order to evaluate the ability of the fluid bed furnace to regenerate Hercules spent carbon, a comprehensive process development program was implemented. The program included bench-scale test work to determine temperature and residence time requirements for suitable regenerated carbon quality. In addition, a full-scale demonstration run with tonnage quantities of spent carbon was conducted in the commercial-sized furnace located at the Charleston, SC research laboratories of Westvaco. The results of this test work provided the data required for the design of a full-scale regeneration furnace. Detailed design and fabrication commenced in May, 1977 for a furnace capable of producing 33,500 pounds of carbon per day. Start-up of the furnace was in July, 1978.

During the certification run for Hercules, the system demonstrated a capability of handling spent carbon feed rates between 29,850 and 45,100 pounds per day. The apparent density of the spent carbon was 39.5 pounds per cubic foot. Regenerated carbon quality averaged the following:

Apparent Density - 33.0 lb/ft^3
Iodine Number - 684
Decolorizing Index - 11.3

In addition, Hercules performs an adsorption isotherm periodically using their regenerated carbon and compares the TOC removal efficiency with that of virgin carbon. During the certification period, the TOC removal efficiency of the regenerated carbon averaged 108.2 percent of that for virgin carbon.

Carbon losses have been monitored by Hercules since start-up by measurement of inventory changes in the spent carbon and regenerated carbon storage tanks. During two test periods of over one month each, carbon losses were about 3% and 5% which include a loss of 1-2% taken across sand removal equipment.

HEAT ECONOMY BENEFITS

The fluid bed furnace at the Hercules plant in Hattiesburg, MS requires a total energy input of only 2100-3800 BTU/lb carbon, depending on the processing rate, as presented in Table 2. About two-thirds of the total energy requirement is derived from the combustion of fuel within the furnace with the other one-third's being used to generate the steam supplied to the furnace. These values for energy were calculated using continuously measured flow rates for the fuel and steam, and for the fuel, were confirmed through use of totalizing meter. As indicated in the table, an afterburner is not used for the Hercules installation. Treatment of the off-gas using only a wet scrubber has proven sufficient for this system.

TABLE 2

MEASURED FUEL AND STEAM CONSUMPTION FOR CARBON REGENERATION AT HERCULES, INCORPORATED, HATTIESBURG, MISSISSIPPI

REGENERATION FURNACE	CARBON PROCESSING RATE		FUEL CONSUMPTION		STEAM CONSUMPTION		TOTAL ENERGY
	Daily Lb/Day	% of Rated Capacity (A)	Furnace BTU/Lb	Afterburner BTU/Lb	Lb/Lb	BTU/Lb	BTU/Lb
FLUID BED AT	37,307	111%	1401	0	0.65	754	2155
HERCULES	31,932	95%	1472	0	0.68	789	2261
	21,000(B)	63%	2474	0	1.14	1323	3797
MULTIPLE-HEARTH AT HERCULES	21,000	63%	6000-7000		1.0	1161	7161-8161
TYPICAL MULTIPLE-HEARTH (1)	--	-	4000	4000	1.0	1161	9161

(A)Rated Capacity - 33,500 Lb/Day

(B)Present Operation

Based on connected horsepower, the power useage for the Hercules installation ranges from about 0.05 to 0.09 KWHr/lb carbon, depending on the processing rate. The requirement for the Fluid Bed is essentially equal to values reported in the literature for the multiple-hearth furnace.

Operation of the multiple-hearth furnace originally used by Hercules required a total energy input of approximately 7000-8000 BTU/lb carbon. This total energy consumption by the Hercules multiple-hearth agrees with the typical values of about 7500-9500 reported in the literature (1). As demonstrated by over one year's operation in the Hercules plant, the Westvaco Fluid Bed Regeneration System uses only one-third to one-half the energy required by a multiple-hearth furnace.

For a mid to large size regeneration system, the savings in energy with the fluid bed could amount to 250,000 to 1,000,000 gallons per year of fuel oil or the equivalent in natural gas. The economic savings in fuel alone would, of course, also be in the 250,000 to 1,000,000 dollars per year range.

The primary factor contributing to the heat economy of the Westvaco Fluid Bed System is the elimination of the requirement for an external afterburner in many cases. This by itself reduces the energy requirement by about 4000 BTU/lb carbon or near one-half the total requirements for a multiple-hearth system. Other factors contributing to the reduced energy requirements include:

1) Utilization of heat from the incineration zone.

The amount of heat generated in the incineration zone by combustion of the volatilized organics, hydrogen, and carbon monoxide is, in many cases, at least equal to the amount of heat generated by combustion of the primary fuel. This raises the temperature in the incineration zone to levels well above the maximum temperature in the lower fluidized regeneration bed, thus allowing a significant amount of heat to be radiated from the hot refractory walls down into the fluidized bed of carbon. The primary fuel requirements are reduced by 10-15% through utilization of this radiant heat.

2) Absence of moving parts in the high temperature zone that require cooling air.

The multiple-hearth furnace has to use large quantities of cooling air to protect the central shaft and rabble arms. This air is normally heated from ambient conditions to 400-600°F before discharge to atmosphere. The absence of moving parts in the Westvaco Fluid Bed that require cooling air results in a fuel savings of about 10% in comparison to the multiple-hearth furnace.

3) Lower off-gas temperatures from the drying section.

The gas supplying the heat for drying in the upper section reaches equilibrium with the bulk bed temperature, thus exiting the drying stage at 250-300°F. This is significantly lower than the normal gas temperature, 400-600°F, above the top hearth of a multiple-hearth unit and thus contributes

to fuel economy for the fluid bed furnace.

REGENERATION OF POTABLE WATER CARBONS IN A WESTVACO FLUID BED REGENERATOR

The Westvaco Fluid Bed Furnace has also been selected for use in two potable water applications, one at Manchester, NH and the other at Cincinnati, OH. Both units are sized to produce six tons per day of regenerated carbon. The Manchester unit is presently being evaluated after a start-up in May, 1979. The Cincinnati system is scheduled for a late 1979 start-up.

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SESSION 8

SLUDGE MANAGEMENT

Chairpersons: James Smith

Dennis Weeter

EVALUATION AND OPTIMIZATION OF THE ANAEROBIC DIGESTION OF MUNICIPAL SLUDGE: SLUDGE AS AN ENERGY RESOURCE

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Rexnord, Inc., Environmental Research Center

G. Selin, F. Nelson
Kenosha Water Utility

INTRODUCTION

Process Description

Anaerobic digestion can be defined as the process of microbiological degradation of organic matter to methane and carbon dioxide under anaerobic conditions. The process consists of two distinct phases. The first phase, or non-methanogenic phase, can be further subdivided into two reactions. The first reaction is the enzymatic hydrolysis of complex organic matter to less complex soluble organic compounds. The second reaction involves the fermentation of the hydrolysis products by a group of facultative and anaerobic bacteria to simple organic compounds (predominantly fatty acids). The second phase, or methanogenic phase, consists of the fermentation of the fatty acids and alcohols to methane and carbon dioxide. A group of substrate-specific, strictly anaerobic bacteria are involved in this second phase.

Despite widespread application of the anaerobic digestion process in the field of municipal sludge management, there is a dearth of information available relative to process fundamentals. The design of anaerobic systems in the past has been based on the engineering judgement of the designer and on empirical knowledge gained through years of satisfactory performances.

Furthermore, few studies have been conducted with the specific objective of process optimization. Since the methane gas produced can be a substantial energy source for the treatment plant, it would be of benefit to optimize the process according to net energy yields. Net energy yield can be defined as latent recoverable energy in the form of methane produced by the system minus heat energy necessary to maintain system temperature.

Study Objectives

The objective of this research was to evaluate and optimize the anaerobic digestion process for municipal sludges. Although the primary optimization variable was net energy yield, other response variables such as sludge stabilization, process stability, digested sludge dewaterability, sidestream quality and land disposal suitability were also documented to provide an overall solids management optimization scheme.

Municipal sludges (primary and waste activated) from the Kenosha, WI Water Pollution Control Plant were used as substrates for three different anaerobic digestion processes. The processes were high rate digestion of mixed primary and waste activated sludge (WAS), segregated primary and WAS; and anaerobic contact with WAS. All processes were first tested on a pilot scale

(150ℓ) and then repeated on a full-scale basis where possible. The control variables were volumetric organic loading rates (0.92 to 6.86 kg VS/m^3·d), predigestion solids thickening, hydraulic retention time (HRT) and sludge source.

Although a fairly good data base exists for the anaerobic digestion of primary and mixed primary and waste activated sludges, very little work has been performed on the digestion of waste activated sludge alone. Another objective of this study, therefore, was to evaluate the digestibility and subsequent dewaterability of WAS alone, both on a pilot and full-scale basis. This is of special significance since it has often been suggested that some of the current problems associated with anaerobic digestion might be attributable to the incorporation of increasing proportions of biological sludges. Handling these two sludges in segregated streams, and optimizing the operating conditions for each, could enhance the efficiency of the overall solids handling train.

RESULTS OF PILOT-SCALE TESTING

High-Rate, Single-Stage Digestion

Tables 1, 2 and 3 summarize stead-state averages for sludge characteristics, gas production rates and dewaterability characteristics, and net energy yields, respectively, for all digestion processes and sludge types. These data are also illustrated in Figures 1 and 2. It is readily apparent that the digestion of primary sludge alone exhibited significantly higher volatile solids removals, gas production rates and net energy yields than both combination sludges and WAS alone. One could infer from this result that blending WAS with primary sludge prior to digestion was antagonistic. The digestion of combination sludges was less efficient than the average of primary and waste activated sludges digested separately. However, this result might have been influenced by the low fraction of volatile solids in the combination sludges (only 46 to 48% of the TS).

Another general conclusion concerns the effect of digestion time on subsequent dewaterability. The cost for chemical conditioning to the optimum dose for dewatering varied inversely with VOLR (directly with HRT) for all sludge types. Once conditioned to the optimum dose, there was little difference in the absolute degree of dewaterability, at least within the sensitivity of the bench-scale tests employed. Vacuum degasification of the digested sludges provided negligible gravity thickening benefits, while chemical conditioning to the optimum dose for filterability significantly enhanced the settleability of digested sludges.

It should be noted that any inter-sludge comparison of digester performance must be tempered by the realization that real-world sludge composition, even within one particular sludge type, can vary with time. Although general conclusions might have broad applicability, specific results pertain only to the test conditions unique to this study. Keeping this limitation in mind, the following conclusions can be drawn from the pilot results for each sludge type:

1. Primary sludge alone - Digestion of this sludge type produced the highest gas and net energy yields, highest volatile solids removals and

lowest land area requirements for ultimate disposal. On the negative side, this sludge type produced the lowest gas quality and highest VA/BA ratios and UVA concentrations. The extremely high VA/BA ratios and UVA concentrations at high VOLR's might have been the result of intermittent acidic industrial discharges to the Kenosha Plant near the end of the pilot study. The uncharacteristically high volatile solids removals were due to the formation of a scum layer in the pilot units. This was verified by grease removals ranging from 80-90% and visual inspection when the units were dismantled. Since such a scum formation did not occur for the other sludge types, the digestion of primary sludge may require a higher mixing energy and/or a different type of mixing (the pilot units were mixed with sludge recirculation at a volume turnover time of approximately 5-7 minutes). A VOLR of 2.0 $kg/m^3 \cdot d$ (15 day HRT) appeared to be the best on the basis of energy yield, dewaterability, and process stability, but not from land application suitability or sidestream loadings.

2. Waste activated sludge alone - Chemical conditioning costs were the highest of all sludge types (with $FeCl_3$ and CaO doses of comparable magnitude). WAS was shown to be amenable to anaerobic digestion at shorter detention times than expected. The optimum VOLR was about 1.5 kg/m^3day (15 day HRT) on the basis of most digestion parameters except volatile solids removal. Some mixing is critical for digester performance, but the degree of mixing required might be less than primary sludge (see results of full-scale tests).

3. Combination sludges - There was a possible substrate limitation due to low volatile solids fractions. Depressed biokinetic reaction rates could account for the gas production plateau effect observed (see Figure 1.b), the comparable volatile solids removals during the higher solids concentration phase and the apparently antagonistic effect of combining primary and WAS prior to digestion. Again, a 15 day HRT (1.8 and 2.3 kg $VS/m^3 \cdot$day, respectively, for the low and high solids concentrations) provided the overall optimum results.

Anaerobic Contact Process (WAS Alone)

The objective of this phase of research was to evaluate the feasibility and performance of the anaerobic contact process (ACP) for digestion of a municipal sludge. In the past, this process has been employed exclusively for dilute, high-strength, highly biodegradable industrial wastes. The process is analagous to activated sludge in that a portion of the effluent is concentrated and recycled as a source of biomass. Thus, longer SRT's can be achieved at short HRT's.

Sludge wasted from the digesters was allowed to settle overnight in gravity thickeners. The concentrated solids were then mixed with the raw WAS at a volumetric ratio of one and then fed to the digester. It can be seen (Table 1) that considerable digester capital cost savings can be realized (or increased sludge stabilization at equal HRT's) with the ACP. VSRT:HRT ratios for HRT's of 7, 12 and 17 days were, respectively, 2.0, 2.4 and 3.2. This increasing ratio is indicative of improved digested sludge settleability in the thickener with increasing retention time. The optimum VOLR was 1.8 kg $VS/m^3 \cdot$ day (12 day HRT, 28.5 day VSRT) on the basis of most indicators of

digestion performance except volatile solids removal.

One of the most significant results of this phase of the pilot testing was that the ACP proved to be one of the most stable operations on the basis of acid/base equilibrium data (VA/BA ratios and UVA concentrations), the coefficient of variation for daily gas production rates, and quality of gas produced. The low net energy yields for the ACP were probably due to lack of gas production metering in the gravity thickeners overnight.

FULL-SCALE VERIFICATION

The full-scale experimental program is still underway at the Kenosha plant, although testing is scheduled for completion in December 1979. Table 4 summarizes the tests which have been completed to date. The remaining tests will provide an additional set of operating conditions for both primary and WAS digested separately.

Comparison of the data in Table 4 with the pilot data presented previously yields a rather surprising result. For all sludge types and indicators of process performance, (gas production, net energy yields and volatile solids removal) the full-scale digesters were significantly more efficient than the pilot units at comparable VOLR's and HRT's, despite the fact that operating temperatures were slightly lower in the full-scale units. For primary sludge alone, this might be attributable to different means of digester mixing (sludge and gas recirculation mixing in the pilot and full-scale units, respectively). For WAS alone, identical modes of mixing were utilized, although the rate of volume turnover in the pilot units was approximately 400 times greater than the full-scale units. Both modes of mixing were used in the full-scale units for combination sludges. Both full-scale digester performances exceeded their comparable pilot tests, despite the fact that the pilot units were fed sludges with higher volumetric primary to WAS ratios.

One final interesting result of the full-scale tests was that sludge recirculation mixing (through the external heat exchangers at an approximate volume turnover rate of 2 days) was sufficient to provide uniform mixing for the digestion of WAS alone, based on a solids' profile.

CONCLUSION

In terms of net energy yields, and most of the other digestion parameters, the optimum VOLR was from 1.5 to 2.3 kg VS/m^3·day (12-15 day HRT) for all sludge types and processes for the pilot-scale tests. This relative uniformity among sludge types does not negate potential advantages of segregated sludge digestion for several reasons: 1)Different microbial populations might be responsible for the optimum operating conditions determined for each sludge type, 2)Segregated sludge digestion can provide increased operating flexibility in both the liquid and solids processing portions of the plant, and 3) Provide increased safety from digestion system upset due to shock loadings of organic or toxic materials.

Although the full-scale verification program is still in progress, the data obtained thus far indicate a poor correlation between the performance of pilot and full-scale digesters. Surprisingly, the full-scale digesters have proven to be far more efficient than the pilot units at comparable loading rates.

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TABLE 1. PILOT-SCALE TESTING -- SLUDGE CHARACTERISTICS -- STEADY-STATE AVERAGES

Sludge Type	Nominal retention time, days HRT	SRT	Feed sludge % TS	VS[a]	Effluent sludge % TS	VS[a]	% VS removed	VOLR, kg VS/m^3-d	Ratio VA/BA[b] HAC/$CaCO_3$	Estimated maximum unionized volatile acids,[e] mg/l HAC
Primary	5	5	4.4	69.7	1.6	54.0	72	6.1	3.98	159.0
	7.5	7.5	4.4	69.7	1.2	57.3	78	3.1	2.40	33.2
	15	15	4.4	69.7	2.2	44.1	68	2.0	0.32	<1
WAS (high-rate)	7.5	7.5	5.1	55.1	4.1	48.3	31	3.7	0.34	5.6
	15	15	3.6	65.1	2.8	52.8	31	1.5	0.04	<1
	20	20	3.6	65.1	2.1	51.1	49	1.2	0.03	<1
	30	30	5.1	55.1	2.6	46.7	58	0.9	0.10	1.4
WAS (Non-mixed-high rate)	15[c]	15	3.6	65.1	3.3	53.1	17	1.5	0.44	2.5
	20[c]	20	5.1	55.1	2.3	49.9	60	1.4	0.13	-
WAS (ACP)	7	13.7	3.8	56.8	2.7	50.5	37	3.1	0.15	<1
	12	28.5	3.8	56.8	2.4	50.4	44	1.8	0.15	<1
	17	53.6	3.8	56.8	1.7	51.5	59	1.3	0.08	<1
Combination[f] (low solids)	5	5	5.7	45.8	5.4	45.6	7	5.3	0.79	11.6[d]
	10	10	5.7	45.8	4.7	43.9	21	2.6	0.07	<1[d]
	15	15	5.7	45.8	4.2	40.5	35	1.8	0.04	<1[d]
Combination[f] (high solids)	5	5	7.2	47.6	6.2	43.4	22	6.9	1.83	18.4
	10	10	7.2	47.6	5.9	42.3	26	3.4	0.78	14.2
	15	15	7.2	47.6	6.6	41.1	23	2.3	0.77	5.0

(a) VS expressed as % of TS.
(b) Volatile acids as acetic mg/l/bicarbonate alkalinity as $CaCO_3$ mg/l.
(c) Temperature control problems.
(d) The pH values for these calculations estimated from the pertinant composite samples.
(e) $pk_A = 4.76$.
(f) Volumetric ratio of primary to secondary sludge equal to 2:1.

TABLE 2. PILOT TESTING -- GAS PRODUCTION, DEWATERABILITY, AND LAND DISPOSAL CHARACTERISTICS -- STEADY-STATE AVERAGES

Sludge type	Nominal retention time, days		Gas Production				Optimum chemical doses for dewaterability[c], cost(g),			Area required for land application[f] ha		Estimated sidestream BOD_5 loading[h],
	HRT	SRT	m^3 gas/m^3 dig. Vol.-day[a]	m^3 gas/kg VS added[a]	% CO_2	COV[b] %	$/dry ton	g/kg dry solids $FeCl_3$	CaO	Cadmium limited	Avail.Nitrogen ltd.	kg/day
Primary	0	0	-	-	-	-	8.72	33	99	22	105	-
	5	5	1.51	0.25	34	14.5	17.25	84	140	12	135	1000
	7.5	7.5	1.41	0.46	37	19.9	17.68	67	201	9	134	525
	15	15	1.13	0.55	31	15.4	20.06	91	183	22	227	55
WAS (high-rate)	0	0	-	-	-	-	12.80	97	0	62	210	17
	7.5	7.5	0.68	0.18	24	22.5	18.83	107	107	83	479	543
	15	15	0.69	0.49	26	17.0	25.60	137	171	48	418	6
	20	20	0.31	0.29	29	29.8	35.78	198	219	41	425	13
	30	30	0.31	0.32	29	15.2	31.15	172	192	56	507	83
WAS (nonmixed-high-rate)	15[d]	15	0.45	0.30	28	26.9	20.46	105	150	56	346	83
	20[d]	20	0.44	0.31	26	74.2	41.93	230	263	40	421	481
WAS (ACP)	0	0	-	-	-	-	24.82	188	0	65	187	40
	7	13.7	0.63	0.15	22	15.6	20.90	102	169	146	508	41
	12	28.5	0.48	0.22	23	15.9	26.05	141	169	159	523	-
	17	53.6	0.31	0.21	23	10.9	22.00	125	125	156	515	40
Combination[e] low solids	0	0	-	-	-	-	-	-	-	-	158	-
	5	5	0.63	0.12	22	26.6	5.63	32	32	83	213	176
	10	10	0.55	0.21	29	20.1	5.46	33	25	90	269	7
	15	15	0.57	0.33	27	15.8	10.78	70	35	94	292	7
Combination[e] - high solids	0	0	-	-	-	-	-	-	-	118	98	-
	5	5	0.92	0.13	33	23.3	13.47	63	117	111	225	227
	10	10	0.60	0.17	28	25.1	13.06	47	156	132	233	286
	15	15	0.61	0.26	32	25.2	16.54	47	235	132	242	164

(a) Primary digester gas production only, $m^3/kg \times 16.02 = ft^3/lb$, $m^3 \times 35.32 = ft^3$.
(b) Daily gas production coefficient of variation, COV = (Standard Deviation/Mean) x 100%.
(c) Based on CST optimum dose, g/kg x 2 = lb/ton.
(d) Temperature control problems, gas production estimated.
(e) Volumetric ratio of primary sludge: WAS equal to 2:1.
(f) First year of application - cadmium and available nitrogen (20% of org.-N + 90% of NH_3-N) application rates are 2.0 and 140 kg/ha respectively.
(g) $44/dry ton CaO, $132/dry ton $FeCl_3$.

TABLE 3. PILOT-SCALE TESTING -- NET ENERGY YIELDS -- STEADY-STATE AVERAGES

Sludge type	Nominal retention time, days HRT	SRT	Net Energy Yield, kJ x 10^6/day[c] Method I[a] Air/Sludge Temperatures, °F 15/40	55/50	90/70	Method II[b] Air/Sludge Temperatures, °F 15/40	55/50	90/70
Primary	5	5	1.8	8.3	20.2	8.4	27.6	63.4
	7.5	7.5	13.8	20.7	32.8	30.7	44.0	68.4
	15	15	39.1	46.6	59.4	41.1	48.5	61.4
WAS (high-rate)	7.5	7.5	-9.5	-2.6	9.6	-17.3	-4.0	20.4
	15	15	11.1	18.6	31.3	12.2	19.6	32.5
	20	20	-7.5	0.3	13.4	-5.6	0.4	10.4
	30	30	3.7	12.3	26.0	1.6	6.1	13.3
WAS (nonmixed-high-rate)	15	15	-4.2	3.3	16.0	-3.6	3.9	16.8
	20	20	3.5	11.4	24.5	3.0	8.9	19.0
WAS (ACP)[d]	7	13.7	-18.2	-11.6	0.3	-27.4	-13.2	12.8
	12	28.5	-12.7	-5.5	6.9	-10.5	-1.6	14.1
	17	53.6	-14.5	-6.8	5.9	-10.6	-3.7	7.8
Combination (low solids)[e]	5	5	-16.9	-10.4	1.5	-49.4	-30.3	5.6
	10	10	-8.4	-1.3	11.1	-11.4	-1.1	17.6
	15	15	3.4	10.9	23.7	4.3	11.7	24.6
Combination (high solids)[e]	5	5	-10.8	-4.2	7.7	-30.4	-11.2	24.7
	10	10	-6.3	0.8	13.2	-8.2	2.2	20.9
	15	15	6.0	13.5	26.2	6.9	14.4	27.3

a. Method I - fixed flow (190 m^3/day), variable digester volume.
b. Method II - fixed digester volume (2939 m^3), variable flow.
c. Assumptions: Latent heat value in digester gas = 22,360 kJ/m^3; Specific heat of sludge = 4.06 J/g-°C; Digesting sludge temperature = 32 °C; Sludge specific gravity = 1.05; Digester diameter to sidewall depth ratio = 2.7; Heat transfer coefficients: roof = 89.9 kJ/day-m^2-°F, sidewalls = 32.7, and floor = 16.4; Heat exchanger efficiency = 80%
   kJ x 0.95 BTU.
d. Around the primary digester only.
e. Volumetric ratio of primary sludge to WAS equal to 2.0.

TABLE 4. RESULTS OF FULL-SCALE EVALUATION OF SINGLE-STAGE, HIGH-RATE DIGESTION OF VARIOUS SLUDGE TYPES

	Sludge Type			
	Primary[a]	WAS[b]	Combination[c]	Combination[d]
Sludge characteristics:				
Feed total solids, %	6.8	3.3	5.1	5.1
Feed volatile solids, % of TS	56	64	62.4	55.6
Digested total solids, %	4.9	1.9	3.5	4.1
Digester volatile solids, % of TS	48	53	56.3	49.1
HRT, days	7.4	18.7	6.2	7.3
VOLR, kg VS/m^3-day	5.1	1.2	5.2	3.9
% VS removed	38	55	38	30
Gas Production characteristics:				
m^3 gas/m^3 dig. vol.-day	1.91	0.58	1.09	1.06
m^3 gas/kg VS added	0.37	0.51	0.21	0.28
% CO_2	-	22	-	-
Net Energy Yields in kJ x 10^6/day at air/raw sludge temperatures of:				
15/40 °F - Method I	29.1	12.5	-2.0	1.8
55/50 °F - Method I	36.0	20.9	3.8	8.7
90/70 °F - Method I	48.0	33.4	16.8	20.8
15/40 °F - Method II	62.8	10.6	-2.5	6.1
55/50 °F - Method II	76.3	17.0	13.2	19.7
90/70 °F - Method II	100.9	27.5	42.5	44.7
Average actual digesting sludge temperature, °C	29	31	32	32

(a) Digester no. 2, October 1978 - March 1979, gas recirculation mixing, steady-state averages.
(b) Digester no. 1, October 1978 - March 1979, sludge heating recirculation mixing only, steady-state averages.
(c) Digester no. 2, January 1976 - May 1977, volumetric primary sludge/WAS ratio = 0.72, gas recirculation mixing.
(d) Digester no. 1, June 1977 - October 1978, volumetric primary sludge/WAS ratio = 0.95, sludge heating recirculation mixing only.

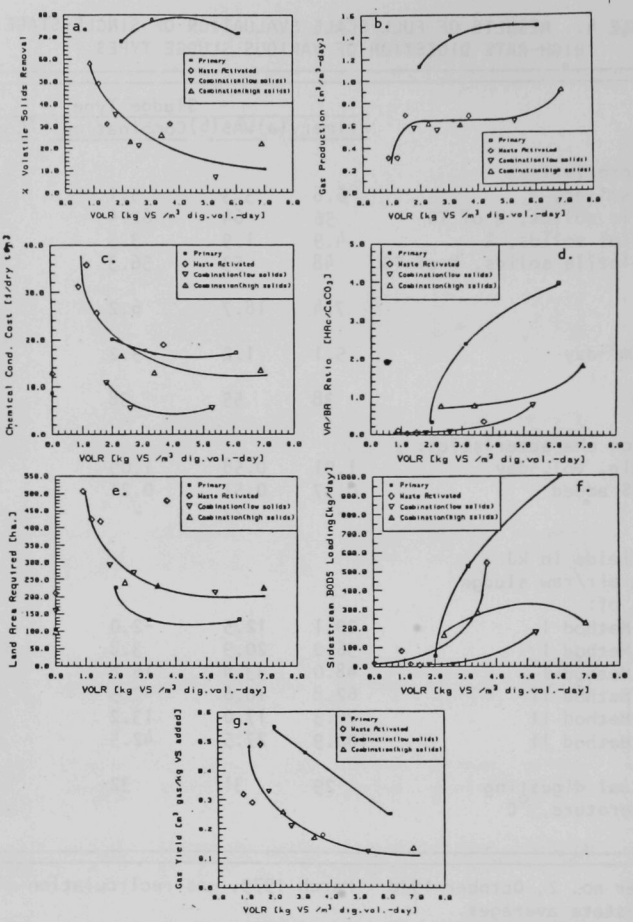

Figure 1. High-rate, single-stage digestion performance - pilot scale:
a. Percent volatile solids removal.
b. Gas production.
c. Chemical conditioning costs (optimum dose).
d. Volatile acid to bicarbonate alkalinity ratio.
e. Land area required for sludge application (nitrogen limited).
f. Dewatering process sidestream BOD_5 loadings.
g. Gas yield.

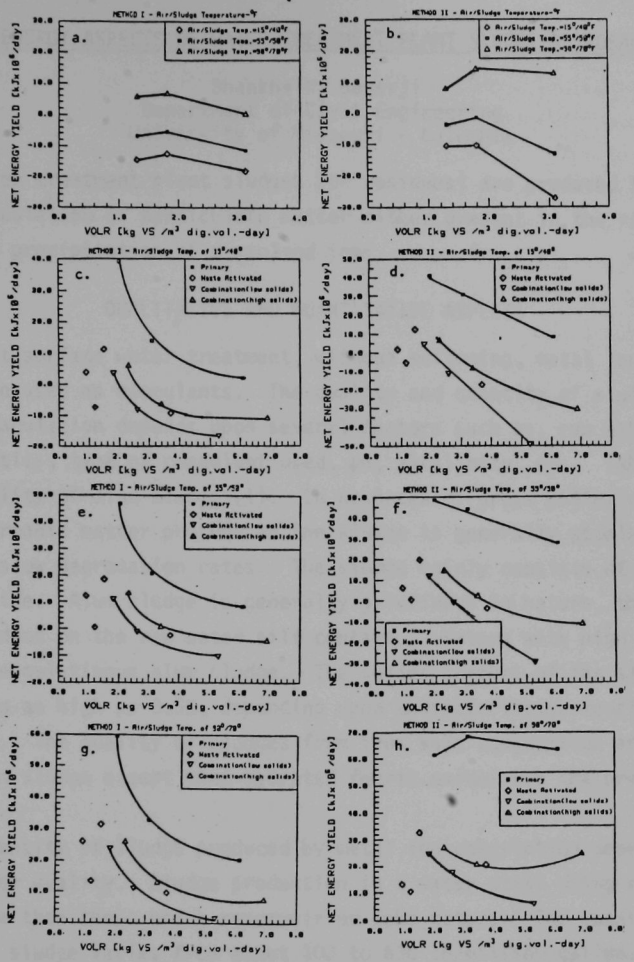

Figure 2. Pilot-scale results - net energy yields.
a. Anaerobic contact process (WAS) - Method 1.
b. Anaerobic contact process (WAS) - Method 11.
c. e. g. High-rate, Method 1 - all sludges.
d. f. h. High-rate, Method II - all sludges.

ENERGY ASPECTS OF WATER TREATMENT PLANT SLUDGE DISPOSAL

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In water treatment plant sludges (or residues) are produced from coagulation-flocculation of particulate matter either present in the raw water or produced by precipitation of dissolved ions.

QUALITATIVE AND QUANTITATIVE ASPECTS

In most surface water treatment, without softening, metal ions and/or polymers are used as coagulants. The quality and quantity of sludges produced by coagulation depends upon several factors such as, raw water characteristics, type of coagulant used, pH, alkalinity, etc. Table 1 shows typical quality of alum sludge (1). In absences of gross pollution of raw water the organic matter present in the sludge is generally stable with relatively slow degradation rates. The sludge mainly consists of hydrated aluminum oxide. Alum sludge is generally gelatinous in nature, the degree being dependent on the raw water silt content. Waters with high silt content may not have gelatinous alum sludge. The solids content of the sludge varies from 0.1% to as high as 3.5%, depending upon characteristics mentioned earlier (2). The quality of sludges from iron salt coagulation are similar to that of alum sludge except that hydrated ferric oxides are the predominant solids.

The quantity of sludge produced by metal ion coagulation depends upon the raw water quality. Sludge production in a water plant using river water is much more than those using reservoir or lake waters. The quantity of coagulation sludge varies from about 100 to 630 lb/million gallons (2).

Water plant sludge produced by lime, or lime and soda ash softening generally consists of relatively pure calcium carbonate, magnesium hydroxide and untreated lime. If surface water supplies are being softened then varying amounts of siliceous and organic particulate matter may also be present in the sludge. Chemical analysis of softening sludge is shown in Table 2. These sludge have a high percentage of $CaCO_3$ and low $Mg(OH)_2$, which would be indicative that mostly calcium hardness was being removed in these systems. However, in situations where larger amounts of magnesium hardness is being removed the proportions of MgO will be higher than those

depicted in Table 2. In some plants to facilitate settlement of precipitated calcium carbonate and magnesium hydroxides, coagulants such as alum and iron salts are added, especially in the secondary basins. This would result in the presence of small quantities of hydrated metal oxides in the settled sludge. Generally, the sludge from softening operations are relatively stable. The solids content of settled softening sludge may vary from 2% to 30% (2). Sludge produced by lime-soda ash softening averaged about 4% in selected Ohio plants (4).

The quantity of sludge produced depends upon the degree of softening attained and the amount of magnesium hydroxide present. The total mass of the sludge can be calculated on the basis of stochiometry of the softening chemical reactions. The volume of the softening sludge produced will depend upon its solids concentration. The volume of sludge produced at 4% solids concentration was reported to be 8540 gal/million gallon water treated/ 100 mg/l $CaCO_3$ hardness removed (4). The corresponding mass of dry solids amounted to 2900 lb/million gallons water treated/100 mg/l $CaCO_3$ hardness removed.

Solids generated from filter backwash operation are of similar characteristics to that produced in pretreatment units. Filter backwash produces a large volume wastewater of low solids concentration. Table 3 shows the quality of filter backwash wastewater. Filter backwash water may contain some organic matter because granular bed filters may support the growth of some microbial forms which would be removed upon backwashing. The settling of the filter backwash water may concentrate the solids and allow the recycling of the supernate.

The amount of solids produced in filter backwash would vary depending upon raw water characteristics, pretreatment efficiency and duration of filter run and backwash cycle. The solids production in filter backwash was reported to vary from 0.5 to 35 lb/million gallons.

WATER TREATMENT PLANT SLUDGE DISPOSAL METHODS

Available water treatment plant sludge disposal methods are:

1. Discharge to Water Course
2. Discharge to Storm or Sanitary Sewer
3. Dewatering on Sand Drying Bed
4. Sludge Lagoons

5. Mechanical Dewatering
   a. Thickening
   b. Centrifugation
   c. Filter Press
   d. Vacuum Filters
   e. Belt Filters
6. Recovery and Reuse of the Sludge

The ultimate disposal of dewatered sludge from the methods 3, 4 and 5 usually consists of transporting to a landfill or, if appropriate, applying on land, or its recovery and reuse. In this paper only mechanical sludge dewatering methods and processes for chemical recovery and reuse will be discussed.

MECHANICAL DEWATERING

Table 4 summarizes the capabilities of various water treatment plant sludge dewatering methods. For reference the performance of sludge lagoon and sand drying bed dewatering are also reported in Table 4.

Thickening: The sludge from the settling tanks may be first thickened in a thickening tank before it is subjected to mechanical dewatering operations.

Alum sludge from settling tank underflow generally is dilute, with solids concentration range of 0.1-3.5% depending upon the raw water characteristics, dosage of alum and other coagulant aids (2). Gravity thickening of alum sludge may achieve a solids concentration in the range of 2-6% depending upon the influent feed sludge concentration and its characteristics.

Softening sludge can be readily thickened in a thickener, when sufficient detention time is provided. With three hours detention time in a thickener, an influent softening sludge of 4% can be thickened to about 10% solids concentration provided supernate is decanted (4).

Centrifugation: Both solid bowl and basket centrifuge have been used for dewatering water treatment plant sludge. However, for alum sludges basket centrifuge may not be a viable option (5). However, Westerhoff and Daly (6) report that conditioned alum sludge may be concentrated from 1.5% to 11% in a basket centrifuge. In another application of basket centrifuge conditioned alum sludge could be concentrated up to 15% (7). Solid bowl (Scroll) centrifuge has been reported to dewater conditioned alum sludge to a

solids concentration of 22% (8). At this concentration the sludge was truckable with a consistency similar to soft wet clay. The type of cake from the centrifuge greatly depends upon the centrifugal force, feed rate, polymer dosage rate, and raw water characteristics (5).

Softening sludge dewaters relatively easily because of the presence of calcium carbonate. Up to 70% solids concentration cake can be achieved using a solid bowl centrifuge with a feed solids concentration of 10-25% (5).

Filter Press: Filter press generally produces higher cake solids compared to other dewatering processes. Conditioned alum sludge with lime or fly ash can be filtered to a solids concentration up to 60% by filter press (9). A fly ash or diatomaceous earth precoat system is often used to extend the interval of filter washing.

Softening sludge can also be concentrated in a filter press to 60% solids concentration with little or no conditioning.

Vacuum Filters: Both traveling medium and precoat medium vacuum filters are used for concentrating water treatment plant sludges. The precoat medium vacuum filter is generally used for alum sludges. With polymer conditioned alum sludge Westerhoff and Daly (6) obtained a cake concentration of 15-17% in a travelling belt vacuum filter, while lime conditioned sludge gave a cake concentration of 30-40%. A precoat vacuum filter with alum sludge can produce a cake concentration of 30-35%.

Lime softened sludge can be dewatered to a cake solid concentration of 40-70% in a travelling belt vacuum filter with multifilament polypropylene medium (5). The performance of this vacuum filter for dewatering softening sludge depends upon feed solids concentration and magnesium content. High magnesium content causes poor cake solids concentration.

Belt Filter: Belt filter press has been evaluated for dewatering conditioned alum sludge. With a raw alum sludge concentration of 4%, it is possible to obtain a cake solids concentration of about 16% with proper polymer conditioning (6).

No reports have come to the attention of the author of softening sludge dewatering using belt filter press.

RECOVERY AND REUSE

Alum Sludge: Recovery of alum from the alum sludge has been shown to be possible in several places in the U.S. (6)(9), especially in view of

escalating market costs of alum. In this process alum sludge from the settling tank and filter backwash operation is equalized in a tank and thickened. The thickened sludge is acidified with sulfuric acid to a pH of 2.0-2.5, which converts the aluminum hydroxide to alum sulfate and hydrolyses some of the organic matter. About 80% recovery of the alum is feasible. The residual solids from the acid treated alum sludge can be removed by filter press to yield about 3.0% alum solution, which can be used in the plant. Care must be taken to see that the recycle of alum does not cause an increase in concentration of iron, manganese and other industrial toxicants in the product water. Periodic wastage of lime neutralized alum sludge cake may relieve the unwanted buildup of these constituents. Figure 1 shows the schematic of the alum recovery process.

A new method of alum recovery is being investigated at Michigan State University which uses organic solvents that selectively extract aluminum from the sludge. The laboratory tests show the method to be quite promising, with 90% alum recovery and much less entrained impurities in the recovered alum (10).

Lime and Magnesium Recovery: Several water treatment plants in the U.S. have implemented lime recovery from water treatment plant softening sludges. These sludges have high proportions of $CaCO_3$ and upon recalcining after dewatering yield CaO, which can be reused in the plant (11). The CO_2 escaping the kiln can be used for recarbonation of the softened water or sludge. In some instances the disposal of softening sludge by recalcining was a profitable operation (12).

High purity magnesium carbonate can be recovered from some softening sludges by recarbonation of sludge and subsequent aeration at a temperature of 35-45°C (13). It has been claimed that recovered magnesium oxide can be economically marketable. Figure 2 shows a schematic of a lime and magnesium recovery system proposed for Johnson County, KS.

ENERGY UTILIZATION BY MECHANICAL DEWATERING UNITS

Table 5 shows the annual energy utilization rates for dewatering sludge in a hypothetical 20 mgd water treatment plant. The amount of alum sludge produced was assumed to be about 632 pounds dry solids per million gallons of water treated, where the raw water turbidity was about 120 turbidity units (5). As mentioned earlier, the quantity of alum sludge depends upon the

turbidity of raw water and alum dosage. For lower raw water turbidities, the sludge produced will be lower. The amount of softening sludge production rate was assumed to be 2900 pounds dry solids per million gallons of water treated for 100 mg/l as $CaCO_3$ hardness removed (4).

The energy utilization values reported are for primary energy only and does not include the energy requirement in the manufacture and transport of chemicals and other materials used during treatment. It should be noted that energy consumption is only one of many factors to be considered before selecting equipment. Capitol and operating costs, operational and maintenance ease and reliability also play a significant role in process selection. However, often an energy efficient choice is also cost effective. Design criteria used for sizing various equipments are also listed in Table 5. Any change in design parameters would change the energy consumption data. Alum sludge concentration by gravity thickening is expected to use 3270 KWH/yr., which is quite insignificant compared to the energy consumption of other devices. The energy costs for sludge pumping are not included in the values listed in the table. The energy usage data are from a soon to be published EPA report by Culp, Wesner and Culp (14).

The energy for Decanter (solid bowl) centrifuges include the power requirements for the centrifuge and the polymer feed system but does not include sludge pumping and dewatering sludge handling. The energy required for heating, ventilation, and lighting of the centrifuge building are also included in the values reported in Table 5. For about 50 gpm sludge flow the building energy requirements were about three times the energy required for sludge processing. The centrifuge sizing was based on sludge gpm flow, which was about 52 gpm for alum sludge and about 47 gpm for softening sludge.

The filter press energy usage data includes energy required for feed pumps, for opening and closing plates, for moving the trays, and for chemical feed system. For sizing the filter press, the solids loading was assumed to be 5 pounds dry solids per cubic feet per hour, with 19 hour operation per day and remaining 5 hours for sludge removal and cleaning. The power required for heating, lighting and ventilating the building were also included in the energy usage figures in the table. The building energy was higher than the process energy for the size of filter press used in the hypothetical example.

The precoat vacuum filter energy usage data for alum sludge dewatering includes energy required for drum drive, discharge roller, vacuum and filterate pumps, precoat pump, tank agitator, and belt conveyor. The alum sludge loading rate for the vacuum filter was assumed to be 1.7 pounds dry solids per square foot per hour (D.S./SF/hr), with 24 hour operation. The energy required for building heat, lighting and ventilation was also included in the total energy requirements reported in the table. For the size of vacuum filter used (310 SF) the building energy requirements were about a third of that required for the process.

The high loading rates (8 pounds D.S./SF/hr) applicable for the softening sludge dewatering permitted the use of the same size vacuum filter as the alum sludge despite its much larger solid content.

Alum sludge is said to be handleable by conventional earthmoving equipment at solids concentration greater than 20% (6). However, for landfill disposal alum sludge between 20-40% solids concentration must be mixed with other materials. At solids concentration greater than 40%, alum sludge can be placed in a landfill by itself. Therefore, from primary energy consumption point of view only it seems for alum sludge solid bowl centrifugation may provide a sludge that is handleable and having a lowest energy requirement.

Softening sludge is generally fluid at 15% solids concentration. Softening sludge containing 15% to 25% solids concentration is either liquid or thixotropic. Thixotropic sludge has a characteristic of a solid as long as it is undisturbed but under pressure it will flow. Softening sludge between 25% to 70% solids concentration is thixotropic and sludge beyond 70% solids is compactable. The centrifugation of softening sludge, which can produce cake solids up to 70%, seems to be the most energy efficient process for dewatering.

In this connection, one has to consider the ultimate disposal and the energy used for transportation of the dewatered cake. The transportation costs will depend upon the haul distance, volume (weight) of sludge hauled, type of vehicle used, topography, etc.

In many areas of this country land disposal of lime sludge is practiced especially where the soil is acidic in nature. In Minneapolis, Minnesota 85% of the lime waste is being applied on land for agricultural purposes. The application rate is about 3.3 tons dry solids per acre for the sandy and

peaty soil predominant in that area (15). The exact application rate will depend upon the type of soil and characteristics of the sludge. The lime sludge has been found to be as effective as commercially available agricultural limestone for improving soil pH. The fineness of the dewatered lime sludge has a distinct advantage over commercial limestone in that the neutralizing reactions are faster and more complete.

Table 6 shows the energy consumption for land disposal of softening sludge from a 20 mgd water treatment plant (having 2900 pounds D.S./mg/100 mg/l $CaCO_3$ hardness removed). The first option is to haul liquid sludge directly from settling tanks at 4-6% solids concentration. The one way haul distance was assumed to be 5 miles. The diesel fuel consumptions by trucks were converted to KWH by appropriate energy equivalent calculations. The second option was to gravity thicken the sludge to a dry solids concentration of 8-15% and then haul it in a semi-liquid form. The last option was to dewater the thickened sludge by centrifuge to about 35% solids concentration before hauling and spreading on the land. It was assumed that most of the energy consumed was during hauling operation and only about 5% was consumed during spreading on land. It can be seen that for this situation it is more energy efficient to dewater sludge to 8-15% solids concentration for land application. The increase in haul distance will proportionately increase the transportation energy usage, and may give results favoring dewatering of sludge to 30-60% solids concentration.

The energy usage for alum recovery shown Figure 1, were obtained from a study reported by Westerhoff and Daly (16) using horizontal vacuum filter for separating residual solids after acidification. For 20 mgd water treatment plant with alum sludge of 630 pounds D.S./mg, the total annual energy required will be about 1,380,000 KWH. The savings in secondary energy for production of alum and its transportation to the water plant must be considered to fully realize the net energy usage.

The energy usage for 50 tons/day CaO and 3 tons/day of MgO recovery was evaluated by Thompson and Mooney (13) to be 4.38×10^6 KWH/yr plus 1.66×10^{11} Btu/yr fuel consumption. At 60 mgd water treatment plant capacity, it was expected that the lime and MgO recovery and sale would give a net cost savings despite the high energy use.

SUMMARY

The paper provides information on energy consumptions for various water treatment plant dewatering methods. For alum sludge dewatering, it appears solid bowl centrifugation may be a viable option from energy consumption viewpoint to produce a handleable sludge (≃20% D.S. concentration). For softening sludge also solid bowl centrifuge is most energy efficient producing a cake solids up to 70%. Energy considerations for land disposal of lime sludge should include energy usage for mechanical dewatering and transportation energy. Design engineers should pay particular attention to energy consumptions while choosing mechanical equipment for water plant sludge treatment. Often an energy efficient process is also a cost effective one.

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Table 1. Characteristics of Aluminum Coagulation Sludge (1)

Plant	BOD, mg/l	COD, mg/l	pH	Total solids, mg/l	Volatile solids, mg/l	Total suspended solids, mg/l	Volatile suspended solids, mg/l
A	41(5-day) 72(7-day) 144(27-day)	540	7.1	1,159	571	1,110(0.1%)	620
B	90(5-day)	2,100	7.1	10,016	3,656	5,105(0.5%)	2,285
C*	108(5-day)	15,500	6.0	16,830	10,166	19,044(1.9%)	10,725
D	44(5-day)		6.0			15,790(1.6%)	4,130

* Activated Carbon in sample

Table 2. Chemical Composition of Dry Solids from Water Softening (3)

Constituent	Percent by Weight		
	Boulder City, Nev.	Miami, Fla.	Cincinnati, OH*
Silica, Iron and aluminum oxides	2.6	1.5	4.4
Calcium oxide, CaO	48.8	52.1	49.5
Magnesium oxide, MgO	7.0	2.8	2.3
Loss on ignition or CO_2	38.4	43.8	40.2
Equivalent $CaCO_3$	87.2	93.0	88.1

*Wright Aeronautical Corporation

Table 3. Characteristics of Filter Backwash Water

Plant Treatment	BOD_5 mg/l	COD mg/l	pH	Solids, mg/l			
				Total	Volatile	Suspended Total	Suspended Volatile
A. Aluminum coagulation	4.2	28	7.8	121	44	47	31
B. Aluminum coagulation	3.7	75	7.2	378	115	104	53
C. Aluminum coagulation	2.8	160	7.8	166	45	75	40
D. Aluminum coagulation	1.8	--	--	--	--	100	60
E. Iron and Manganese Removal	--	--	6.9	1487	343	1370	400

Table 4. Performance of Water Treatment Plant Sludge Dewatering Methods

Method	% Solid Out	Remarks/Reference
1. Sand Bed Drying		
Alum Sludge	20%	After 70-100 hrs.
Softening Sludge	--	
2. Sludge Lagoon		
Alum Sludge	Up to 10%	Depending upon design and local conditions
Softening Sludge	25-40%	
3. Gravity Thickening		
Alum Sludge	2-6%	
Softening Sludge	10%	3-Hr. detention
4. Centrifugation (with conditioning)		
Alum Sludge	12-22%	
Softening Sludge	Up to 70%	
5. Filter Press (with conditioning)		
Alum Sludge	30-60%	
Softening Sludge	Up to 60%	
6. Vacuum Filter (with conditioning)		
Alum Sludge	15-17% (30-35%)	With Precoat
Softening Sludge	Up to 50%	
7. Belt Filter (with conditioning)		
Alum Sludge	15%	
Softening Sludge	--	

Table 5. Energy Usage by Various Sludge Dewatering Methods

20 mgd Capacity Plant

Method	Energy Usage KWH/Yr	Remarks
1. Gravity Thickening		
Alum Sludge	3,270	Loading Rate 40# D.S./SF/Day
2. Decanter Centrifugation		
Alum Sludge	206,000	Feed in @2% Solids 24-hr. operation
Softening Sludge	197,000	Feed in @10% Solids 24-hr. operation
3. Filter Press		
Alum Sludge	520,000	Loading Rate
Softening Sludge	1,400,000	5# D.S./CF/hr 19-hr. operation
4. Vacuum Filter		
Alum Sludge	810,000	Feed in @1% Solids Feed out @20%; Loading rate 1.7# D.S./Hr/SF; 24-hr. operation
Softening Sludge	810,000	Loading Rate 8# D.S./Hr/SF 24-hr. operation

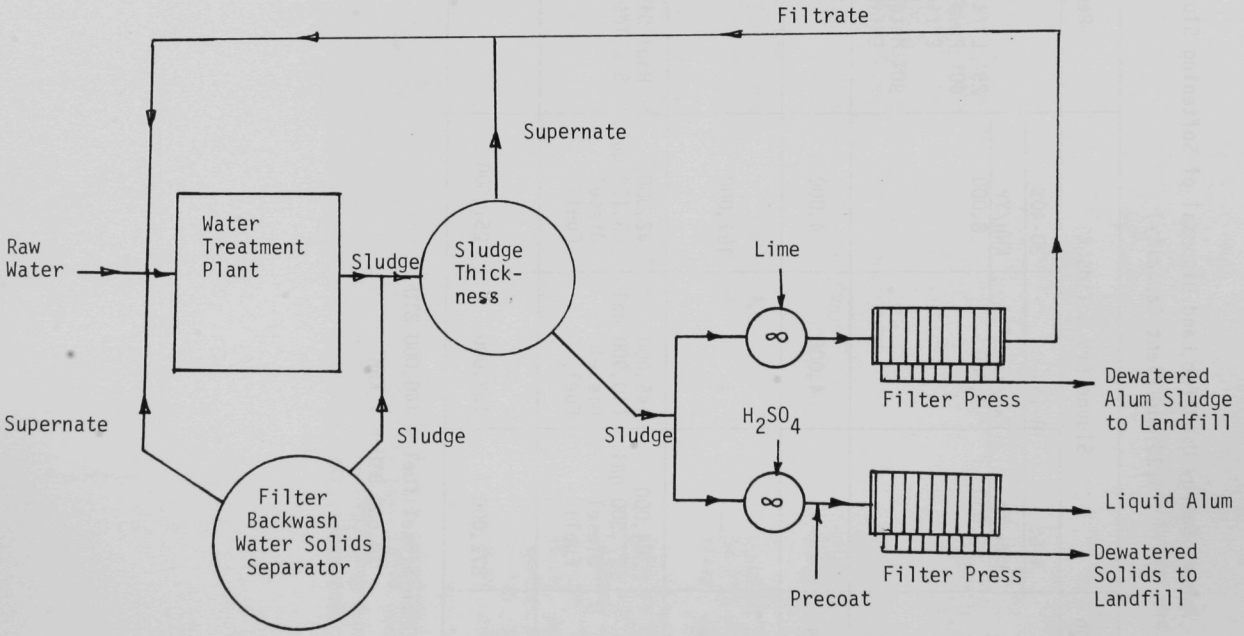
Note: 1. Assumed Alum Sludge production rate 632 #/mg based on average raw water turbidity = 120 JTU; settling tank solid conc. = 0.5%

2. Assumed Softening Sludge production rate 2900 #/mg for 100 mg/l as $CaCO_3$ hardness removal. Settling tank solids conc. = 4%

Table 6. Annual Energy Used for Land Disposal of Softening Sludge
(20 mgd Plant Capacity)

Unit Operation	Sludge Dry Solids %			Remarks
	4-6%	8-15%	30-40%	
1. Pumping (inplant)	KWH/Yr 8,000	KWH/Yr 8,000	KWH/Yr 8,000	25' Lift 60% Pumping Efficiency 90% Motor Efficiency
2. Thickening	--	4,000	4,000	
3. Dewatering Decanter Centrifuge	--	--	197,000	
4. Hauling & Spreading	364,000 (27,300 gal. Diesel Fuel)	145,000 (10,900 gal. Diesel Fuel)	42,000 (3,150 gal. Diesel Fuel)	Haul Distance 5 miles one way
Total Energy Used KWH/yr	372,000	157,000	251,000	

Note: 1 gallon diesel fuel = 140,000 BTU
10,500 BTU = 1 KWH



Energy Utilization for Alum Recovery: 0.3 KWH/# D.S. Input
(For 10,000 # D.S./Day;
Annual KWH = 1,905,000)

Figure 1. Alum Recovery Process - (Fulton Process) (8).

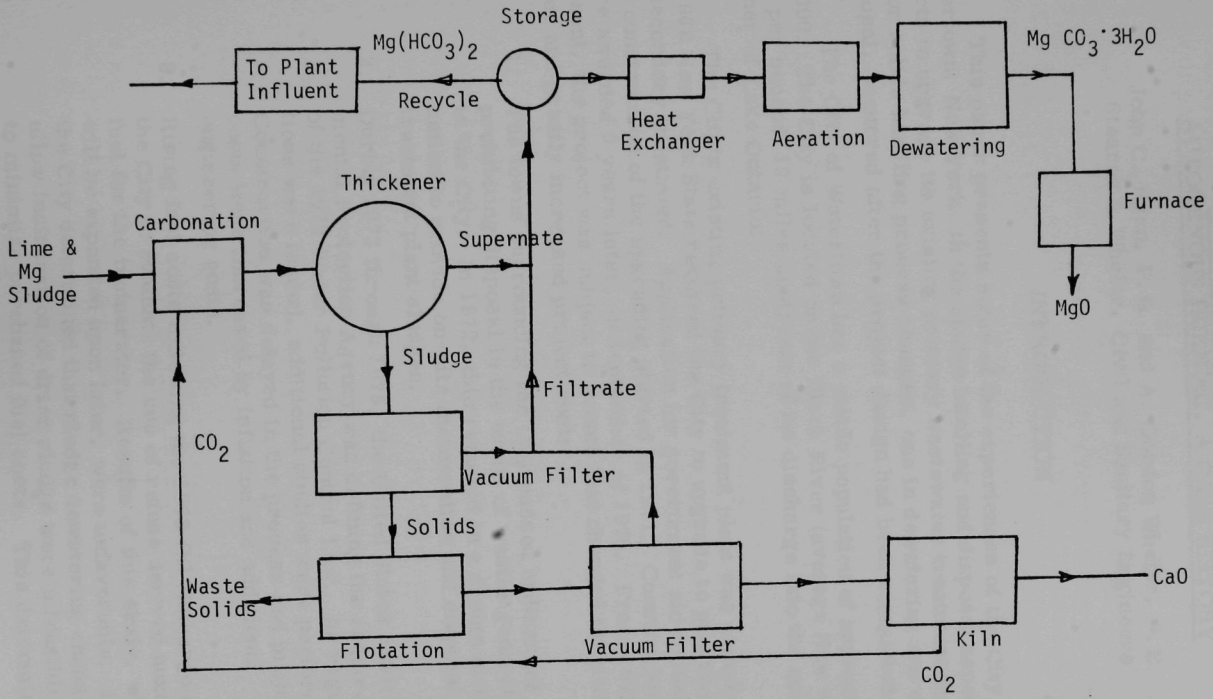


Figure 2. Lime and Magnesium Compounds Recovery from Water Treatment Plant Sludge (13)
(50 Tons/Day CaO, 3 Tons/Day MgO) Johnson County, KS. Plant
Energy Consumption:
Electricity: 3,610,000 + 775,000 = 4.385.000 KWH/yr
Fuel (Natural Gas): $1.6 \times 10^{11} + 6.5 \times 10^{9} = 1.66 \times 10^{11}$ BTU/yr

SLUDGE DEWATERING FOR AUTOGENOUS BURNING, A CASE HISTORY

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INTRODUCTION

This paper presents some of the experiences of the City of Watertown, New York, in the sludge handling and disposal aspects of its efforts to upgrade its existing primary wastewater treatment plant. Two major solids handling process changes, one in dewatering and one in disposal, occurred after the project design had been completed.

The City of Watertown has a stable population of approximately 30,000. The City is located on the Black River (average flow 4,000 cubic feet per second) 12 miles upstream of its discharge into the northeast corner of Lake Ontario.

The City's existing primary treatment plant was completed in 1966. In 1968 New York State required the City to upgrade to provide a minimum of secondary treatment. Application for government aid to fund the design and construction of the upgrading started in 1971. Construction contracts were awarded 5 years later, in September of 1976. From 1971 to the present, the project was subject to events that changed the process design twice and greatly increased project cost:

1. The towns surrounding the City passed ordinances prohibiting disposal in the towns of wastes generated in the City. In 1972, this resulted in a design modification to provide on-site incineration and disposal of treatment plant sludges.

2. During 1973 through 1975, the United States Environmental Protection Agency was defining the requirements of the 1972 Water Pollution Control Law. As regulations were issued, additional studies were performed. Construction was delayed in the process and project costs were increased by inflation and additional engineering costs.

3. Rising fuel costs following the 1973 Oil Embargo caused the City to evaluate the use of refuse derived auxiliary fuel for the incinerator. Results of this study, which will be expanded upon later, were unfavorable. However, the City determined that sludge dewatering changes to allow incineration of drier sludge were a feasible means to minimize purchased fuel costs. This determination

required replacement of vacuum filters with pressure filters. In June 1977, a stop work order was issued on an existing sludge disposal system contract to allow design modifications to be made. Also in June 1977, in the face of rising labor cost, the City required that the design modification for pressure filtration be expanded to provide a 40-hour per week solids disposal schedule, rather than the 160-hour per week schedule contemplated in the 1971 design.

5. In March 1978, the contractor on the sludge disposal system was declared bankrupt.

BACKGROUND

The 1966 treatment plant process includes sludge stabilization by anaerobic digestion and ultimate disposal of grit and sludges by on-site burial and lagooning. Methane gas generated by digestion was used for sludge digester and space heating and for raw wastewater pumping. The plant is connected to 76.5 miles of combined sewers, serves a 1970 population of 30,525, and is designed to receive 8 million gallons per day on the average with capacity to handle a 27 million gallon per day peak flow.

Existing and design year wastewater characteristics are shown on Table 1. The population growth noted is entirely residential resulting from service being extended to the surrounding towns. Industry contributes 20 percent of the average flow and organic loadings. The major industrial contributor is a milk and cheese processing plant.

Figure 1 shows the flow diagram for the upgrading project on which construction is now 90 percent complete. Solids to be generated will include primary, trickling filter and chemicals for phosphorus removal. Figure 2 shows solids handling for the upgraded treatment plant. This includes the final processes selected to minimize fuel oil requirement, allow 40-hour per week operation, and provide for on-site disposal. As designed, sludge will be pumped from the clarification units essentially on a continuous schedule with sludge conditioning also operating approximately 20 hours per day. Conditioned sludge will be stored in one of the new 50-foot sludge thickeners which will serve as a holding tank. Withdrawal will be on demand signal from the filter press batch feed tanks. If sufficient heat value exists in the sludges, some of the unconditioned thickened sludge may be fed to the digesters daily to maintain their operation. The digesters are to serve as backup when the reactor is shut down for repair or maintenance. After filter pressing, the cake will be conveyed to a surge bin which feeds the incinerator. A bypass conveyor provides flexibility to transfer sludge cake to a truck for hauling to landfill when the

incinerator is down. Construction has not started on this portion of the project. Also noted on Table 2 are the expected characteristics of the solids to be handled.

Next the selection steps will be reviewed that led from the initial design of filter press with land disposal through vacuum filtration with incineration, to incineration with refuse derived fuel for auxiliary fuel, and finally to the present design of pressure filtration with fluid bed incineration.

INCINERATION SELECTION

In 1972, when neighboring town ordinances precluded City waste disposal outside the City limits, the City's solid disposal problem became critical. With sludge cake production projected at 9,500 cubic yards annually, it was estimated that only one year's storage of cake was available on site. To complicate the problem, the City's landfill was closed to sewage sludge disposal by a City health ordinance. Even with changes in the ordinance, space remaining at the landfill was estimated to be 6 to 8 years, without the treatment plant sludges. The City, therefore, required that the design be changed to provide an on-site sludge incineration system.

There are two basic types of sludge incinerators: multiple hearth incinerators (MHI) and fluidized bed incinerators (FBI). The FBI was selected due to four inherent advantages:

1. Excess Air: FBI requires only 40 percent excess air compared with 80 percent to 100 percent for standard MHI.

2. Heat Transfer: The violent internal mixing of solids and gases in the fluidized bed results in uniform conditions and rapid heat transfer.

3. Shutdown: Due to the large heat sink provided by the bed, a fluidized bed reactor cools at a rate of approximately 15° F per hour so that overnight shutdowns normally will not require preheating. Incinerator capacity may then be adjusted by varying the operating hours. This characteristic was particularly well suited to the City's requirement to minimize operating hours per week.

4. Odor Control: Off gases from the fluidized bed exit the incinerator at 1,500° F and are, therefore, odor free. MHI's, on the other hand, sometimes require after-combustion reheating.

At the same time the City decided on incineration, vacuum filters were substituted for the filter presses recommended in the initial Facilities Report. This was done to partially offset the cost increase due to adding the incinerator.

CO-DISPOSAL

In 1977, because of rising fuel costs and shrinking landfill capacity, the City authorized an evaluation of preparing and utilizing a refuse derived fuel (RDF) for auxiliary fuel instead of oil. Figure 3 records the trend in oil prices that were being experienced by the City. The present worth of auxiliary oil for the design life using vacuum filters was calculated in 1977 to be $1,800,000 at a constant unit price of $0.3976 per gallon and $5,300,000 allowing 10 percent annual inflation from the 1977 price. First-year auxiliary fuel costs were then estimated to be $120,000 ($4 per capita).

The present average solid waste generation figure for the City is 76 tons per day. A preliminary layout and cost estimate indicated that an additional 12,000 square foot building would be required to house the needed solid waste conditioning equipment; screens, pulverizers, classifier, and metal separators. A new standby incinerator and associated conveyors would also be required. The cost to accomplish this pretreatment and modify the reactor and the building, on which a contract already had been awarded, was estimated at over $4,500,000.

Due to questionable Federal and State participation in funding, to the probable negative impacts on plant construction already in progress, and to high cost compared to auxiliary oil; the consideration of co-disposal was replaced with an evaluation of how to reduce the auxiliary fuel requirement.

DEWATERING OPTION

The amount of auxiliary fuel required in the combustion of a particular sludge cake depends on the characteristics of the cake, the percent moisture, and the amount and type of volatile solids. Table 3 lists volatile content and heating values for general sludge categories. The primary method of reducing auxiliary fuel requirements is to decrease the percent moisture in the sludge cake. To achieve independence from auxiliary oil, or autogenous combustion, furnace suppliers were contacted to determine at what percent sludge cake solids concentration this could be achieved. Since the sludge to be incinerated did not yet exist and could not be tested for heat content, trying to get a specific answer on this point required a fair amount of crystal ball gazing. Complicating the evaluation for Watertown was the limited amount of information on heating values of

trickling filter sludges, which on this project calculated to be 60 percent by weight of the total non-chemical solids generated. A conservative heat value of 6,000 BTU per pound of dry sludge solids (DSS) was used.

Figure 4 shows the relationship between heat (volatile) content of a sludge and the point at which autogenous combustion may be expected. For the previously selected average heat value of 6,000 BTU per pound of DSS this data would indicate that autogenous combustion should occur at 25 percent solids. However, to achieve such a solids concentration, the sludge requires chemical conditioning prior to dewatering. Since the City required a design based on a 40-hour week, overnight and weekend sludge storage is necessary, and one of the two 50-foot diameter new gravity sludge thickeners is to be used as a weekend holding tank. To minimize septic conditions, the sludge will be conditioned prior to storage by a lime and ferric chloride addition step. This step will increase sludge pH to approximately 10.5. This conditioning step has a negative impact on the heat value of the final sludge cake since the chemicals added have no heat value. In fact, the lime added will have a heat of calcination demand of 765 BTU per dry pound. The net effect of chemical addition is a reduction of heat value to 4,400 BTU per pound of DSS when using a conditioning load of 20 percent lime and 5 percent ferric chloride by weight. This means that the Watertown dewatering process has to be cabable of achieving 35 to 40 percent dry solids concentration. A requirement of the New York State Department of Environmental Conservation for grant eligibility was that costs associated with modifications would have to total less than the present worth value of the auxiliary fuel oil cost under the previous vacuum filtration proposal.

There are many options available on dewatering equipment and more become available each year. Table 4 summarizes the better known mechanical means. Only pressure filtration was thought to be dependably capable of achieving the percent solids required for autogenous combustion of the expected Watertown sludges. Within the technology of pressure filtration, there are options that provide capabilities for greater cake solids concentration and for processing more tons per day per unit of machinery. These options enhanced the attractiveness of this process for Watertown, since actual sludge characteristics are unknown.

The production rate of filter presses can be varied by:

1. Adding or deleting plates depending on press frame size. The City of Watertown will be starting its presses with spool pieces on the hydraulic closing rams that allow for 30 percent future expansion.

2. Increasing terminal pressure. At Watertown a 225 psi terminal pressure system is being provided. This is the maximum pressure presently marketed.

3. Changing the type of plate. Two plate designs are now available: fixed volume (recessed plate) and the diaphragm (membrane) plate. Diaphragm plate manufacturers present full scale test reports indicating that by changing to diaphragm plates the output per press may be increased by 50 to 100 percent. This is accomplished by decreasing the length of the press cycle.

The percent cake solids may be varied by:

1. Increasing terminal pressure.
2. Increasing the time that maximum pressure is maintained. (In Watertown it is planned that at the end of the operating day both presses would be charged, brought up to pressure and shut off to hold that pressure overnight.)
3. Changing the type of plate to diaphragm plates and varying what is referred to as "squeeze" time.

Pressure filtration certainly is not without its limitations. The process is a high energy consumer, requires large capital expense, and has the main disadvantage of being a batch process, which in the municipal sludge disposal business is like fitting a square peg into a round hole.

In preparing bidding documents for autogenous combustion by pressure filtration, the final design of the incineration system was included as part of the bidder's work. Only incinerator manufacturers were allowed to bid. The primary obligation of the successful bidder was to provide a dewatering/incineration system that would operate autogenously on the sludge described. Prior to the bid, conferences were held with prospective bidders to decide on specifications to be incorporated into the documents that would define the system sufficiently for bidding without eliminating design flexibility needed by the bidders to meet the autogenous combustion requirement. Items such as press volume, pressure and cycle time, incinerator solids capability, excess air requirements, feed points and temperatures were defined.

OTHER FEATURES

In addition to the modifications in dewatering to achieve the City's objectives of reduced fuel and labor operating costs, two other energy related features have been incorporated.

The first feature is the continued use of the existing high rate anaerobic sludge digesters. Sludge digestion is an incongruous process to sludge incineration in that both compete for the heat value of the sludge

input. However, the pressure filtration equipment being provided on this project may obtain cake solids high enough that the existing digesters could be maintained in operation as standard rate digesters. Facilities that were provided in the 1966 plant to use sludge gases for digester heating and space heating will continue in operation if digestion can be maintained. The digesters will serve as backup when the incinerator is down for maintenance or repair.

The second feature is that waste heat from the incinerator is returned to the process line. Scrubber cooling water is returned to the head of the plant, which should improve both clarifier and trickling filter efficiency. Incinerator off gases are piped to the trickling filters to maintain higher than ambient temperatures during the winter months. These 122-foot diameter filters are covered by insulated fiberglass domes. Trickling filter removal efficiencies are very temperature dependent.

CONCLUSION

Two conclusions may be drawn from the Watertown experience

1. Due to political limits on land disposal, which required incineration on a small scale, and the additional equipment required for the 40-hour per week operating schedule, the Watertown unit cost for sludge disposal is $155 per dry ton including capital, operation and maintenance costs. This figure does not compare well with other published values for these reasons. (No consideration was made in this figure for State and Federal grant monies.)

2. By changing the method of dewatering to achieve autogenous combustion, the City of Watertown has increased control over their operations budget by minimizing their dependence on fuel oil. At full system capacity and 1980 oil prices, the sludge disposal unit cost for the former vacuum filter/incineration design would be $195 per dry ton this year and would escalate annually at the fuel cost inflation rate.

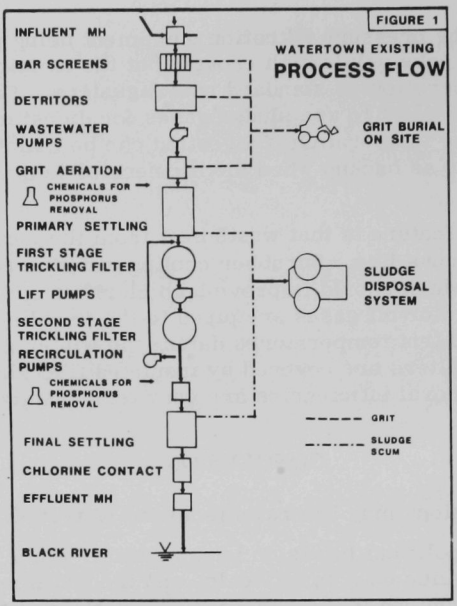


FIGURE 1
WATERTOWN EXISTING
PROCESS FLOW
INFLUENT MH
BAR SCREENS
DETRITORS
WASTEWATER PUMPS
GRIT AERATION
CHEMICALS FOR PHOSPHORUS REMOVAL
PRIMARY SETTLING
FIRST STAGE TRICKLING FILTER
LIFT PUMPS
SECOND STAGE TRICKLING FILTER
RECIRCULATION PUMPS
CHEMICALS FOR PHOSPHORUS REMOVAL
FINAL SETTLING
CHLORINE CONTACT
EFFLUENT MH
BLACK RIVER
GRIT BURIAL ON SITE
SLUDGE DISPOSAL SYSTEM
GRIT
SLUDGE SCUM

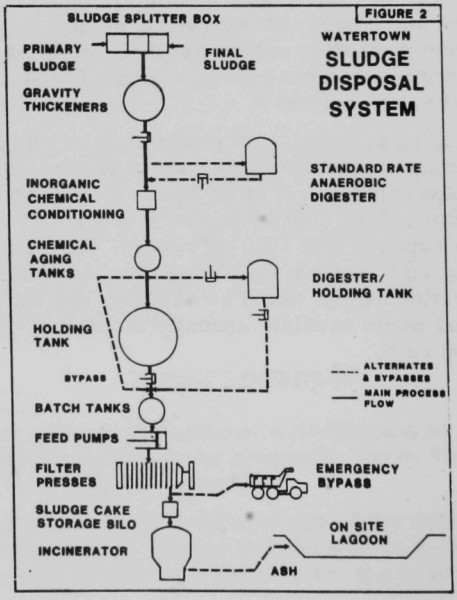


FIGURE 2
SLUDGE SPLITTER BOX
WATERTOWN
SLUDGE DISPOSAL SYSTEM
PRIMARY SLUDGE
FINAL SLUDGE
GRAVITY THICKENERS
STANDARD RATE ANAEROBIC DIGESTER
INORGANIC CHEMICAL CONDITIONING
CHEMICAL AGING TANKS
DIGESTER/ HOLDING TANK
HOLDING TANK
BYPASS
ALTERNATES & BYPASSES
MAIN PROCESS FLOW
BATCH TANKS
FEED PUMPS
FILTER PRESSES
EMERGENCY BYPASS
SLUDGE CAKE STORAGE SILO
INCINERATOR
ON SITE LAGOON
ASH

TABLE 1

INFLUENT WASTEWATER CHARACTERISTICS

Item	Existing	Present	Design
Year	1970	1979	1995
Average Flow (mgd)	5	9.5	8.0
Peak Flow (mgd)	20	20.0	27.0
Suspended Solids (ppm)	240	100	250
BOD_5 (ppm)	290	326	300
mg $(PO_4)/l$	20	2.2*	25

* Since 1970 New York State has passed a ban against phosphates in detergents.

TABLE 2

AVERAGE CHARACTERISTICS OF SLUDGES GENERATED (POUNDS DRY SOLIDS PER DAY)

Item	Existing	First Year	Design
Year	1970	1980	1995
1. With Digestion:			
Primary	6,000	6,650	8,900
Secondary	---	9,500	12,800
Digested	4,500	2,500	3,400
Chemical*	---	1,000	1,400
Conditioners**	---	4,800	6,500
Net	4,500	20,600	27,800
2. Without Digestion:			
Primary	6,000	6,600	8,900
Secondary	---	9,500	12,800
Chemical*	---	1,000	1,400
Conditioners**	---	5,100	6,900
Total	6,000	22,200	30,000

* For phosphorus removal.
** Ferric chloride and lime.

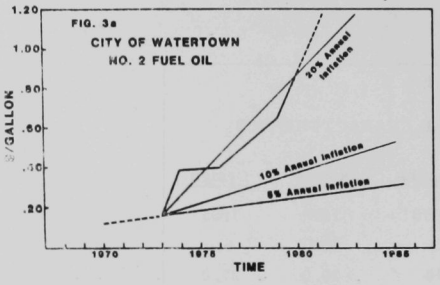


TABLE 3

TYPICAL HEAT VALUES[2]

Material	Combustibles	BTU Per Pound of Combustibles	BTU Per Pound
Grit	33.2	12,000	4,000
Digested	59.6	8,900	5,290
Primary	74.0	13,900	10,285
Activated	---	---	8,500 - 10,000
Biological Filter	---	---	8,500 - 10,000
Grease/Scum	88.5	18,900	16,750
Fuel Oil[1]	---	---	18,500
Natural Gas[1]	99	22,850	22,800

1 "North American Combustion Handbook", First Edition (1965).

2 Balakrishnan, S.; Williamson, D. E., and Okey, R. W.; "State of the Art, Review of Sludge Incineration Practice", Federal Water Quality Administration, Department of the Interior.

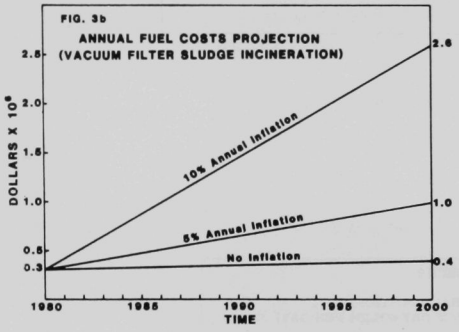


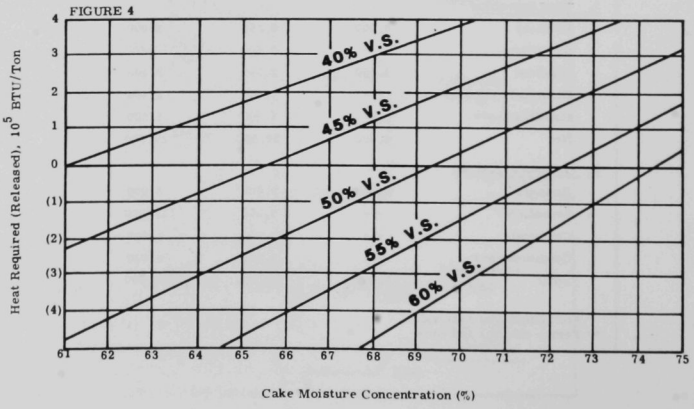
TABLE 4

SUMMARY OF MECHANICAL DEWATERING OPTIONS

Dewatering Method	Cake Solids Concentration (%)
Vacuum Filter	20 - 26
Centrifuge	20 - 26
Filter Press (fixed volume)	30 - 45
Filter Press (diaphragm)	30 - 60
Belt Press	15 - 30
Belt Press* (high pressure)	25 - 40

* Generally not available before 1978.

FIGURE 4



ENERGY CONSERVATION BY REDUCTION OF CHEMICAL USAGE WITH A DIAPHRAGM FILTER PRESS VERSUS OTHER SLUDGE DEWATERING DEVICES

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INTRODUCTION

As more stringent regulations are applied to sludge solids disposal more efficient processes and equipment will be necessary to reduce their volume so that they may be ultimately disposed of at the least cost and in compliance with the regulatory agencies. Energy considerations to the reduction of chemical consumption as well as power requirements should be closely evaluated.

The objective of this article is to investigate the reduction of chemical dosage using a diaphragm filter press comparing to other sludge dewatering or thickening devices such as:

1. Conventional filter press
2. Centrifugation (decanter and basket)
3. Belt press
4. Vacuum filtration

Of all the dewatering techniques described above, the diaphragm filter press produces the driest cake solids which reduces hauling costs. Also the diaphragm filter press produces sludge cake dry enough for autogenous combustion and compositing.

Dewatering of sludges from water and wastewater treatment plants using filter presses is still a relatively new technique in the United States although they have been used extensively in Europe and Japan. The advanced filter press technology stems from the fact that Europe and Japan generally have limited land and ocean areas for sludge disposal. This explains the fact that their dewatering technology is more advanced than in the United States.

Until recently there were two basic types of filter presses in the United States: 100 and 225 psi fixed volume designs. The main components of a filter press are: two stationary end supports, a movable head, sidearm supports, and a multiplicity of filter plates placed vertically adjacent to one another. These presses dewater sludge, chemically conditioned with lime and ferric chloride, by pumping it into filtering chambers lined with a filter cloth. Filtrate passes through the cloth and is carried away from the filter press and returned to the treatment plant. Sludge is usually pumped into the chamber until the terminal pressure is reached or the filtrate flow rate is sufficiently low. Depending upon the type of sludge and its chemical conditioning, the pumping time can last from one to four hours or more.

Recently, a new design of filter press called a "diaphragm" or variable volume type press was introduced into the United States. This type of press is different from conventional presses in the method of dewatering sludge. With the diaphragm press, sludge is dewatered in two distinct steps:

filtration and squeezing. The filtration phase is essentially the same as conventional presses except that terminal pressure ranges from 56 to 98 psi. Upon reaching terminal pressure or minimum filtrate flow rate, the squeezing phase is initiated. Water is pumped into cavities or voids (in the plates) adjacent to the dewatering chambers. This causes the flexible diaphragms to expand, squeeze the cake, and drive out additional filtrate; terminal squeezing pressure is 215 psi.

Centrifugation is essentially a sedimentation device where solids-liquid separation is achieved by gravitational forces applied to the wall of the rotating bowl. Centrifuges have been used for both sludge thickening and dewatering. The two types of centrifuges investigated were the solid bowl decanter type and the basket or imperforate bowl which is a batch dewatering unit. The decanter is a continuous sludge dewatering or thickening device.

Dewatering sludge using the belt press is usually utilized in smaller wastewater treatment plants. The feed sludge is placed on a porous moving belt. Dewatering occurs as the sludge moves through a series of rollers which squeeze the sludge between two belts. The cake is discharged from the belt by a mechanical scraper.

Sludge dewatering by vacuum filtration has been used extensively for many years but other new and innovative dewatering methods are being considered in place of vacuum filtration.

DEWATERING VARIABLES

Diaphragm filter press tests were conducted on various types of sludges throughout the United States to determine the dewaterability of each particular sludge. Tests were conducted to produce optimum cake solids, filtrate quality, minimum chemical dosage, and display good cake discharge characteristics. The dependent and independent variables to be optimized were:

Dependent Variables	Independent Variables
Cake solids	Feed solids
Solids recovery	Chemical dosage
Cake discharge characteristics	Filtration time
Yield	Squeeze time

At each site, all these parameters were varied to obtain the best results. Prior to any testing a capillary suction test (CST) was performed on the unconditioned sludge. The CST test is a quick way of determining the most effective amounts of chemicals required for sludge conditioning prior to dewatering. Chemical preconditioning alters particulate surface charges and permits agglomeration, or development of particle size. Bound water is freed up and permits compaction of sludge, at higher solids recovery rates. The CST unit measures the rate of release of bound water at a specific chemical(s) dosage. Thus, a faster CST time means a more dewaterable sludge. Large numbers of tests have shown 95 percent confidence limits of a single determination to be $\pm$ 10 to $\pm$ 20 percent of the mean value.

FILTRATION THEORY

In addition to running CST's, a specific resistance (r) was run. Specific resistance is primarily useful in comparing the filtration characteristics of different sludges. The theoretical approach to the equation using Darcy's law is,

$$\frac{dV}{d\theta} = \frac{P}{\mu} \frac{AK}{L} \qquad (1)$$

where:

$\frac{dV}{d\theta}$ = rate of flow, volumes (V) per time (θ), P = pressure difference, A = area, μ = viscosity, K = permeability, L = thickness,

Resistance can be defined as $F = \frac{1}{K}$

This is reduced to

$$\frac{dV}{d\theta} = \frac{PA^2}{\mu\ (wrV + R_f A)} \qquad (2)$$

where:

w = weight of dry cake solids per unit volume of filtrate

r = specific resistance

assuming the pressure is constant with time

$$\int_o^\theta d\theta = \int_o^V \frac{\mu w r V}{PA^2} + \frac{R_f \mu}{PA}\ dV \qquad (3)$$

The slope of θ/V versus V is defined as

$$b = \frac{\mu\ r\ w}{2\ PA^2} \qquad (4)$$

from this the specific resistance equation is:

$$r = \frac{2\ PA^2 b}{\mu\ w} \qquad (5)$$

TEST RESULTS

Figure 1 displays results of testing at the various sites throughout the United States. The three lines represent "difficult" to "easy" to dewater sludges based on CST and specific resistance values at various feed solids concentrations.

The plot of K_s/F_s vs $(C_L + C_F)/F_s$ shows the effect of chemical dosage on cake solids produced as a function of feed solids, and CST and specific resistance values.

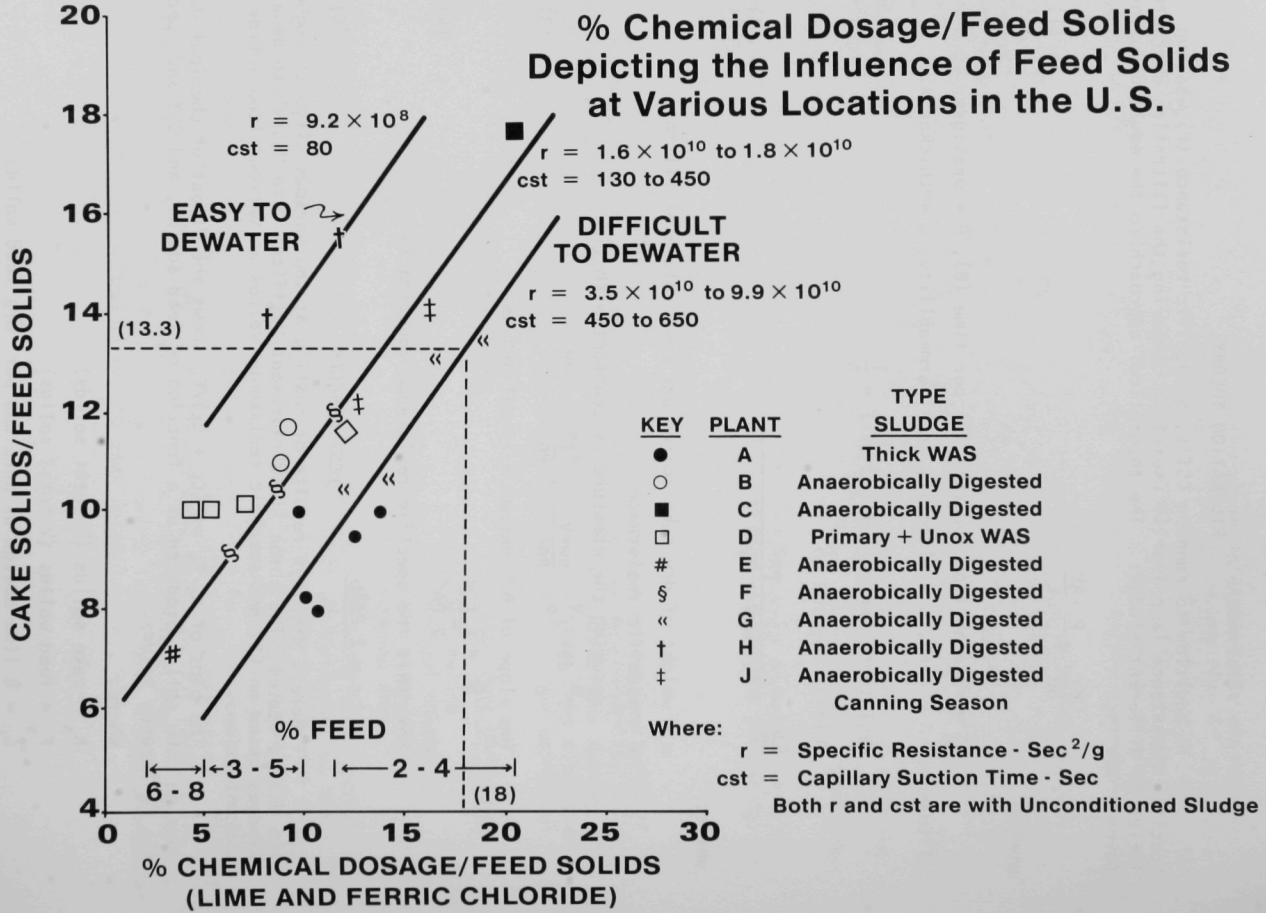
Where:

K_s = cake solids (% total solids)

F_s = feed solids (% total solids)

C_L = % lime dosage as pure CaO per dry feed solids

FIGURE 1 Ratios of Cake Solids/Feed Solids Vs. % Chemical Dosage/Feed Solids Depicting the Influence of Feed Solids at Various Locations in the U.S.



C_F = % ferric chloride dosage as pur $FeCl_3$ per dry feed solids

CST = capillary suction test (seconds)

The significance of this plot is apparent when estimating chemical dosage. For example, if a particular plan had a CST time of 450 to 650 seconds on their unconditioned sludge and a specific resistance value in the range of 3.5×10^{10} to 9.9×10^{10} sec^2/g with a feed solids of 3% total solids, the following prediction could be made. Assuming this plant required a cake of 40%, one could predict the chemical dosage required by finding the value of 13.3 (40÷3) on the ordinate and following across to the "difficult" to dewater sludge and obtaining a value of 18 on the abscissa. This results in total chemical requirement of 18 x 3 = 54% lime and ferric chloride addition. Figure 1 can also be used to compare chemical dosage for different sludges, i.e. "difficult" versus "easy" to dewater.

EFFECT OF FILTRATION TIME

One of the advantages of the diaphragm filter press is its flexibility of operation. The filtration and squeeze times can be adjusted to obtain optimum chemical requirements to produce a desirable yield and cake solids concentration. A shorter filtration cycle will result in a higher cake solids, but at the expense of yield. Therefore, an optimum filtration to squeeze time was determined at each site with each type of sludge. Table I shows the dramatic effect on total yield while keeping all other variables constant.

TABLE I. Data from Plant G

Cycle Time (min)	% T.S. Cake	Process Yield ($kg/hr \cdot m^2$)	Total Yield ($kg/hr \cdot m^2$)
36	33.5	2.75	1.70
20	34.0	2.55	1.20
15	36.6	2.60	1.05

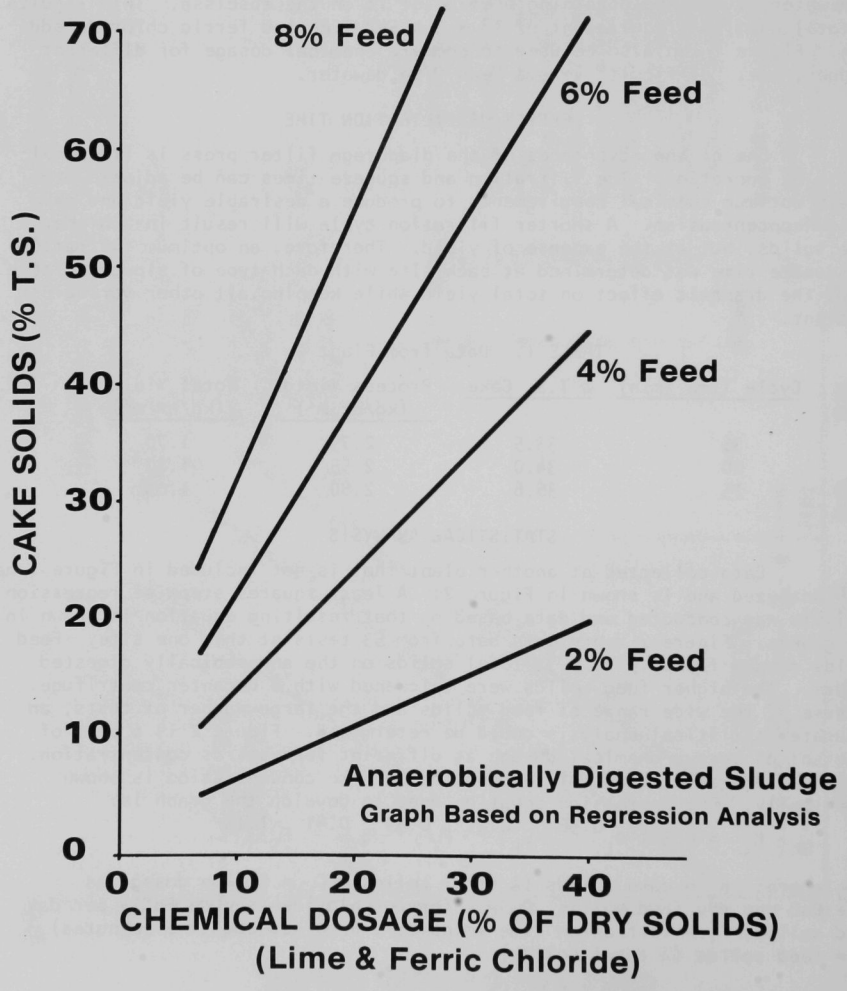
STATISTICAL ANALYSIS

Data collected at another plant that is not included in Figure 1 has been analyzed and is shown in Figure 2. A least squares stepwise regression analysis was conducted and data based on that resulting equation is shown in the graphs. Figure 2 represents data from 53 tests at this one site. Feed solids ranged from 0.9 to 9.3% total solids on the anaerobically digested sludge. The higher feed solids were thickened with a decanter centrifuge. Because of the wide range of feed solids and the large number of tests, an accurate statistical analysis could be determined. Figure 2 is a plot of cake solids versus chemical dosage at different feed solids concentration. Again the dramatic effect of feed solids on cake concentration is shown graphically. The regression equation used to develop the graph is:

$$K_s = 0.121\ C_L^{0.56}\ C_F^{0.25}\ F_T^{0.39}\ S_T^{0.41}\ F_S^{1.15}$$

where: K_s = cake solids (% total solids), C_L = % lime dosage as pure CaO per dry feed solids, C_F = % ferric chloride as pure $FeCl_3$ per day feed solids; F_T = filtration time (minutes), S_T = squeeze time (minutes), F_S = feed solids (% total solids)

FIGURE 2
The Effect of Feed Solids on Chemical Reduction Vs. Cake Solids



The correlation coefficient is 0.919 and the F-ratios are:

Variable	F-Ratio
F_s	39.3
S_T	33.8
C_L	17.9
F_T	9.4
C_F	2.3

Feed solids is the most important variable affecting cake solids as displayed by the F-ratios above. A similar regression analysis was conducted with the same data to develop an equation for yield. The results of this equation also showed feed solids as the most important independent variable. The equation for process yield (Y_P (kg/hr·m^2) is:

$$Y_P = \frac{0.080\ C_L^{0.41}\ C_F^{0.65}\ F_s^{1.37}}{F_T^{0.078}\ S_T^{0.050}}$$

The respective F-ratios are:

Variable	F-Ratio
F_s	36.5
C_F	10.5
C_L	6.1
S_T	0.3
F_T	0.2

The correlation coefficient is 0.902. The chemical dosage can be reduced significantly at the higher feed solids concentrations. In fact, the process yield is increased from 2.0 to 9.3 kg/hr·m^2 at 30% chemical dosage when increasing the feed solids from 2% to 6% total solids.

CHEMICAL DOSAGE

Previous data shows the relationship between yield and chemical dosage and that there are some definite trade-offs. Cassel and Johnson found that these trade-offs could be used to one's advantage. Cassel's work covered three states of chemical conditioning: marginal, optimum, and excess. The results showed that the diaphragm press could handle marginally conditioned sludge, such that cake discharge would be acceptable. The significance of this is that during periods of low sludge production chemical savings can be realized by dosing the sludge with less than optimum chemical quantities. Another more subtle use of this capability is the fact that most sludges vary widely and frequently in dewaterability. Unless the CST is checked regularly and chemical dosage changed accordingly, the possibility of marginally conditioned sludge being fed to the press is very high. With the diaphragm press this situation can be rectified by extending the squeeze time.

At several plant sites where sludge was tested, conventional filter presses were tested alongside the diaphragm press. Both presses were to optimize chemical dosage, cake solids, and yield. The results of this testing is found in Table II.

Table II. Chemical Dosage for Diaphragm and Conventional Filter Presses[9]

Plant	E		F		G		J	
Type of Press	D	C	D	C	D	C	D	C
% Lime	12	22	20	45	36	50	18	27
% Ferric Chloride	4	3.5	5	15	11	15	9	12
% DS	45	39	45	40	35	35	35	35

NOTE: D = diaphragm press
C = Conventional press

The plausible explanation as to why the diaphragm press used less chemicals is that the localized, mechancial squeezing pressure of the diaphragm is more efficient than the discharge pressure of the sludge feed pump. One will recall that cake thickness is a very important parameter when defining filtration. In essence, the distance from the sludge feed pump to the filter press acts to resist dewatering.

POWER REQUIREMENTS

Power requirements for the Envirex diaphragm filter press is dependent on various factors such as:

1. Size of the press - number of chambers,
2. Type of sludge to be dewatered,
3. Feed solids and other inherent sludge characteristics.

For the largest press offered by Envirex the range of power consumption is 15 to 25 kw hrs/ton dry solids dewatered. The power consumption increase with the smaller chamber press.

Values as high as 80 kw hrs/ton of dry solids are reported by others with a conventional filter press.

CENTRIFUGE TEST RESULTS

The Burlington, Wisconsin Wastewater Treatment Plant uses a basket centrifuge to dewater aerobically digested sludge.

Evaluation of the following alternative methods of dewatering and disposal were conducted by the consulting engineer. These various methods include:

1. centrifugation
2. filter press
3. belt press
4. vacuum filter
5. air flotation

Of these methods, basket centrifugation and land disposal were selected at the most cost-effective.

Burlington has one Envirex 48" bottom feed basket centrifuge. Flow rates, polymer dosages, type of polymers, combination of polymers and dosing points were varied to very effectively optimize the dewaterability of the aerobically digested sludge. Typical instantaneous feed rates are 60 to 90 gpm.

Without polymers, two truck loads per day at 6-8% solids and 24 hours of dewatering time were required. With polymers a dryer more haulable cake - (13-14%) results in one truck load per 8 hour shift and the centrifuge feed rate is increased almost 4 times. Thus polymer rates of up to $30/ton are more than offset by savings in manpower, sludge handling and electrical costs. Table III itemizes operating costs with and without polymer.

TABLE III

WITHOUT POLYMER		WITH POLYMER	
Instantaneous feed rate of 23 gpm @ 1.4% feed		Instantaneous feed rate of 88 gpm @ 1.4% feed	
150,000 gal/week to dewater		150,000 gal/week to dewater	
17,500 lbs/week to dewater		17,500 lbs/week to dewater	
can dewater 104 lbs/hour		can dewater 397 lbs/hr	
total hours of operation per week = 168		total hours of operation per week = 44	
labor cost + trucking per week @ 45% of the time = $378.00		labor cost + trucking per week @ 45% of the time = $99.00	
Electricity 4888 KWH @ $0.3/KWH = $146.63/week		Electricity 1585 KWH @ $0.3/KWH = $47.52/week	
Chemical Cost = 0		Chemist Cost = $30/ton	
Cake solids 6-8% untruckable		Cake solids 13-15% truckable	
Skimmings up to 50% of Basket Capacity		Skimmings @ 14% of Basket Capacity	
Labor & Trucking	$44.47	Labor & Trucking	$11.65
Electrical	17.25	Electrical	5.59
Chemical	-0-	Chemical	30.00
	$61.72/ton		$47.23/ton

$14.48/ton
Cheaper with the use of polymer

At the design rate of 40 gpm and a sludge age of 5 days the operating cost would be reduced to $25/ton with polymer.

A statistical analysis of the data collected at Burlington showed the importance of plant operating variables as well as machine variables. Sludge age was one of the most important variable affecting sludge dewaterability. The projected polymer costs at 40 gpm and a 5 day sludge age is $7.50/ton. The following table illustrates the effect of feed rate and sludge age on polymer cost.

TABLE IV

	Polymer Cost $/Ton	
	(optimum) 5-day sludge age	12 day sludge age
88 gpm feed rate	15	30
40 gpm (design) feed rate	7-1/2	15

Data have been collected from 28 sites throughout the U.S. on basket and decanter centrifuges. The range of cake solids produced and the anaerobically digested sludges varied from 9 to 27% total solids using 1 to 3 lbs/ton of polymer with a basket centrifuge. The decanter centrifuge required more polymer (3 to 18 lbs/ton) than the basket to achieve the same cake solids concentration. The power consumption for the Envirex decanter varies according to the ranges listed below.

Capacity		
lb/hr	gpm	hp/gpm
550 - 1100	60	0.35 - 0.40
4500	150	0.25 - 0.30
--	180	0.11

BELT PRESS

Data is not yet available on the Envirex belt press. Information by others have reported cake solids ranging from 12 to 23% cake solids concentration as an anaerobically digested sludge. Polymer dosage varied from 0 to 24 lbs/ton dry solids. Other variables evaluated were belt speed, belt tension, feed rate and feed solids concentration. A multiple regression analysis was conducted on this data and showed the relative importance of each variable. The following Table illustrates the effect of each variable on cake solids and recovery.

TABLE V

	Cake Solids*	Recovery*
Belt Tension	2	3
Belt Speed	2	2
Feed Rate	3	1
Feed Solids Concentration	1	4
Polymer Dosage	3	5

* Ranking of importance of independent variables

The other conclusions drawn from the regression analysis are as follows:

As,	Cake Solids	Recovery
Belt speed increases	decreases	decreases
Belt tension increases	increases	decreases
Feed solids increases	increases	increases
Feed rate increases	no change	increases

Pilot belt press tests with a one meter press were run at the Burlington Wastewater Treatment Plant which produced cakes ranging from 12 to 15% total solids. The maximum throughput with the one meter press is 336 lb/hr. The press required less energy consumption ($\simeq$ 3.5 Hp for operation) than the basket centrifuge. Polymer usage varied from $13 to $31/ton (1978 prices) dry solids.

VACUUM FILTRATION

Full scale top feed vacuum filtration tests have been conducted at a site in Wisconsin on straight waste activated sludge. Performance of the 10 foot vacuum filter can vary widely depending on sludge type, sludge characteristics, conditioning, type of vacuum filter, and loading rates. Typical loading rates varied from 1.0 to 1.8 lbs/hr/ft^2. Cake solids produced a range of 12 to 17% total solids with ferric chloride dosages of 600 to 1600 mg/l. Typical electrical costs have been reported ranging from 1.3 to 2.0 dollars per ton dry solids.

CONCLUSIONS

1. Comparison of the diaphragm filter press to a conventional filter press demonstrates the marked reduction in chemical consumption with the diaphragm filter press. The range of chemical consumption with the conventional press varied from 25 to 60% lime and ferric chloride versus 16 to 47% chemical usage with the diaphragm filter press.

2. In each of the dewatering devices evaluated, machine and process variables should be thoroughly optimized to provide the desired result with the least amount of chemical and power requirements. A statistical analysis of all the independent variables is necessary to determine the variables of most importance.

3. The diaphragm filter press has a high degree of operating flexibility and produces cake solids dry enough for autogenous combustion which is an energy savings.

4. The extensive polymer evaluation conducted at Burlington with the basket centrifuge resulted in the optimum selection of polymers, combination of anionic to cationic, and dosing points. The polymer cost per dry ton of solids is optimized at $30/ton at present. The high polymer costs is attributed to the fact that an aerobically digested sludge is an extremely difficult sludge to dewater as opposed to anaerobically digested, straight waste activated or primary sludge. By reducing the sludge age to five days, this polymer cost can be decreased by at least 50% and significantly less at the design rate. The hydraulic feed rate at present is 88 gpm, which is actually more than twice the design feed rate of 40 gpm. The polymer cost would be reduced to $5 to $10/ton at the design rate and at a five day sludge age.

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THE SOLAR DRYING OF SEWAGE SLUDGE ON AN INCLINED PLANE

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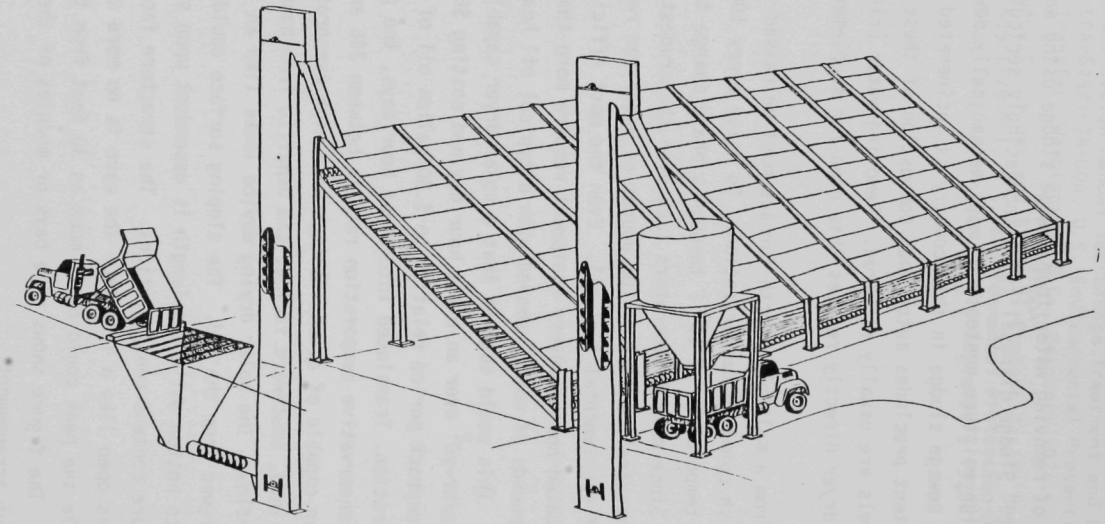
The ramifications of removing water from sewage sludge using energy from the sun are significant. Sludge disposal is an increasingly serious problem; especially given the high moisture contents of most mechanically dewatered sludges. A large portion of the sewage sludge in this country is incinerated. Since this high moisture content precludes autogenous combustion of these sludges, supplemental fossil fuels are usually required. Removing water prior to combustion using a solar dryer directly reduces this need for supplemental fossil fuels.

The moist gases from a multiple hearth furnace must be raised in temperature to 1200° F to accomplish complete deodorization. To raise every 100 pounds of water from an ambient temperature of 60° F through a phase change to 1200° F requires 211000 BTU. Since many incinerators lack even the crudest heat recovery systems, this marginal removal of 100 pounds of water can require as much as 3.0 gallons of supplemental fuel oil. Even the most efficient incinerators with sophisticated heat recovery equipment require more than 1.5 gallons of oil for every 100 pounds of water removed with a typical oil heat content of 145000 BTU/gallon. This would suggest that a solar dryer capable of removing an average of 1.28 lbs/hr-yd^{2} over an eight hour day and costing $85/yd^{2} installed would have a payback period relative to $.80/gallon oil of between 2767 and 5534 hours of operation. Translated into eight hour days, the payback period resulting from this conservative evaporation rate is between 346 and 692 days.

The configuration capable of accomplishing these high evaporation rates at such low cost is inclined, southward facing, and benefits from both free and forced convective airflow. The solar drying device looks like an inclined plane under glazing when viewed from the end. The sloping surface could be as much as 30 feet from top to bottom while its length is dependent upon sludge volume flow and final moisture content considerations. The structure faces south while its north side remains open like a lean-to. The eave is no more than 3 feet from the ground, while the peak could be as much as 30 feet from the ground as shown in the figure. The figure shows nine bays or modules of the device with a typical input-output arrangement.

Solar drying serves as an intermediate step between mechanical dewatering and waste heat drying. In the loosest sense, solar drying includes evaporation



MODULAR SOLAR DRYER OF SEWAGE SLUDGE

resulting from a series of solar related phenomena. For simplicity, consider evaporation rate as a function of the ambient air temperature, speed, and humidity as well as the equilibrium surface temperature of the material being dried.

Unfortunately, when large volumes of a material are to be dried, it is difficult to physically move the material such that: 1) the airspeed above the material is high; 2) the surface temperature is high due to uniform exposure to solar radiation; 3) the materials' surface moisture content is sufficiently high to sustain maximum evaporation rates; and 4) the air temperature and humidity directly adjacent to the material are high and low respectively. Spreading material on the flat to dry is seriously handicapped by the tendency for air to develop a stagnant high humidity boundary layer drastically reducing the mass diffusion of water into the atmosphere.

In light of these difficulties with conventional solar drying on the flat, a dryer design optimally should expose wet material on a slope more nearly normal to the sun's incident radiation. Material movement should be facilitated as often as possible using gravity. Coupled forced and natural convection should be encouraged on a slope to reduce the thickness of the boundary layer and thereby increase the mass transfer of water into the convective airstream. Finally, as is always the case, both investment and operations costs should be minimized per square yard of dryer area.

This solar dryer configuration addresses each of the problems enumerated above. The slope is between twenty and thirty degrees, dependent on the natural angle of repose of the sludge to be dried. There is no vertical wall so air can enter under the eave. The wet sludge is placed just under the glazing so that the drying surface is parallel to the glazing and no more than two feet below it. This sloping surface can be the exposed southern face of a long pile, or a sloping platform can be constructed as shown. This platform is parallel to the glazing and slightly less than the natural angle of repose of the sludge. Since this platform is inclined at an angle just less than the material's angle of repose, no more than 5 inches of material needs to be applied to the surface to completely cover it.

Most of the sun's radiation passes through the glazing striking the wet sludge surface. Depending on the absorptivity of the sludge, a certain portion of this energy is used to heat up both the surface of the sludge, and the airspace between the material and the glazing. Much of the rest of the energy is

either reflected away or reemitted by the grey body wet material. The glazing, however, is not transparent to most of this longer wave radiation and reflects much of it back at the wet material. This greenhouse effect greatly improves the efficiency of the dryer, while the chimney effect created by buoyant warm air between two inclined surfaces encourages evaporation. Water diffuses into the warm airspace above the sludge and is carried away by natural convection. Relatively dry outside air enters at the bottom under the eave and leaves at the top carrying away some moisture. Since the wet material is moved from the top of the device to the bottom, the drying air flow is counter-current. Counter-current drying results in a drier finished product since the drying potential of the air is at a maximum when the sludge is least willing to release its remaining moisture.

In addition to utilizing natural convection, the device benefits from wind regardless of its direction. Since evaporation is a function of the airspeed just above the material, any additional speed generated by external wind is helpful. When the wind, or some component of the wind, is from the south, air flows under the eave in the direction of natural convective flow. There is a wind barrier, not shown in the figure, which runs the length of the device and protects the opening at the top from a wind out of the north. Air flowing over the barrier creates a negative pressure under the glazing at the top of the device. This negative pressure sucks air from the top of the dryer increasing the airflow rate in the same manner that a wind improves the draft on a chimney. Although it is clear that evaporation rate is a function of air velocity, the importance of doubling the airspeed inside the dryer cannot be overemphasized. Furthermore, this doubling is not difficult when the natural convective airspeed between inclined parallel plates under typical solar conditions is less than three feet per second. Since the external airspeed is usually greater than this, the dryer exploits this available energy through the use of a baffle.

A moving rake is mounted between the surface of the wet sludge and the glazing. It grooms the slope turning the material over periodically to expose wetter material underneath while gradually moving material down the slope from the peak to the bottom, insuring that the surface remains parallel to the glazing. This rake traverses the length of the dryer automatically. To accomplish this, an iron rail is mounted just under the peak and another just under the eave running parallel to the length of the solar dryer. A light truss spans the distance between top and bottom resting on trolleys at both

ends that run on these rails. This truss moves from one end of the device to the other in the same manner that a light bridge crane moves. The trolleys are powered by a small electric motor that derives its power from bus bars mounted parallel to one of the rails just like a bridge crane. Spikes are mounted every three inches on the bottom flange of the truss. These spikes drag through the wet material as the rake truss progresses from one end of the device to the other. The spikes dislodge the material allowing the larger clods to roll a few inches downhill on each pass. The action of the rake depletes the material at the top of the slope unless it is replenished. At the same time material piles up along the length of the device at the bottom of the slope unless it is taken away. Therefore, in equilibrium the rake maintains the southward facing inclined surface of drying material parallel to the transparent glazing which itself is at an angle slightly less than the natural angle of repose of the material to be dried.

To supply wet material at the top of the slope continuously along the length of the device, a troughless screw conveyor is used. The screw conveyor sections span the ten feet between columns with the short shaft between sections supported by hanger bearings. These hanger bearings are mounted to the steel column-beam structure that supports the rails, cover, and drying surface. A small electric motor drives the screw from one end. The power requirements are minimal since the screw only moves material horizontally. Wet material supplied to the screw at one end by means of a bucket elevator is moved by the screw in a trough of this same wet material. As the raking action depletes the material at the top, the screw automatically supplies new material along the length of the device. As a gap develops in the screw conveyor's "trough" because of the raking action, material is immediately added to fill the hole. If the rake stops and material is no longer drawn away from under the screw, the trough will remain intact and new wet material will be moved all the way to the far end of the device.

In a similar fashion another screw conveyor with a half trough is mounted along the length of the device at the bottom of the slope beneath the eave. As the last spike of the rake prods dried material into this screw, the conveyor takes this dried material back to a point below the feed point of the top conveyor. Some of this dried material might be mixed with the wet feed to obtain an optimal feed consistency, but most of it will go directly to the waste heat dryer and then to the incinerator.

Since the construction is so simple and low cost, and the input-output arrangements so flexible, modules could quickly be installed on all available land adjacent to sewage treatment plants incinerating sludge. Following the initial construction of a few bays, additional bays could be added as the funds became available. Unlike most other processes, this dryer configuration allows for easy, low cost, expansion. Given the ample seasonal storage capabilities underneath the dryer, the conversion of the sun's abundant energy into a dried pulverized fuel might ultimately be considered as more than a method of reducing oil consumption in existing oil burning incinerators. If the land is available these dryer arrays are quite capable of producing a pulverized fuel that is 85% to 90% solids with virtually the same overall efficiency as that which prevailed throughout the higher moisture content drying range. Such a dried fuel would have value.

During the spring of 1979 a twenty-five foot long open ended, glazed, inclined plane, testing apparatus was constructed and operated at Princeton University. The purpose of this work was to gain an understanding of the determinants of heat and mass transfer between parallel planes inclined from the horizontal. A model was developed mathematically and corrolated with empirical data. What resulted was a mathematical model useful for comparative purposes but not sufficiently accurate for predictive purposes.

Test conditions were constant and simulated the average conditions over an eight hour february day in New Jersey. The incident radiation was constant at 716 BTU/hr-yd^2, (244 Watts/m^2), with an ambient air relative humidity of 68% and a temperature of 55° F. Coal slack and two different primary sludges were tested. Interestingly, drying runs with coal produced more immediate success in terms of high evaporation rates than did the drying runs with sludge. This was attributed to the difference in surface area between loose coal and compacted sludge off a rotary vacuum filter. The sludge's handling characteristics varied widely among different sludges and across the range of moisture contents. Again, what seemed critical was the tendency on the part of the sludge to remain compact and maintain a minimal surface area. The raking action in the actual dryer that facilitates material movement down the slope is critical to high evaporation rates because it serves to break up the material exposing the wet interior of each clump. It might even be advisable to include a small pugmill right before the input bucket elevator which would "fluff up" the sludge as required. Such a pugmill would also allow the operator to mix a small

portion of the dried sludge with the incoming wet sludge to obtain an optimal initial moisture content. The test data did not suggest a significant deviation from the maximum evaporation rate with either very wet sludge (less than 25% solids) or with drier sludge up to 45% solids. However, good materials handling properties might very well dictate a solids content range between 25% and 90%. The upper range, above 90% solids, is limited by a serious reduction in the evaporation rate and dusting. Throughout what is known as the constant drying moisture content range of 25% to 45% solids, the evaporation rate exceeded 1.12 lb/hr-yd^2, with a maximum of 1.35 lb/hr-yd^2 and a mean of 1.28 lb/hr-yd^2.

These rates are the best estimate of how a full size device would operate in the field. Minimizing the problems of similitude, I regard these evaporation rates as conservative, measured as they are using conditions simulating an average hour in an average February day in New Jersey. It is impossible to simply extrapolate up, but many of the present incinerators are located in areas more conducive to solar drying than New Jersey in February.

Throughout the test work done at Princeton it was observed that the odors emanating from the hundreds of pounds of sludge during drying were minimal in comparision to those driven off when a few pounds of sludge were heated to 500° F. This observation is essential to the understanding of the principal advantage of the dryer. Only a very small percentage of the objectionable volatiles are vaporized at the operating temperatures within the solar dryer. Since the sludge never exceeds 100° F, due to evaporative cooling, the few volatiles driven off with the water are not sufficient to require the deodorization of the moist air stream. Low operating temperatures mean a more efficient use of the sun's abundant yet extremely diffuse energy.

The use of the sun's energy to remove water from a material intended for combustion constitutes effective energy generation. Test data prove that a solar dryer of this configuration is capable of achieving evaporation rates in excess of 1.28 lb/hr-yd^2 across a range of moisture contents with an incident radiation of 716 BTU/hr-yd^2. This amounts to 1805 BTU/hr-yd^2 of energy saved by not allowing that water to be vaporized during combustion and heated to final stack temperatures of 500° F. The solar dryer has a minimum "efficiency" of 252% when rated in this manner.

SLUDGE MANAGEMENT AND ENERGY INDEPENDENCE AT THE FRANK E. VANLARE WATER POLLUTION CONTROL PLANT

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MONROE COUNTY DIVISION OF PURE WATERS

The Frank E. VanLare Water Pollution Control Plant, a conventional activated sludge plant, located in Monroe County, New York provides wastewater treatment service to over half the County's population including the City of Rochester. The 430,000 people in the service area and connected commercial and industrial establishments generate an average flow of 90 mgd. The plant design flow is 100 mgd.

PRESENT OPERATION

Solids removed in the primary and secondary systems are thickened in eight (8) gravity thickening tanks and pumped to day tanks. The day tanks are operated on the fill and draw basis and provide sludge to five (5) vacuum filters. Dewatered sludge is incinerated in three (3) multiple hearth furnaces and residual ash is pumped to on-site lagoons. Figure 1 is a simplified schematic of the plant.

Approximately 60 tons* of dry sludge solids are processed daily. In order to process this quantity of solids three vacuum filters and all three operable furnaces (#3, #4, and #5) must be run continuously. In the event of shutdown of one of the large (11 hearth) furnaces sludge must be stored in holding tanks, which are of limited capacity (344,000 cf)**. Consequently any prolonged incinerator outage results in storage in the treatment system and deterioration of sludge quality.

The sludge processing limitations caused by restricted incinerator capacity are further aggravated by the low solids content of the sludge cake produced by the vacuum filters which averages 16 percent solids. This high

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* In addition 12 tons per day of sawdust are used for vacuum filtration conditioning.

** 361.2 tons dry solids @ 3 percent.

OPERATIONAL SLUDGE HANDLING FACILITIES BASIC DATA

Process Units	No. Units	Physical Dimensions
Sludge Thickening Basins	8	10' swd x 70' diam.
Sludge Holding Tanks	2	30' swd x 80' diam.
Sludge Day Tanks	4	10.5' swd x 20' sq.
Sludge Dewatering Units (Vacuum Filters)	5	11.5' diam x 12' face (430 sq. ft.)
Incinerators	1	22'-3" diam. x 6 hearths
	2	22-3" diam. x 11 hearths

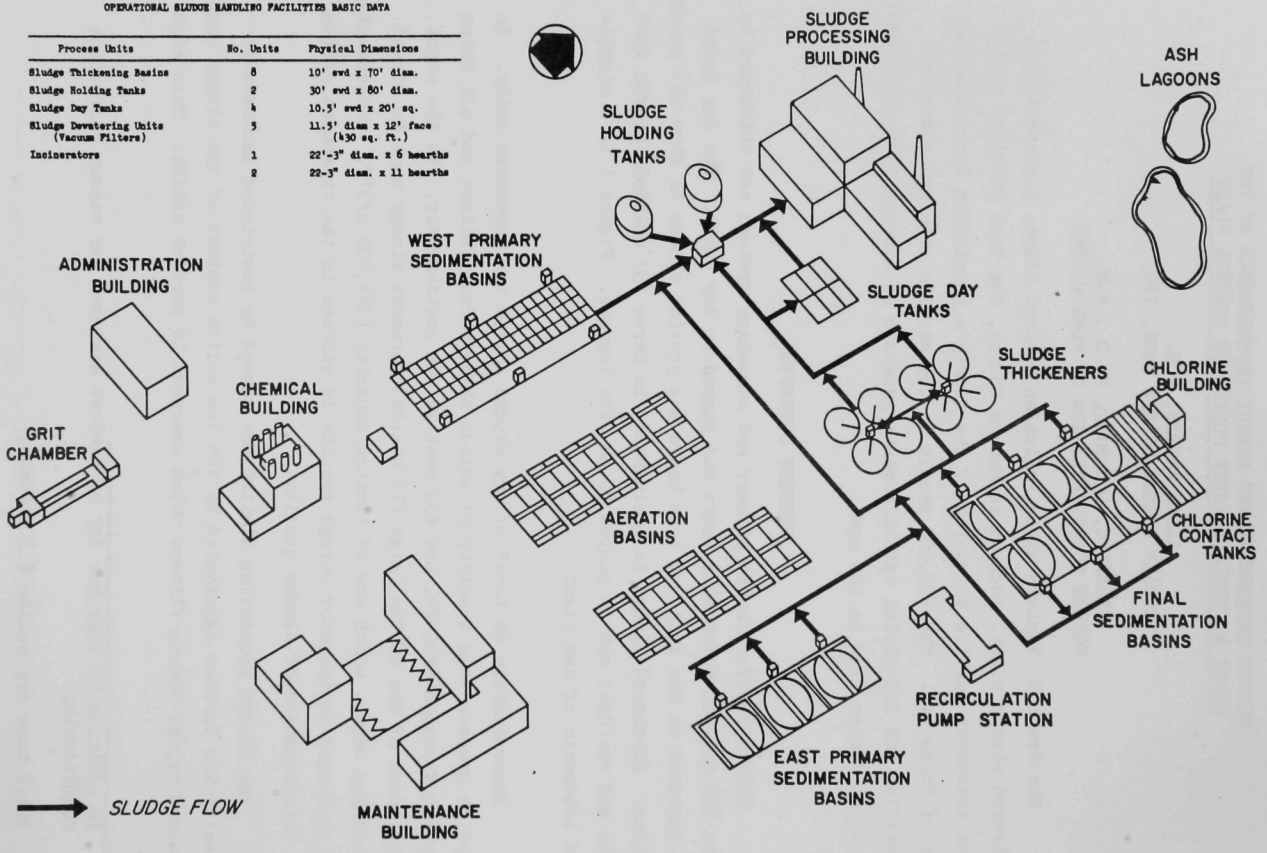

FIG. 1 SCHEMATIC DIAGRAM OF FRANK E. VANLARE WATER POLLUTION CONTROL PLANT

moisture burden limits furnace capacity and requires high auxiliary heat input to dry the sludge and to reach odor destruction temperatures of 1400°F. Fuel use averages approximately 200 gallons per dry ton or 12,000 gallons per day***. At the August, 1979 fuel oil cost of 72 cents per gallon this amounts to over $3,000,000 per year. A summary of the operating costs for solids processing is shown in Table 1.

TABLE 1

PRESENT OPERATING COSTS FOR SLUDGE PROCESSING AT THE FRANK E. VANLARE WATER POLLUTION CONTROL PLANT

Manpower	$480,128.19
Electrical	173,336.76
Chemical	211,550.38
Fuel	3,133,014.00
Water	3,710.00
Sawdust	217,096.55
Total	$4,218,835.90
Unit Cost	$192.64/TDS

Notes: 1. Based on processing 60 TPD dry solids and August, 1979 unit costs as follow:
   a. Electrical cost at $0.0263/kWh
   b. Chemical costs at $1.27/lb of polymer
   c. Fuel oil cost at $0.7153/gal
   d. Sawdust cost at $5.65/cy
   e. Supervisory labor at 9.40/hr and other labor at $7.00/hr including fringe benefits.

2. Maintenance costs are excluded.

The Frank E. VanLare plant is not unique in being faced with increasing sludge disposal problems. Difficult to dewater biological sludges limit furnace capacity. High auxiliary fuel requirements and expensive fuels have

*** Based on 60 tons per day of dry solids presently being processed.

made conventional incineration of wet sludges very expensive. Increasing future sludge quantities and ever-increasing fuel costs will further complicate problems and increase costs. Recognizing these problems, the Monroe County Division of Pure Waters initiated studies in 1977 to develop a long-term plan for cost-effective environmentally responsible disposal of sludge.

SLUDGE MANAGEMENT PLAN

The sludge management plan for the Frank E. VanLare Water Pollution Control Plant consists of (1) replacement of four vacuum filters with continuous belt filter presses (CBFP) capable of obtaining 25 percent sludge cake solids, (2) modification of the two large multiple hearth furnaces (22'-3" x 11 hearth) for starved air combustion mode of operation, (3) provision for addition of refuse derived fuel to the two large furnaces and addition of waste heat boilers and steam turbine generators for electrical power generation for in-plant use. Upon implementation of the plan total installed operable dewatering capacity will be 180 TPD dry solids and the furnace capacity will be 181 TPD.

Figure 2 is a simplified schematic of the plan. Primary sludge, generated in the eight primary sedimentation basins (1), will be pumped by new sludge pumps (2), to either the existing thickeners (3), storage tanks (4), or the day tanks (5). Waste activated sludge from the secondary sedimentation tanks (6) will continue to be pumped by the existing waste activated sludge pumps (7) to the thickeners. Primary and waste activated sludge will be either thickened separately or together in the thickeners. The day tanks will continue to be used prior to dewatering.

From the day tanks, sludge will be pumped by progressive cavity pumps (8) to any combination of eight new continuous belt filter presses (9) and one existing filter (10). The existing vacuum filter will only be used in combination with the existing six-hearth furnace (11) which will be retained as standby.

Dewatered sludge from the new belt filter presses will be transported by the existing conveyor systems (12) to the two eleven-hearth furnaces (13). The existing conveying systems will be modified to feed either of the two furnaces or to transport the dewatered sludge outside the building.

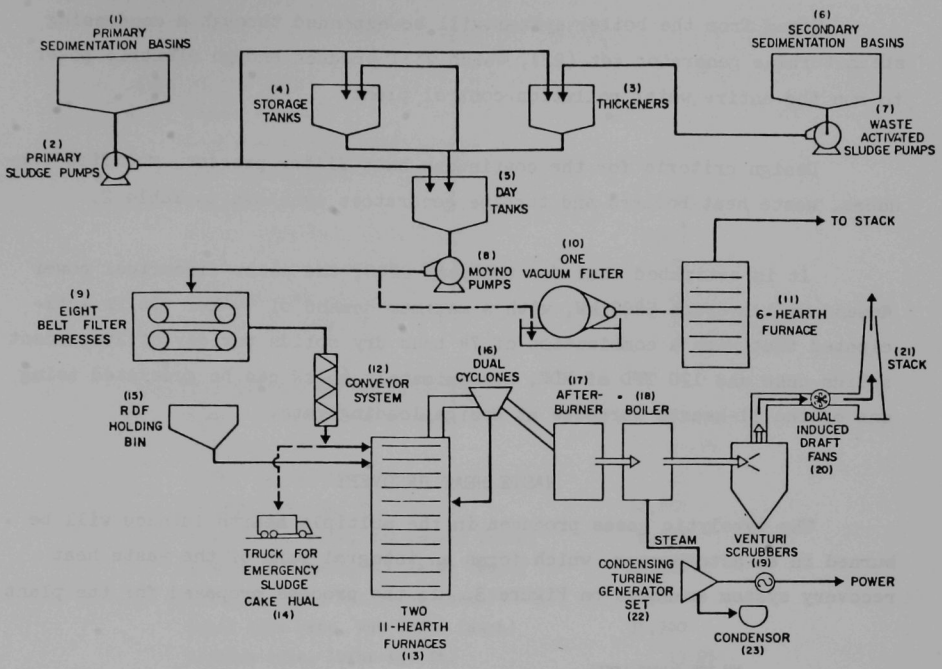

FIG. 2 SLUDGE MANAGEMENT PLAN - FRANK E. VANLARE WATER POLLUTION CONTROL PLANT

The two existing eleven-hearth furnaces will be modified to permit operation in a starved air combustion mode utilizing RDF addition. RDF, heavy and/or light fraction, will be trucked from the County's resource recovery plant and stored in a holding bin (15). A metered amount of RDF will then be fed to the furnaces for co-firing with the dewatered sludge.

Particulates from the pyrolytic gases produced by starved air combustion of sludge and refuse will be removed in dual cyclones (16) and the pyrolytic gases then ignited in an afterburner (17). Exhaust gases will then pass through the boiler (18) including superheater and economizer sections prior to passing through venturi scrubbing (19). The clean exhaust gas will then be discharged through the stack (21) by an induced draft fan (20).

Steam from the boiler system will be expended through a condensing steam turbine generator set (22), which will produce enough electric power to run the entire water pollution control plant.

Design criteria for the continuous belt filter presses, modified furnaces, waste heat boilers and turbine generators is shown in Table 2.

It is estimated that for the year 2000, the plant electrical power demand will average 5400 kW, with a maximum demand of 74 kW. It is anticipated that with a combination of 74 tons dry solids per day of 25 percent solids cake and 170 TPD of RDF, approximately 65 kW can be generated using one of the 11-hearth furnaces at design loading rate.

WASTE HEAT RECOVERY

The pyrolytic gases produced in the multiple hearth furnace will be burned in an afterburner, which forms an integral part of the waste heat recovery system as shown on Figure 3. In the process proposed for the plant,

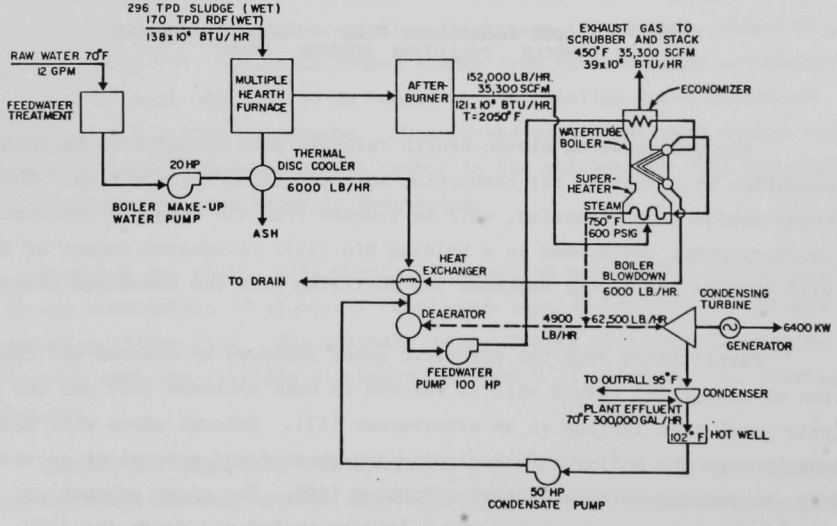

FIG. 3 WASTE HEAT RECOVERY SCHEMATIC - FRANK E. VANLARE PLANT

TABLE 2 DESIGN CRITERIA

BELT PRESS FILTERS	
Number of units	8
Feed solids concentration (%)	4
Belt width, meters (each)	2
Polymer, $/TDS	6-12
Solids capacity, lb/hr (each)	1,600
Total system capacity, TDS/day*	134.4
Expected cake solids, %	25
Solids recovery, %	97
MODIFIED MULTIPLE HEARTH FURNACES	
Number of units (existing)	2
Diameter - ft.	22.75
Number of hearths	11
Hearth area, sq. ft. (each)	2,860
Sludge feed rate, dry lbs/hr (each)	6,150
Sludge feed rate, wet lb/hr (each)	24,600
RDF feed rate, wet lb/hr (each)	13,300
Total feed rate, wet lb/hr (each)	37,900
Loading rate, lb/hr-sq. ft.	13.25
Afterburner detention time, sec.	2.00
WASTE HEAT RECOVERY	
Number of boilers	2
Type of boilers	Watertube boilers with separate superheater and economizer
Steam pressure, psi	600
Steam temperature, ^{o}F	750
Rated boiler steam capacity lb/hr (each)	75,000
Turbine type	Condensing steam
Number of units	2
Capacity, KW (each)	7,500
Condenser vacuum, in. Hg	2
Generator voltage, volts	4,160
Average electrical output, KW	6,400

* With one unit as standby

hot gases from the afterburner pass through the waste heat boiler which consists of a first stage boiler, superheater, second stage boiler, and economizer sections. This arrangement is used to limit superheater tube temperatures. The resultant 750°F, 600 psi steam is expanded in a condensing turbine, which drives a generator.

PHASED IMPLEMENTATION

The modification previously described will be implemented in stages. Stage 1 installation of belt presses will reduce fuel oil consumption from approximately 200 gallons per ton of dry solids (gal/TDS) to approximately 80 gal/TDS, a 7200 gallon per day reduction. This reduction in oil consumption results mainly from the reduced water content of the sludge cake. The reduced water content of the sludge cake will also result in an increase of the sludge handling capacity of each furnace. Only one large furnace (#4 or #5) will be necessary to process daily sludge production up to 64 tons per day. Therefore, reserve capacity consisting of the other eleven-hearth furnace and the six-hearth furnace will be available, which greatly increases sludge processing reliability.

Stage 2 consists of conversion of furnaces #4 and #5 to the starved air combustion mode of operation. Modifications will be made to one furnace at a time and operation will be on sludge only (no RDF). Conversion of the second furnace will not start until the first is on-line and successfully operating for six months. Conversion to the starved air combustion mode of operation will further reduce fuel oil consumption to 40 gallons per ton of dry solids or 2400 gallons per day, thus effecting a further savings of 2400 gallons per day. Operation in the starved air combustion mode will further increase furnace capacity. Hearth loading of up to 10 lbs/hr/sf will be achievable. The resulting capacity of each 11 hearth furnace will be 85.8 TPD. The higher furnace loading results from the reduced heat release in the furnace when operating in starved air combustion mode.

Stage 3 consists of construction of RDF receiving, storage and conveyance facilities and power generation facilities. Addition of RDF will essentially eliminate fuel oil use. Seventy-four TPD of sludge and 170 TPD of RDF will produce 62,500 lbs/hr of steam at 600 psi and 750°F. When expended in a condensing turbine this will produce 6400 kw of electric power. This

quantity is sufficient to essentially power the entire plant. A summary of the operating parameters of each large (22'-3" x 11 hearth) furnace under present conditions and as modified by each stage is shown in Table 3.

TABLE 3

FURNACE OPERATING PARAMETERS (22'-3" x 11 hearth)

	Present Operation Incineration @ 16% Solids	Stage 1 Incineration @ 25% Solids	Stage 2 S.A.C.* @ 25% Solids	Stage 3 S.A.C.* @ 25% Solids+RDF
Hearth Area, sf	2,860	2,860	2,860	2,860
Wet Solids Loading, lb/hr	21,450	21,450	28,600	38,933
Dry Solids Loading, lb/hr	3,432	5,362	7,150	17,775
Hearth Loading lb/hr-sf	7.5	7.5	10.0	13.3
Theoretical Excess Air-%	225	150	30-50	30-50
Auxiliary Fuel MM BTU/hr	50.2	30.0	20.0	-
Equivalent #2 Fuel Oil, gal/hr	359	214	143	-
gal/TDS	209	80	40	-
Furnace Heat Release, MM BTU/hr	42.5	41.6	37.5	69.3
Afterburner Heat Release, MM BTU/hr	21.4	26.3	36.1	69.3

* Staved air combustion.

The effect of implementation of Stages 1 and 2 on fuel oil consumption is shown in Table 3. As can be seen fuel oil will be reduced from about 200 gal/TDS presently, to about 80 gal/TDS upon completion of Stage 1 and further to 40 gal/TDS after Stage 2. Stage 3 will result in essentially complete elimination of fuel oil usage.

COSTS

Table 4 shows the operating requirements and costs for Stages 1, 2 and 3. All costs have been based on processing 74 TPD of sludge (the year 2000 estimate). As can be seen, implementation of each stage significantly reduces operating costs. Table 5 shows the capital cost required for implementation of each stage in present day dollars and the resulting pay back period for the investment. These costs DO NOT include consideration of Federal or State grants which are anticipated.

TABLE 4 ANNUAL OPERATING COST

Item	Units	Stage 1 Quantity	Stage 1 $	Stage 2 Quantity	Stage 2 $	Stage 3 Quantity	Stage 3 $
Operating Labor	MEN	24	445,000	24	445,000	33	610,000
Power Consumption	KW	510	118,000	610	141,000	1000	230,000
Power Production	KW	-	-	-	-	6000	(1,382,000)
Fuel Oil	GAL/TDS	80	1,546,000	40	773,000	-	-
RDF	TONS	-	-	-	-	170	309,000
Chemicals	$		215,000	-	215,000	-	215,000
Maintenance	$		150,000		180,000		500,000
Total	$		$2,474,000		1,754,000		482,000
Unit Cost	$/TDS		92		65		18

Notes: 1. Based on processing 74 TPD dry solids in each stage
2. All costs in present day (August 1979) prices as follows:
   a. Electric power @ $0.0263/kWh
   b. Fuel oil @ $0.7153/gal.
   c. RDF @ $5.00/ton.
   d. Manpower @ $18,500/man year incl. fringes.
   e. Chemicals-Polymer @ $8.00/TDS.

TABLE 5 PAY BACK ANALYSIS

Stage	Description	Capital Cost ($)	Operating Cost ($)	Incremental Operating Cost Saving ($)	Pay Back (Yrs)
	Present Operation	-	5,213,000 (1)	-	-
1	Belt presses replace vacuum filters	2,600,000	2,474,000 (2)	2,739,000	0.9
2.	Conversion of furnaces #4 & #5 to SAC (sludge only)	2,300,000	1,754,000 (2)	720,000	3.2
3	Waste heat recovery w/power gen. (sludge+RDF)	14,700,000	482,000 (2)	1,310,000	11.2
Total Project		19,600,000			6.0

(1) Based on processing 74 TPD at same unit costs shown in Table 8.

(2) Refer to Table 8 for operating cost details.

PRESENT STATUS

At the present time four belt presses are being installed. These units are being installed with local funds in view of the anticipated fuel oil reduction and short pay back period. A Step 2 grant application for the remainder of Stage 1 and Stage 2 is expected to be approved shortly. Based on our schedule Stage 2 will be completed in 1982. Stage 3 will await an update of the State-of-the-Art of RDF handling and waste heat power generation.

ENERGY CONSIDERATIONS IN SLUDGE MANAGEMENT AT THE RENO-SPARKS WASTEWATER TREATMENT PLANT

By John G.M. Gonzales, Chester W. Auckly, Eugene Davis
City of Sparks, Nevada

INTRODUCTION

The cities of Reno and Sparks share the same valley in the Western part of the State of Nevada. The Reno area which consists of Sparks, Lake Tahoe, and Carson City is second only to Las Vegas, Nevada, in gross revenues from the state's largest industry, gaming.

The population in the metropolitan area of Reno and Sparks is approximately 160,000. Because of the area attraction, an added tourist population may exceed 50,000, giving a total equivalent population of the metropolitan area of 210,000. Because of the proximity of Reno to San Francisco and Sacramento, the number of tourists visiting Reno has increased substantially over the past few years. The expansion of the tourism industry and the gaming has also affected a substantial increase in the permanent population of the Reno-Sparks metropolitan area. Reno is, therefore, one of the fastest growing cities in the country.

THE RENO-SPARKS JOINT WASTEWATER TREATMENT FACILITY

Early in the 1960's decade, the cities of Reno and Sparks agreed to discontinue using individually owned sewage treatment facilities. A joint treatment facility was constructed to serve the sewage treatment needs of the entire metropolitan area. The Reno-Sparks Joint Water Pollution Control Facility began operating in 1967.

The present capacity of the Reno-Sparks joint sewage treatment facility is 20 million gallons per day. The design is a standard activated sludge secondary treatment facility using anaerobic sludge digestion for sewage sludge conditioning. Methane gas produced by the anaerobic sludge digestion process is utilized to fire boilers that provide hot water and steam to heat the buildings that are part of the sewage treatment facility. The methane gas is also

utilized as the fuel to fire heat exchangers for heating the anaerobic sludge digesters to a temperature of 96° Fahrenheit (mesophyllic range).

SLUDGE DISPOSAL AND/OR BENEFICIAL REUSE

The original plan for sludge disposal at the Reno-Sparks wastewater treatment facility included air drying in shallow sludge beds and hauling of the dried sludge to farmland and new park sites for use as a soil conditioner. Unfortunately, because of the rocks and other debris including plastics which were dispersed throughout the dried sludge, the desirability of using the sludge was extremely low. Sludge accumulated on site until after 11 years of operation, more than 110,000 cubic yards of sludge was stockpiled in a storage area. Prompted by fears of flooding over the sludge storage site and the possible contamination of nearby ground water supplies, the State of Nevada Division of Environmental Protection issued an order to dispose of all stored sludge and cease any further sludge storage operations.

After the issuance of the order to remove the stored sludge, the treatment plant staff began investigations into various alternatives for immediate disposal of the large quantity of digested sludge. Alternatives considered included composting, vermicology (composting and degradation using earthworms), and hauling the sludge to an approved landfill. Because of the short time frame allowed for the completion of the final sludge disposal, the only solution feasible was to haul the sludge to an approved sanitary landfill.

INTERIM SLUDGE DISPOSAL

In order to avoid sludge problems in the next few years, an interim disposal plan was studied. The interim plan could allow for sludge disposal and/or reuse until a permanent sludge disposal solution could be designed and constructed. Four alternatives were considered:

1) Hauling digested sewage sludge by tanker truck to an approved sanitary landfill.
2) Mechanical dewatering and hauling of the dewatered sludge to an approved

sanitary landfill.

3) Mechanical dewatering, incineration, and hauling the ash to an approved sanitary landfill.
4) Direct injection of the digested sludge into the top 24 inches on non-irrigated ground.

The alternative selected for interim sludge disposal was direct injection of digested sludge into the land surface of acreage adjacent to the joint treatment facility. Analysis of the remaining alternatives indicated prohibitive costs or disposal problems. Mechanical dewatering would require a large capital outlay for construction and would require excessive construction time. Hauling liquid sludge to an approved sanitary landfill was projected to be more than four times the annual cost of soil injection. It was also anticipated that the only approved landfill near the wastewater treatment facility would not be able to handle the large quantities of liquid sludge that would be produced from the Reno-Sparks sewage treatment facility. Mechanical dewatering, incineration, and ash hauling required a substantial capital outlay and excessive construction time.

COST COMPARISON AND ENERGY CONSUMPTION ANALYSIS OF SLUDGE DISPOSAL OPTIONS

Four sludge disposal alternatives were analyzed to include a comparison of energy utilization and overall cost. Figure No. 1 and Figure No. 2 illustrate the cost and energy evaluations of the alternatives. Capital construction costs and the resultant annual debt repayment were excluded from the analysis.

The energy use comparison, depicted in Figure 1, illustrated relative electrical power and fuel consumption for the four sludge disposal alternatives that were analyzed. The projected fuel and power requirements for the sludge injection option were considerably less than those of the three remaining alternatives.

The annual costs associated with sludge disposal alternatives illustrate the cost-effectiveness of the sludge injection option. However, the mechanical

COMPARISON OF ENERGY USE OF 4 SLUDGE DISPOSAL ALTERNATIVES

(ORDINATE MEASURE-RELATIVE SCALE)

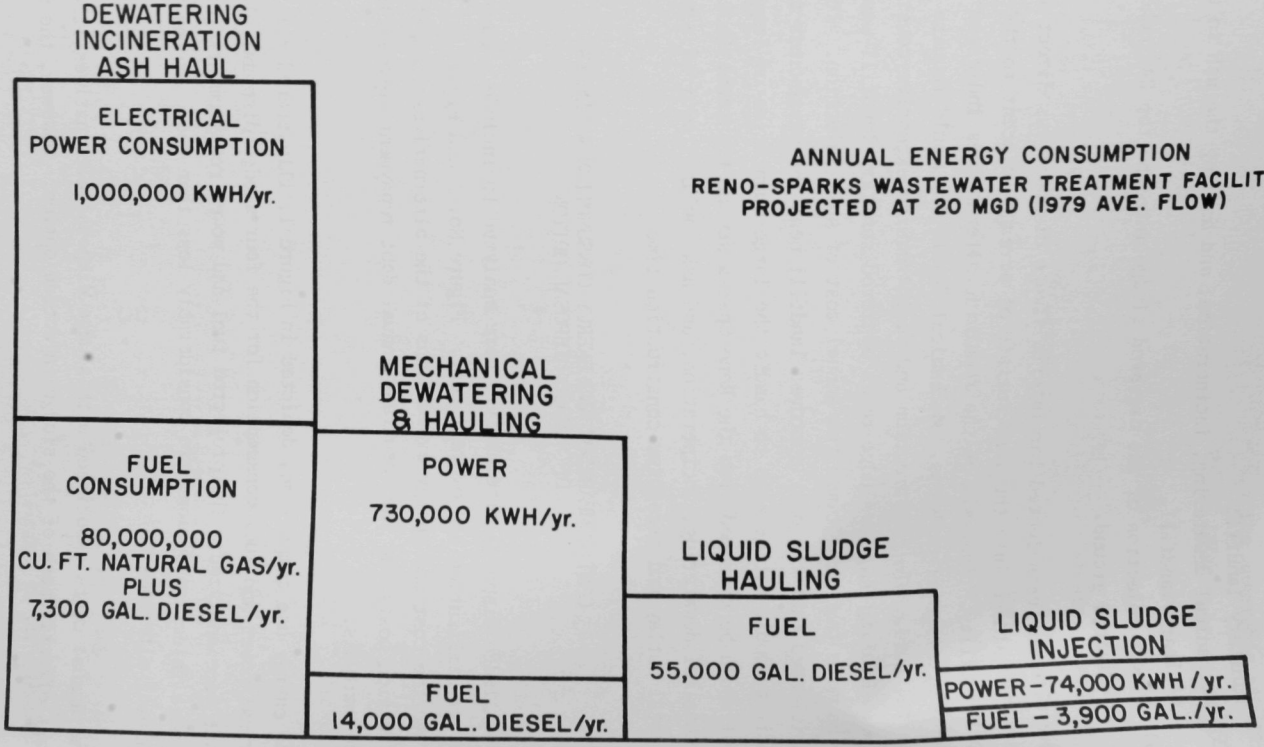

FIGURE 1

COMPARISON OF PROJECTED COSTS OF 4 SLUDGE DISPOSAL ALTERNATIVES

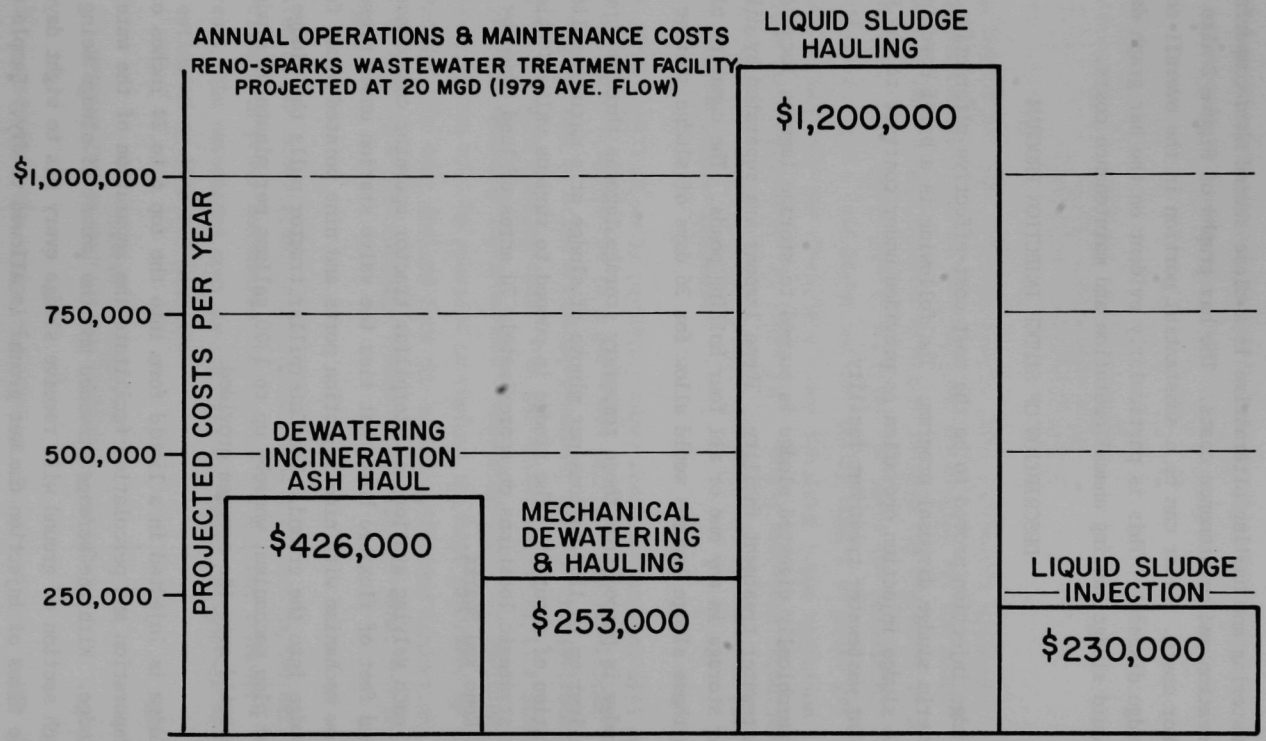

FIGURE 2

dewatering and hauling alternative is a close second when comparing annual operation and maintenance costs. The bar graphs of Figure 2 also includes labor costs. Labor can be a substantial portion in the overall cost for sludge disposal. This is particularly evident on the bar graph depicting liquid sludge hauling annual operation and maintenance costs.

DESCRIPTION OF SLUDGE INJECTION PROGRAM

Sludge injection proved to be the most cost-effective alternative for the interim sludge disposal program. The following is a brief description of the sludge injection operation as provided under contract to the Reno-Sparks joint wastewater treatment facility.

Anaerobically digested sludge is pumped to storage lagoons located near the wastewater treatment facility. These lagoons are separated by dikes to allow for storage in any one or all four holding ponds. The capacity of storage in these sludge lagoons would allow for 26 days of sludge storage if needed.

Sludge is pumped from these temporary storage lagoons through a pump that can deliver up to 1200 gallons per minute of sludge at a maximum solids concentration of about 6%. The sludge is pumped to various valve stations located at strategic locations on approximately 70 acres of land owned by the cities of Reno and Sparks.

At each valving station, a Caterpillar tractor operator can connect a thousand feet of flexible hose that ties the valve station under pressure to a plow mechanism with nine injection ports and nine pointed plows for injecting sludge into the ground. The Caterpillar tractor pulls the sludge injection and plow mechanism, whereby up to 1200 gallons per minute of sludge can be injected over the acreage provided.

Sludge is injected in a liquid form into the top 6 to 24 inches of ground. Evaporation and percolation facilitate the separation of the water from the sludge. With the acreage provided and the amount of sludge being injected, each section of ground will receive sludge every six to eight days. Between the times of injection the wet ground is allowed to dry. Sample wells have

been provided as an early warning system to detect any possible changes in the ground water quality within the sludge injection area and immediately downstream.

METHANE GAS UTILIZATION AT THE RENO-SPARKS JOINT WASTEWATER TREATMENT FACILITY

Under the present operation, the Reno-Sparks joint wastewater treatment facility utilizes methane gas to fire boilers for heating all the plant buildings and to fire sludge heaters necessary to keep the digesters at a high temperature. Even though gas is utilized for these two beneficial purposes, there has been a continuous and increasing supply of waste gas that has been burned into the atmosphere since the plant began operation. Gas production has increased by 22% since 1967. Plans have been formulated for the beneficial use of the wasted methane gas produced by the anaerobic processes within the sludge digesters.

A treatment plant modification is presently under construction that will convert an electrically-driven motor to a gas-fired engine. The electrical motor is presently used to drive a turbine-producing air for the activated sludge process and post-aeration tank. The gas-driven engine will be fired by methane gases produced by the anaerobic sludge digesters. In addition, a heat recovery system will be provided to capture the heat generated by the gas-fired engine. The heat derived from the heat transfer and conduction system will be used in a heat exchange system to provide hot water for treatment plant use.

The conversion of the electrically-driven motor to a gas-fired engine will provide an electrical power consumption savings of 3.5 million kilowatt-hours per year. The associated cost savings for the power consumption reduction is estimated at $105,000 annually.

PROGRAM TO INCREASE METHANE GAS PRODUCTION AND COMBUSTION RATING

The combustible gases from the sludge digesters at the Reno-Sparks joint wastewater treatment facility, produce 586 BTU per cubic feet of digester

gas. Presently, 60% of the volatile gas produced by the anaerobic digestion process is methane. The projected total available methane gas energy, upon completion of a 10-million gallon per day expansion of the facility, is 13,800,000 BTU per hour. Approximately 5,000,000 BTU per hour are utilized for sludge heating and building heating at the wastewater treatment plant. It is anticipated that the methane-driven blower engine will require 5,800,000 BTU per hour. Therefore, 3,000,000 BTU per hour will be available for additional uses. It is anticipated that a portion of this gas could be stored for emergency use if the production of methane decreased due to problems in the operation of the treatment plant.

An additional phase of expansion is slated for completion in 1983. At that time, an additional 3,000,000 BTU per hour of methane gas production will be available for beneficial uses based on a continued operation of 60% methane. The additional methane gas will be adequate to fire a 700-horsepower engine, similar to the engine currently being installed under the present construction program.

The joint treatment plant staff are currently working on methods to increase the methane content from 60% to 75%. The increase could increase the available BTU of the digester gases from 586 BTU per cubic foot to 732 BTU per cubic foot. If the attempts to increase the percentage of methane prove successful, an additional 1,326,000 BTU per hour will be available for beneficial use within one year, and 1,785,000 BTU per hour will be available in addition to the expected 11,900,000 BTU per hour produced when the wastewater treatment facility has reached a capacity of 40 million gallons per day.

SUMMARY AND CONCLUSIONS FOR FUTURE USE OF SEWAGE SLUDGES

Although the present sludge injection program currently utilized at the Reno-Sparks joint wastewater treatment facility is an interim procedure, the use of this sludge disposal technique is cost-effective. In areas where land is available for providing adequate acreage for the sludge injection program, this alternative may be the most cost-effective and energy efficient means of sewage sludge disposal. Unfortunately, there is not enough available acreage

to use sludge injection at the Reno-Sparks joint treatment plant on a permanent basis. It is anticipated that sludge dewatering and composting, followed by a beneficial reuse as a soil conditioner, may be the alternative selected for permanent sludge disposal at the Reno-Sparks facility. Other alternatives are currently being investigated for the permanent solution including piping the sludge several miles into the desert and using spray irrigation for disposal.

Methane produced as a by-product from the anaerobic sludge digestion process can provide an energy-efficient alternative for conserving electrical power and other fuels. The operators of all wastewater treatment plants that utilize anaerobic sludge digestion have an obligation to produce the highest possible concentration of methane gas from the digesters they operate. Efficiently operating sludge digesters can produce methane gas which can save hundreds of thousands of dollars each year in electrical power costs. The optimum use of anaerobic sludge digester produced methane gas, represents progress toward self-sufficiency in the operation of a wastewater treatment facility.

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SESSION 9

ENERGY RECOVERY AND REUSE

Chairpersons: Robert Mourninghan

Russ Eberhart

AN ENERGY USE AND RECOVERY SURVEY OF WASTEWATER TREATMENT PLANTS

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1. INTRODUCTION

An energy use and potential energy recovery survey of a small number of wastewater treatment plants has been accomplished for the Department of Energy in preparation for a national survey. The energy recovery information was sought for digester gas generation, grease and oils collected, hydroelectric potential, and thermal energy extraction potential from the effluent using heat pumps.

In preparation for the national survey a model has been developed for initially estimating the potential hydroelectric power output, annual energy output and costs of hydroelectric development at wastewater treatment plants. The model has been applied to information for the two largest plants in the country with some general assumptions for many of the variables.

2. ANALYSIS METHODS

A copy of the questionnaire sent to 13 wastewater treatment plants for a preliminary survey is shown in Appendix A. Four computer programs were written to assist in the analysis of the returned questionnaires.

The first computer program is for calculating the energy used at the plants. The inputs to this program come from the answers to question number seven on the questionnaire. The total energy used at the plants was calculated in British thermal units (Btu's). The conversions used for this calculation were the following.

TYPE	CONVERSION
Electricity	10,500 Btu's per kWh
Oil	140,000 Btu's per gallon
Natural Gas	1,030 Btu's per ft^3
Digester Gas	600 Btu's per ft^3

It should be noted that the electrical energy used is converted to the amount of thermal energy required to generate the electricity.

After determining the total annual energy used, the average power used in kW is calculated by dividing by 9.2×10^7 ($10{,}500 \frac{\text{Btu}}{\text{kWhr}} \times 24 \frac{\text{hr}}{\text{day}} \times 365$ days). This power is used to calculate an equivalent hydraulic head required for a theoretical hydroelectric plant which would generate this power using the average water flow received by the wastewater plant. The basic equation for hydroelectric power is:

$$\text{Power} = \eta \times \text{HEAD} \times \text{AFLOW} \div 11.8 \quad [\text{kW}] \qquad (1)$$

where η is efficiency of conversion

HEAD is hydraulic head in feet

AFLOW is the average flow in $ft^3/sec.$

If AFLOW is initially in MGD (million gallons per day) it can be converted to cubic feet per second by multiplying by 1.547. The equivalent HEAD required is thus:

$$HEAD = Power \times 9.5 \div AFLOW \quad [ft] \qquad (2)$$

assuming the efficiency is 80% ($9.5 = 11.8 \div 0.8 \times 1.547$). This equivalent HEAD can be easily adjusted for actual plant specific differences in level of influent versus level of effluent so the energy use at different plants can be compared on a more common basis: the case where the level of influent at the head of the plant equals the level of effluent in the last holding tank of the plant.

The energy use per million gallons (MG) of influent was calculated from the total energy used annually divided by the total annual influent processed (365 x average daily flow). The energy use per MG is important in comparing plants with one another.

The second computer program is for identifying the potential energy output. The inputs to this program are the amount of digester gas, and grease and oils recovered by the wastewater plant annually, the potential hydroelectric hydraulic head available at the plant (difference in elevation level of effluent in the last holding tank in the plant and the elevation level of the water body for final discharge), average daily flow and the fraction of flow available for hydroelectric production.

The energy potential from digester gas and grease and oils is assumed to be 600 Btu/ft^3 and 18,000 Btu/lb respectively. The hydroelectric potential is calculated using equation (1) assuming the efficiency of hydroelectric conversion is 80%.

The energy recovery potential from sludge, grit, and screenings was not determined because of the wide range of several variables. Also, incineration of sludge typically requires auxiliary fuel and is a net user of energy rather than a source of energy. The variables which determine the energy recovery potential or the need for auxiliary fuel to incinerate sludge, grit and screenings include (Ref. 1):

1. Water content
2. Percentage of volatile matter in the dry solids
3. Heat content of the volatile matter
4. Flue gas exit temperature
5. Radiation and convective losses from the incinerator or furnace
6. Amount of excess air supplied for combustion
7. Heat loss in the ash
8. Thermal cycling requirement

The total energy recovery potential per MG of influent was calculated by first summing the annual energy recovery potential from digester gas, grease and oils and hydroelectric development, and then dividing by the annual amount of influent processed. This number was then compared to the total energy use per MG processed.

Table 1 is a summary of the energy use and potential energy recovery as analyzed by these two computer programs for the nine wastewater treatment plants which responded to the questionnaire sent to them.

The third computer program is for identifying the hydroelectric potential energy output and its estimated costs per kWh. This program is a modification of a previously developed program for evaluating the hydroelectric energy supply for existing dams in the United States. The computer program and analysis method are discussed in a report by Taylor and Green (Ref. 2).

This program has been applied to data for the Back River, Chicago and Detroit wastewater treatment plants. The results from this application are given in Section 3, Analysis of Preliminary Results.

The fourth computer program written for analysis of energy use and recovery at wastewater treatment plants is for determining the potential heat recovery via heat pumps from the effluent. The coefficient of performance, COP, for a heat pump is defined as the total thermal energy output from the heat pump divided by the energy used to operate it. A previous study of heat pumps by Powell (Ref. 3) found that the COP can be approximated by the following formula:

$$COP = 0.4 \times \frac{T_o}{T_o-(T_E-\Delta T)} \qquad (3)$$

where T_o is the temperature output from the heat pump in degrees Rankine

T_E is the temperature of the effluent in degrees Rankine

ΔT is the temperature differential between the effluent and the working fluid of the heat pump (approximately 10°F).

For normal heating purposes, T_o is assumed to be 120°F, or 580° Rankine.

The heat extracted from the effluent is assumed to be five Btu's per pound (approximately a five degree fahrenheit temperature drop). This results in 41.7 x 10^6 Btu/MG (five Btu/lb x 8.34 lb/gal x 1,000,000 gallon) thermal extraction from the effluent.

The thermal energy output from a heat pump is the heat extracted from the effluent (HEAT = 41,700,000 x AFLOW x DAYS) plus the work done (WORK) by the heat pump to extract the HEAT. Thus the coefficient of performance, COP, for the heat pump is defined as the following:

$$COP = (HEAT + WORK) \div WORK \qquad (4)$$

The WORK necessary for heat extraction can thus be calculated as the following:

$$WORK = HEAT \div (COP - 1) \qquad (5)$$

TABLE 1. Energy Use and Potential Recovery at Nine Wastewater Treatment Plants

	Joint Water Pollution Center	Back River WWTP	Little Patuxent WWTP	Cox Creek WWTP	Hagerstown WWTP	Parkway WWTP	Patuxent WWTP	Ocean City WWTP	Broadneck WWTP
ENERGY USE									
Flow [MGD]	345	177	9	8	6.5	4.9	3.7	2.8	1.7
Electrical Energy [Btu]	4.01×10^{11}	2.06×10^{11}	6.37×10^{10}	3.60×10^{10}	3.71×10^{10}	4.60×10^{10}	1.84×10^{10}	1.24×10^{10}	1.65×10^{10}
Total Energy [Btu]	9.45×10^{11}	2.94×10^{11}	6.37×10^{10}	4.23×10^{10}	4.25×10^{10}	4.60×10^{10}	1.98×10^{10}	1.50×10^{10}	2.02×10^{10}
Required Hydraulic Head [ft]	284	172	733	548	681	967	556	562	1233
Energy Use per MG [Btu/mG]	7.51×10^{6}	4.55×10^{6}	19.38×10^{6}	14.49×10^{6}	18.01×10^{6}	25.5×10^{6}	14.69×10^{6}	14.86×10^{6}	32.6×10^{6}
ENERGY RECOVERY									
Digester Gas[Btu]	1.19×10^{12}	2.21×10^{11}	No Digesters	2.64×10^{9}	Meters Not Working	No Digesters	2.89×10^{9}	No Digesters	Meters Not Working
Oils & Grease [Btu]	0	0	0	0	0	1.67×10^{8}	1.08×10^{9}	0	Unknown
Hydro-electric [Btu]	0	1.55×10^{10}	8.69×10^{8}	9.5×10^{8}	5.8×10^{8}	1.43×10^{8}	1.18×10^{8}	6.66×10^{7}	1.70×10^{8}
Total Energy [Btu]	1.19×10^{12}	2.36×10^{11}	8.69×10^{8}	3.59×10^{9}	Unknown	3.1×10^{8}	4.09×10^{9}	6.66×10^{7}	Unknown
Energy per MG [But/MG]	9.48×10^{6}	3.66×10^{6}	0.265×10^{6}	1.23×10^{6}	Unknown	0.17×10^{6}	3.03×10^{6}	0.066×10^{6}	Unknown
Digester Gas [ft^3/MG]	15.8×10^{3}	5.7×10^{3}	0	1.5×10^{3}	Unknown	0	3.6×10^{3}	0	Unknown
Sludge [tons/MG]	0.276	0.395	0.871	2.334	0.230	2.02	1.01	0.372	0.392
Total Solids [tons/MG]	0.276	0.418	0.874	2.353	0.230	2.10	1.04	0.481	0.582
Potential Energy Recovery Energy Use	1.259	0.803	0.0136	0.0849	Unknown	0.0067	0.207	0.00444	Unknown

The thermal energy, (THERMAL), required to generate the electrical energy, WORK, can be approximated by dividing WORK by the thermal-electric conversion efficiency (3413 Btu's electrical per 10,500 Btu's thermal). Thus the THERMAL energy required to generate the electrical energy WORK is:

$$\text{THERMAL} = 3.08 \times \text{WORK} \qquad (6)$$

The total thermal energy output from the heat pump can be approximated by the following equation (using equation 5).

$$\text{WORK} + \text{HEAT} = \text{HEAT} \times \text{COP} \div (\text{COP} - 1) \qquad (7)$$

The net thermal energy output from the effluent is the difference between the thermal energy output (7) and the thermal energy (6) used to generate the electrical energy, WORK.

3. ANALYSIS OF PRELIMINARY RESULTS

Energy Use

The total energy use at the plants investigated varied from 4.55×10^6 British thermal units per million gallons (Btu/MG) to 32.6×10^6 Btu/MG, while the average flow at each plant varied from 345 MGD to 1.7 MGD. Thus the energy per MG varied by about a factor of 10 while the average flow at each plant varied by approximately a factor of 100. This implies that the energy use per MG is approximately proportional to the inverse square root of the average flow, or that the total power required is approximately proportional to the square root of the average flow.

Since the electrical energy is mainly used for pumping water and water handling, it is important to determine the relationship between electrical power requirements and average daily flow. The theoretical hydraulic head required for electrical energy required can be determined by multiplying the electrical energy use divided by the total energy use times the total required hydraulic head given in Table 1. This hydraulic head should be adjusted so that plant specific variations in the level of the wastewater at the head of the plant and the level of the effluent at the last holding tank are set to zero. The net electric hydraulic heads shown in Figure 1 are for the plants as if they had no differences between the level of influent and level of effluent. The fitted curve in Figure 1 gives the following relationship for the average electrical power required at a wastewater treatment plant versus the average flow at the plant:

$$\text{Electrical Power} = 100 \times \text{AFLOW}^{0.6} \quad [\text{kW}] \qquad (8)$$

Some of the plants' average electrical power use deviated by as much as 40% from the results given in equation (8). Part of this variation may be due to the different types of wastewater treatment such as trickling filter, activated sludge, and filtration. In the proposed questionnaire for the national survey, the type of treatment at each plant will be determined.

The importance of determining relationships such as given in equation (8) for the different types of treatment is that it allows identification of plants which have unusual electrical energy consumption. Plants with abnormal electrical energy use can then be investigated further.

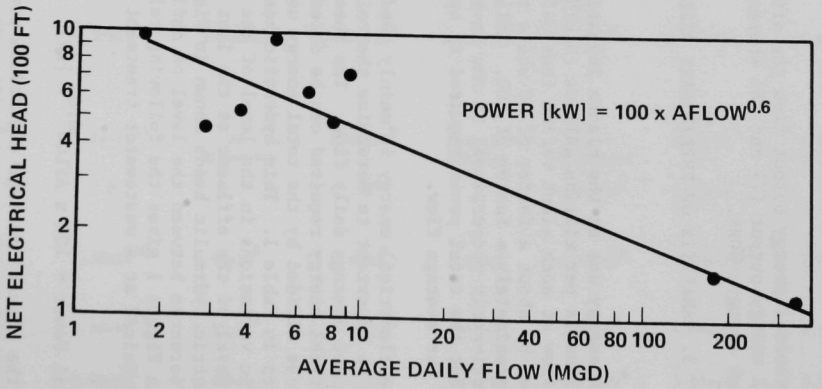

Fig. 1 Net electrical head versus average daily flow for different plants

It should be noted that the two plants which are significantly higher than the fitted curve in Figure 1 are the Little Patuxent WWTP and the Parkway WWTP which are activated sludge plants which do not have anaerobic digesters. The only other plant without anaerobic digesters is the Ocean City WWTP, but that plant uses clarigesters instead of activated sludge. Its electrical energy use is significantly below the estimate using equation (8).

Another important area to be considered during the National Survey is the amount of energy required for heating anaerobic digesters. The importance of this area was discovered while analyzing the estimated energy use for the new Back River Wastewater Treatment Plant. A consulting engineering firm had used EPA's publication 430/9-77-011, entitled Energy Conservation in Municipal Wastewater Treatment, (Ref. 4) to estimate the energy required for anaerobic digestion. The answer was over three times the rate of digester energy use at the present plant.

Energy Recovery

The major potential independent source of energy recovery from wastewater treatment plants is digester gas from anaerobic digesters. Unfortunately, three of the nine plants surveyed did not have digesters. Also, two of the remaining six plants did not have working gas meters for their digesters, therefore only four data points exist upon which to make some preliminary findings. Of these four plants with digester gas production information, the digester gas constituted 70% to 100% of the total energy recovery potential of each plant. The potential independent energy recovery per MG from the three plants without anaerobic digesters was from 0.01 to 0.1 of the potential energy per MG for facilities with digesters.

A plot of the total potential independent energy recovery per MG versus average flow is given in Figure 2. Although there may be a relationship between potential energy recovery per MG and the average plant flow, certainly four data points are insufficient to even reasonably guess such a relationship when the scatter is as large as shown in Figure 2. For our preliminary results, it is probably best to assume the energy recovery per MG is independent of the flow and just use the average, 4.3×10^6 Btu/MG. This translates into about 17 kW of recoverable electric power/MGD for potential independent energy recovery from wastewater treatment plants. From Table 1 we find the total power required at a plant is approximately 1.5 times equation (8).

Using these two approximations, it appears that a 230 MGD plant would require about the same amount of energy as is potentially recoverable. Above this size more energy is generally recoverable than is required, while below this size, the opposite is generally true. This is consistent with the data shown on the last row of Table 1. The national survey will refine these estimates and more precisely determine the variation of these results with different types of wastewater treatment.

The potential for hydroelectric development was analyzed for three plants, two of which are the largest plants in the U. S., Chicago and Detroit wastewater treatment plants.

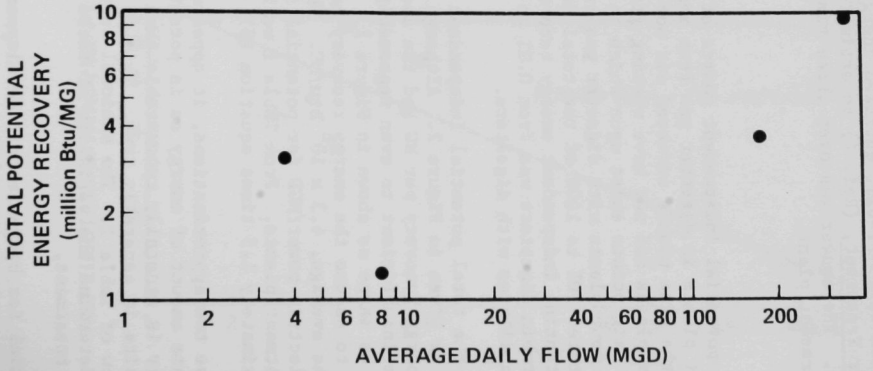

Fig. 2 The total potential independent energy recovery per MG versus average flow for four different plants with anaerobic digesters.

The principal data inputs are the following:

Plant	Hydraulic Head	Average Flow
Chicago	15.5 ft	900 MGD
Detroit	17 ft	700 MGD
Back River	20.8 ft	77 MGD*

* 100 MGD at Back River is sent to Bethlehem Steel and is not available for hydroelectric development.

For an annual fixed charge rate of 10.4% some of the model outputs are the following:

Plant	Potential Capacity [kW]	Annual Energy [kWh]	Capital Cost [$1978]	Cost/kWh [¢/kWh]
Chicago	1400	10.8 million	2.5 million	3.4
Detroit	1200	9.2 million	2.1 million	3.3
Back River	170	1.3 million	0.4 million	4.4

A dependent energy source at wastewater treatment plants is heat in the effluent. This energy is called dependent because it typically requires electrical energy via the use of a heat pump in order to recover it. The amount of energy which could be extracted via heat pumps from the effluent and used for heating, HOUT, between October and March (six months) was calculated for six of the plants which responded with effluent temperatures. The ratio of potential HOUT to the total annual energy actually used at each plant varied between 0.9 and 6.7. To obtain the potential HOUT, electrical energy has to be expended. To generate this electrical energy, thermal energy, THERMAL, has to be used. The ratio of HOUT-THERMAL to the annual energy used at each plant varied from 0.056 to 1.2. The ratio of 1.2 was for the Joint Water Pollution Control Plant in Los Angeles. The remaining five plants were in Maryland. The highest value of this ratio was 0.21 for these plants.

4. CONCLUSIONS

Although no reliable conclusions can be drawn from this limited survey it does provide the basic analysis approach for the National Survey. It also indicates the relationships between energy use and energy recovery as a function of average wastewater flow. These relationships will be more reliably determined from the national survey of approximately 500 wastewater treatment plants.

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APPENDIX A

QUESTIONNAIRE
NATIONAL SURVEY OF WASTEWATER TREATMENT PLANTS
U.S. Department of Energy
Urban Waste and Municipal Systems Branch
Energy Use and Potential Energy Recovery

Plant Name: _____

Mailing Address: _____

_____
City State Zip Code

Telephone Number: _____
Area Code

Prepared by: _____
Date

1. Average Daily Influent _____ MGD.* Range of Influent _____ MGD to _____ MGD.

2. What is the typical flow of influent on a two hour basis? Please give the typical flows in MGD.
   2am _____ MGD, 4am _____ MGD, 6am _____ MGD, 8am _____ MGD, 10am _____ MGD, 12noon _____ MGD
   2pm _____ MGD, 4pm _____ MGD, 6pm _____ MGD, 8pm _____ MGD, 10pm _____ MGD, 12midnt _____ MGD

3. What is the elevation of the following:
   a. Wastewater influent pipe at head of plant _____ msl° (ft)
   b. Last holding tank in plant _____ msl (ft)
   c. Elevation of water body for final discharge _____ msl (ft)

4. What is your best estimate of the amount of solids such as paper, rags, wood, plastics, grit, etc., which were screened out of the wastewater during 1978 _____ tons? What percent of this material is non-combustible _____ %?

5. What is your best estimate of the amount of grease and oils collected by your plant during 1978 _____ lbs?

6. What is the average temperature of your effluent on a monthly basis? (°F)
   Jan. ____, Feb. ____, Mar ____, Apr. ____, May ____, June ____, July ____
   Aug. ____, Sept. ____, Oct. ____, Nov. ____, Dec. ____

7. What was your annual energy consumption during 1978?

	Electricity kWh	Oil Gallons	Natural Gas Cubic Feet	Digester Gas Cubic Feet	Other Specify
Units					
Dollars					

* MGD is million gallons per day
° msl is mean sea level in feet

Call Robert J. Taylor (301-953-7100, Ext. 2339) for assistance with this questionnaire

COLUMBUS, OHIO--AN EXAMPLE OF SMALL HYDROPOWER POTENTIAL IN THE MIDWEST

David Pritchard
Burgess & Niple, Limited

I am a Civil Engineer specializing in water resources, planning, and design at the Columbus, Ohio consulting firm of Burgess & Niple, Limited. We have recently completed a detailed feasibility assessment for two sites at the existing Griggs and O'Shaughnessy Dams[1], operated by the Columbus Division of Water and located on the Scioto River in Columbus, Ohio. We have also conducted preliminary feasibility reviews for several other sites in the state, and have reviewed the preliminary results of ongoing studies by the Army Corps of Engineers. Our involvement with these projects and related review of other such projects have convinced us that small scale, low-head hydroelectric power development[2] is a feasible and worthwhile source of energy for municipalities in todays market.

I would like to consider the topic of hydroelectric power development according to the following outline:

1. General characteristics and history.
2. Potential in the midwest.
3. Applicable regulations.
4. Examples--Sites at Columbus.
5. Special implications of using water supply dams.
6. Incentive programs.
7. Future needs.

General Characteristics and History

At present, hydroelectric generating capacity in the United States is about 15 percent of total generating capacity, or 70,000 megawatts. Studies[3] indicate that an additional 55,000 megawatts can be developed at existing dams throughout the United States, generating 160,000 million kilowatt-hours per year, the equivalent of 266 million barrels of oil annually. This is a greater amount than solar energy contributions generally targeted for the year 2000. However, development of the hydropower resource is feasible with existing technology and within a shorter time frame. These are technically developable sites; their economic feasibility must be shown within the existing and expected energy market before a decision on implementation can be made.

The use of hydroelectric power in preference to thermal power, especially gas, oil, or coal fired thermal involves the substitution of capital, and engineering technology, for depletable resources. The physical variables are the height of the dam (head) and the flow of water (Figure 1). The advantage of this substitution, especially during a period of time when fuel costs are inflating rapidly, is that the majority of costs (typically over 90 percent) are locked in at the time of development in the form of capital expended and interest rates applicable to the bonds or other forms of financing utilized. A comparison of these capital costs, plus relatively minor operating costs, to the capital and fuel costs for a thermal plant, for a life cycle period of 30 to 50 years provides the basis for evaluating a project; Figure 2 illustrates this advantage.

Hydraulic turbines were first used to drive an electric generator at Appleton, Wisconsin in 1882, and began to find wide use after the value of such projects was demonstrated by the Niagara Falls project about 1895. Development of small sites was a major source of new power until about 1915 when steam plants became generally more economical. Some development of smaller sites continued through the 1930s, but most hydropower development since the 1920s has been at larger sites in the mountainous areas of the United States. There were historically 12 sites in Ohio, a state with modest resources, with a maximum generating capacity of 15-20 megawatts.

The only remaining operating hydroelectric power site in Ohio, on the Great Miami River at Hamilton, produces a maximum of 1.5 megawatts (Figure 3). It was built in 1919, and is now operated by the City of Hamilton as a part of their municipal system. An operating plant of a newer design is located on the Ohio River at Markland Dam, about 50 miles downstream from Cincinnati. It is a 56 megawatt facility, built in 1967 (Figure 4).

Regional Hydroelectric Power Development Potential

Looking at the State of Ohio in some detail, hydroelectric power development is being actively considered, or developed at five sites at various locations within Ohio and at four sites on the Ohio River with a capacity of about 14 megawatts in Ohio and 236 megawatts on the Ohio River.

The national potential for hydroelectric development is the topic of a current Corps of Engineers nationwide study, by state. The Ohio potential is relatively small in proportion to the United States total, on the order of 1 percent, however, the value of energy produced and the dollar cost of construction are substantial.

From the preliminary results, of which have recently been made public, I have excerpted the following information on existing dams in Ohio, excluding those sites which generate less than 100,000 kilowatt-hours per year (0.1 megawatt per year).

Site Ownership	Number of Sites	Total Feasible Capacity, mw	Total Annual Output, Million kw-hr
Corps of Engineers	24	61.2	134
State of Ohio	17	185.8	471
Miami Conservancy District	5	13.3	45
Cities and Counties	14	40.6	75
Other	15	14.0	25
Total in Ohio	75	314.9	750

The total energy production noted in the table would have a value of about $22.5 million per year (at 30 mills per kilowatt-hour), and could involve construction costs in the range of $300 million (at $1,000 per kilowatt-hour). The total of the potential energy production, about 750 million kilowatt-hour per year, amounts to about 1 percent of annual production in Ohio.

Throughout the midwest, preliminary Corps estimates are as listed below:

	Less Than 15 Megawatts		Over 15 Megawatts	
	Present Capacity	Potential Additional	Present Capacity	Potential Additional
West Virginia	46	18	102	2,952
Kentucky	0	64	636	9,207
Indiana	28	58	0	37
Illinois	100	52	32	678
Michigan	283	303	203	830
Ohio	2	105	0	209
Wisconsin	220	219	210	492
Total	679	819	1,183	14,405

Applicable Regulations

The Federal Energy Regulatory Commission (FERC) has control of hydropower development on all streams of significant size in the United States. Its authority in some areas supercedes state authority, but it does, through the review and assessment process, include the needs expressed by state and local agencies. State permits may also be required. The aims of the FERC licensing procedures are to assure that potential sites are made available (an entity can obtain a license on a dam owned by another entity), and to assure that development is economically efficient and environmentally sound. The Commission can also assist the municipality in gaining rights to transmit generated electricity from a site to a point of use via utility lines.

Columbus, Ohio Hydroelectric Study Sites

The two sites on the Scioto River near Columbus provide examples of potential hydroelectric power development in the midwest. Feasibility studies were largely funded by the U.S. Department of Energy, with partial funding by the Ohio Department of Energy and the City of Columbus. The study addressed the technical, environmental, economic, and institutional feasibility of the projects. Hydroelectric power development was found to be feasible in all four categories for both of the sites.

Both sites had originally been constructed with plugged conduits through the dam with the expectation of adding turbines at a later date. Several studies had in fact been made, but economic viability was poor due to the low cost of alternative fuels.

O'Shaughnessy Dam is about 65 feet high, controlling a drainage area of 980 square miles; Griggs Dam, located 9 miles downstream is about 35 feet high, and controls 1,044 square miles. A 15' x 19' conduit was provided through the west end of O'Shaughnessy Dam, built in 1925 (Figure 6). Griggs Dam, built in 1905, was provided with a 9 foot diameter conduit also in the west end.

Site development plans would call for modification to the existing intake structure at O'Shaughnessy Dam, and construction of a penstock and powerhouse which would be approximately 30 by 55 feet (Figure 7), and would be relatively small in proportion to the dam in both cases.

In typical annual operation the two sites would produce over 21 million kilowatt-hours of energy, at an average rate of 2.4 megawatts, and a maximum rate of 7.2 megawatts, the capacity of proposed equipment. The table below gives the cost breakdown of projects, $7.7 million for the entire project.

PRELIMINARY PROJECT COST ESTIMATES

	O'Shaughnessy	Griggs	Total
Turbine, Generator and Electrical Equipment	$1,975,000	$1,814,000	$3,789,000
Civil Works	1,244,000	1,036,000	2,280,000
Electrical Transmission	30,000	37,000	67,000
Total Estimated Construction Cost	$3,249,000	$2,887,000	$6,136,000
Add for Interest During Construction & Design	831,000	738,000	1,569,000
Total Project Cost	$4,080,000	$3,625,000	$7,705,000

In terms of financial feasibility, the following table shows that both sites are feasible individually.

FINANCIAL ANALYSIS - SUMMARY OF RESULTS*

	Capacity	Project Cost $million	Unit Cost $/kw	Net Gain(1) (PV) $million	ROI(2)	BCR(3)
O'Shaughnessy Dam	5 Mw	4.080	800	11.16	15%	2.31
Griggs Dam	2 Mw	3.625	1,800	2.38	8%	1.37
Combined	7 Mw	7.705	1,100	13.54	12%	1.91

*Based on future cost and value projections of Figure 7

(1)PV - The sum of the present values of annual income

(2)Return on investment - Assumes inflation rate of approximately 6 percent

(3)Benefit cost ratio

Because the O'Shaughnessy site has a higher dam height, it can produce much more energy for approximately the same development cost as the Griggs site. The financial parameters, net gain, return on investment, and benefit cost ratio are therefore much more favorable. In terms of environmental, recreational, and water supply uses, a program of development and operation has been designed to allow these uses to continue relatively undisturbed.

The completed feasibility study, has allowed the city to compete for partial funding of project development in the U.S. Deparment of Energy demonstration grant program.

Special Implications of Using Water Supply Dams

Generally all dams built primarily for water supply are or will be used to their full water supply reliable yield. Consequently, during dry periods, the maximum possible amount of inflow is stored in the reservoir and the total outflow which is drawn from storage is limited to the raw water supply to the treatment plant plus the minimum flow required downstream for environmental reasons or for riparian rights.

The mode of transmission of this water is important. Raw water may be conveyed by releasing the water from the dam into the streambed where it flows by gravity to an intake pool (usually formed by a low head dam) near the water plant and is then pumped into the water plant. Raw water may also be conveyed to the water plant through a pipeline extending from the dam to the water plant. Griggs and O'Shaughnessy Dams use the streambed to convey the water to the water plant.

Figure 8 illustrates two extremes in outflow-duration curves that might exist at equal height dams with identical inflow-duration curves. The curve on the left is for a dam providing little storage relative to the streamflow. This dam affects the outflow-duration curve significantly only during periods of extreme low flow where water is drawn for storage to meet the raw water demands. The curve on the right is for a dam of large storage relative to the streamflow which highly regulates the outflow.

The raw water flow-duration curves for these dams are also shown on Figure 8 assuming the full water supply potential is used. The reliable yield for each dam is shown as well. The reliable yield for the dam with large storage is greater and the raw water flow-duration curve is higher than for the dam with small storage.

If the raw water were conveyed to the water plant via the streambed, the annual energy potential at these two hypothetical dams would be the same, but more dependable power could be produced at the dam with the greater storage. If, however, the raw water supply is conveyed by a pipeline to the water plant, and if a power plant were to use only the flow discharged into the stream below the dam (flow in excess of the raw water needs) the dam with small storage would have the greater hydroelectric potential.

Using a raw water pipeline as a penstock, the gross head would be the difference in elevation between the pool above the dam and the hydraulic gradient at the water plant. If the hydropower development was not considered in the design of the raw water pipeline, it is likely that the maximum discharge would be limited to about the maximum anticipated raw water demand due to friction losses in the system. In such a case, the dam with the large storage has the greater hydropower potential. If, however, a penstock/raw water line and inlet structure were designed considering hydropower development, it would be possible to pass much more than the raw water supply through the penstock/raw water line for greater power production by dumping the flow in excess of raw water needs back into the river at the hydroplant. At Griggs and O'Shaughnessy Dams the water supply outlets are too small for significant hydropower development.

The ability of electric energy source to meet peak demands (peaking) is important. Peaking herein will refer to a daily variation of the water release through the turbine resulting in a variation in the power produced. Peaking increases the annual energy produced and reduces the owner's demand charge for purchased supplemental energy. During dry periods, the annual energy production can be increased by releasing the daily raw water demand in a few hours at a rate equal to or greater than the turbine's minimum required discharge. This permits using a turbine of greater capacity while still utilizing the low flow for power production. Peaking increased the estimated potential typical year energy production at the dams by about 18 percent, and allowed peak reductions of 7 megawatts in winter and 3 megawatts in summer (Figure 9).

The possible sources of peaking at water supply dams are as follows:

1. Flow in excess of raw water demand.
2. Variations in finished water demand.
3. Clear well capacity at water plant.
4. Storage in stream, intermediate reservoirs, or intake pools.

The economic feasibility of hydropower development is enhanced if the owner of the hydropower facility uses the power produced because the worth of the power can be compared with a retail electric rate, because an owner-user could better coordinate power production with power use to reduce the demand charge, and may face lower transmission distances and related costs. Most water supply dams are municipally owned and most municipal governments use more energy than could be produced at their water supply dams. The City of Columbus could use all the power that could be produced at their three dams; they use or retail over 300 million kilowatt-hours per year. The water plants use about 91 million kilowatt-hours per year whereas the hydropower potential is about 26 million kilowatt-hours per year for three sites.

Economic feasibility of city-owned water supply dams is favorably affected by the low interest rates, often 6 to 8 percent, that cities enjoy due to their tax free bonds. This is particularly an advantage at a time when the prime interest rate is about 13 percent.

Hydroelectric power development for communities much smaller than Columbus may have a wider range of applicability because of numerous existing dams in the smaller size range.

For a small community, say 10,000 population, with combined water and sewer plant electric needs of about 300 kilowatts, an economical site development program might be as follows:

Dam Height	20-30 feet
Drainage Area	200 square miles
Design Flow	200 cubic feet/second
Plant Factor	50%
Capacity-peak	300 kilowatts
Output-average	150 kilowatts
Value at 30 mills	$40,000/year

Based on life cycle costing, this could support an investment of $800,000 or about $2,500 per kilowatt capacity by the municipality. Many sites in the midwest can be developed at this cost and in larger capacities and higher dams, at 1/2 to 1/3 this cost.

Development of Incentive Programs

The U.S. Department of Energy has initiated a program for promoting hydroelectric power generation where appropriate until with a potential for bringing substantial new hydroelectric power on-line in the next decade. The program has involved grants and loans for feasibility studies and regulatory filings, partial funding of construction for demonstration projects, and research in hydropower technology and economics. Another program of low interest loans for project construction is authorized but funding has not yet been provided.

To develop hydropower at an existing dam, loans for feasibility studies and license applications can be of value as they are offered for 90 percent of study and application costs at an annual interest rate of 7 1/8 percent, for up to $50,000 per site. The loans may be canceled by the U.S. Department of Energy if the site is found not to be technically or economically feasible for development. To qualify, a developer must show via a feasibility study that it has reasonable assurance of use of the site by the owner, that development is in the range of technical and economic feasibility and that the organization and its consultants are able to carry out the proposed study, design, financing, construction (contracted), and operation of the project in an economical and timely manner.

Important factors in the feasibility study are site data and performance characteristics, potential for use of generated power, capital investment and return on investment, preliminary environmental and social impact evaluation, and a technical feasibility review.

Also, private bond placement organizations are actively interested in placing bonds for constructing such projects under appropriate conditions.

Future Needs

In summary it appears that small scale development of hydroelectric power will increase substantially in the United States in the next few years because of the higher costs of other forms of energy and because of promotional and incentive efforts by the U.S. Department of Energy, manufacturers selling equipment, consultants selling engineering services, and other believers in the concept. Questions remaining are:

1. What are appropriate means of accelerating development of feasible sites?

2. What are appropriate means of improving the economy of hydroelectric power site development?

Essential areas of action are:

1. Stepped-up information programs for cities, public utilities, and electric cooperatives.

2. Increased manufacturing capacity and improved technology in the United States.

3. Reduced licensing requirements or elimination of licensing under certain conditions.

4. Interest subsidies where appropriate.

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[1]Study funded by U.S. Department of Energy and performed cooperatively with the City of Columbus and the Ohio Department of Energy.

[2]Small scale hydroelectric development in the midwest generally address sites producing less than 25 megawatts of power, and with low heads less than 65 feet. Also, the discussion is limited to sites at existing dams.

[3]R. J. McDonald et. al. "Estimate of Hydroelectric Power Potential at Existing Dams," Army Corps of Engineers, Institute of Water Resources Ft. Belvoir, Virginia (July 20, 1977).

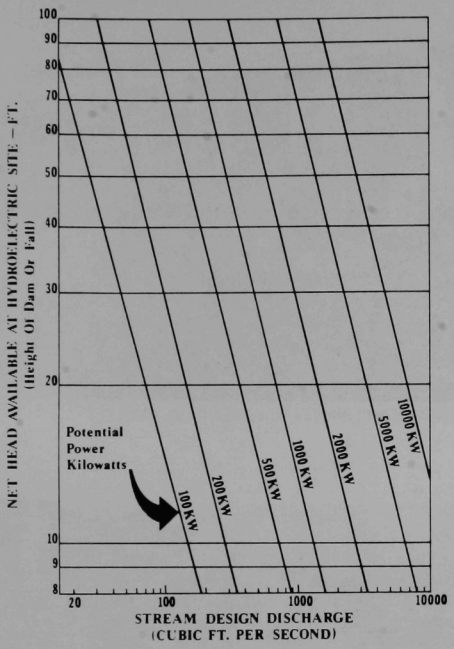


FIGURE 1 HYDROELECTRIC SITE POWER POTENTIAL

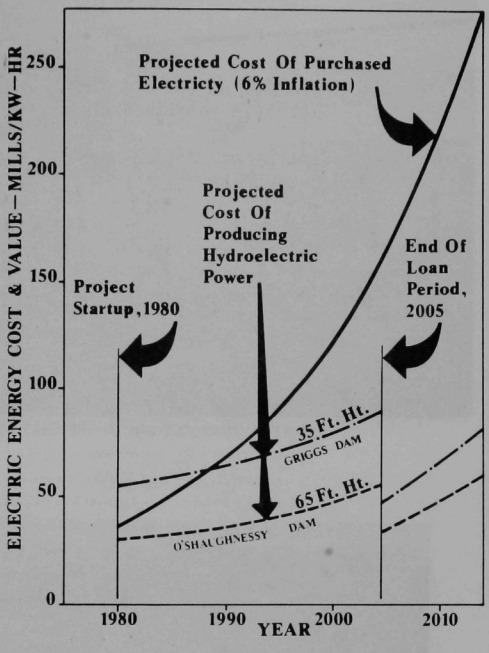


FIGURE 2 HYDROELECTRIC SITE ECONOMICS LIFE CYCLE COST BASIS- 35 YEARS

FIGURE 3 EXISTING HYDROELECTRIC PLANT AT HAMILTON, OHIO - BUILT IN 1919 MAXIMUM OUTPUT AT PRESENT 1.5 MEGAWATTS

FIGURE 4 MARKLAND DAM ON THE OHIO RIVER NEAR CINCINNATI - POWERPLANT ON FAR LEFT

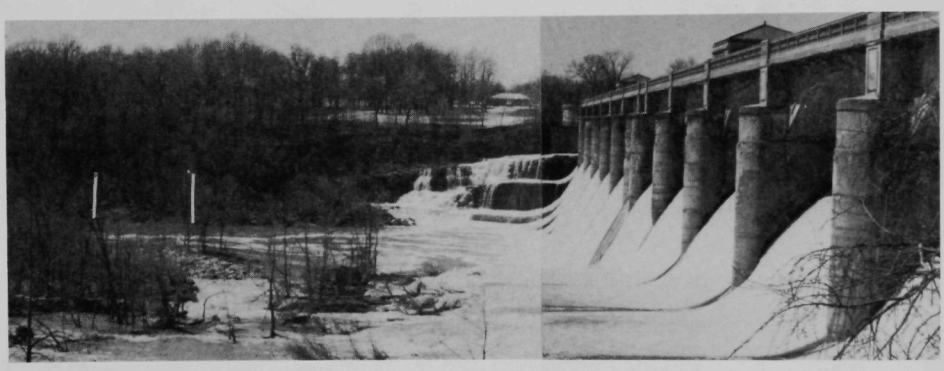


FIGURE 6 O'SHAUGHNESSY DAM ON THE SCIOTO RIVER NEAR COLUMBUS - PROPOSED POWERPLANT SITE ON LEFT

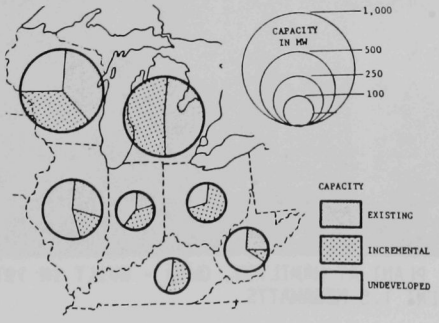
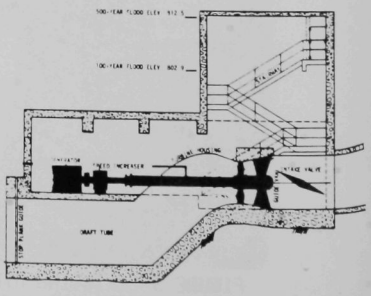


FIGURE 5 MIDWEST HYDROELECTRIC POTENTIAL AFTER CORPS OF ENGINEERS -1979



SECTION

PROPOSED POWER PLANT

FIGURE 7 O'SHAUGHNESSY DAM DETAIL

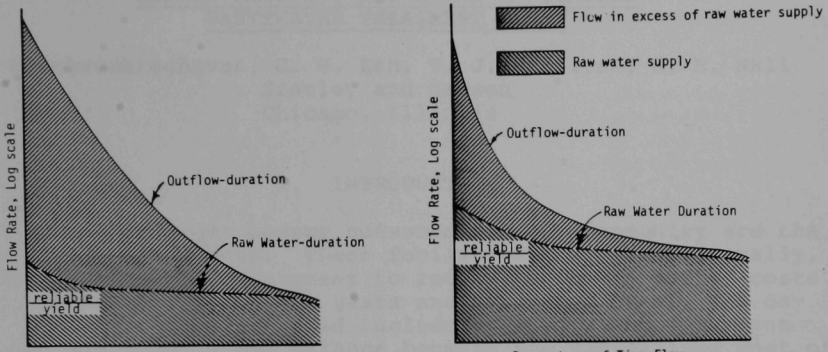


FIGURE 8
Schematic Flow-Duration Curves

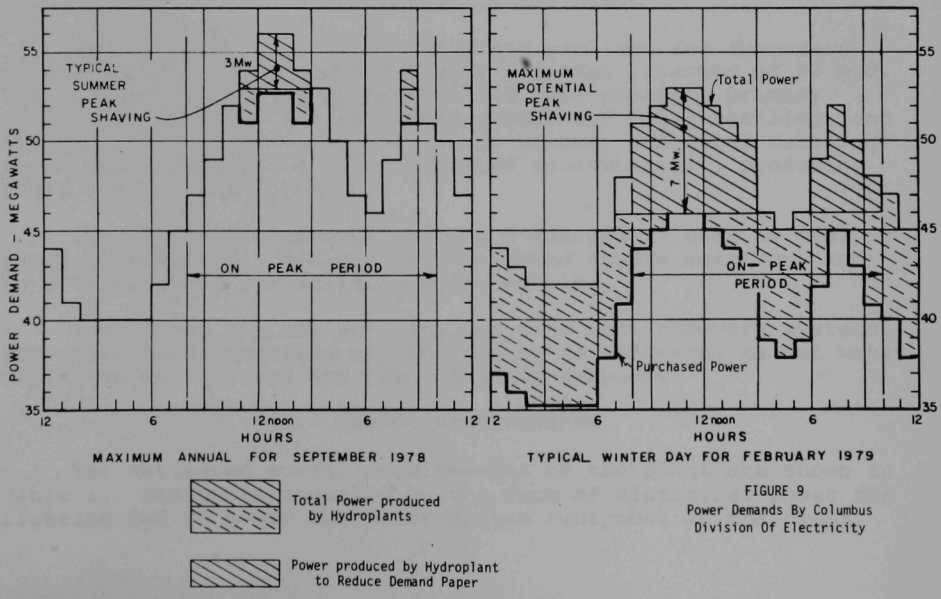


FIGURE 9
Power Demands By Columbus Division Of Electricity

ENERGY RECOVERY AND REUSE AT YONKERS WASTEWATER TREATMENT PLANT

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Greeley and Hansen
Chicago, Illinois

1. INTRODUCTION

Among the larger energy consumers in the community are the sewage treatment plants. These facilities have, traditionally, included systems and equipment to reduce purchased energy costs by making use of by-product gases and recovered heat. The degree to which designers could include energy saving equipment has been governed by the balance between the capitalized cost of the equipment and the annual costs of purchased energy.

The Joint Treatment Plant located along the Hudson River at Yonkers, New York represents a case history of a construction program wherein the owner of the facility, the Westchester County, New York Department of Environmental Facilities, has invested in large horsepower, dual fuel reciprocating engines to use by-product methane gas to offset the annual costs for purchased energy. The treatment process is typical of many in the northeast; however, the case history shows the trend in design is towards greater utilization of by-product gas and bettering overall efficiencies of equipment.

2. DESCRIPTION OF THE PLANT

The Yonkers Joint Treatment Plant provides for secondary treatment of sewage quantities with a design capacity of 92 mgd. The treatment includes screenings and grit removal, primary settling, step-aeration activated sludge and final settling, and disinfection. Solids removed in the primary and final settling tanks are thickened and then digested anaerobically, producing sludge gas as a by-product.

The sludge gas produced in the digesters is used as fuel to power engine-driven blowers for activated sludge aeration, and to produce steam for building space heating.

The blower engines are equipped with heat recovery systems. The waste heat generated by the engines is recovered as hot water and steam and is used for plant heating purposes.

3. ENERGY REQUIREMENTS

The estimated energy requirements of the plant are shown in Table 1. Energy is required in the form of electrical power for lighting and to power the motor-driven equipment in the plant,

and as fuel to power the engine-driven process air blowers and for building space and sludge heating.

The connected electrical loads shown for lighting and motors are the totals of rated capacities of the equipment installed. The estimated operating loads are based on an analysis of the frequency and duration of the operation of the units and an estimate of the power draw in kw or horsepower.

The fuel requirements shown for the process air engine-driven blowers are based on average process air consumption, and may vary depending on the method of operation. The blowers are driven by diesel engines with a fuel consumption of 7,000 Btu per horsepower-hour. Each blower requires about 1,350 brake horsepower to operate at average air conditions.

Energy requirements for heating and air conditioning are shown for winter and summer conditions. Winter space and sludge heating requirements are shown for the 40-degree F day (average winter) and the zero-degree F day (design). Summer air conditioning and sludge heating requirements are based on the 90-degree F day.

Sludge heating is required to maintain adequate temperatures for sludge digestion. The bulk of the space heat is required to heat large quantities of outside air for ventilation purposes with lesser quantities required for building losses. Air conditioning is by a steam regenerated type unit. The space heat and air conditioning requirements are based on maintaining inside spaces at 70 degrees F in the winter and 75 degrees F in the summer.

4. SLUDGE GAS PRODUCTION

Sludge gas is produced as a by-product of the anaerobic digestion of the primary and waste activated sludges. Both sludges are thickened prior to digestion to increase the solids concentration and reduce the volume of sludge to the digesters.

The production of sludge gas in the digesters is dependent on the feed volatile solids, adequate mixing of digester contents and maintaining digester temperature at about 95 degrees F. The estimated quantities of sludge gas produced are shown in Table 2.

The sludge gas quantities shown are based on a cubic foot of gas having a low heat value of 600 Btu and a yield of 18 cubic feet of gas from each pound of volatile sludge solids destroyed in the digesters.

5. ENERGY RECOVERY AND UTILIZATION

A. Sludge Gas

Sludge gas is the primary source of recoverable energy at the plant. Sludge gas is contained within the digesters and is collected for use. All sludge gas quantities produced in the digesters are recoverable and available for use in the plant. Sludge gas is used as fuel for the engine-driven process air blowers and for building space heating in the winter. Sludge gas utilization for summer and winter conditions is shown in Table 3.

Most of the sludge gas produced is used as fuel for the engine-driven blowers. The total fuel requirement for the engine-driven blowers, as shown in Table 1, is 680 million Btu per day. Engine operation requires the use of pilot fuel oil for ignition of the sludge gas. Fuel oil requirements are 5 percent of the total engine fuel input or 34 million Btu per day. Sludge gas provides 95 percent of the total engine fuel requirements or 646 million Btu per day.

B. Engine Heat Recovery

The efficiency of the blower engines is about 38 percent; 260 of the total 680 million Btu per day fuel input produces useful engine work. Most of the energy input is generated as heat which is recovered by the following engine-blower systems:

- Engine jacket water system
- Engine lube oil system
- Engine exhaust gas system

Heat is recovered from the engine jacket water and lube oil systems as hot water, and is used to heat the waste activated sludge. Approximately 25 percent of the total engine fuel input or 170 million Btu per day is recovered in this form. Heat is recovered from the engine exhaust gas system as steam, and is used for primary sludge and building space heating in the winter and for primary sludge heating and air conditioning in the summer. Approximately 30 percent of the total engine fuel input or 204 million Btu per day is recovered as steam.

Utilization of the heat recovered from the engine-blower systems is shown in Table 3 for both summer and winter conditions.

C. Total Energy Recovery and Utilization

The total quantity of recoverable energy, as shown in Table 3, is 1,112 million Btu per day. 785 million Btu per day or about 70 percent of the total recoverable energy is utilized in

the plant at average summer day conditions, and 1,055 million Btu per day or 95 percent is utilized at average winter day conditions.

6. ECONOMIES OF ENERGY RECOVERY

Estimated annual energy costs for the plant utilizing sludge gas and engine heat recovery are shown in Table 4. Costs are also shown for plant operation using 100 percent purchased energy.

Purchased energy costs include the costs for electrical power and fuel oil. Purchased electrical power costs are based on the electrical power requirements shown in Table 1 at a unit cost of $0.08 per kwh. Fuel oil costs are based on the heat requirements, for the engine-driven process air blowers and for heating and air conditioning, shown in Table 1, at a unit cost of $0.85 per gallon for fuel oil with a heat value of 140,000 Btu per gallon.

The total cost of energy based on plant operation using 100 percent purchased energy is $3,333,000 per year. The total energy cost for the plant utilizing sludge gas and engine heat recovery is $1,389,000 per year. The annual savings in energy costs with energy recovery and utilization is $1,944,000 or about 58 percent of the total energy cost.

7. SUMMARY

Sludge gas and engine heat are recovered at the Yonkers Plant and are used to offset the costs of purchased energy. Sludge gas is used as fuel for the engine-driven process air blowers and for building space heating. Recovered engine heat is used for sludge heating and building space heating and air conditioning.

A total of 1,112 million Btu per day of energy is recoverable for use in the plant. 785 million Btu per day or about 70 percent of the recoverable energy is used during the summer, and 1,055 million Btu per day or 95 percent is used during average winter day conditions.

The annual savings in energy costs utilizing sludge gas and recovered engine heat is about $1,944,000. This is about 58 percent of the total energy cost of $3,333,000 for the plant without energy recovery and utilization.

TABLE 1

Plant Energy Requirements

1. Electrical Power	
a. Lighting	
1) Load in kw:	
a) Connected	1,458
b) Estimated Operating	169
2) Operating Time, hrs per day	12
3) Power Consumption, kwh per day	2,028
b. Motors	
1) Load in Hp:	
a) Connected	4,398
b) Estimated Operating	1,881
2) Unit Power Consumption, kw per hp	0.9
3) Operating Time, hrs per day	24
4) Power Consumption, kwh per day	40,632
c. Total Power Consumption, kwh per day	
1) Lighting	2,028
2) Motors	40,632
3) Total	42,660
2. Engine-Driven Process Air Blowers	
a. Number of Units	
1) Installed	4
2) Operating	3
b. Average Blower Power Requirements, Hp:	
1) Per Blower	1,350
2) Total Operating	4,050
c. Average Engine Fuel Consumption	
1) Btu per Blower, hp-hr	7,000
2) Total, million Btu per day:	
a) Per Blower	227
b) Total Operating	680
3. Heating and Air Conditioning Requirements in Million Btu per Day	
a. Winter	
1) Space Heating:	
a) 40-Degree F Day (Average Winter)	287
b) Zero-Degree F Day (Design)	667
2) Sludge Heating:	
a) 40-Degree F Day	147
b) Zero-Degree F Day	172
b. Summer (90-Degree F Day)	
1) Air Conditioning	34
2) Sludge Heating	68

TABLE 2

Sludge Gas Production

1. Primary Sludge Gas	
a. Sludge Gas, 1,000 CF per Day	735
b. Heat Value, Million Btu per Day	441
2. Waste Activated Sludge Gas	
a. Sludge Gas, 1,000 CF per Day	495
b. Heat Value, Million Btu per Day	297
3. Total Sludge Gas	
a. Sludge Gas, 1,000 CF per Day	1,230
b. Heat Value, Million Btu per Day	738

TABLE 3

Energy Recovery and Utilization

	Million Btu per Day		
	Summer	Winter	
	90° F Day	40° F Day	0° F Day
1. Recoverable Energy			
a. Sludge Gas	738	738	738
b. Engine Heat Recovery:			
1) Jacket Water-Lube Oil	170	170	170
2) Exhaust Gas	204	204	204
3) Total	374	374	374
c. Total Recoverable Energy	1,112	1,112	1,112
2. Utilization of Recovered Energy			
a. Sludge Gas			
1) Engine-Driven Process Air Blowers	646	646	646
2) Space Heating	0	92	92
3) Total Sludge Gas Used	646	738	738
b. Engine Heat Recovery			
1) Jacket Water-Lube Oil:			
a) Waste Activated Sludge Heating	46	96	113
b) Piping Losses	17	17	17
c) Total	63	113	130
2) Exhaust Gas:			
a) Primary Sludge Heating	22	51	59
b) Space Heating	0	133	125
c) Steam Regenerated Air Conditioner	34	0	0
d) Piping Losses	20	20	20
e) Total	76	204	204
3) Total Engine Heat Used	139	317	334
c. Total Recovered Energy Used	785	1,055	1,072
3. Surplus Recovered Energy			
a. Sludge Gas	92	0	0
b. Engine Heat Recovery:			
1) Jacket Water-Lube Oil	107	57	40
2) Exhaust Gas	128	0	0
3) Total	235	57	40
c. Total Surplus Energy	327	57	40

TABLE 4

Estimated Annual Energy Costs

	Energy Cost in $1,000's	
	All Energy Purchased	With Sludge Gas & Heat Recovery
1. Electrical Power		
a. Lighting	$ 59	$ 59
b. Motors	1,186	1,186
c. Total	$1,245	$1,245
2. Fuel Oil		
a. Engine-Driven Process Air Blowers	$1,507	$1,507
b. Space Heating	318	318
c. Sludge Heating	238	238
d. Air Conditioning	25	25
e. Total	$2,088	$2,088
3. Value of Sludge Gas and Recovered Engine Heat		
a. Engine-Driven Process Air Blowers	$ 0	$(1,432)
b. Space Heating	0	(249)
c. Sludge Heating	0	(238)
d. Air Conditioning	0	(25)
e. Total	$ 0	$(1,944)
4. Total Annual Energy Cost	$3,333	$ 1,389

"ENERGY CONSERVATION AT AN ADVANCED SECONDARY TREATMENT PLANT"

by
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Oak View Sanitary District
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The intent of this paper is to demonstrate that in many cases fixed film treatment systems, particularly the rotating biological contactor (RBC) system, can produce an effluent of comparable quality to an aerobic suspended solid treatment system with significantly less energy and less total operation and maintenance (O&M) requirements. The authors recognize and acknowledge that suspended solid systems, particulary activated sludge (AS) systems, have some significant advantages over fixed film systems in terms of treatment efficiency and reliability. However, this does not negate the fact that fixed film systems by themselves, or in conjunction with suspended soilds systems, can produce an effluent of comparable quality to suspended solids systems with less energy.

ENERGY REQUIREMENTS AND COSTS AT WASTEWATER TREATMENT PLANTS

Next to labor costs, the most expensive component of operation and maintenance of wastewater treatment plants is energy. Depending upon the level and method of treatment, energy can account for in excess of 25 percent of the total O&M budget (EPA-2/78). An EPA survey of over 130 activated sludge treatment plants indicate that average energy costs account for between 14 and 27 percent of the total annual O&M budgets depending upon treatment plant size (Table 1).

As treatment requirements become more stringent, treatment plant O&M budgets for the more complex and energy intensive processes and equipment will significantly increase. Table 2 contains the results of a recent EPA survey on O&M costs and clearly indicates that total O&M costs, particularly energy costs, increase dramatically as the level of treatment increases. Future increases in the cost of energy will not only increase total energy costs but will also result in energy costs becoming a more significant portion of the total O&M budget. According to U.S. Bureau of Labor Statistics, the cost of electrical energy is anticipated to increase at a faster rate than labor or chemical costs resulting in energy costs becoming a larger percentage of the total O&M budget.

ADVANTAGES OF FIXED FILM SYSTEMS OVER SUSPENDED SOLIDS SYSTEMS IN CONSERVING ENERGY

The most significant portion of the energy requirement for secondary treatment is in conversion of soluble organics to settleable material and in handling and treating these solids. Table 2 indicates that well over 60 percent of the energy at an activated sludge secondary treatment plant is used for these specific purposes. The energy required for this conversion process is dependent upon the type of process selected. Table 3 shows that

the energy required to treat the same waste to the same level varies significantly with the method of treatment utilized. Though there are numerous processes for affecting this conversion, the processes can all be categorized as either fixed film systems or suspended solids systems. Typical fixed film systems include trickling filters and rotating biological contactors. Typical suspended solid systems include many variations of the air and pure oxygen activated sludge systems and numerous variations of lagoon systems.

In general, for the same level of treatment, fixed film system require less energy than suspended solid system. Some of the basic reasons for this are:

1. In most fixed film systems the majority of oxygen required for converson of BOD and NH_3-N is supplied at ambient pressure rather than at the much higher pressure needed for oxygen transfer in a suspended solids system. Some energy is required to pump the wastewater over the trickling filter media or to turn the rotating biological contractors but this is much less than is required to mechanically supply oxygen for suspended solids systems.

2. Solids produced in a fixed film system are generally much heavier and settle out to produce a thicker sludge than those from a suspended solids system. This results in a smaller volume of sludge from fixed film systems and, hence, a smaller energy requirement to further handle and treat the sludge.

3. Many treatment plants today are being required to nitrify their effluent prior to discharge. Many of these plants, however, need only achieve partial nitrification; say 10 mg/l NH_3-N. As opposed to fixed film systems, studies of suspended solid systems have demonstrated that partial nitrification cannot be reliably achieved due to the instability of the system. In order for a suspended solids system to meet a partial nitrification requirement, it is generally necessary to completely nitrify a portion of the flow which, when combined with the nonnitrified flow, will meet the partial nitrification requirement. Thus, if only partial nitrification is required, a suspended solids systems must either invest more capital to construct a system capable of handling two sludges or nitrify the complete flow, thereby utilizing significantly more energy.

An advantage often claimed for activated sludge systems over fixed film systems is that they have much greater process flexibility and can thus produce a more reliable better quality effluent. To an extent, this is true, particulary when comparing activated sludge systems to trickling filters. However, rotating biological contactor systems have more process control factors than trickling filters and can reliably produce an effluent of comparable quality to that of an activated sludge systems with less energy and less operator attendance and skill. This fact has been demonstrated in numerous pilot studies and full scale treatment plants.

In recent pilot studies conducted for the City of San Francisco, rotating biological contactors were selected over both air and high purity oxygen activated systems for treatment of domestic wastewaters from the southwest

San Francisco area. One of the main reasons cited for the selection of the rotating biological contactor system was its simplicity of operation and its ability to adequately treat large fluctuations in hydraulic and organic loads with minimal plant upset.

In a full scale installation at LeSourdsville, Ohio, a 3 mgd rotating biological contactor treatment plant is meeting stringent treatment requirements of 10 mg/l BOD, 12 mg/l SS, and 2 mg/l NH_3-N. Secondary clarifier effluent quality, prior to filtration, is 15 mg/l BOD, 15 mg/l SS, and 2 mg/l NH_3-N; greater than 94% removal for all these constituents. After two years of operation, the plant superintendent says discharge requirements have been reliably met despite significant fluctuations in hydraulic and organic loading.

An additional advantage of fixed film systems is their greater resistance to toxicity and ability to recover more quickly from "biological kills" than suspended solid systems. A recent well publicized failure of the activated sludge system at the San Jose Water Reclamation Plant, San Jose, California, is illustrative of the difficulty in restarting an activated sludge system after a biological kill.

CASE STUDY-OAK VIEW WASTEWATER TREATMENT PLANT

Oak View Wastewater Treatment Plant (OVTP) is a 3.0 mgd trickling filter-activated sludge treatment plant located approximately five miles north of Ventura, California. The plant is presently hydraulically and organically underloaded so that the activacted sludge system is only occassionally utilized. Recent changes in Oak View's NPDES permit, particularly the inclusion of an NH_3-N requirement, necessitates an upgrading of the present treatment plant. Though the NH_3-N discharge limitation is only 10 mg/l, the presence of unusually high influent concentrations of NH_3-N (35-40 mg/l) necessitates construction of substantial facilities.

The most obvious method of upgrading OVTP, and the method selected in the facility plan, was to upgrade the activated sludge to achieve nitrification. Figure 1 is a schematic of the proposed system. Major improvements included an equalization basin upstream of the activated sludge system, intermediate clarifiers to remove trickling filter solids, and significant aeration system modifications, including increased electrical instrumentation and controls.

This system, though feasible, did not meet the District's requirements of operational simplicity and low O&M costs. Additional alternatives were evaluated to develop a system that would reliably meet effluent requirements with a lower O&M cost and with less operator attendance and skill. The alternative selected was air drive rotating biological contactors. Figure 2 is a schematic of the proposed RBC system. Major improvements include equalization basins upstream of the oxidation tower, conversion of the slag media trickling filter system to a plastic media oxidation tower, and replacement of the activated sludge system with air drive rotating biological contactors.

DESIGN COMPARISON

Though both of the proposed treatment systems seemed capable of meeting effluent requirements, a comparison of the two designs led to selection of the RBC system for the following technical reasons:

1. The existing final clarifiers are only 8-1/2 feet deep. This is generally considered to shallow for proper operation of an AS system but is adequate for the RBC system. Despite the decreased variations in hydraulic loading resulting from the equalization basins, the AS system will be more susceptible to wash out than the RBC system.

2. The AS system would be designed to achieve both carbonaceous and nitrogen removal, while the RBC system would essentially be utilized only for nitrogenous removal. Studies have shown that while it is possible to achieve removal of BOD and NH_3-N in a single stage system, it is less reliable than a two stage system (Stover - 1978).

3. AS systems have demonstrated difficulty in achieving partial nitrification, as prescribed at OVTP. Without significant process modifications to allow for splitting flows to nitrify only a portion of the flow before recombining, complete nitrification will result. While this will not pose any problems it will needlessly O&M costs.

4. The AS system is more complicated than the RBC system and will require increased operator attendance and skill. Additionally, the AS system is operationally and conceptually different than the existing TF system while the RBC system is quite similar. This will also result in greater operator attendance and skill for the AS system.

COST COMPARISON

A comparison of costs for the two systems illustrated that, while the AS system requires significantly less capital investment, it requires substantially higher O&M costs. The RBC system requires only 40% the energy of the AS system. The net result is that on a present worth analysis, the RBC system is the most cost-effective alternative. Table 4 clearly show this fact.

An additional benefit of the RBC system from Oak View's point-of-view, is that it qualifies for additional finding as innovative technology. Under the Innovative and Alternative Technology Program, EPA will increase their fundings from 75 percent to 85 percent for projects determined to be innovative alternative technology. When this is coupled with 12-1/2 percent funding from California, Oak View's total share for the RBC system is only 2-1/2 percent.

CONCLUSION

Energy costs presently constitute a significant portion of the O&M budgets at wastewater treatment plants. More stringent treatment requirements and

increased energy costs are anticipated to increase total energy costs in the future. One of the most significant ways that this energy requirement can be reduced is in selection of the secondary treatment method. Fixed systems, particularly the RBC system, can produce comparative effluent quality to activated sludge systems with significantly less energy and total O&M. To maximize energy conservation at wastewater treatment plants fixed film systems should be given serious consideration. At Oak View Wastewater Treatment Plant installation of an RBC system for nitrification, was shown to require significantly less energy and was overall more cost-effective than upgrading the existing activated sludge system to achieve nitrification.

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TABLE 1

AVERAGE PERCENTAGE DISTRIBUTION OF O&M BUDGET FOR TYPICAL ACTIVATED SLUDGE TREATMENT PLANT[a]

Classification	1.0-5.0 mgd	5.1-20.0 mgd	>20 mgd	All Plants
Personnel	54	48	47	52
Power[b]	(22)	(27)	(14)	(22)
Total Utilities	23	30	18	24
Chemical Disinfection[c]	(2)	(3)	(3)	(2)
Total Chemicals	6	9	8	7
Equipment	4	4	2	4
Materials	6	6	9	6
Contractual	3	1	8	3
Other	4	2	8	4
Total	100	100	100	100
Number of Plants Surveyed	95	30	12	137

[a]Adapted from "Analysis of Operations and Maintenance Costs for Municipal Wastewater Treatment Systems".

[b]Power costs are also included in total utility costs

[c]Chemical disinfection (usually chlorine) costs are also included in total chemical costs.

TABLE 2

AVERAGE COST PER MILLION GALLONS TREATED[a]

Category	Primary	Secondary[b]	Tertiary[c]
Personnel	$ 94	$139	$183
Power	22	59	76
Chemical	16	19	48
Others	27	51	91
Total	159	268	398

[a]Adapted from "Analysis of Operations and Maintenance Costs For Municipal Wastewater Treatment Systems." Costs shown are an average of all plants surveyed.

[b]Typical activated sludge treatment plant

[c]Degree of treatment varied with plant surveyed.

TABLE 3

BRAKE HORSEPOWER REQUIREMENTS FOR VARIOUS SECONDARY TREATMENT PROCESSES[a]

Flow	Aero-Surf[b] RBC	Oxidation Ditch	Plastic Media Trickling Filter	Activated Sludge	
				Fine Bubbles	Course Bubbles
1.0	12	60	9	34	59
5.0	52	265	43	174	275
10.0	98	510	80	305	502

[a]Assumes standard secondary treatment removal efficiencies of 85% and influent BOD and SS of 200 mg/l. Data for all requirements are at sea level and standard conditions.

[b]Assumes air at 3.2 psg

[c]Assumes air at 7.0 psg. Power requirement for return activated sludge pumping not included.

TABLE 4

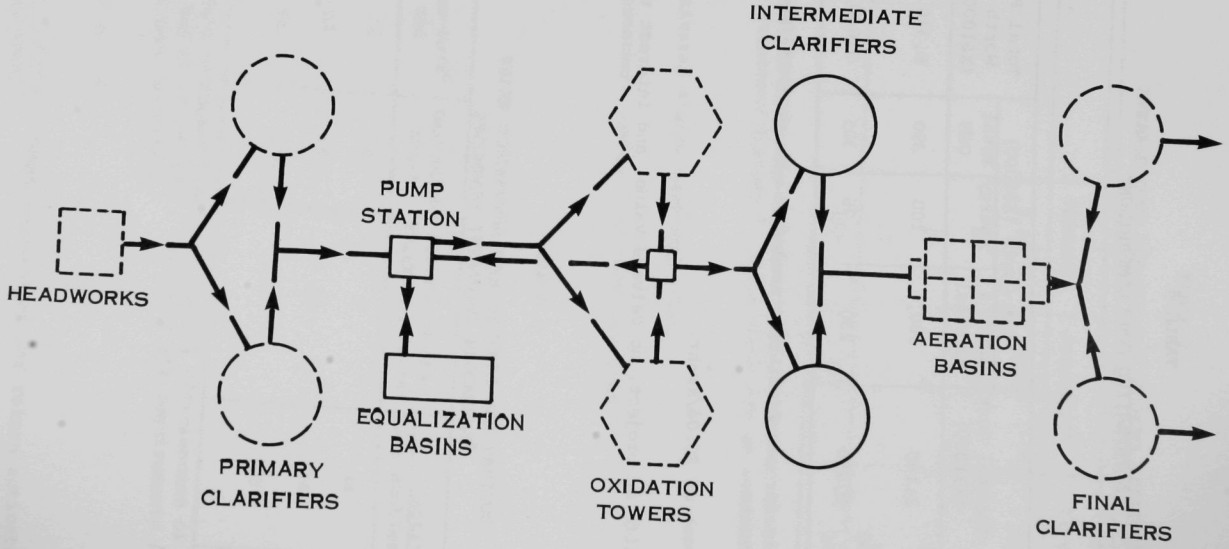
ALTERNATIVE COST COMPARISON[a]

Alternative	Capital Cost (X$1000)	O&M Costs (X$1000) Labor & Material	Energy	Total O&M	Total Present Worth (X$1000)
Activated Sludge	2,180	290	100	390	6,310
Rotating Biological Contactors	2,450	230	75	305	5,680

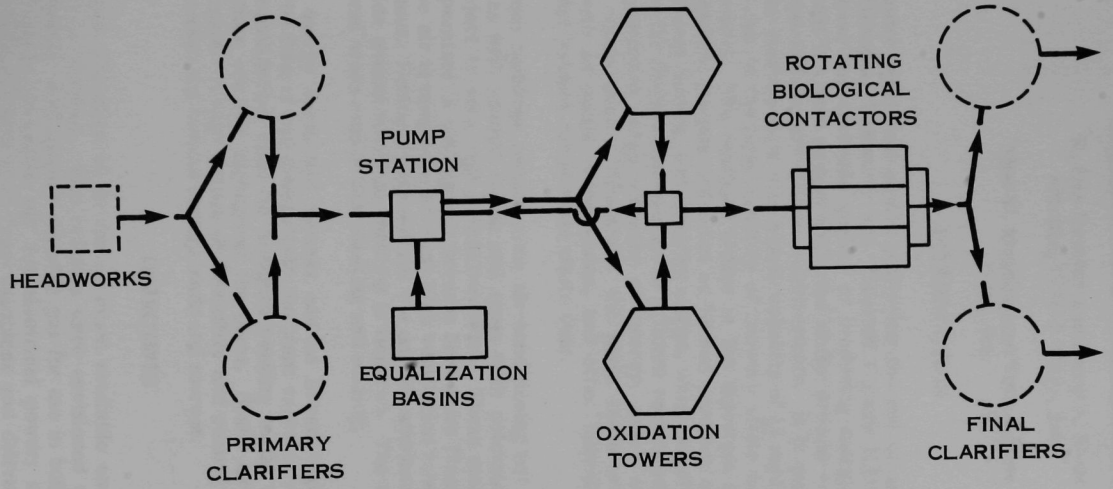
[a]Comparison is only for new Facilities needed to meet more stringent effluent requirements.

[b]Assumes energy cost of $0.04/kw-hr.

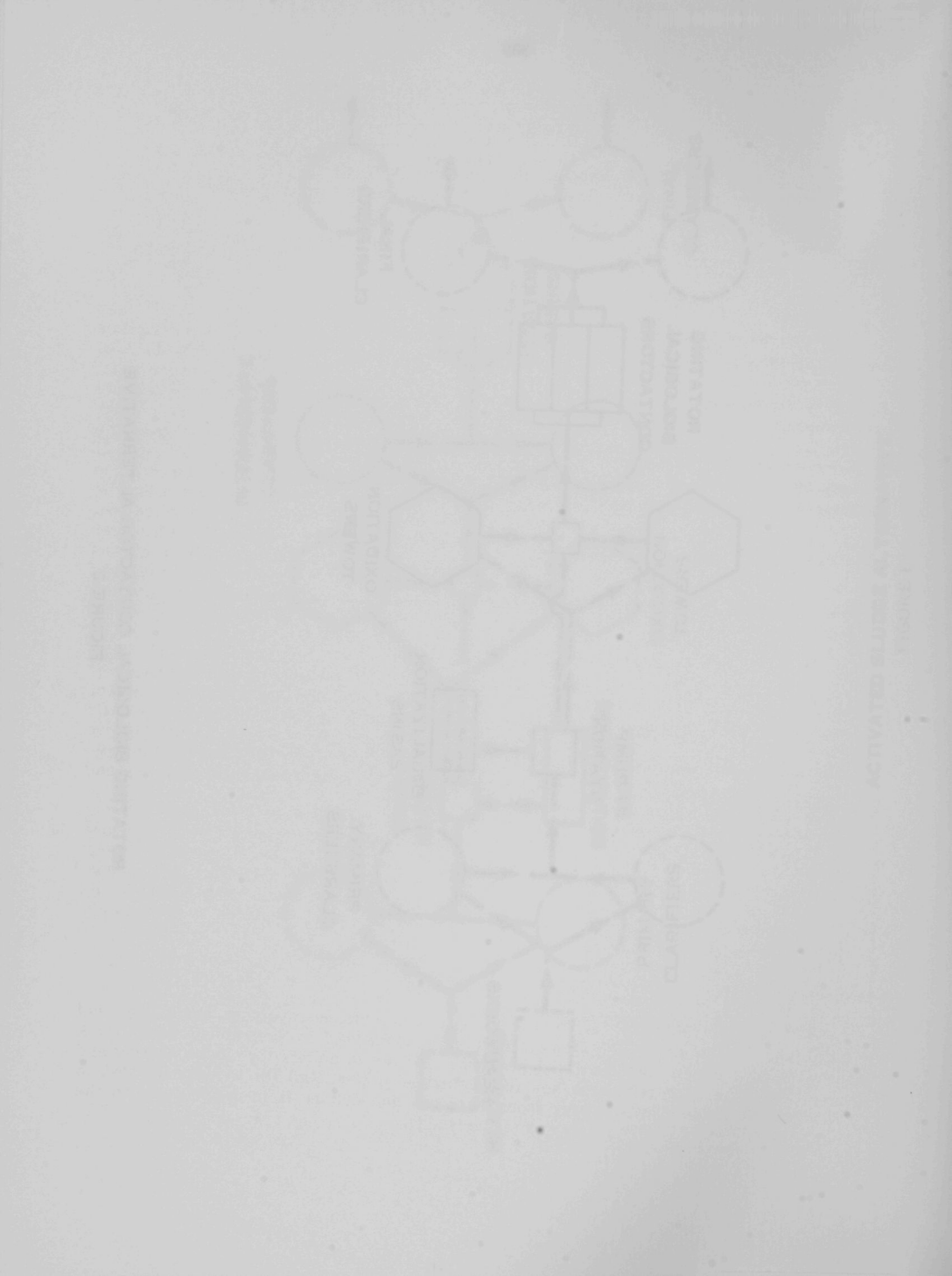
[c]Assume 20 year life of project, no salvage value, and interest rate of 7 percent.



ACTIVATED SLUDGE ALTERNATIVE
FIGURE I



ROTATING BIOLOGICAL CONTACTOR ALTERNATIVE
FIGURE 2



RECOVERY AND USE OF WASTE HEAT AND DIGESTER GAS AT THE LOTT WASTEWATER TREATMENT PLANT

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INTRODUCTION

Wastewater treatment plants employing the use of anaerobic digesters such as the Lacey, Olympia, Tumwater and Thurston County (LOTT) plant in the state of Washington have the desirable quality of producing energy as well as consuming it. The LOTT plant is a pure oxygen activated sludge process compactly designed because of the site restrictions and effluent requirements. It is capable of 14 million gallons per day (mgd) average flow with a peak capacity of 36 mgd. The solids loading on the plant is high due to the large quantity of brewery waste in addition to more typical sources of waste. The resulting sludge in the digesters is heated to speed up the anaerobic digestion process which has as its end products digester gas, which can be used as an energy source, and digested sludge, which is suitable for land application. Ozone is used for disinfection because of effluent requirements. Production of oxygen and ozone consumes large amounts of energy, much of which is given off as recoverable waste heat. Throughout the plant there are numerous other energy consumers such as pumps, compressors, and other miscellaneous pieces of process equipment that produce recoverable waste heat.

The plant buildings require some air-conditioning but mostly heating and ventilation. Areas with noxious fumes and areas with potentially explosive atmospheres that are subject to very high ventilation rates. These spaces are kept near ambient outside temperature in the summer because heat gains from the equipment have little effect on the air temperature at these high ventilation rates, 15 to 30 air changes per hour continuous. During winter months the outside ambient temperature is unacceptable for inside environments so heating is required. The administration building and other personnel areas require both heating and cooling.

These energy loads and sources can be linked together, however, there are variables. The flow of raw sewage into the plant varies in volume and content. There is a large equalization basin to mitigate peaking loads, but variations still occur. These variations are felt throughout the plant. Space air-conditioning requirements are also variable. A high degree of flexibility and control is required to successfully interface between the various energy loads and sources.

OBJECTIVES

The main objective is to use as much available energy from waste heat and methane gas as possible. Two systems were envisioned to accomplish this: (1) a system to produce high quality digester gas for use in boilers for sludge heating and possibly an engine/generator set for electrical power; and (2) a hydronic system designed to collect waste heat from machines and deliver it in usable form to the various building HVAC systems and to the sludge.

DESIGN PARAMETERS

There are many requirements that must be met for proper process function while conserving energy. The sludge is kept at an average temperature of 95°F in the digesters. The ozone generators must be supplied with 76°F cooling water for maximum efficiency. The efficiency of the ozone generators drops off rapidly when the temperature varies from 76°F. The building process spaces require 55°F to 60°F temperatures in winter. The plant will operate at less than capacity at start-up and gradually increase. These parameters are directly connected to energy conservation. There are other process parameters that indirectly affect energy use.

SYSTEM CONFIGURATION

The amount of digester gas available is more than required for just sludge heating. The challenge is to use all the digester gas in such a way as to minimize energy costs. The method selected uses digester gas to generate heat and electricity. Optimum utilization calculations show that operating a 400 kW engine/generator (e/g) set for 16 hours per day and boilers the other 8 hours uses the excess digester gas most cost-effectively.

The e/g set produces 400 kW that is used to reduce diurnal peak electrical demand for 16 hours per day. Waste heat from the engine cooling water and exhaust gases is sufficient for the steady state heat required by the digesters. Note that digester start-up and peak loads require boilers assist for heating the sludge. During the remaining hours per day the dedicated electrical loads are switched to utility power and the sludge heating is accomplished by boiler alone. Whenever sludge gas production does not meet this 24-hour schedule then interruptable natural gas or propane is substituted.

This operating schedule is a result of relative costs for utility electricity, natural gas, and sludge gas production, and the relative efficiencies of the boilers vs. the e/g set/heat recovery system. Electricity presently costs more per million BTU than natural gas, but the boiler uses fuels at 80 percent efficiency whereas the e/g set heat recovery system attains 60 percent efficiency. The sludge gas could be used for 20 hours of electrical generation after which the sludge gas would be expended, and natural gas and utility electical power would be substituted. This or any alternative would increase energy costs over the chosen schedule of 16 hours per day of electrical generation.

The fuel system is designed for use of sludge gas, natural gas, and propane. The methane is cleaned by gas purifiers that remove sulphur compounds to prevent corrosion in the boiler or engine. The sludge gas still has impurities and thus less energy content than natural gas, so the boilers and e/g set are derated. Use of natural gas just increases the capacity of these pieces of equipment. Propane is available for emergencies only and must be blended with air to reduce effective energy content as supplied to the boilers and e/g set.

The system referred to as the High Heat Loop operates at 180°F water temperature. The high temperature-constraint for this system is the engine-jacket water temperature of 210 °F. In order to avoid caking of sludge, the heating water-temperature must be limited to 155°F which is the low temperature-constraint. The

sludge heating water inlet temperature of 155°F is maintained by recirculating most of the water and adding 180°F water as needed. The engine-jacket water temperature of 210°F is maintained by using a heat exchanger with the High Heat Loop water at a temperature of 180°F. Heat is dumped to reclaimed water (plant effluent) by means of another heat exchanger when heat supply is greater than demand and there is no other use for the energy. Another heat exchanger provides transfer of heat to the Low Heat Loop if it has an insufficient supply.

The Low Heat Loop operates at a water temperature of 80°F and collects waste heat from miscellaneous machines making it available for space-heating requirements. The water temperature restriction in this system is due to the ozone generators. Cooling-water inlet temperatures to these generators must be maintained in a narrow band around 76°F or efficiency will drop off significantly. These generators use large amounts of energy so any reduction in efficiency has a large effect on energy consumption. Outlet water temperature from the ozone generators is 90°F. In addition to the ozone generators, this loop is used to gather heat from the large air compressors used to supply the oxygen-generating plant with compressed air. Other pieces of equipment used as heat sources include methane, compressors, oxygen compressors, compressed gas coolers, lube oil coolers, and sludge centrifuges. The heat exchange with all of these units is designed to produce a 90°F water temperature.

This temperature is marginal for use with standard HVAC coils and air-handling units. The winter building-loads due to the large ventilation rates are larger than the available waste heat. Large exhaust air heat recovery wheels are used to minimize these loads. As a result 80 percent of the building space heating demand and 95 percent of annual energy use can be met by available waste heat. Oversize coils are used to compensate for the low water-temperature and are used in spaces where only heating is required. Spaces requiring heating and cooling are served by water to air heat pumps that use the Low Heat Loop as a heat source or a sink for heating and cooling respectively.

The balance between available and required heat varies greatly with fluctuations in influent and weather conditions. Many machines operate independently causing fluctuations in available waste heat. Fluctuation in weather conditions and occupancy cause variation in building space-heating loads. A large volume of water is circulated for the purpose of reducing the potential range in temperatures in the Low Heat Loop due to these fluctuations. One heat exchanger provides a heat dump to the effluent similiar to the High Heat Loop. As noted earlier there is also a heat exchanger that provides the extra heat from the High Heat Loop when the extra 20 percent demand is required.

The complexity in all of these systems requires a flexible, multifaceted control system. The Low Heat Loop operates independently of the High Heat Loop and Methane Gas System except for interfaces at the heat exchangers. Control is based on temperature at the ozone generators and pressure drop through the loop. The High Heat Loop and Digester Gas System are highly interdependent requiring a complex system controls designed for the flexible control of many options.

CONTROL SYSTEMS EVALUATION - A SYSTEM APPROACH

The plant processes produce continuously varying and unpredictable sludge load (flow and density) to the digesters. Although anaerobic digester operation is a relatively stable process, it must accept the sludge load it is supplied and make necessary compensation to optimize the gas production. Several interrelated processes are involved in generation and usage of the digester gas. The digesters are charged with thickened primary and secondary sludge. Stabilized sludge is concurrently withdrawn from the digesters. Digester gas generation and management for optimum utilization is a continuous process tied closely to the sludge control. Operation of the heat exchangers for sludge heating is part of this system. The boilers and engine generator utilize the digester gas to supply heat to the High Heat Loop. The electric generator is used for electrical demand peak shaving.

Anaerobic digestion requires careful control and monitoring of intermediate variables in order to recognize and react correctly to upset conditions. On the other hand, the electrical demand control is a very fast process and one needs to monitor all related loads and correct them as soon as the upset occurs. Figure 1 shows a simplified block diagram of the processes involved. The process interdependencies are apparent.

INSTRUMENTS FOR CONTROLLING AND MONITORING

The following instruments are used to monitor different process parameters for control purposes or for logging the information for future use for improving process strategies.

1.	Sludge density	Optical sludge density meter
2.	Sludge flow	Magnetic flowmeter
3.	Gas flowmeter	Orifice plate and differential transmitter
4.	Digester sludge temperature	Resistance bulb in a thermowell
5.	High Heat Loop and heat exchanger loop temperature	Thermocouple in a thermowell
6.	Digester sludge level	Bubbler with differential transmitter
7.	Digester cover level	Electromechanical devices
8.	Plant electrical demand and energy usage	Signal is received from the power company's metering equipment

The following process variables are controlled:

1. Sludge flow to each digester
2. The digester gas pressure to the boilers and engine generator
3. Temperature of the High Heat Loop
4. Excess gas burner temperature
5. Plant's electrical demand
6. Digester sludge temperature

CONTROL PHILOSOPHY

The control system was selected based on meeting a number of objectives. The sludge is stabilized for easier dewatering and to produce methane gas constantly at the highest possible digester efficiency in all different environment and loading conditions. Thus optimization of the availability of the digesters is attained. The usage of the digester gas produced is simultaneously optimized while providing for sludge heating 24 hours a day and 365 days a year and supplying electrical power to some of the electrical loads in the plant. The plant electrical power demand peaks are minimized by controlling the timing of the generator operation.

RECOMMENDED CONTROL PHILOSOPHY

As a result of the process interdependencies and the possibility for optimization, it was concluded that all control actions for the process system shown in Figure 1 should be centralized. Centralization is attained by adhering to several basic concepts specific to this plant. The operator in charge of digester gas sludge and High Heat Loop systems has all information about the processes involved available to him at all times, and shall have an access to control of all the critical equipment involved in the process. One control operator station provides all of these functions. The control system is provided with sufficient redundancy to keep the process running at degraded level with an absence of the central control system. A high degree of flexibility is built into the control system for addition, deletion, or modification of control points, the control loops for the process, or whole control programs. Therefore one can implement new control strategies or better monitor the processes. The control system provides simplified man-process interface equipment for convenient and efficient control of the process.

NECESSARY CHARACTERISTICS

The control system has several basic characteristics in order to tailor the needs for the processes involved. For example: the process may expand in its size; new processes or new instruments may be added. The management may require more information or may need tighter control to increase the efficiency of the process or to meet government standards for waste disposal. The system can accommodate all these probable modifications at a minimum added cost.

The complexity of the process dictates that flexibility in control strategy is an absolute must. Except for a static process, it is generally not practical to offer the operator only one control alternative. Therefore, the operator is able to apply a variety of strategies, depending upon the particular conditions with which he is faced at any point in time. The operator is also able to control and monitor all the processes involved from a control location. This minimizes communication requirements at the local level. The monitoring of the processes and equipment involved is by an exception rule. This means the operator is informed about either impending or actual failure of a sensor or equipment only when it occurs. However, status of all equipment, each sensor, and each sensor output is available to the operator on demand at all times.

The control system provides the maximum possible availability. If the central system fails for some reason, the critical processes continue to operate in their last control mode and status. If a process has to be controlled 100 percent of the time,

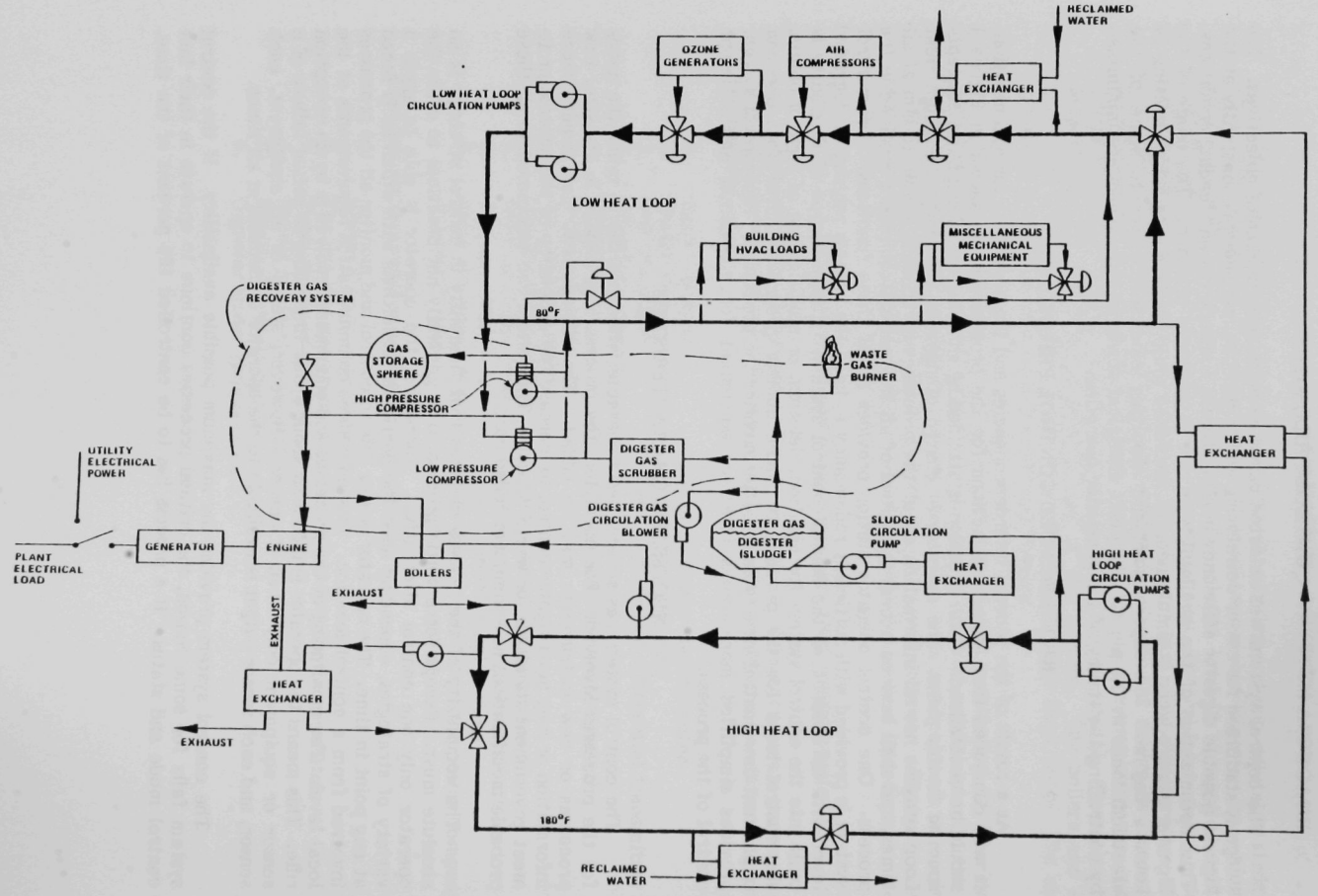

FIGURE 1 DIGESTER GAS AND WASTE HEAT RECOVERY SYSTEM

then a local controller is provided and an arrangement is made to switch over the control to local controller without interruption or a redundant central control and/or redundant local control system is provided.

DIGITAL SCHEMES

To obtain all requirements, stated in the above section, a digital control system is used. There are three major varieties of digital control systems available. In each case analog signals representing process variables are converted to a form which can be read and understood by a digital computer. The computer makes control decisions based on preprogrammed logic and operator input, and issues field-compatible control outputs.

In the microcomputer system all of the control and sensor outputs are brought back to a single computer through multiplexers located in the field. This computer may be backed up by another similar computer. The multiplexers are hardwired to the computer with only a pair of wires. All control decisions are made soley by the computer. If the on-line computer is unavailable, the backup computer will take over the controls of all the processes and equipment.

The central digital control system requires a main computer located in a central control room. It may be backed up by another computer. This computer is connected to all the processes and equipment in a plant through remote multiplexers. The two computers are linked together via communication lines and share various pieces of information. Each computer also monitors the status of the other computer. A failure of the on-line computer will switch the control of all processes and equipment over to the backup computer.

The remote multiplexers may be equipped with a micro- or minicomputer. These multiplexers would typically perform all process input and output, error checking, and conversion to or from engineering units. Some sequencing logic and other analog logic might also be included.

The distributed digital control system is a hierarchial system which includes multiple levels of control and control decisions. This system is composed of a number of individual computers which control unit processes or process. These computers are hardwired to a central computer system.

Different types of control and process decisions are made at each level. The objective is to distribute the execution and processing among the various computers in order to increase control responses and to increase reliability and availability. Each subsystem computers' responsibility is to control the process for which it is responsible. Each process computer will communicate with the central computer to obtain related information about other processes. The central computer collects and records all the data through the process computers. The central computer links the entire control system. This includes the basic optimization changes necessary to achieve high performance in the total plant.

Power demand monitoring and control, and the reporting of all data for operations or regulatory agency are performed by the central computer.

THE SELECTED CONTROL SYSTEM

The system chosen for the process is a digital control system with remote multiplexers. It is equipped with an on-line central computer, a backup computer, and five remote multiplexers (for the whole plant). The remote multiplexers are not equipped with mini- or microcomputers. They are designed only to perform multiplexing functions between the field signals and the central computer.

The central computer system is supplied with necessary peripherals like programmer's console, CRT units with keyboard for entering and retrieving information, printers for alarm printing and report printing, the data storage equipment and keypunch machine. Two central computers are connected through a communication controller.

The processes involved in this project are connected to the computer through two multiplexers. Each multiplexer is connected to the local control devices and sensors through their local panels.

ANTICIPATED RESULTS

The present system is designed to produce 100,000 standard cubic feet (scf) digester gas at start-up (in 1981) and 165,000 scf digester gas by 1985. The system will use the methane gas to generate heat for sludge and space heating and to generate electricity for plant equipment. Significant savings will be realized from this as illustrated in the following paragraphs.

The following information is considered in estimating the savings in the plant operating cost.

- Engine generator produces 400 kW at full load. Additional 2,100 MBH heat can be recovered from the engine exhaust gases. It consumes methane gas at a rate of 134 scfm maximum at full load.
- The hot water boiler at full load provides 4,184 MBH heat. It consumes methane gas at a rate of 105 scfm maximum at full load.
- Sludge heating requirement equals 1,600 MBH average.
- Maximum space heating requirement equals 2,430 MBH (mostly in winter time).
- Average available equipment waste heat equals 1,870 MBH.

Available gas will be shared by both the engine generator and the boilers to meet the following requirements.

- Provide necessary heat for the digester sludge heating at all times
- Decrease the maximum electrical demand charges on the overall plant
- Provide low-cost electrical energy
- Utilize all sludge gas produced by the digester
- Meet space-heating requirements when needed

The Low Heat Loop will collect available waste heat, use what is needed, and dissipate the rest.

Based on the available gas, in summer time (assumed 6 months), the engine can be operated for 16 hours a day at full load and one boiler will run at (32 percent) minimum load for 8 hours a day.

SAVINGS FROM THE HIGH HEAT LOOP

1. From electrical charges per month:

$$\text{demand cost} = \frac{\$2.05}{\text{kW/month}} \times 400 \text{ kW} = \$820/\text{month}$$

$$\text{energy cost} = \frac{\$.012}{\text{kWh}} \times 400 \text{ kW} \times 16 \text{ hr/day} \times \frac{365 \text{ days}}{12 \text{ months}} = \$2,336/\text{month}$$

2. From methane gas utilization for heating per month:

$$\text{cost} = 0.32 \times \frac{105 \text{ scfm} \times 60 \text{ min/hr}}{95 \text{ scf/therm}} \times \frac{\$0.28}{\text{therm}} \times \frac{8 \text{ hr}}{\text{day}} \times \frac{365 \text{ days}}{12 \text{ months}}$$

$$\text{cost} = \$1,445/\text{month}$$

In winter time (assumed 6 months), the engine will be running at 16 hours a day at full load. One boiler will be running at 60 percent load for 8 hours a day. Space heating demand is less than or equal to available waste heat 95 percent of the year. The remainder is met by waste heat from the engine/generator set or heat from a boiler, both assumed to be operating on sludge gas.

SAVINGS REALIZED PER MONTH

1. From electrical charges:
   (Same as summer period which is \$820 + \$2,336 = \$3,156)

2. From utilizing methane gas for heating:

$$\text{cost} = \$1,445/\text{month} \times \frac{.60}{132} = \$2,709/\text{month}$$

SAVINGS FROM THE LOW HEAT LOOP

Annual waste heat energy used for space heating = 6.80×10^9 BTU

Annual sludge gas energy used for space heating = 400×10^6 BTU

These energy sources replace energy that would normally be provided by a natural gas boiler. Assuming 80 percent efficiency the total space-heating energy saved equivalent to natural gas is:

$$\text{Energy} = \frac{6.80 \times 10^9 \text{ BTU} + 400 \times 10^6 \text{ BTU}}{.80}$$

$$= 9.00 \times 10^9 \text{ BTU}$$

$$\text{Cost} = 9.00 \times 10^9 \frac{\text{BTU}}{\text{Year}} \times \frac{95 \text{ scf}}{100,000 \text{ BTU}} \times \frac{.28}{95 \text{ scf}}$$

$$= \$25,200/\text{year}$$

TABLE 1
SUMMARY: COST SAVING

	Engine Operator	Boiler Operation	Low Temp Heating	Total
Summer 6 months	$18,936	$ 8,670		$27,606
Winter 6 months	$18,936	$16,254	$25,200	$60,390
TOTAL PER YEAR	$37,872	$24,924	$25,200	$87,996

Cost savings for sludge heating will not be realized because it is an inherent part of this treatment process.

The authors acknowledge the following people and agencies for their continued support and encouragement throughout this project: Allan L. Kimbel and Leonard Esteb of the City of Olympia and other members of the Technical Subcommittee for LOTT Wastewater Treatment Plant Improvements; Cloyd R. Jackson, Ralph R. Federspiel, Victor C. Oblas, and Dr. Paul Liao of Kramer, Chin & Mayo, Inc.; Washington State Department of Ecology, U.S. Environmental Protection Agency Region X, and LOTT agencies for funding of system design and construction.

ANAEROBIC DIGESTER GAS UTILIZATION AT KNOXVILLE'S NEW ADVANCED WASTEWATER TREATMENT FACILITY

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1. INTRODUCTION

In March, 1978, construction began on the expansion of the Kuwahee Waste Water Treatment Facility in Knoxville, Tennessee. This expansion was the result of a county-wide 201 Wastewater Facilities Plan; both the Plan and the plant design were prepared by Envirodyne Engineers, Inc. The expanded plant is still under construction and is scheduled to be completed in August, 1981. Envirodyne is currently providing construction inspection services and any additional design work required.

The treatment processes that will be provided by the expansion consist of screening, grit removal, primary treatment, oxygen-activated sludge secondary treatment, air-activated sludge nitrification treatment, disinfection by chlorination and dechlorination.

The Kuwahee Facility will have an average design flow of 40 MGD and is expandable to 50 MGD on the existing site. The plant has also been designed to treat peak daily flows of 70 MGD and stormwater flows of 120 MGD. Stormwater treatment is required because the plant receives a large amount of infiltration and inflow.

Waste activated sludge will be thickened by dissolved-air flotation, mixed with thickened primary sludge, and then anaerobically digested. The digested sludge will be dewatered by filter presses and disposed of in a landfill. A rendering of this facility is shown in Figure 1.

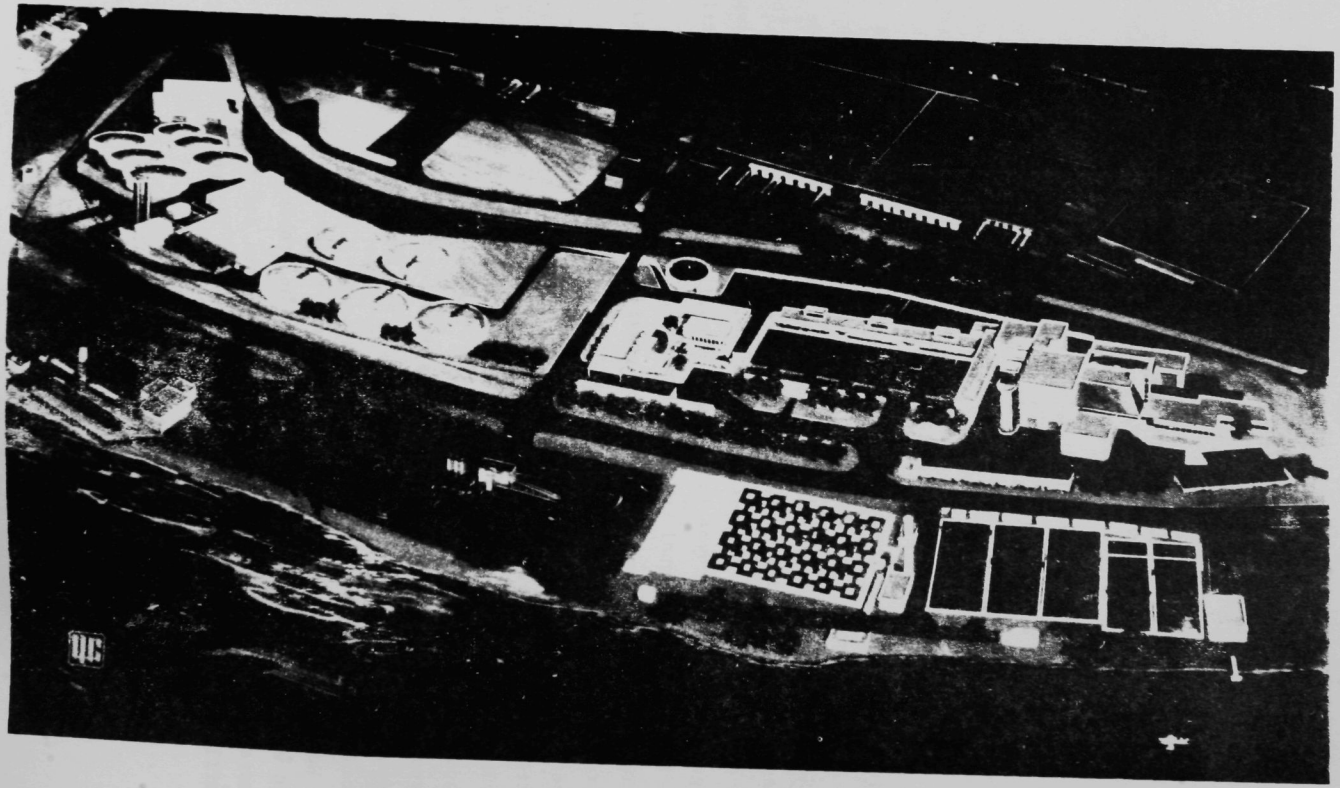

Figure 1. Kuwahee Waste Water Treatment Facility

One major criterion in the design of this facility was the minimization of operating costs. This goal will be attained through computer-assisted control of plant operations, thereby reducing salary costs and conserving energy. Highly efficient treatment equipment will be employed throughout the facility and the methane gas produced in the anaerobic digesters will be used as a primary energy source.

2. GAS UTILIZATION DESIGN CONSIDERATIONS

Although the primary objective of anaerobic digestion is to produce a uniformly stablized sludge, the byproduct of methane gas was recognized as an available energy source (a commodity recognized to be in short supply and high demand). This energy source has been intermittently used in boilers, heaters and internal combustion engines, with the majority being burned off. The use of digester gas as mixing media is common practice.

However, with the projected amount of digester gas production at the Kuwahee Plant, it was determined to utilize this gas as a primary power source for the digester area with the excess to be available for the main plant site. The gas will be used in turbine driven generators. The turbine engine selection was made based on two features: 1) maintenance requirements, and 2) heat recovery capabilities, increasing total efficiency.

The digester gas at this facility can also be burned in the sludge heaters, but the amount of heat exhausted from the turbine generators makes waste heat recovery and transfer to the sludge heaters a most attractive alternative. A schematic flow diagram (Figure 2) shows gas process piping.

The use of digester gas for the good of the plant and/or the public is nothing new, but new and innovative engineering has made it a viable alternative. No matter what material is used: sewage, shredded waste, animal waste, or vegetable matter; methane production can be substantiated in energy conservation.

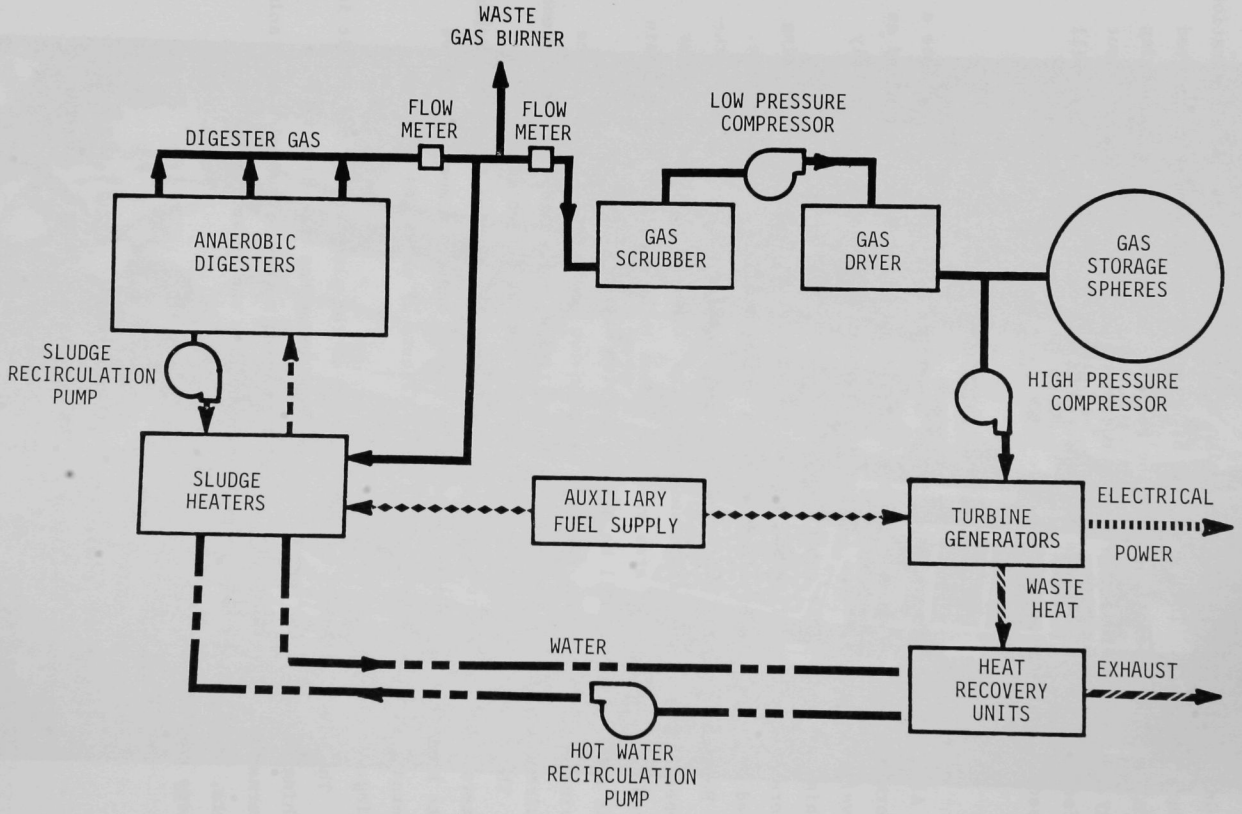

FIGURE 2
PROCESS FLOW SCHEMATIC

3. UNIT PROCESS DESCRIPTIONS
(Refer to Figure 2)

Anaerobic Digesters

Five digesters, each with floating cover, supernatant removal piping, and a maximum capacity of approximately two million gallons, were designed to operate as either a primary or secondary unit, or in any combination. The digester complex will include sludge recirculation, transfer and withdrawal pumps and piping, gas mixing equipment, and a laboratory control room for process monitoring and control. The primary digesters will be heated by three sludge heaters, which are sized to maintain sludge temperature at 95°F. The secondary digesters will not be heated. Maximum gas production is expected to be 27 MMBTU/hr. based upon a design sludge feed quantity of 164,100 lbs./day total solids, volatile solids destruction of 54,000 lbs./day, and gas production of 15 - 18 cu.ft./lb. volatile solids destroyed.

Gas Production Rate Measurement

Gas Mixing and production flow will be measured by five venturi meters installed in the discharge piping of each digester. Another gas meter will measure the flow to the turbine generators. All six of these meters will be connected to the treatment plant's central computer for continuous indication and data storage. Any gas flow to the sludge heaters or waste gas burner will be determined by subtracting the flow rate to the turbines from the flow rates out of the digesters, which are adjusted for recirculation.

Scrubbing and Drying

The normal digester gas stream contains such impurities as hydrogen sulfide (H_2S), carbon dioxide (CO_2) and moisture that are harmful or detrimental to a High Heat Value (HHV). Therefore, the removal or reduction of these impurities is required to prevent the formation of sulfuric acid and enhance or sweeten the gas to an acceptable HHV. The estimated increase is from approximately 635 BTU/cu.ft. to approximately 900 BTU/cu.ft., or approximately a 42 percent increase. Gas burned in the sludge heaters or waste gas burner will not require scrubbing or drying.

The gas scrubber will use sodium hydroxide (NaOH) flowing through a packed tower to provide contact with the digester gas, containing H_2S and CO_2.

The H_2S and CO_2 will be absorbed by the caustic NaOH. The pH of the scrubbing solution will be monitored and NaOH automatically added to the scrubber by a pump to maintain a required pH. The NAOH solution will be replaced periodically.

The gas dryer will utilize a dessicant to remove moisture in the gas stream. The dessicant will be routinely recycled by oven drying.

Storage Comprssors and Spheres

Two 633 cfm low pressure storage compressors will operate automatically to maintain pressure in the two 30-ft. diameter storage spheres between 45 psi and 50 psi. However, the storage compressors will operate only if the suction pressure between the compressors and digesters is between five and six inches water column. This feature, utilizing pressure switches, will prevent the development of a negative pressure in the digestion tanks.

Waste Gas Burner

The waste gas burner will be an open flame-type activated by a pressure regulated valve set to open when pressure increases above six inches water column between the digester covers and the storage compressors. A line will bypass the pressure relief valve to provide constant gas flow to the waste gas burner pilot light.

Turbine Compressor

The high pressure (300 psi) turbine compressors will supply clean, dry digester gas from the storage spheres to the turbine generators. The turbine compressor will be turned off and on by demand from the turbine generators. Additionally, this compressor will automatically shut down because of low pressure in the gas storage spheres and turbine operation will switch to auxiliary fuel.

Turbines and Generators

The gas turbine/generator system will consist of three sets, each with a 650 HP single shaft turbine and a generator rated at 475 KW and 480V, with brushless-type exciter and static voltage regulator. Control panels and starting battery packs will be included with each unit. Although gas turbines are not as fuel efficient as reciprocating engines, they are much more reliable and require less frequent and demanding maintenance. A cutaway drawing of the turbine is shown in Figure 3.

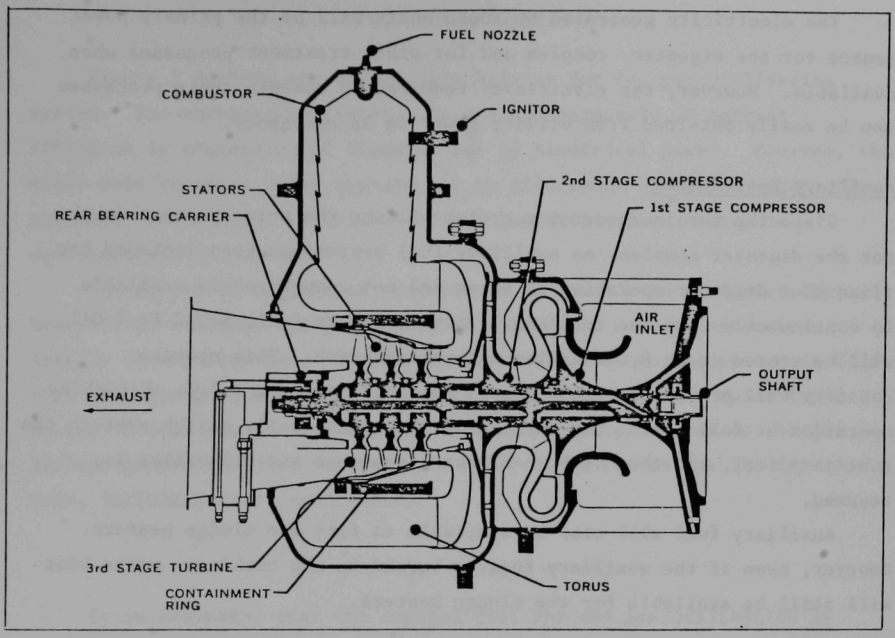

FIGURE 3
CUTAWAY DRAWING
OF GAS TURBINE

(Courtsey of
North American Turbine
Corporation, Houston,
Texas)

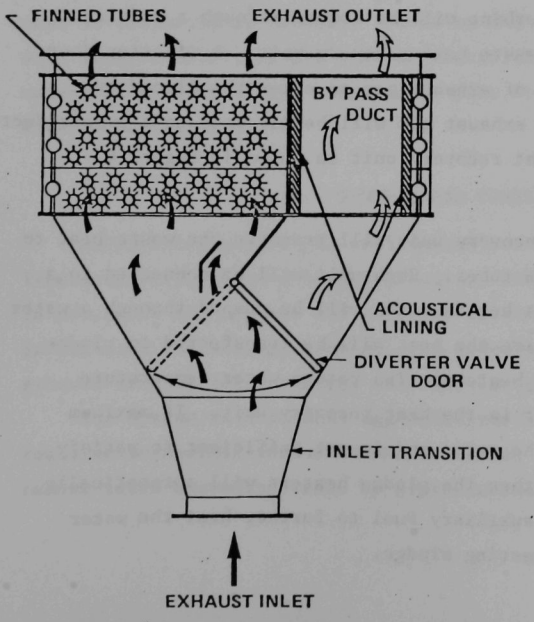

FIGURE 4
WASTE HEAT RECOVERY
UNIT FLOW DIAGRAM

The electricity generated by these units will be the primary power source for the digester complex and for other treatment processes when available. However, the electricity required to operate these processes can be easily obtained from utility power in an emergency.

Auxiliary Fuel System

Since the turbine/generator system will be the primary power source for the digester complex, an auxiliary fuel system has been included for times when digester operation is upset and not enough gas is available to continuously fire the turbines. This auxiliary fuel, No. 2 Fuel Oil, will be stored in an 8,000-gallon underground tank. This storage capacity will provide enough fuel oil for approximately 3 days of turbine operation at full load. The turbines will automatically switch over to the auxiliary fuel, and then back to digester gas when gas production is resumed.

Auxiliary fuel will also be available to fire the sludge heaters. However, even if the auxiliary fuel is burned in the turbines, waste heat will still be available for the sludge heaters.

Waste Heat Recovery Units

Exhaust gases from each turbine will be routed through a modular, finned tube, waterwall design waste heat recovery unit. A diverter door will allow the required amount of exhaust gas to enter the finned tube section. The remainder of the exhaust gas will be diverted to a bypass duct. A flow diagram of the waste heat recovery unit is shown in Fugure 4.

Hot Water Recirculation System

The finned tubes in the recovery unit will transfer the waste heat to water recirculating through the tubes. Each unit will be connected to a 300 gpm centrifugal pump. This heated water will be pumped through a water bath in the sludge heaters, where the heat will be transferred to sludge being recirculated through the heaters. The return water temperature will regulate the diverter door in the heat recovery unit. If maximum heat is being transferred to the water and is not sufficient to satisfy the sludge heat requirements, then the sludge heaters will automatically begin burning digester gas or auxiliary fuel to further heat the water bath, and subsequently the digesting sludge.

4. PERFORMANCE

Figure 5 depicts the energy mass balance for the gas utilization system. The turbines/generator sets are approximately 18 percent efficient in conversion of digester gas to electrical power. However, the waste heat recovery units operate at an efficiency of approximately 57 percent. The overall efficiency of the gas utilization system is approximately 65 percent.

The system is capable of producing considerably more electricity; however, it has been designed to be in continuous service and to run totally on digester gas. Therefore, the power load to the system is less than the average gas production. This means that gas will normally be wasted through the gas burner, but that the connected loads to the gas turbine generators will continuously serve those facilities 365 days per year, including peak requirements.

5. COSTS

It is estimated that the capital cost for the gas utilization by turbines for the production of energy is $607,000 and the annual operating cost is estimated at $28,000. Based on the projected loads the generators will generate $177,000 per year of electricity. This amounts to approximately 12 percent of the total power cost for the City of Knoxville's Waste Water Treatment Facility

6. SUMMARY AND CONCLUSIONS

Anaerobic digester gas utilization at the Knoxville Kuwahee Waste Water Treatment Facility is a highly effective energy conservation process that is adaptable to both new and existing treatment facilities. Digester gas is utilized for mixing, heating, and primary power production. Waste exhaust heat is recovered and used for sludge heating.

The system will produce approximately 12 percent of the total power required to operate this advanced wastewater treatment plant and will reduce total operating costs by approximately $150,000 per year.

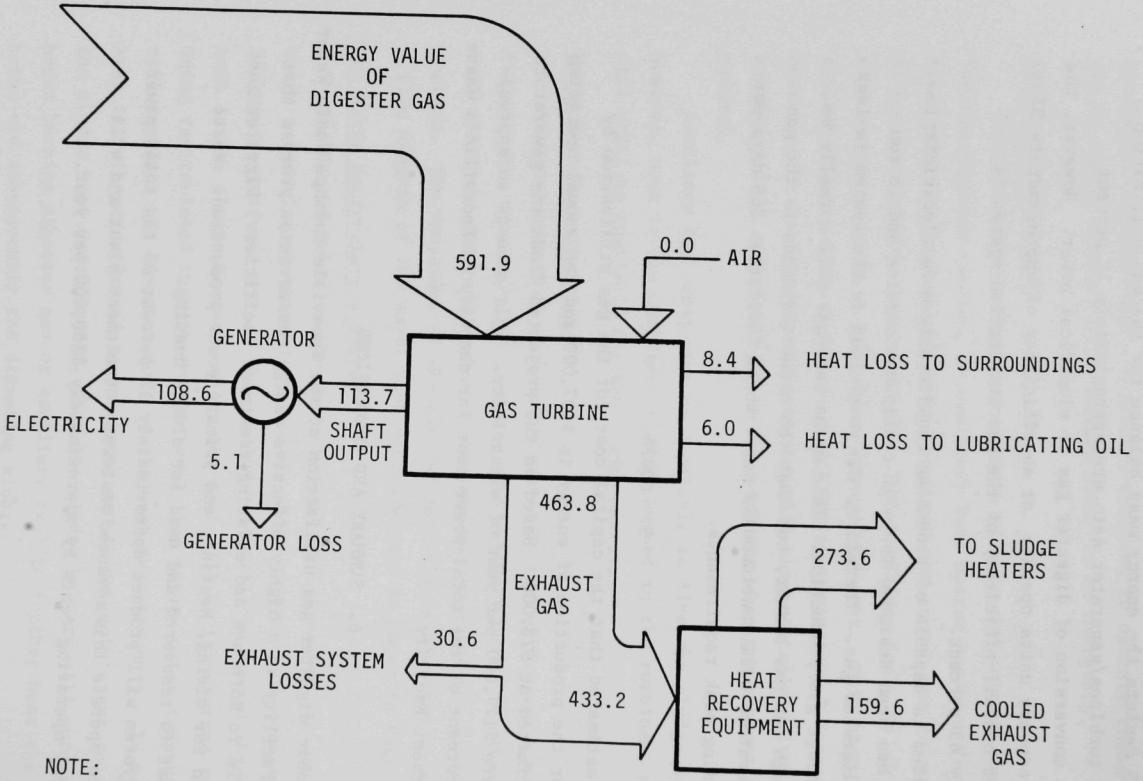

FIGURE 5
ENERGY FLOWSHEET FOR
GAS UTILIZATION SYSTEM

SESSION 10

TRANSMISSION AND CONVEYANCE

Chairpersons: Gary Yoshioka

Peregrine White

MULTIVARIATE ANALYSIS OF ENERGY CONSUMPTION FOR WATER PUMPING STATIONS

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Department of Water and Sewers

1. INTRODUCTION

In most metropolitan areas, water is supplied to the consumer through facilities comprised of treatment plants, pumping stations, reservoirs and distribution networks. The largest proportion of energy consumed in a water utility is usually at the pumping stations. It is at this stage in the water supply process that this study is concerned to analyze energy consumption. Methods are presented to aid the Energy Manager in understanding the factors involved in the energy requirements for pumping water in the aggregate system. Knowledge of the energy consumption patterns and the associated effect of each contributing factor for the water system becomes important for purposes of system planning for conservation and forecasting for anticipated future demand.

The area of the Study was the water system for the City of Chicago. The Chicago Water System serves approximately 4.6 million people in the City and 74 neighboring communities. The system is comprised of two (2) water filtration plants and eleven (11) water pumping stations. Beyond the pumping stations there are no facilities for direct supply water storage except at the points where suburban communities tie into the system. All of the pumps currently being used in Chicago Pumping Stations are centrifugal type with the exception of the Southwest Station which has two stage vertical circulating type pumps.

The pumping stations are classified by types of prime mover used to drive the pumps. Six of the eleven stations use electric motors (Electric Stations) and the remaining five stations use steam turbines (Steam Stations). The majority of boilers at the Steam Stations are gas/oil fired, and two boilers are fueled by coal. Thus, it was necessary to develop two models for energy consumption, one for total Electric Stations and the other for total Steam Stations, since all BTUs are not provided at the same unit cost. An aim of this paper is to develop a predictive model for monthly energy requirements for pumping water, using historical data to develop the model.

The power requirement for a single pump is formulated as a function of quantity of water pumped, total head and efficiencies of the pump and driver used, or:

$$HP = \frac{QH}{3960}(e) \qquad (1)$$

where Q = pumping rate in gpm; H = total dynamic head; and e = combined pump and driver (motor or turbine) efficiency. Functionally stated, Power = f (Q, H,e). Since energy is simply the total power over a time period, a functional relationship for energy can be represented as, Energy = $f(Q_t,H,e)$, where Q_t is the quantity pumped over time period t, and the variables are linear.

The performance of a single pump or multiple pumps at a Station is defined by the associated head-capacity, efficiency-capacity, and horsepower-capacity relationships. The amount of energy necessary to deliver a given quantity of water is specified by the system-capacity function. If the Water System consisted of one pumping station, analysis of energy requirements for pumping would merely consist of finding points on the characteristic pump curves. When multiple plants are used in the System, formulated methods for the aggregate become complex if not impossible.

This Study is directed at those Systems where multiple pumping stations are used. The proposed method for analysis is multiple linear regression.

2. DEVELOPMENT OF MODELS

Factors assumed to have a measurable impact on energy consumption at a pumping station were total pumpage (million gallons), total head (feet), and pumpage efficiency (per-cent). Monthly data for each of these parameters was collected along with the energy consumption information on the utility bills.

The pumpage efficiency is a measure of energy output/energy input, where energy output is measured in million-foot-gallons pumped and converted into million BTUs.[1] Changes in the pumpage efficiency are reflected by the individual efficiencies of pumps, motors, turbines and boilers along with other factors that are characteristic to the operation of the equipment, such as valve throttling at the pump discharge. Energy input is a measure of million-BTUs consumed in this process during the billing period. For energy inputs at

[1] Million-foot-gallons (MFG) is the product of gallons pumped and the corresponding discharge pressure of head.

Electric Pumping Stations, total Mil-BTUs was derived by using the factor 3413 BTU/KWH. With regard to Steam Pumping Stations the heating value of the individual fossil fuels was used for the conversion. This factor accounted, in part, for the wide differences in the efficiencies calculated between Electric and Steam Pumping Stations.

The Total Head figure is the average system total dynamic head generated by the pumping units during the period. Variations in system head are caused by changes in seasonal demand pressures and fluctuations in the reservoir levels at the filtration plants. Both total head and total pumpage for each Station were available directly from records of the Department of Water and Sewers.

The regression models were developed with monthly figures of these variables. All data corresponded to the span of days of the various utility billing periods (i.e. a 33 day billing period would include pumpage for 33 days and not the calendar month). Data from 30 consecutive billing periods was collected, thus, allowing seasonal variations in the operating parameters to be reflected in the construction of the models.

The mean value and standard deviation of the data set for each variable considered resulted as follows:

Variable	Mean	Standard Deviation
Electric Stations		
Energy Consumption (KWHs)	6,273,890.00	860,910.00
Pumpage (Mil. Gals.)	12,800.90	1,709.20
Total Head (Feet)	107.75	2.38
Efficiency (%)	69.19	2.51
Steam Stations		
Energy Consumption (Therms)	2,311,610.00	301,148.00
Pumpage (Mil. Gals.)	18,539.60	2,437.20
Total Head (Feet)	123.06	4.49
Efficiency (%)	10.60	0.47

This information provides perspective concerning the dimension and variability of each parameter recorded.

The two data sets of 30 observations each (N=30) were entered into the multiple regression computer program with energy consumption, the dependent variable, regressed against pumpage, head and efficiency, the independent variables. A stepwise regression was used to analyze the relationships of the data.

The resulting correlation matrix was used to list the independent variables according to significance in explaining the variation in the dependent variable, energy consumption.

Table No. 1

CORRELATION MATRIX - ENERGY CONSUMPTION (ELECTRIC STATIONS)

PARAMETERS	Y	X_1	X_2	X_3
Y: KWH	1	.968	.077	.405
X_1: Pumpage (Q_t)	.968	1	.284	.356
X_2: Efficiency (eff)	.077	.284	1	.390
X_3: Total Head (H)	.405	.356	.390	1

CORRELATION MATRIX - ENERGY CONSUMPTION (STEAM STATIONS)

PARAMETERS	Y	X_1	X_2	X_3
Y: Therms	1	.962	.282	.505
X_1: Pumpage (Q_t)	.962	1	.445	.448
X_2: Efficiency (eff)	.282	.445	1	.649
X_3: Total Head (H)	.505	.448	.649	1

The above Correlation Matrix shows the strengths of the relationship between each independent variable (X_1, X_2, X_3) and the dependent variable (Y). Also, the correlation between the independent variables is shown (i.e. the effect of monthly pumpage on system efficiency, X_1 on X_2).

Pumpage shows a very strong positive linear correlation with energy consumption ($r = .96$) for both Electric and Steam Stations. Also, total head exhibits a moderate positive correlation with energy use. This is expected, since energy consumption has to increase as more pumps are added to the system to increase either pumpage or head. However, the low positive correlation between system efficiency and energy consumption is somewhat unexpected. From the functional relationship for a single pump the efficiency is negatively correlated to energy consumption. Or as efficiency would increase, the energy input should decrease. This low correlation coefficient between efficiency and energy consumption is probably due to a couple of effects when system conditions change.

System efficiency would improve with increasing energy input in the following cases:

1) Output is increased to higher efficiency levels of the equipment in service.

2) Equipment with higher efficiency ratings are utilized.

In either of these cases an increase in energy consumed can be complimented by an increase in efficiency. On the other hand, operation of equip-

ment at points on the pump curves other than the maximum efficiency point will result in an inverse relationship between energy required and equipment efficiency. Both of these conditions appear to exist in the study system.

Six different regression models were used to analyze the relationship between energy consumption and the explanatory variables. Table 2 provides a summary of results from the stepwise technique.[1]

Single Variable Models - When pumpage alone was regressed against energy consumption $E = f(Q_t)$ the coefficient of determination, R^2, for both Electric and Steam Stations was very high (i.e. for Electric Stations $R^2 = 0.93$ or 93 per cent of the total variation in energy consumption was explained by the pumpage). The high F-values are used to conclude that the model is significant at the 0.01 confidence level. However, a plot of residuals against time and the statistical test based on the Durbin-Watson statistic were used to conclude that the residuals were autocorrelated. Correlation in the error terms suggests that there is additional explanatory information in the data that has not been exploited in the model, such as significant variables missing.[2]

Two Variable Models - Total head was selected to be included as the second independent variable $E = f(Q_t,H)$. This action did very little to increase R^2, and serial correlation in the error terms was not removed. The intercorrelation between pumpage and head was insignificant. Thus, it was concluded that this model was even less desirable than the single variable model.

Three Variable Models - Overall system pumpage efficiency was included as the last step in this procedure. Thus, the model assumed the form $E = f(Q,H,eff)$. The resulting R^2 values increased to 0.99 for both models. The F-value for the model and the resulting t-values for each regression coefficient was statistically significant at the 0.01 confidence level. In addition, the inclusion of efficiency eliminated the serial correlation which affected the one and two variable models. The only limitation was the moderate multicollinearity that existed among the variables in the Steam Stations Model. This result affects the interpretations of the individual regression coefficients but does not alter the value of the complete model.

From Table 2, the estimated linear equation for the three variable model was:

Aggregate Electric Stations (6 Stations)

$$KWHs = 95{,}538 + 497.6(Q_t) + 56{,}587.6(H) - 90{,}885.5(eff) \qquad (2)$$

TABLE 2
COMPARISON OF REGRESSION RESULTS

ALL ELECTRIC STATIONS

	$E = f(Q_t)$	$E = f(Q_t,H)$	$E = f(Q_t,H,eff)$
N	30	30	30
B_0	30,613	-2,485,410	95,538
B_1	487.7	475.4	497.6
B_2	-	24,813.9	56,587.6
B_3	-	-	-90,885.5
F	420.7	218.0	18,000.0
R^2	0.93	0.94	0.99
S.E.	218,874	215,443	19,945
d	0.47	0.39	2.06
x^2	-	3.74	8.97

ALL STEAM STATIONS

	$E = f(Q_t)$	$E = f(Q_t,H)$	$E = f(Q_t,H,eff)$
N	30	30	30
B_0	105,728	-559,566	40,898
B_1	118.9	113.9	122.1
B_2	-	6,174.3	19,125.5
B_3	-	-	-221,314
F	356.8	191.0	6,075.0
R^2	0.92	0.93	0.99
S.E.	82,672	80,181	12,004
d	1.27	1.39	1.84
x^2	-	6.17	22.4

N - Number of observations - monthly energy bills
B_0 - Constant term
B_1 - Coefficient for pumpage variable
B_2 - Coefficient for total head variable
B_3 - Coefficient for efficiency variable
F - Statistic used to test significance of the model
R^2 - Coefficient of determination
S.E. - Standard error of the estimate for the model
d - Durbin-Watson statistic
x^2 - Chi-square statistic to test multicollinearity of variables

Table 3. PERFORMANCE OF MODELS

Basic Equation: Y (KWHs or Therms) = $B_0 + B_1 (Q_t) + B_2$ (Head) + B_3 (Eff)

		ENERGY CONSUMPTION	
		Electric Stations (KWHs)	Steam Stations (Therms)
JULY	Actual	6,953,280	2,368,945
	Calculated	7,000,685	2,367,900
	|Act. - Cal.|	47,405	1,045
	% Diff.	0.68	0.04
AUGUST	Actual	7,026,400	2,371,526
	Calculated	7,059,474	2,369,515
	|Act. - Cal.|	33,074	2,011
	% Diff.	0.47	0.08
SEPT.	Actual	6,842,800	2,229,238
	Calculated	6,868,182	2,218,350
	|Act. - Cal.|	25,382	10,888
	% Diff.	0.37	0.49
OCT.	Actual	5,523,200	2,181,104
	Calculated	5,474,899	2,186,025
	|Act. - Cal.|	48,301	4,921
	% Diff.	0.87	0.23
NOV.	Actual	6,117,695	1,992,868
	Calculated	5,931,282	2,004,504
	|Act. - Cal.|	186,680	11,636
	% Diff.	3.05	0.58
DEC.	Actual	5,624,191	1,904,509
	Calculated	5,613,456	1,907,560
	|Act. - Cal.|	10,735	3,051
	% Diff.	0.19	0.16
Root Mean Square Error		82,948	6,983
Mean Absolute % Error		0.94	0.26

Aggregate Steam Stations (5 Stations)

Total Equivalent Therms = 40,898 + 122.1(Q_t) + 19,125.5(H) - 221,314(eff) (3)

Table 3 shows a comparison of the models to the actual results during a six-month period. The essence of the procedure is to develop the models based on a data set and then validate the models on subsequent samples. Overall, each model performed quite well using the independent data sets.

3. USING THE MODELS

After the model has been developed and tested, the next stop is to put it to use. Such a model can provide reasonably accurate answers to such questions as : What are total KWHs or total therms expected to be at various levels of system pumpage and average Station discharge head? What effect on energy consumption can we expect by making minor changes in system total head? What are the probabilities associated with energy costs at various system pumpages and utility rates charges by the Gas and Electric Companies?

If the regression coefficients in the model are accurate and precise (i.e. no serial correlation with the residuals or multicollinearity with the independent variables), then these estimators represent the marginal effect on the dependent variable by a unit change in its independent variable, while holding the other independent variables constant. For example, holding total head and pumpage constant for the aggregate of Electric Stations in the study model, an increase in system efficiency of 1% would result in a decrease in energy consumption by 90,885 KWHs per month. The limitation to marginal analysis of regression coefficients can be seen in the model for Steam Stations. Since moderate multicollinearity exists between the independent variables in the model, the associated regression coefficients may be over or understated. However, the coefficients given for the Steam Station Model are good approximations of the true marginal effect on unit changes in the independent variables.

In addition to marginal analysis, the results of the regression models could serve as inputs in an Energy Simulation Model. Simulation is an approxmation tool that determines projected results subject to a margin of error. This technique has been used quite successfully by Financial Analysts in projecting cash balances [3], and could be adapted to energy requirements and costs when preparing the energy budget for pumping needs.

A short range (12-month) energy simulation could be sketched out as follows:

(1) Using historical data, determine the expected value for each variable in the regression model and a measure of variation about that value for each monthly period.

(2) Plug these values into the regression equation to project a range of energy requirements to which relative probabilities could be attached.

(3) To determine energy costs, attach relative probabilities to anticipated unit energy costs (¢/Therm or ¢/KWH) from the Utility Companies and combine with the projected energy requirements.

(4) The resulting monthly values are summed for the 12-month forecast resulting in a budget to which probability statements could be attached (i.e. Optimistic, Pessimistic and Most Likely Costs for energy).

These are the basic steps necessary to construct a budget using Simulation Methods. This technique also allows the Energy Manager to experiment with different operating policies (i.e. shifting pumpage from Steam to Electric Stations or increasing or decreasing average system head) to determine impact on energy consumption and overall cost.

4. CONCLUSION

The purpose of this study was to develop and test a model for predicting energy requirements for pumping water at a plant or set of plants. The application of multiple linear regression was found to be a simple and accurate tool in forecasting energy requirements for pumpage. The three (3) variable models resulted in equations which contained statistically significant variables and were highly accurate and reliable in explaining energy consumption. Testing the models using an independent data set for 6 months resulted in predictions for energy consumption that averaged more than 99% of the actual consumption during the periods. This accuracy though uncommon in statistical analysis can occur when time series data is used to develop the model.

Stepwise Regression techniques were used to illustrate that both the single and two variable models resulted in high serial correlation of the residual terms. This was an indication that a significant independent variable was missing from the regression model even though the coefficient of determination was very high (R^2 = .94 for the one variable model). Thus, from a statistical point of view, the addition of Total Dynamic Head and System Efficiency to the System Pumpage as independent variables were justified. This small increase

in explanatory value of the three variable model was significantly complemented by the fact that the serial correlation was removed. In addition, the level of correlation between the independent variables for the Electric Stations Model was insignificant from the results of the Chi-Square test for orthogonality of these variables. There was moderate correlation between the independent variables for the Steam Stations Model.

These findings suggest that energy consumption for an aggregate system of plants can be analyzed using methods such as these to produce quick and reasonably accurate forecasts. The modeling concept could be applicable to both large and small water supply systems.

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OPTIMUM ENERGY COST DESIGN OF PUMPING SYSTEMS

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INTRODUCTION

A significant amount of energy is spent in transporting water and wastewater through pipelines in water and wastewater management for municipal and industrial applications. With the increase of energy cost in recent years and prospective increases in the future, energy cost consideration in optimizing a pumping system has gained recognition.

In the design of a pumping system, the costs of the system consist of the cost of pumps, pipes, operation and maintenance (O & M), and energy. The size of pipeline is an important factor in the whole pumping system. Smaller diameters of pipeline will result in large friction heads and more energy. Again, with the increase of pipe size, energy cost will be decreased, but the cost of pipelines will be increased. Cost benefits from saving energy will be gradually reduced with the increase of pipe size and will reach an optimum size beyond which energy cost is not sensitive to the total cost of the system.

This paper develops a method for the direct least-cost solution of pipe size in a pumping system, consisting of pumps and transmission pipeline, for a known-flow condition, incorporating various cost functions.

MODEL FORMULATION

In developing a mathematical model of a pumping system, the total cost is divided into two parts: capital costs and O & M costs (i.e., maintenance and energy costs).

Total capital cost of an installed pipeline (including laying, jointing, etc.) can be expressed as a function of diameter: [1,2,3]

$$y_1 = k_1 D^{m_1} \qquad (1)$$

where: y_1 is the capital cost of the pipeline
D is the diameter of the pipeline
k_1 is a coefficient
m_1 is an exponent of the cost function

On the basis of average 1976 cost data, when y_1 is expressed in dollars/ft length and D is in inches, the value of $k_1 = 1.01$ and $m_1 = 1.29$.

The capital cost of an installed pump was expressed as a function of flow and total head:[5]

$$y_2 = k_2 H^{m_2} Q^{m_3} \quad (2)$$

where: y_2 is the capital cost of the installed pumps in dollars
k_2 is the coefficient
Q is the flow rate in gpm
H is the total head in feet (it is equal to the summation of static and friction heads)
m_2 and m_3 are exponents of the cost function

On the basis of average U.S. cost data updated to 1976, the values of $k_2 = 16.14$, $m_2 = 0.642$, and $m_3 = 0.453$.

The annual cost of energy is related to the unit cost of energy, operating head, and flow rate; indirectly, it is related to the horsepower of the pump. The horsepower (hp) of the pump can be expressed when transporting water as:

$$HP = \frac{Q (H_s + H_f) 8.33}{33000 E_p} \quad (3)$$

where: Q is the flow in gallons per minute
H_s is the static head in feet
H_f is the friction head
E_p is the pump efficiency

Friction water head, H_f, is dependent on the flow through the pump, the pipeline diameter, and the length of the pipeline. The relationship between friction head loss in the pipe and the flow through it can be given by many formulas. For water supply, the Hazen-Williams equation is widely used and can be given as:

$$H_f = \frac{L Q^p}{0.0955 C^p D^q} \quad (4)$$

where: H_f is the friction head loss in feet
L is the length of pipeline in feet
Q is the flow in gallons per minute
C is the Hazen-Williams coefficient
D is the diameter of the pipeline in inches
$p = 1.85$
$q = 4.86$

he value of the Hazen-Williams coefficient, C, is not constant; for a ew cast iron pipe, the value of C may be about 130, and for old pipe = 100. For the calculation of design horsepower, hp, of the pumps, t is rational to consider the value of C at the end of the design life (that is, C = 100 for cast iron pipe). For other pipes, appropriate values of C should be taken.

For the calculation of annual electrical energy cost, the average of the C values during the design period may be taken. If a head loss equation other than the Hazen-Williams equation is used, the C value will be replaced by the appropriate friction coefficient. Combining Equations (2) and (4), the capital cost of pumps can be expressed as:

$$y_2 = k_2 Q^{m_3} \left[H_s + \frac{L Q^p}{0.0955 C^p D^q} \right]^{m_2} \tag{5}$$

Average annual energy cost can be calculated using the equation for horsepower of the pump (Equation 3) as:

$$Y_3' = \frac{0.746 \, Q \, 24 \, c \, 365 \, (8.33) \, (H_s + H_f)}{33000 \, E_m \, E_p} \tag{6}$$

where: c is the cost of energy per kW-hr
E_m is the mechanical efficiency of the pump

Combining the Hazen-Williams equation for head loss, Equation (4), with Equation (6), the annual energy cost Y_3' can be expressed as:

$$Y_3' = \frac{0.746 \, Q \, 24 \, c \, 365 \, (8.33)}{33000 \, E_m \, E_p} \left(H_s + \frac{L Q^p}{0.0955 C^p D^q} \right) \tag{7}$$

The operation and maintenance other than energy cost can be assumed to be f times the energy cost. Thus, the total annual O & M cost, including energy cost, can be written as:

$$Y_3 = \frac{(1 + f) \, (0.746) \, Q \, 24 \, c \, 365 \, (8.33)}{33000 \, E_m \, E_p} \left(H_s + \frac{L Q^p}{0.0955 C^p D^q} \right) \tag{8}$$

For water pumping systems, the value of f has been suggested by Linaweaver[1] as equal to 0.08. To obtain the total annual cost of the system, the capital costs are converted into annual capital recovery cost considering an interest rate of i and repayment period of n years (useful life of equipment) and added to the annual maintenance and operation costs. If the capital is to be paid in equal annual payments

during the life period (n years) of the equipment at an interest rate of i, and considering s as the salvage value of the equipment at the end of the useful life as a ratio of actual value, the annual capital recovery factor is expressed as:

$$R = \frac{i(1+i)^n}{(1+i)^n - 1}(1-s) + is \tag{9}$$

where R is the annual capital recovery factor.

Considering the useful life of a pipeline as N_1 years, a salvage value factor s_1, rate of interest i, and annual capital recovery R_1, then the initial cost of the pipeline Y_1 can be obtained by combining Equations (1) and (9):

$$\begin{aligned} Y_1 &= y_1 L R_1 \\ &= k_1 D^{m_1} L \left[\frac{i(1+i)^{n_1}}{(1+i)^{n_1}-1}(1-s_1) + i s_1\right] \end{aligned} \tag{10}$$

Similarly, the initial cost of pumps can also be converted to annual capital recovery cost by combining Equations (5) and (9).

$$\begin{aligned} Y_2 &= y_2 R_2 \\ &= k_2 \left[\frac{i(1+i)^{n_2}}{(1+i)^{n_2}-1}(1-s_2) + i s_2\right] Q^{m_3} \left(H_s + \frac{L Q^p}{0.0955 C^p D^q}\right)^{m_2} \end{aligned} \tag{11}$$

Now, the total annual cost of the pumping system is the summation of Y_1, Y_2 and Y_3, which can be obtained from Equations (ü), (10), and (11) as follows:

$$\begin{aligned} Y &= Y_1 + Y_2 + Y_3 \\ &= R_1 k_1 D^{m_1} L \\ &\quad + R_2 k_2 \left[Q^{m_3} \left(H_s + \frac{L Q^p}{0.0955 C^p D^q}\right)^{m_2}\right] + \\ &\quad + \frac{(1+f)(0.746) Q\ 24\ c\ 365\ (8.33)}{33000\ E_m\ E_p}\left[H_s + \frac{L Q^p}{0.0955 C^p D^q}\right] \end{aligned} \tag{12}$$

Equation (12) can be rewritten as:

$$Y = R_1 k_1 D^{m_1} L + R_2 k_2 \left[Q^{m_3}\left(H_s + \frac{XL}{D^q}\right)^{m_2}\right] + k_3 F \left(H_s + \frac{XL}{D^q}\right) \quad (13)$$

where:

$$F = \frac{Q}{33000\, E_p}$$

$$X = \frac{Q^p}{0.0955\, C^p} \text{ and } k_3 = \frac{(1+f)(0.746)\ 24\ c\ 365\ (8.33)}{E_m} = \frac{(1+f)\ 54436\ c}{E_m}$$

Equation (13) represents the total annual cost of the pumping system. For a given flow Q, length L, and static pumping head H_s, the total annual cost of the pumping system can be obtained in terms of pipeline diameter using Equation (13).

To obtain the optimum diameter of the pipeline, Equation (13) can be differentiated with respect to diameter D. Equating this to zero yields:

$$\frac{dY}{dD} = R_1 k_1 L m_1 D^{m_1-1} - R_2 k_2 m_2 \left[Q^{m_3}(H_s + \frac{XL}{D^q})^{m_2-1}\right](q X L D^{-(q+1)}) - q F k_3 XL D^{-(q+1)} = O \quad (14)$$

By solving Equation (14), the optimum diameter of the pipeline of the pumping system can now be obtained. But Equation (14) can be solved only by trial and error. By sensitivity analysis, however, it was found that the second term on the right-hand side of Equation (14) does not have a significant effect on the value for optimum diameter. Therefore, the second term of Equation (14) can be neglected to obtain the optimum diameter of the pipeline:

$$\frac{dY}{dD} = R_1 k_1 L m_1 D^{m_1-1} - q F k_3 XL D^{-(q+1)} = O \quad (15)$$

The value for optimum diameter obtained from the simplified equation, i.e., Equation (15), is not significantly different from the value for optimum diameter achieved through the trial-and-error solution of Equation (14). Using the optimum diameter from Equation (14) or Equation (15) will not affect the final selection of pipe size because pipe available commercially comes only in fixed sizes.

Equation (15) can now be solved for diameter D:

$$D = \left[\frac{q\, F\, k_3\, X}{R_1\, k_1\, m_1}\right]^{\frac{1}{m_1 + q}} \tag{16}$$

Replacing the values of F and X and expressing in terms of flow Q, Equation (16) can be rewritten as:

$$D = K\, Q^{(p\,+\,1)/(m_1\,+\,q)} \tag{17}$$

where:

$$K = \left[\frac{q}{R_1\, k_1 m_1}\ \frac{1}{33000\, E_p}\ \frac{k_3}{0.0955\, C^p}\right]^{\frac{1}{m_1 + q}} \tag{18}$$

Equation (17) results in an optimum diameter of a pipeline of a pumping system, a diameter which will produce the minimum total cost (capital and O & M) of the system.

The theoretical horsepower of the pump, obtained using Equation (3), will be converted to design horsepower by multiplication of a stand-by factor which can be determined from the following relationships.[6]

$$\left.\begin{array}{rl} Q' \leq 2.0 & AJ = 2.08 - 0.18Q \\ 2.0 < Q' \leq 5.0 & AJ = 1.9666 - 0.1233Q \\ 5.0 < Q' \leq 10.0 & AJ = 1.42 - 0.014Q \\ 10.0 < Q' \leq 20.0 & AJ = 1.30 - 0.002Q \\ 20.0 < Q' & AJ = 1.25 \end{array}\right\} \tag{19}$$

where Q' = flow in million gallons per day.

SOLUTION

To illustrate the applicability of the method of analysis developed and the sensitivity of energy cost on pumping systems, one example with the following data is considered: flow rate = 1,500 gpm of water; length of pipe = 6,000 ft; static head to be pumped = 160 ft; rate of interest i = 0.06; useful life of pipeline, n_1 = 30 yrs; useful life of pumps n_2 = 15 yrs; salvage value ratio for pipeline and pumps = 0.1; Hazen-Williams coefficient = 100; p = 1.85; q = 4,86; E_p = 0.8; E_m = 0.8; energy cost, c = \$0.03 and \$0.10/kw hr; additional cost of maintenance as a fraction of energy cost, f = 0.08; coefficients and exponents of cost functions k_1 = 1.01, m_1 = 1.29, m_2 = 0.642, m_3 = 0.453, k_2 = 16.14.

Using these data in Equation (12), the annual costs of pipeline (Y_1), pumps (Y_2), and energy and maintenance cost (Y_3) have been calculated for various values of pipeline diameter. Using Equation (16), the least-cost diameter of the pipeline of the pumping system has been obtained as 13.8 inches.

For a water transportation system using pumps and pipeline, the optimum diameter of a pipeline is found to be proportional to $Q^{(p + 1)/(m1 + q)}$ (Equation 17). For p = 1.85, m_1 = 1.29 and q = 4.86, the optimum diameter is proportional to $Q^{0.463}$. Equation (16) has also been solved for various interest rate (i) values, and the optimum diameter of a pipeline is found to decrease with the increase of interest rate. The theoretical horse-power required, calculated by using Equation (3), is 17.78; to obtain the design horsepower, this is multiplied by the stand-by factor: 1.7 x 17.78 = 30.21 hp.

ENERGY COST SENSITIVITY

Annual costs of pipeline (Y_1), pumps (Y_2), energy and maintenance of (Y_3), and total cost of the system (Y) have been plotted with size of pipeline in Figure 1. It is apparent that the total cost of the system decreases initially with the increase of pipeline diameter, reaches a minimum cost, and then increases with the increase of pipeline diameter. Variation of annual energy consumption of the pumping system with the diameter has also been plotted in Figure 1.

A sensitivity analysis of cost of energy on the total cost of a pumping system and on the optimum size of the pipeline has been made. In the example discussed, when considering the cost of energy as \$0.10/kwh instead of \$0.03/kwh, solutions of the system have been made for various sizes of pipeline and the result has also been plotted in Figure 1. It can be seen that additional costs of energy due to change of unit energy cost from \$0.03 to \$0.10/kwh are more at lower pipe sizes and less as the size of pipe increases. Since the cost of pipe is not affected by the change of energy cost, in order to balance the rate of change of energy cost with the rate of change of pipe cost, a larger pipeline will be required at higher energy prices.

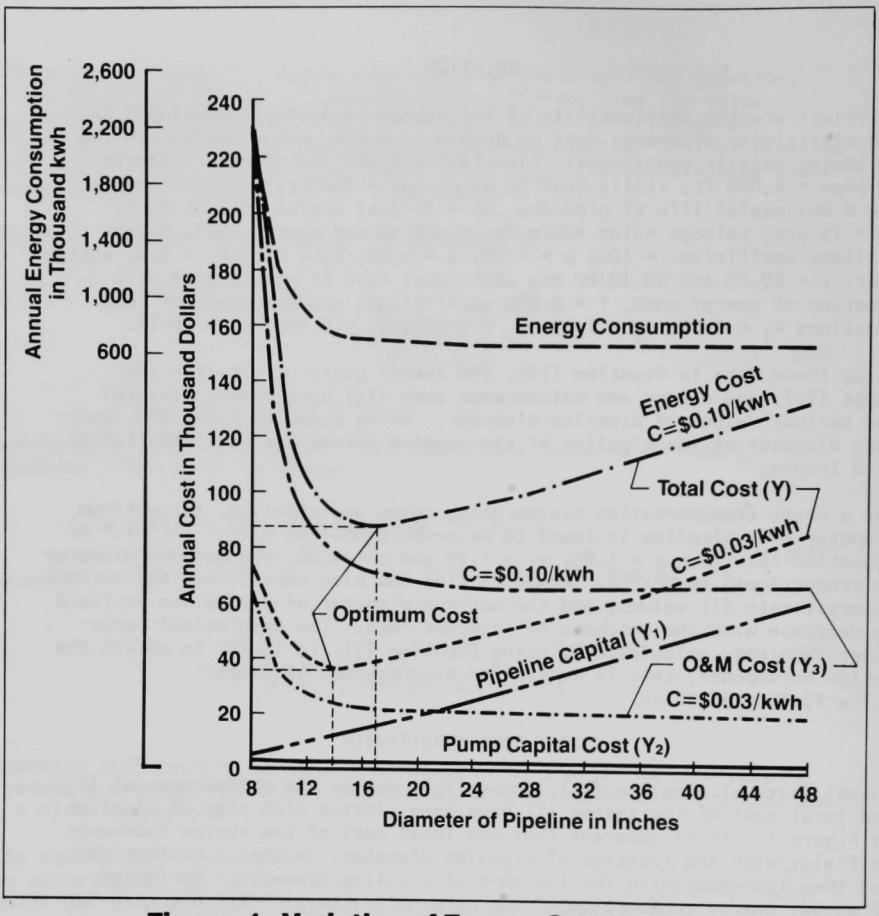

Figure 1 Variation of Energy Consumption and Costs With Pipeline Diameter

Using Equation (16), it can also be shown that the optimum diameter of a pipeline is proportional to $c^{1/(m_1 + q)}$ or $c^{0.16}$. If the unit energy cost changes from \$0.03 to \$0.10/kwh, then the optimum size of the pipeline in the example changes from 13.8 inches to 17 inches. Thus, for a least-cost system, the pipe size would be increased with the increase of the energy cost.

The rate of change of energy cost with the change of pipe size has been plotted in Figure 2. It is clear from Figure 2 that the rate of change of energy cost per unit increase of pipe size is very high at lower pipe sizes and decreases very rapidly and stabilizes with higher diameters of pipe. Therefore, the advantage of energy cost savings by changing the pipe size is high at lower pipe sizes, and there is no appreciable energy cost savings at higher pipe sizes.

SUMMARY AND CONCLUSIONS

A method of pipe size optimization in a pumping system for known flow has been developed, incorporating various cost functions. The cost of energy has been explicitly incorporated in the objective function of the system, and the sensitivity of energy cost on the total cost of the system and on the optimum size of the pipeline has been studied. The method outlined herein is simple and can be efficiently used for optimum energy cost design of a pumping system. However, it is always advisable to check the validity of the model under circumstances of application. In this connection, attention should be given to the following points:

1. The coefficients and exponents of the cost functions used to derive the mathematical model should be valid for the locality of use.

2. Values of variables such as interest rate, useful life of equipment, salvage value, and head loss equation coefficient should be properly selected.

3. When pumping fluids other than water, appropriate head loss equation, viscosity, and density values should be taken.

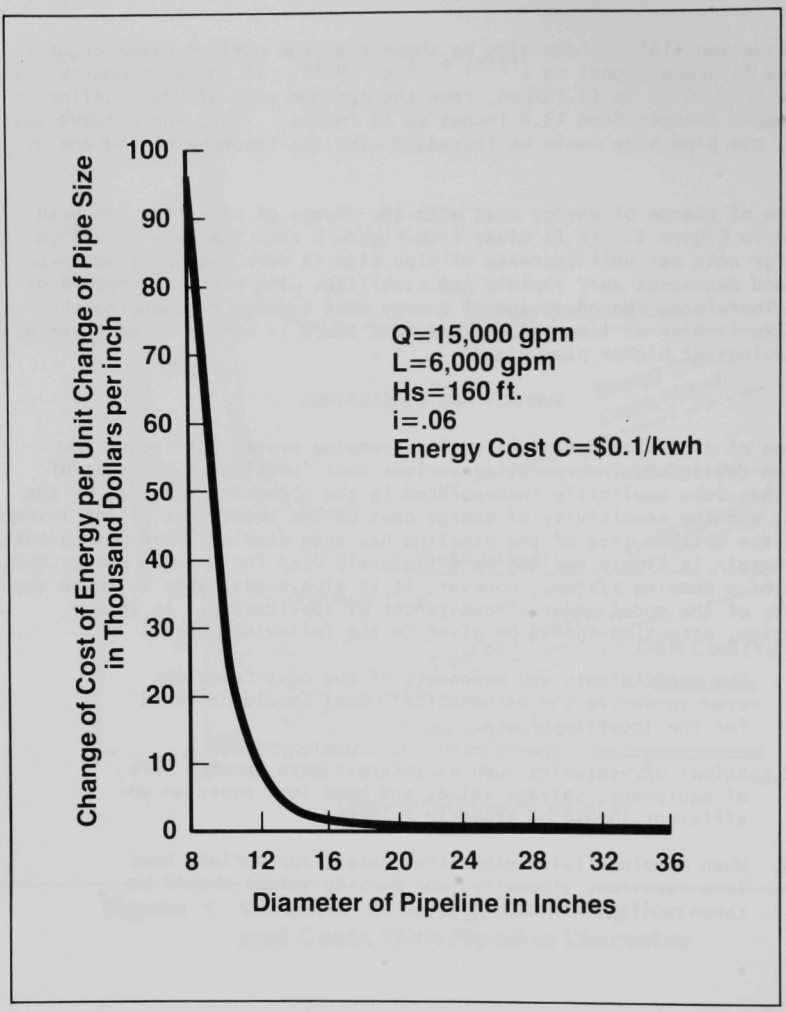

Figure 2. Variation of Rate of Change of Energy Cost With Pipe Size

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MAKING WASTEWATER PUMPING MORE ECONOMICAL

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Because energy was both plentiful and relatively cheap prior to 1973, only moderate consideration was given to its cost when designing systems, particularly pumping systems. The main concern in those times was investment cost. Now, however, energy conservation has become a priority item and, with increasing frequency, higher capital investment in more efficient equipment can be justified and materially offset by reduced running costs over the life of the system. With energy costs now a significant portion of the operating costs, meaningful economical analysis must be applied to take this into account and investment decisions should be based on the lowest total cost estimate.

This concept is illustrated by examining various pumping systems in general and calculating the energy costs for a typical application.

BASIC COMPARISON OF PUMP RUNNING COSTS BASED ON SYSTEM CURVE

Both one time capital expenses (investment costs) and accumulative energy expenditures (running costs) will depend on the number and relative sizes (capacities) of pumps in a station where inflow varies between "average" and "peak." To present the problem as simply as possible--we will exclude standby pumps (required for safety and reliability) and disregard extra requirements (such as units added to handle stormwater) and examine a typical pumping station with flow requirements varying between average and maximum (peak) and the ramifications of selecting pumps for either of these two specific conditions.

The basic energy equation of pumping is

$$\text{Power} = \frac{\text{Head x Quantity}}{\text{Efficiency}}$$

Assuming their efficiencies to be equal for comparison between pumps, power is proportional to Head and Quantity.

$$\text{Power} = \text{Head x Quantity}$$

or, introducing the time element to express accumulated energy use.

$$\text{Power Consumption} = \text{Head x Quantity x Time}$$
$$\text{KWH} = H \times Q \times T \qquad (1)$$

Let's present now average flow (Q_A) at average head (H_A) versus max. flow (Q_M) at max head (H_M) on a typical system head curve:

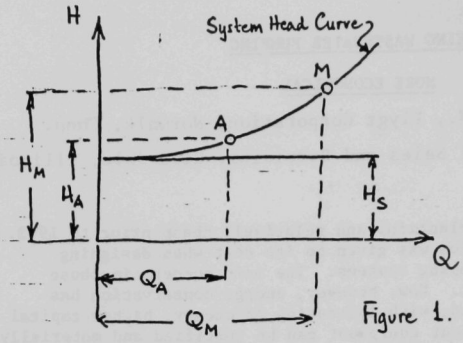


Figure 1.

Q_A Average Flow
Q_M Maximum (Peak) Flow
H_S Static Head
H_A System Head at Q_A
H_M System Head at Q_M

To handle these two conditions with the minimum number of pumps (excluding standby and stormwater requirements) we have two basic choices:

(1) two units - one smaller size to handle average flow and one larger size to handle max. flow. or

(2) one unit large enough to handle <u>both</u> max. flow and average flow.

Assuming now that the smaller pump is rated at P_A horsepower and the larger pump at P_M horsepower and their respective running times are T_A and T_M, the power consumptions, at equal efficiencies, will be

$$\text{KWH Pump}_A = Q_A \times H_A \times T_A \qquad (2)$$

$$\text{KWH Pump}_M = Q_M \times H_M \times T_M \qquad (3)$$

There is, however, an inverse relationship between pump capacity and running time--it takes longer to handle the same amount for a smaller capacity pump than for a larger capacity pump and this relationship is not only inverse but also proportional, in our case:

$$T_A = T_M \times \frac{Q_M}{Q_A} = \frac{T_M \times Q_M}{Q_A} \qquad (4)$$

Please note, that head (H) does not enter into this relationship, but substituting $\frac{T_M \times Q_M}{Q_A}$ for T_A in Equation (2) we obtain

$$\text{KWH Pump}_A = \frac{Q_A \times H_A \times T_M \times Q_M}{Q_A} = H_A \times T_M \times Q_M \qquad (5)$$

Now comparing Equations (5) and (3), we find that the ratio between the power consumptions of $Pump_M$ and $Pump_A$ is

$$\frac{KWH\ Pump_M}{KWH\ Pump_A} = \frac{Q_M \times H_M \times T_M}{H_A \times T_M \times Q_M} = \frac{H_M}{H_A}$$

Conclusion No 1: The specific power consumption (KW/GPM) of the larger pump ($Pump_M$) is higher than that of the smaller pump and the proportionality is identical with the ratio between their operating heads.

Conclusion No. 2: For the same reason, as the ratio $\frac{H_M}{H_A}$ increases, it becomes more and more disadvantageous to run larger pumps at lower than optimum capacity.

Conclusion No. 3: The ratio $\frac{H_M}{H_A}$ will primarily depend on the shape of the System Head Curve, see Fig. 1 and the following three different cases:

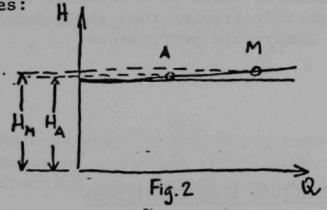


Fig. 2

Flat System Head Curve, practically all static and no friction head. H_M/H_A ratio close to 1.0 Static head 60 - 100% of H_M

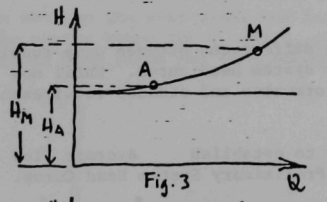


Fig. 3

Moderately steep ("average") System Head Curve with both static and friction head H_M/H_A ratio between 1.33 and 1.66 Static Head 30 - 50% of H_M

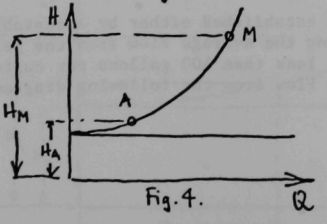


Fig. 4.

Steep System Head Curve, almost entirely friction head. H_M/H_A ratio above 1.66 Static Head less than 30% of H_M

Conclusion No. 4: With a steep system curve (pipe friction losses predominant) it would be very uneconomical to use an oversized (max-flow) pump, in such cases it is imperative that a pump of proper size is selected to take care of the average flow.

Based on the preceding we can now establish two main rules to use as a guide in the primary process of pump selection.

Rule #1: If the Static Head (H_S) is more than 60% of the anticipated Maximum Head (H_M) on the System Head Curve, a pump sized for Maximum Flow (Q_M) is economically acceptable to handle both Maximum and Average Flows and therefore all units (standby, stormwater, etc.) can be of the same size to minimize number of units and to maximize interchangeability.

Rule #2: If the Static Head (H_S) is less than 60% of the anticipated Maximum Head (H_M) on the System Head Curve, it is imperative that at least one pump is selected to handle Average Flow (Q_A) and one to handle Maximum Flow (Q_M). In such a case the pumps will be of different size.

Comment: The above selection process will result in multi-pump constant speed systems with very high economical (running cost) efficiencies. Only extremely steep system curves (pipe friction losses dominant) can justify varispeed pumping systems in terms of operation costs. However, the investment costs of a variable speed system will in most cases be considerably greater than those for a constant speed multipump system of comparable performance.

SELECTION OF PUMPS FOR ACTUAL FLOW CONDITIONS

In the preceding we pointed out the differences between pump running costs depending on the shape of the system head curve. Until now we have deliberately disregarded stormwater and standby requirements and we will continue to do so.

To proceed with actual data we have to establish Average Flow (Q_A) and Maximum Flow (Q_M) and plot the Preliminary System Head Curve.

The Average and Maximum Flow can be established either by acceptable actual measurements or by calculating the Average Flow from the per capita flow (recommended value not less than 100 gallons per capita per day) and estimating the Maximum Flow from the following diagram:

Ratio of Extreme Flow to Daily Average Flow

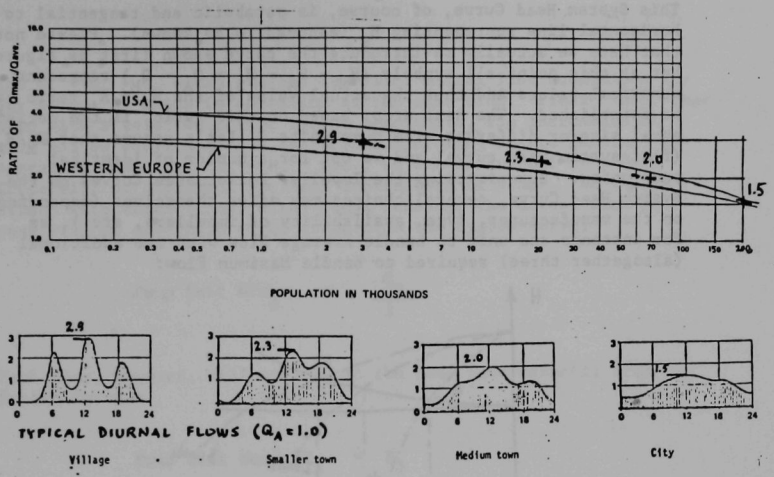


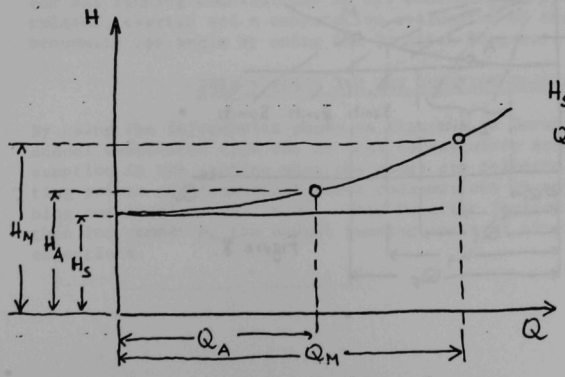
Fig. 5

Multiplying the Average Flow (Q_A) value by the $\frac{Q_M}{Q_A}$ ratio obtained from this diagram, we arrive now at the Maximum Flow (Q_M) value:

$$Q_M = R \times Q_A$$

The next step is sizing the sewer piping based on this Q_M value and observing standards and minimum requirements set by established engineering practice and regulatory agencies.

As soon as the pipe sizes and lengths (system parameters) are available, the system heads for Q_A and Q_M can be calculated and a Preliminary System Head curve can be plotted:



H_S, H_A and H_M in feet,
Q_A and Q_M in gpm.

Figure 6.

This System Head Curve, of course, is parabolic and tangential to the horizontal line representing H_S (assumed to be known). Please note that here we actually re-introduce the curve shown first as Figure 1 but at this point all symbols (H_M - H_A - H_S - Q_A - Q_M) represent numerical values and thus the actual value of the H_M/H_A ratio can be established. The next step, based on this ratio, is the decision--equal size or different size pump units ? Let's assume that $H_M/H_A < 1$. (flat system head curve) and we opt for a number of identical pump units. Superimposing the Impeller Performance Curves on the System Head Curve, several choices may offer themselves (depending on the manufacturer, type, availability of impellers, etc.), as for instance one unit to handle Average Flow with two additional (altogether three) required to handle Maximum Flow:

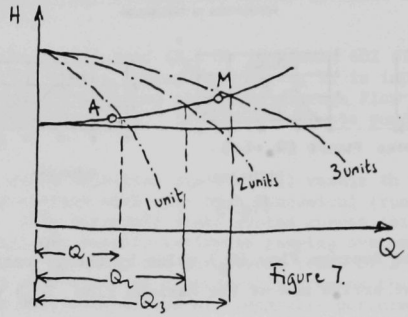


Figure 7.

or two units in parallel to handle Average and three more (altogether five) to pump Maximum Flow:

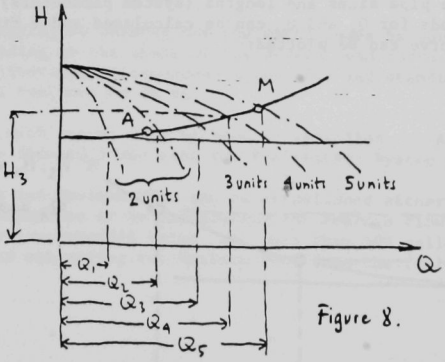


Figure 8.

The Q_1 - Q_2 - Q_3 - etc. values represent actual flow conditions for any number of units in operation at the same time. Please note, that even with a relatively flat System Head Curve, the increments of Q will not be equal and thus the actual operating points (defined by operating heads H_1 - H_2 - H_3 - etc.) on the individual Pump Head Curve will be different for each combination of pumps. As for instance with three (equal size) pumps running, the actual flow per pump will be one-third of Q_3 (read from the System Curve) =

$$\text{Pump Unit GPM}_3 = \frac{Q_3}{3}$$

Each pump, however, will operate at the head read directly from the System Curve =

$$\text{Pump Unit Head FT}_3 = H_3$$

and each pump will use as much power as the KW input corresponding to the operating head of that particular combination but read from the individual Pump KW Input Curve =

$$\text{Pump Unit KW Input} = KW_{\text{Pump Curve}}^{\text{at H System Curve.}}$$

(Note: The scope of this paper does not allow more detailed discussion of the ramifications and limitations of centrifugal pumps running in parallel. For those interested we highly recommend the article "Operating Limits of Centrifugal Pumps in Parallel" by J. P. Messina).

At this point, the specific power consumption values (KW_1-KW_2-KW_3- etc.) for all running combinations in all contemplated systems can be calculated, charted and a comparative evaluation of their long range economics can begin by using the Duration Diagram.

ENERGY COSTS AND THE DURATION DIAGRAM.

By using the information shown on Figure 5, a Duration Diagram of annual wastewater flow can be made to calculate annual power consumption in the station once the pumps are selected and the operating points and individual power consumptions in any possible running combinations are known. This Duration Diagram can be used then for comparing the annual running costs of alternate pump selections.

To make the construction of the Duration Diagram easier, the typical diurnal flow patterns shown on the lower half of Figure 5 have been converted into the following nomogram:

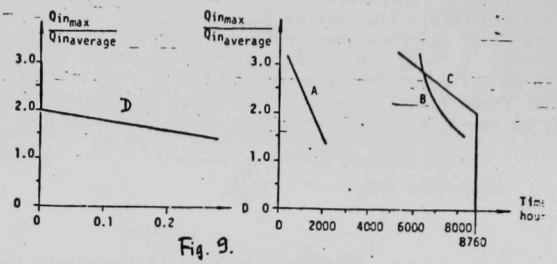


Fig. 9.

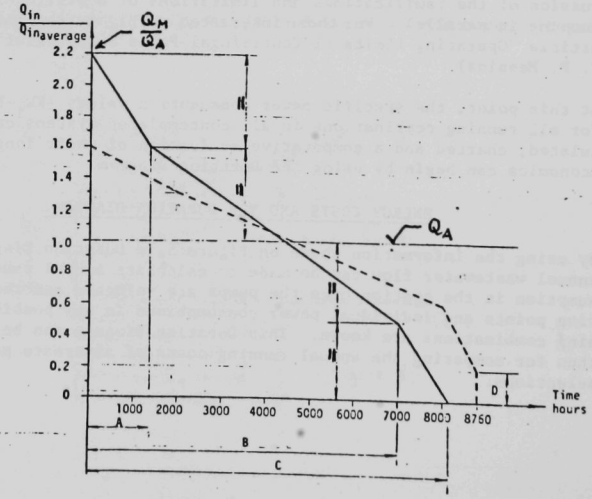
Entering this nomogram at the selected $\frac{Q_M}{Q_A}$ ratio (on axis Y) horizontally we can read the accumulative number of hours ("durations") per year on the time (X) axis for the following flows:

A = duration of daily flows exceeding the half value between maximum flow (Q_M) and average flow (Q_A)

B = duration of daily flows exceeding the half value between average flow and minimum flow.

C = duration of daily flows exceeding minimum flow

Introducing these values now in a similarly constructed coordinate system we obtain a Typical Duration Diagram for the selected $\frac{Q_M}{Q_A}$ ratio:



The running time for any combination of pumps can now be calculated from this diagram. In simple terms, it can be found by dividing the area under the Duration Diagram Curve by the term

$$K_0 = \frac{Q_{out}}{Q_{in\ ave}}$$

representing the combination of pumps running simultaneously, applied to that portion of the diagram.

The total annual energy consumption will be equal to the sum of all the energy segments under the Duration Diagram Curve. In mathematical terms:

$$KWH_{Total} = KWH_1 + KWH_2 + KWH_3 \ldots\ldots = \sum_1^n KWH_n$$

A more detailed explanation of this principle and of the procedures, illustrated by numerical examples and including the necessary nomogram is available from Flygt Corporation, Norwalk, Ct. 06856.

ENERGY EFFICIENT PUMP SYSTEMS--PART I--PUMP DRIVES

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Burns & McDonnell Engineering Company

There is little doubt that the electric motor is the most practical pump-drive system available. Its widespread use is testimony enough to this fact.

But the increasing cost of electrical power casts a new light on the budgetary impact of pump-drive systems. Electrical pump-drive systems can no longer be purchased, installed and forgotten. They have a growing impact on operating expenses.

A close look at pump system design may lead to increased pump-drive efficiency and some surprising cost savings.

Electric Motors: What Cost Efficiency?

Electric motors convert electrical energy into mechanical energy and, in the process, lose some energy to heat. Most motors in the 1-200 hp range average 90 percent efficiency; therefore, they consume approximately 10 percent of all the energy they convert.

To reduce power costs, motor efficiency must be known and maximized where possible. Motor efficiency is calculated by the formula:

$$EFF = \frac{746 \times \text{hp Out}}{\text{Watts in}} \qquad 1.)$$

Table 1 shows efficiency comparisons among five different motor sizes from 1 to 100 hp. As seen in the table, the industrial efficiency

TABLE 1

EFFICIENCY-
INDUSTRY AVERAGE FOR
FIVE DIFFERENT RATINGS

HP	Ind. Avg.	Prem.	Points of Eff. Dif.
1	73%	81.6%	8.6
5	80	87	7
10	85	89	4
50	90	93	3
100	91.5	94	1.5

average for a 1 hp motor is 73 percent. For the same size premium (high-efficiency) motor, the efficiency average is 81.6 percent, an improvement of 8.6 percent. The same comparison of the 100 hp motor shows an efficiency improvement of 2.5 percent. (The different in efficiency improvements is

a result of the lower standard efficiency rates of small horsepower motors. But it is important to get even a small efficiency improvement in the larger motors since they use more power.)

The technology is available to build high-efficiency motors, but they cost more to manufacture--and to purchase--than a standard motor. Consumer demand for smaller, more powerful, less expensive motors brought on the trend for less efficient motors. However, high-efficiency motors are available and, with increasing power costs, are becoming more cost effective.

The benefits of high-efficiency motors are: a) cooler operation, typically 20 degrees C with a resultant increase in insulation life (every 10 degrees C in reduction in operating temperature theoretically doubles insulation life); b) ability to accelerate larger inertial loads; and c) less power factor sensitivity to variation in line voltage and motor loads.

Newer, high-efficiency design motors cost 15-25 percent more than standard models. The relative length of time necessary to pay back the premium cost for a high-efficiency motor is usually two to four years. A typical 25 hp pump motor of high-efficiency design will pay for the extra premium in two to three years (if operating only 50 percent of the time), and the motor will continue to save money over its anticipated 10-15 year operating lifetime.

Savings are real once operating cost for the energy-efficient motor and standard motors are beyond the break-even point, see Figure 1. The Owner is then into long-term savings, which can be significant over the remaining life of the motor. It is good economics for the specifier or purchaser to indicate a preference for high-efficiency motors.

But how can the Owner be assured of getting a high-efficiency motor? Fortunately, standards have been developed to assure high-efficiency motor designs, see Figure 2. Two standards are IEEE Standard 112A, test method B, and NEMA Standard MG1-12.536. The NEMA standard provides for an efficiency index letter on the motor nameplate which corresponds to a minimum and nominal efficiency.

Efficiency vs. Power Factor

The efficiency of an electric motor extends over a wide range of loads, but it must be built into the motor and is costly and difficult to improve once the unit is completed. As shown in Figure 3, motor efficiency remains fairly high from 50-125 percent of rated output, but drops significantly below 50 percent. This means that a motor oversized by 100 percent for the system in which it operates will constantly be operating at 50 percent of its rated load, at which point efficiency is marginal and efficiency improvement becomes costly and difficult. As also shown in Figure 3, the power factor falls off rapidly as the motor's load decreases below 100 percent. However, the power factor can be economically boosted by adding capacitors to an existing system.

When specifying a motor, efficiency is by far the most important design consideration. Not every electrical customer is penalized for a poor

power factor, but everyone pays for poor efficiency through higher-than-necessary power bills.

System Efficiency

Although it is important to improve the efficiency of the motor, it is equally important to improve the efficiency of the mechanical load the motor is driving, in this case, the pump. As previously stated, electric motors are high efficiency machines having efficiency ratings up to 94 percent in the 125 hp range. However, many of the loads driven by motors have efficiency ratings of only 50-90 percent, which is typical of pumps.

Improving system efficiency may cost less at the pump than at the motor. When a pump is throttled to reduce its discharge flow, more energy is wasted in the load than in the motor. A more efficient way to reduce flow is to use multi-speed motors or variable-speed drives. Also, a motor should be directly coupled to a pump wherever possible since connections through gears, belts or slip couplings reduce system efficiency.

Variable Speed Drives

Figure 4 shows the comparison in efficiency for several variable speed drives at 50 percent to 100 percent of rated speed at 100 hp. Efficiency is expressed as a percent, power line to output drive shaft. (The two horizontal bars at the top of the graph represent a high-efficiency, constant-speed motor and a standard 100 hp constant-speed motor. These are included for reference.)

As shown, the conventionally wound rotor motor and eddy current (magnetic coupling) variable speed drives are only about 50 percent efficient when pumping at 60 percent of rated speed. Figure 4 also shows two variable-speed drive systems that are more efficient than the others. One is controlled current variable frequency; the other is a wound rotor motor with energy recovery circuits. Either of these systems (when driving a pump from 60 to 90 percent full speed) exhibits line-to-shaft efficiencies of 83 to 90 percent. This is the total drive system efficiency including variable-speed controller and variable-speed motor.

Adjustable Speed Drives

Constant-speed drives should be used where possible because they are the least complicated and have the highest efficiency. However, there are several reasons for using adjustable speed drives: 1) a savings on wet-well size; 2) minimizing the cycling of a constant-speed motor; 3) exact speed or pumping rate can only be determined experimentally; 4) improvement in system efficiency by reducing flow via the adjustable-speed drive instead of by throttling a valve; 5) the need for the "soft start" capability which some variable-speed drive systems provide because of utility or emergency generator design criteria.

Summary

There are many factors to consider when selecting a motor drive system, and efficiency is becoming more important every day as energy cost continued to escalate.

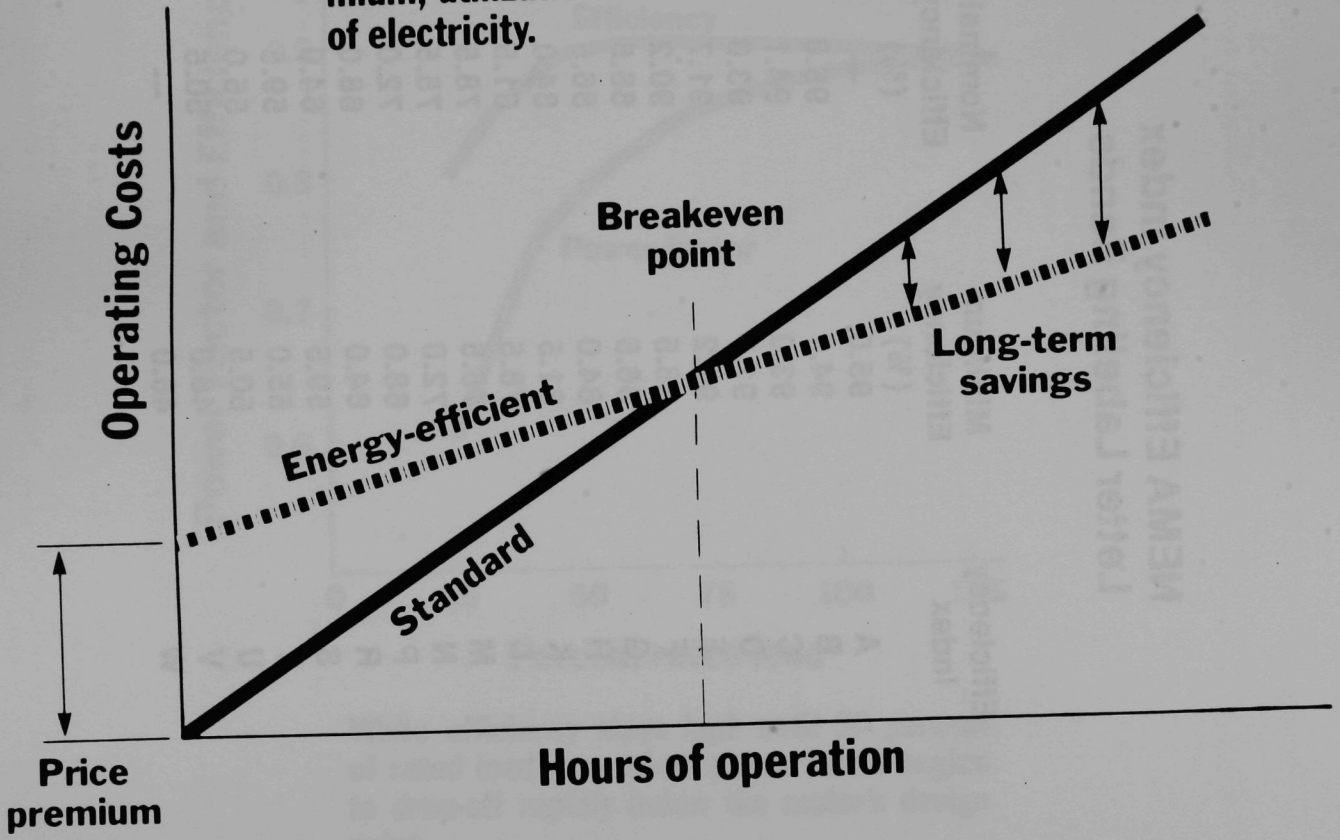
References

MONTGOMERY, DAVID C. Efficiency--A New Emphasis in Motor Selection, GER-3052 General Electric Co., Hendersonville, Tennessee.

KLEIN, A. Rising Electricity Cost Will Make Energy Efficient Motors Pay Back Faster. Control Engineering, July 1977.

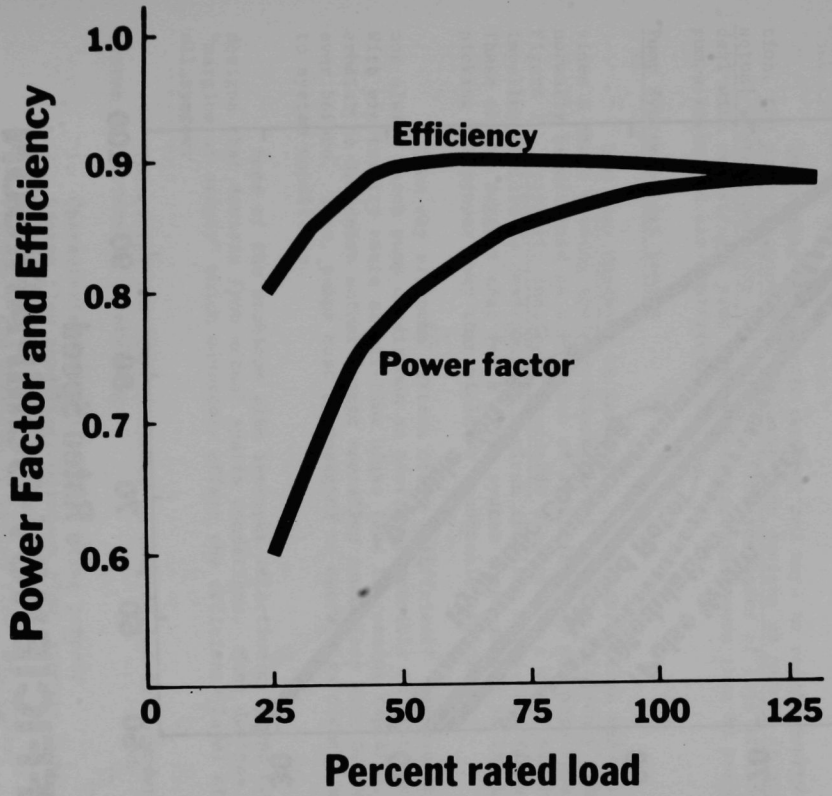
LINDHORST, PAUL K. High Efficiency Motors Generate Substantial Savings. Water & Sewage Works, January 1978.

Time to breakeven will depend on price premium, utilization of the motor, and local costs of electricity.

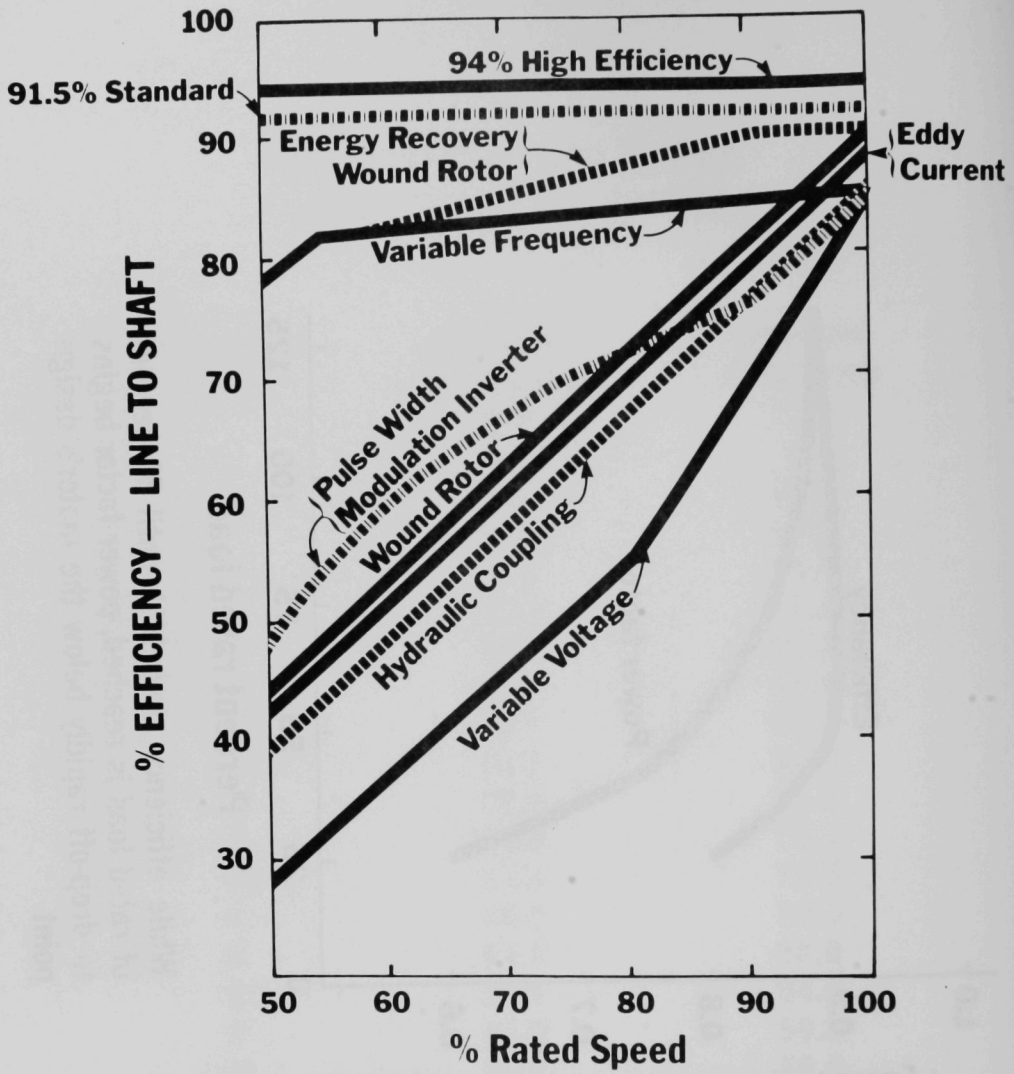


NEMA Efficiency Index Letter Labeling Bands

Efficiency Index	Minimum Efficiency (%)	Nominal Efficiency (%)
A	95.0	—
B	94.1	95.0
C	93.0	94.1
D	91.7	93.0
E	90.2	91.7
F	88.5	90.2
G	86.5	88.5
H	84.0	86.5
K	81.5	84.0
L	78.5	81.5
M	75.5	78.5
N	72.0	75.5
P	68.0	72.0
R	64.0	68.0
S	59.5	64.0
T	55.0	59.5
U	50.5	55.0
V	46.0	50.5
W	46.0	—



While efficiency stays high until 50 percent of rated load is reached, power factor begins to drop-off rapidly below the motor's design point.



EFFICIENCY COMPARISON GRAPH AT 100 HP

Some of the information shown on this chart was taken from "Engineer's ACI Handbook" by Robicon Corporation, April, 1977.

ENERGY EFFICIENT PUMP SYSTEMS--PART II--THE PUMP SYSTEM DESIGN

John M. Price, P. E.
Burns & McDonnell Engineering Company

Improper selection or oversizing of pumps and pump drives can be the major cause of excessive power consumption in existing pump systems. Because of this, system designs must be based on a total system analysis, from development of accurate system operating criteria to selection of the proper pump and driver.

In examing pump system designs and ways to reduce energy consumption, the primary concern is matching pump designs as nearly as possible to actual system operating conditions. (The remainder of this article will deal with centrifugal pump system designs, since more than 80 percent of all pumps installed are centrifugal.)

Pump System Design Basics

The Pump Handbook by Karrassik, Krutzsch, Fraser and Messina provides a chart showing the approximate level of pump efficiency which can normally be attained as a function of specific speed and pump capacity, see Figure 1. Hydraulic Institute Standards also offers a chart showing which impeller profile is desirable at a given specific speed, see Figures 2 and 3. These charts suggest that for a given system there should be little problem picking the proper pump that yields a reasonable efficiency.

Then why are pump systems often inefficient? Because designers do not always match pump conditions as nearly as possible to system conditions. With electricity costs at all-time highs (and increasing rapidly), it is crucial to determine actual system operating conditions more accurately than ever before. Then, pumps must be designed to operate as closely as possible to system conditions.

Some of the problems with improper selection of pumps come from designs that deviate from actual system conditions. Many designs include "margins of safety" which adversely affect the efficiency level of the overall system.

Therefore, the first priority in pump design is to understand the system operations by determining:

1. Characteristics of the fluid being pumped;
2. System flow conditions--minimum flow, average flow, normal maximum flow and extreme maximum flow;
3. Frequency and duration of system flow conditions;
4. System head conditions;
5. Future system operation.

A system head curve is a plot of head versus capacity, or flow, through a particular pipe system. This plot should include the minimum and

maximum static head conditions and friction curves for old and new pipe conditions or present and anticipated pipe conditions. Figure 4 shows what a system head curve might look like for a particular pipe system design for a clear water pump system. Actually, the system head curve is the area enclosed by the old and new pipe friction curves. Normally, this type of curve would be carried through the entire pump system analysis, but in order to simplify comparisons, the system head curve will be represented with a single-line system head curve as shown in Figure 5.

Single-Pump System

A single-pump system design for the particular pipe system shown in Figure 5 requires a 4,000 gpm (15,140 liter per minute) pumping rate. In a single-pump design, a system head curve is not always developed. Usually, a "design" head at the capacity desired is calculated and no curve is drawn. The tendency in single-pump design is to calculate the maximum head required and increase this "just a little" as a safety margin to assure sufficient head for the system.

For example, a designer might calculate 110 feet (33.5 meters) of head at 4,000 gpm (15,140 lpm), and increase the head to 130 feet (39.6 meters) in the specifications just to provide a little margin of safety. Figure 6 shows what this margin of safety does to pump operation. The pump purchased would be designed for maximum efficiency at 130 feet (39.6 meters) of head and 4,000 gpm (15,140 lpm) based on the specifications. Since only 4,000 gpm (15,140 lpm) are required, the pump discharge valve must be throttled. Then, the pump is operating at 130 feet (39.6 meters) instead of the required system head of 110 feet (33.5 meters). That additional 20 feet (6.1 meters) of head would cost approximately $5,500 the first year based on $0.03 per kWh at 4,000 gpm (15,140 lpm), assuming continuous pump operation.

The key to single-pump design is determining actual system operating conditions as accurately as possible and matching the pump to those conditions. A complete system head curve should be drawn for the above system, and the pump designed to provide for the anticipated system operation.

Multiple Pump System: Constant Speed

Figure 7 shows the same pipe system described previously with the following conditions: average continuous flow of 1,500 gpm (5,677.5 lpm), normal maximum flow of 3,000 gpm (11,355 lpm), and extreme maximum flow of 6,000 gpm (22,710 lpm).

The best design for this system might at first appear to be three 50 percent capacity pumps, each designed similar to the pump curve shown in Figure 7. One of these pumps would be a standby, but all three would be designed for 3,000 gpm (11,355 lpm) at the maximum system head of 160 feet (48.3 meters). Then two pumps could be operated simultaneously to obtain 6,000 gpm (22,710 lpm) at 160 feet (48.3 meters) of head, the extreme maximum condition. In terms of overall maintenance, three similar pumps are ideal; installation is fairly simple, all pumps have interchangeable parts, and the system is easy to operate. But what happens during average system operation?

The normal flow requirement is 1,500 gpm (5,677.5 lpm), so the pump discharge valve or line valve must be throttled to create the artificial system head curve as shown in Figure 8. The system will function this way with plenty of capacity; but instead of pumping 1,500 gpm (5,677.5 lpm) at the required system head of 64 feet (19.5 meters), the system will be pumping 1,500 gpm (5,677.5 lpm) at 193 feet (60.3 meters). Also, the pump will be operating at much less than peak efficiency since it must be throttled to produce the 1,500 gpm (5,677.5 lpm). Assuming this pump operates continuously throughout the year, operating costs would be approximately $26,600 this first year.

Based on the Pump Handbook specific speed curves, a pump properly designed to supply 1,500 gpm (5,677.5 lpm) at 64 feet (19.5 meters) of head should be able to provide maximum efficiency of about 84 percent. The operating cost for this properly sized pump would be about $6,500 the first year, less than one quarter the operating cost of the improperly designed system. Note that although the system head is only about three times as great for the throttled system vs. the properly designed system, 193 feet vs. 64 feet (60.3 meters vs. 19.5 meters), because of pump inefficiency, energy consumption is four times as great, $26,600 vs. $6,500.

The conclusion is that although throttled systems may be easier to operate and maintain, they are not energy efficient. The best alternative for this kind of system is at least one small pump designed for maximum system efficiency at 1,500 gpm (5,677.5 lpm) and 64 feet (19.5 meters) of head. Then, larger pumps or a combination of pumps may be provided to fulfill the normal maximum and extreme system requirements, depending on the kind of system reliability required and on the frequency and duration of the 3,000 gpm (11,355 lpm) and 6,000 gpm (22,710 lpm) rates.

Multiple-Pump System: Variable Speed

Another possibility for this same system is a three-pump installation with one variable-speed drive to meet lower capacity system requirements and two constant speed units to meet maximum system requirements, as seen in Figure 9. The type of variable-speed drive selected has a major effect on overall system efficiency. A magnetic (eddy current) coupling or standard wound rotor motor with liquid rheostat would substantially lower the variable speed unit's efficiency at reduced speeds, see Figure 10.

Note that the maximum efficiency point along the pump curve at 3,000 gpm (11,355 lpm) and 1,450 rpm is not of particular concern because this is not the average system operation. However, the maximum efficiency point should be chosen fairly close to the average system capacity of 1,500 gpm (5,677.5 lpm) at 64 feet (19.5 meters) of head and 1,075 rpm. (If it were closer to the end of the 1,785 rpm curve in Figure 9, operation would be fairly inefficient at average pump conditions.)

Assume that the maximum efficiency of the pump chosen is about 85 percent and that it is operating at about 80 percent efficiency at average pump conditions. In order to obtain the normal maximum capacity of 3,000 gpm (11,355 lpm) at 88 feet (28.8 meters) of head, the speed of the variable unit

must be reduced about 20 percent to 1,450 rpm, and to obtain average system capacity and head, speed must be reduced about 40 percent to 1,075 rpm.

For a magnetic coupling or standard wound rotor unit to operate at 60 percent speed (1,075 rpm), the efficiency level is about 50 percent. Therefore, even though the pump is operating at 80 percent efficiency, the overall system efficiency is only 40 percent. And instead of paying approximately $6,500 for the first year's operation at 1,500 gpm (5,677.5 lpm), the owner must pay approximately $14,000. If the first few years additional operating expenses were added to the original cost of the magnetic coupling or wound rotor unit, there would be a sufficient amount remaining to purchase a variable-frequency drive; and operating costs for this variable-speed system would be about $8,400 per year.

Multiple-Pump System: Base-Flow Pumps

Assume that two 100 percent capacity pumps (one standby) for the system head curve shown in Figure 11 are pumping to an elevated storage tank. Assume also that the demand flow from the storage tank varies from 1,500 gpm (5,677.5 lpm) to 6,000 gpm (22,710 lpm) throughout the year, with the average flow from the tank at 3,000 gpm (11,355 lpm). Two 6,000 gpm (22,710 lpm) pumps have been provided for the system. Normally, when the storage tank level is low, one of these pumps operates to fill the tank to the maximum level and then shuts off.

If a "base flow" (minimum flow) of 1,500 gpm (5,677.5 lpm) is established, a pump could be installed to provide this continuous flow to the storage tank, see Figure 12. Depending on the frequency and duration of the base flow, a pump could be selected to provide 1,500; 2,500; or 3,000 gpm (5,677.5; 9,565.5; or 11,355 lpm). Two base-flow pumps may even be required if base flows vary substantially during the year. These pumps could then be combined to provide additional base flow capacity.

The important consideration is that if a base-flow pump (or pumps) operates continuously, a major portion of the annual system demand can be pumped at a reduced head. A substantial amount of energy and money can be saved by pumping a base flow at 64, 88 or 100 feet (19.5, 28.8 or 30.5 meters) of head as opposed to pumping all of the system requirements at 160 feet (48.8 meters) of head. The maximum capacity pump could still operate when the base-flow pumps cannot keep up with tank demand requirements.

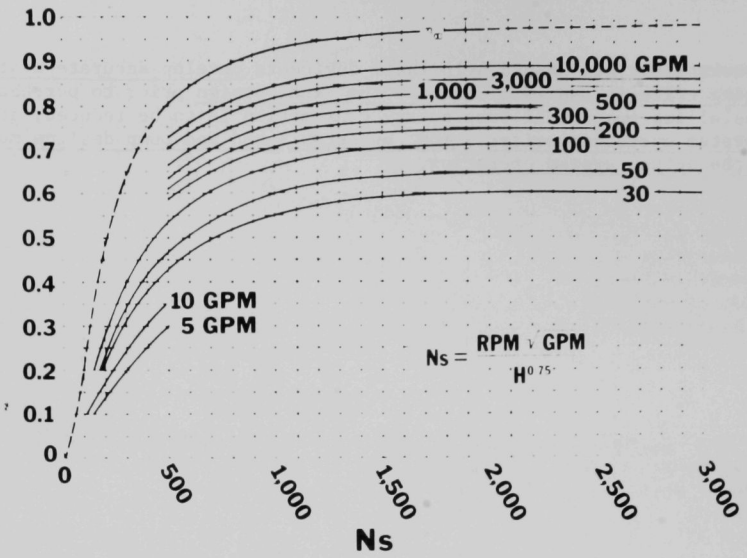
The Pipe System

One more energy efficiency consideration is this: if it is difficult to obtain slightly more flow through a pipe system without increasing the head substantially, it may be necessary to clean out an existing line or install a new or parallel line for the system. Sometimes this can be a real energy saver.

Summary

The author believes that both engineers and waterworks managers have a responsibility to the public to design energy efficient pump systems.

Waterworks officials should help their engineers develop accurate system operating criteria and should review the final design prior to purchasing and installing pumps. If pump energy consumption is to be reduced, accurate pump system operating criteria must be established and pump designs must match the actual system operation.



EFFICIENCY AS A FUNCTION OF SPECIFIC SPEED AND CAPACITY.

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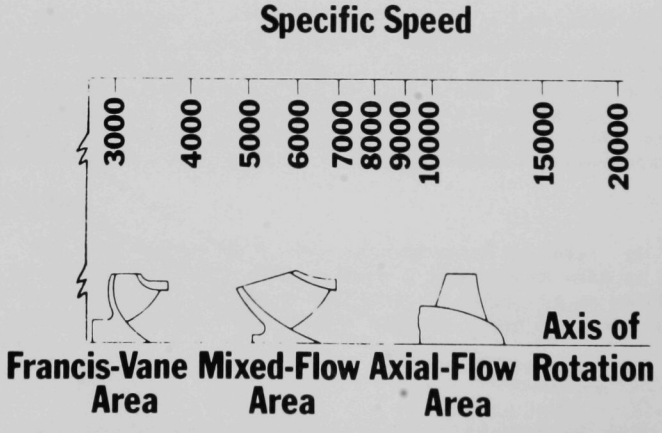
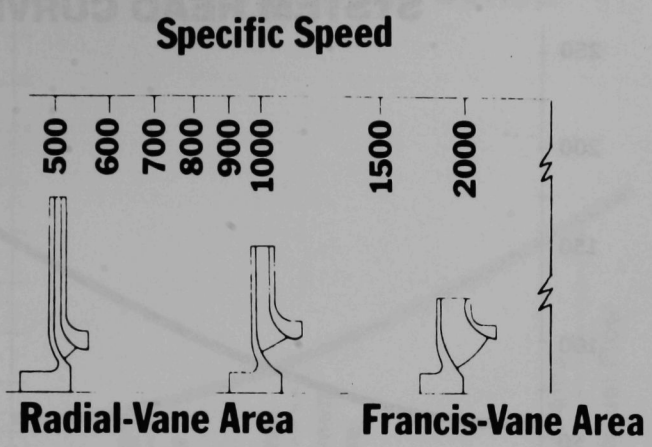


COMPARISON OF PUMP PROFILES



COMPARISON OF PUMP PROFILES

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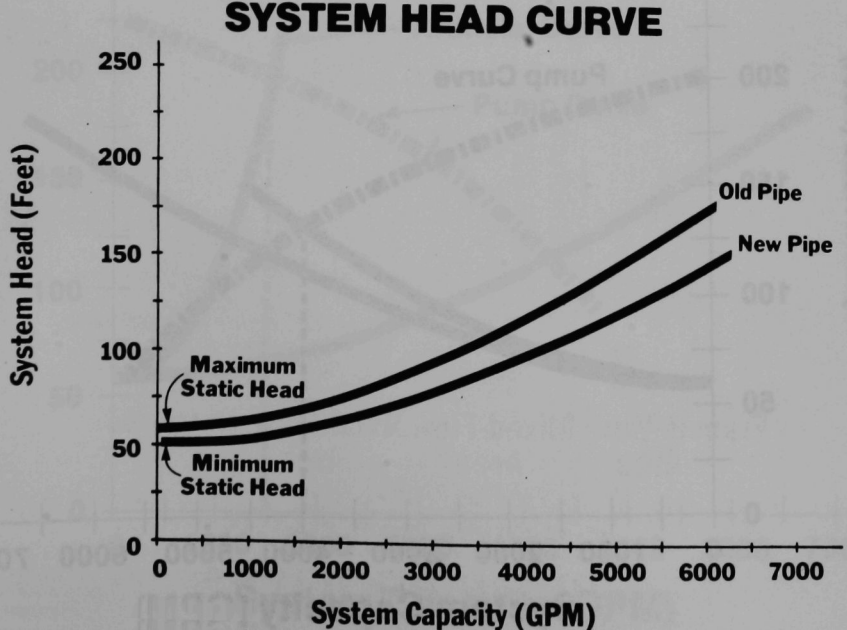
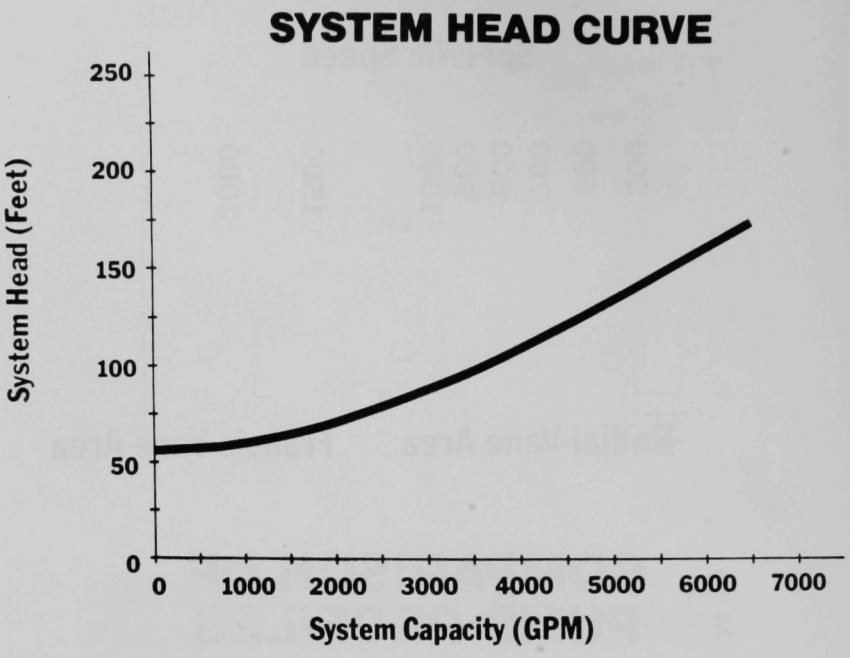
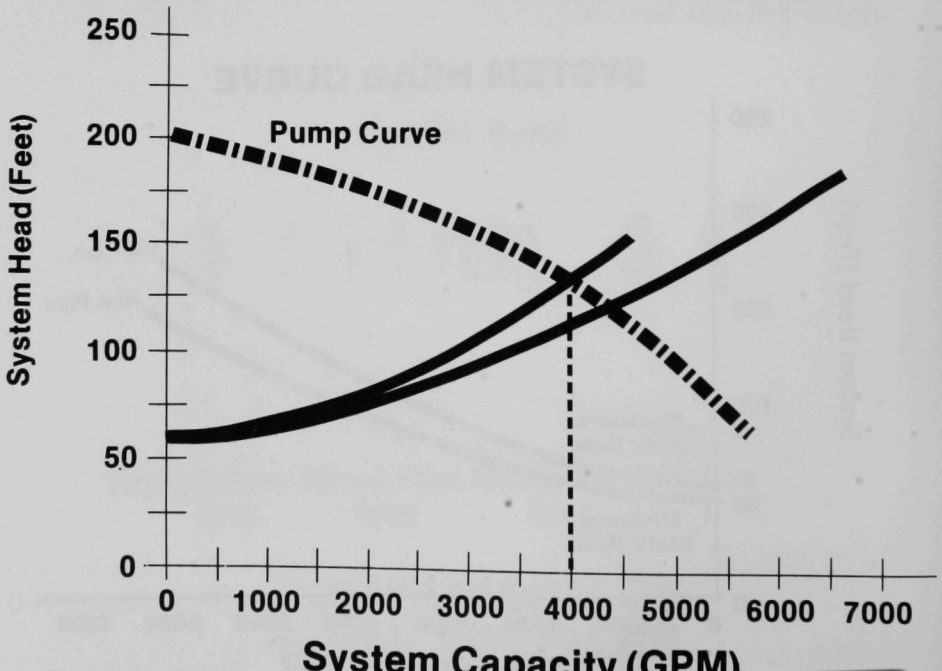
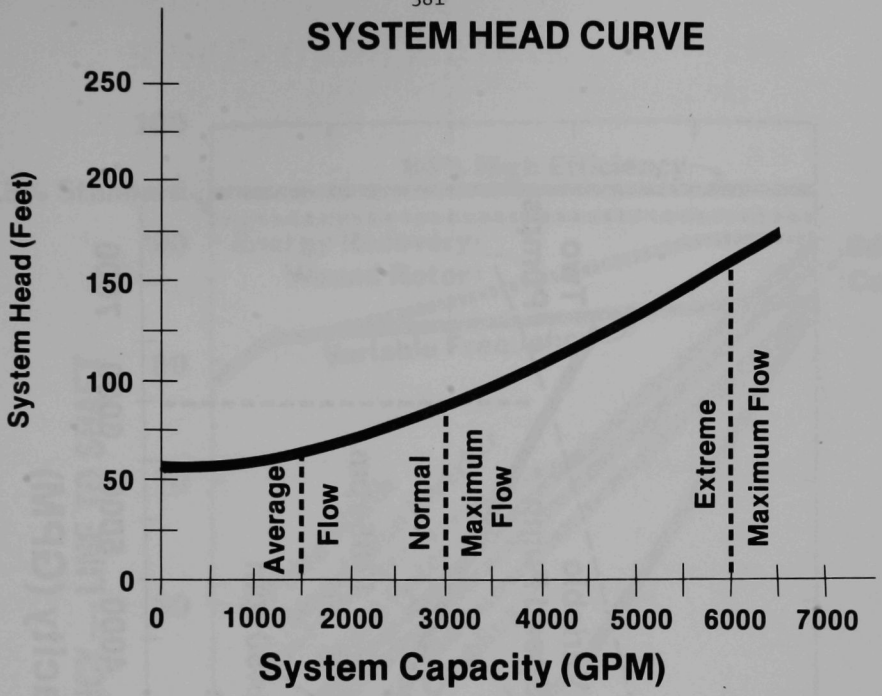
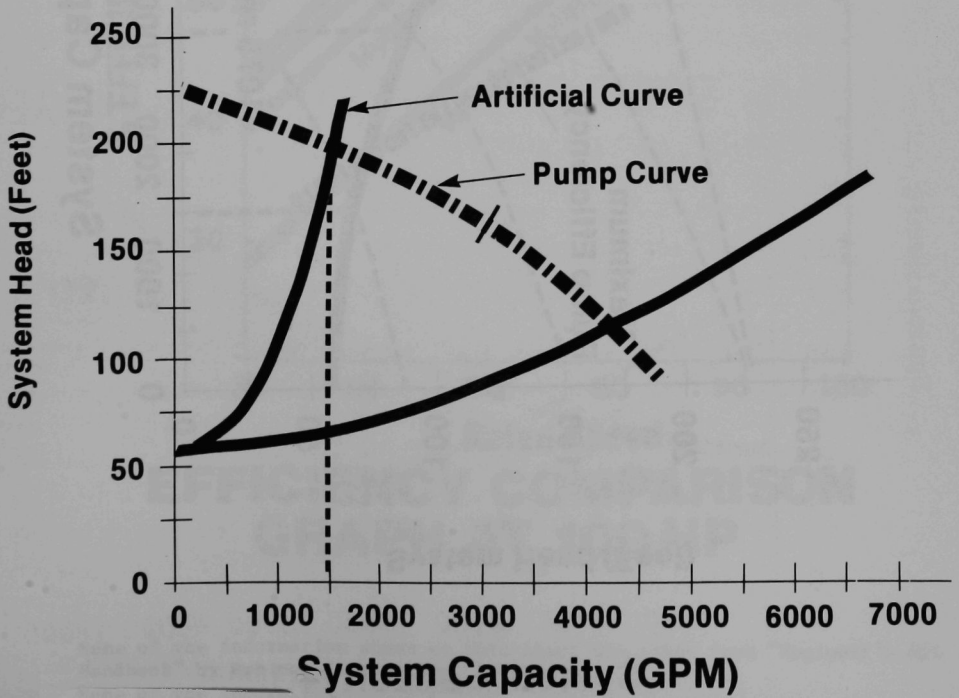
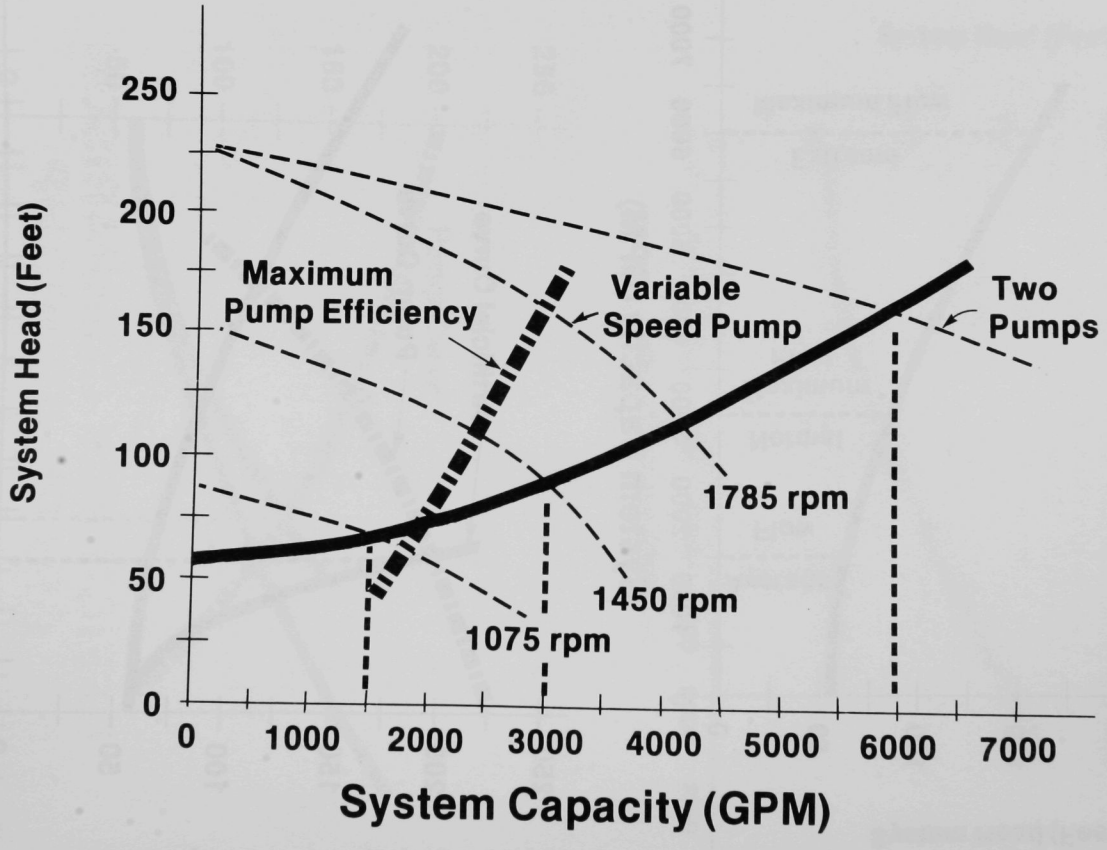


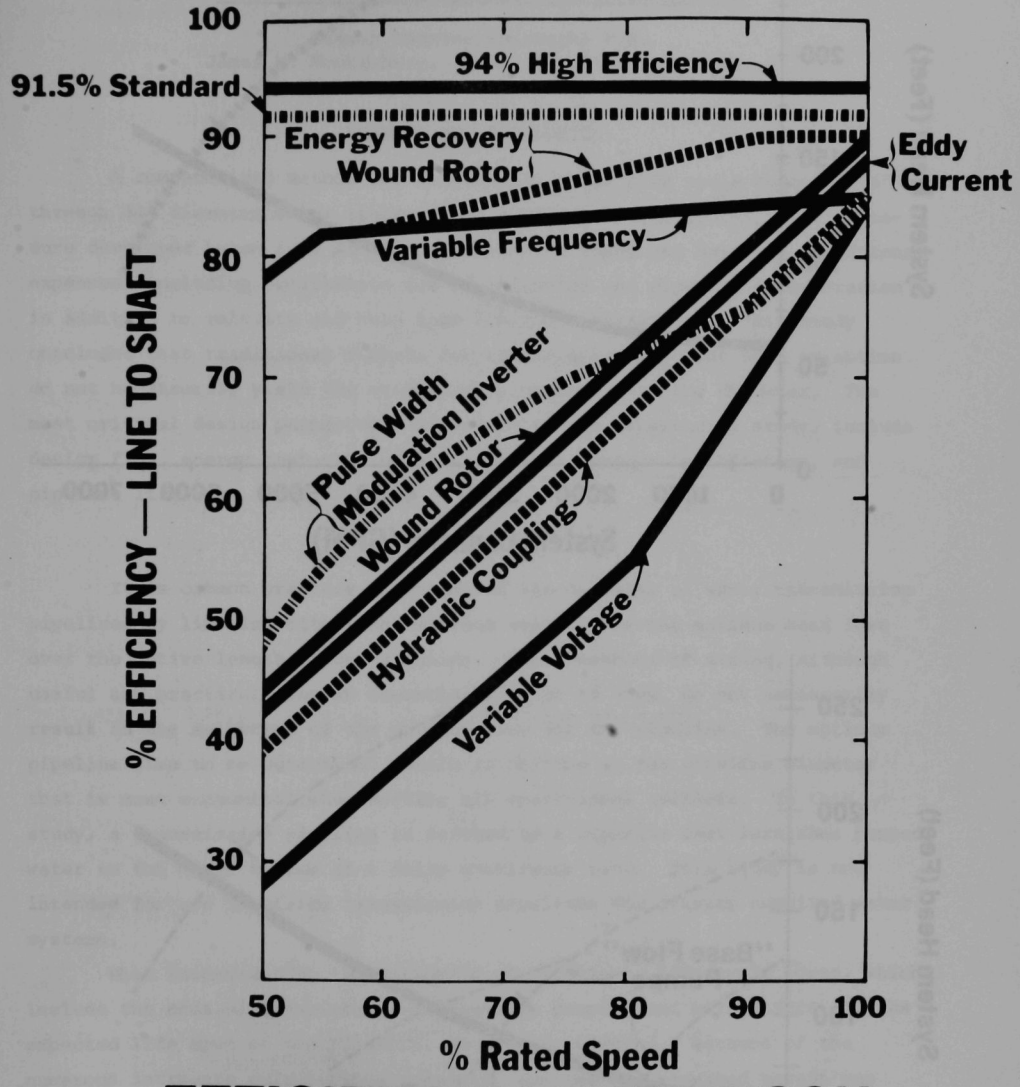

SYSTEM HEAD CURVE
System Head (Feet)
250
200
150
100
50
0
0
1000
2000
3000
4000
5000
6000
7000
System Capacity (GPM)


System Head (Feet)
250
200
150
100
50
0
Pump Curve
0
1000
2000
3000
4000
5000
6000
7000
System Capacity (GPM)


SYSTEM HEAD CURVE
System Head (Feet)
250
200
150
100
50
0
Average Flow
Normal Maximum Flow
Extreme Maximum Flow
0
1000
2000
3000
4000
5000
6000
7000
System Capacity (GPM)

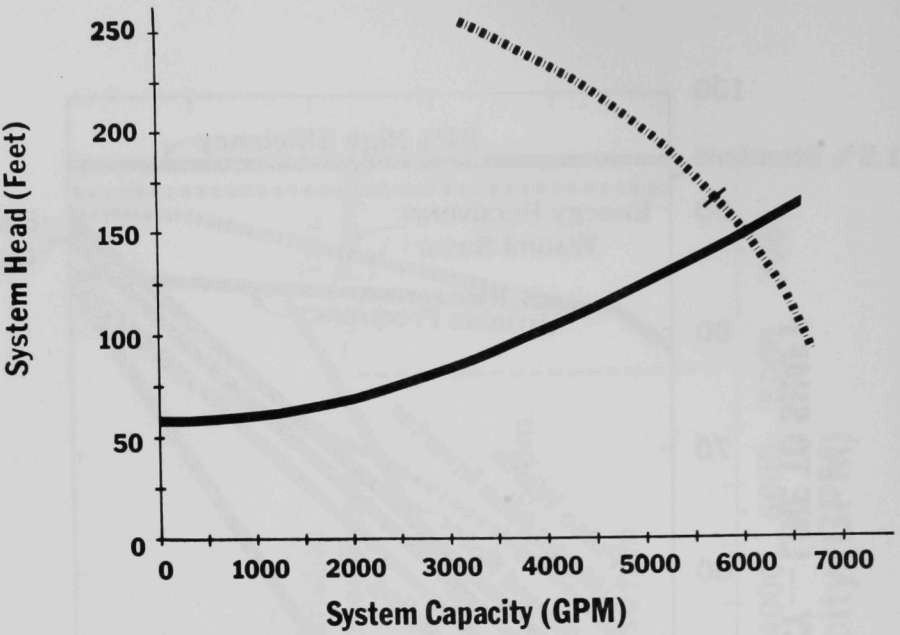

System Head (Feet)
250
200
150
100
50
0
Artificial Curve
Pump Curve
0
1000
2000
3000
4000
5000
6000
7000
System Capacity (GPM)

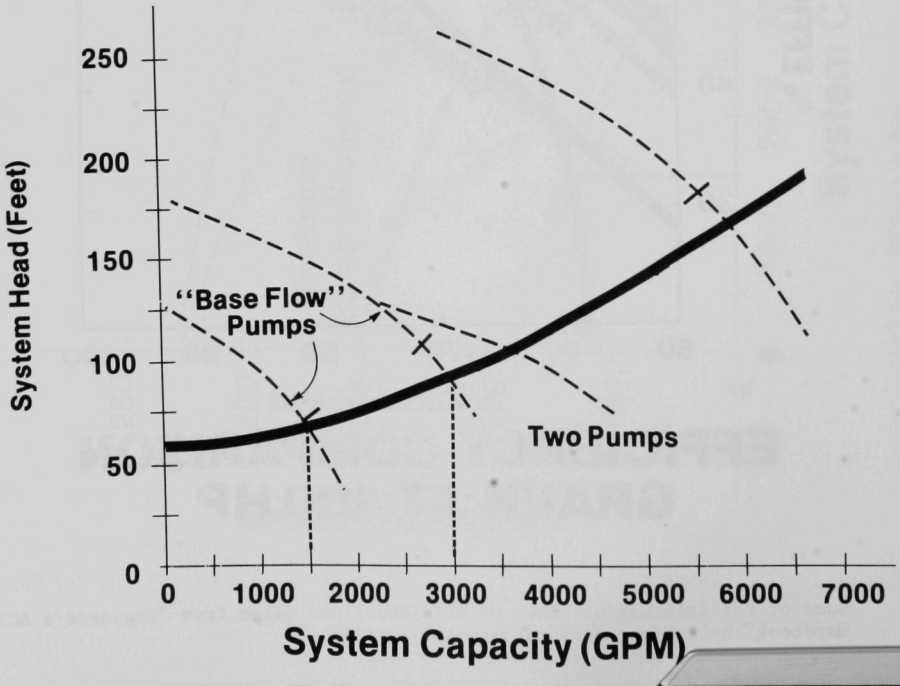

System Head (Feet)
250
200
150
100
50
0
Maximum
Pump Efficiency
Variable
Speed Pump
Two
Pumps
1785 rpm
1450 rpm
1075 rpm
0
1000
2000
3000
4000
5000
6000
7000
System Capacity (GPM)



EFFICIENCY COMPARISON GRAPH AT 100 HP

Some of the information shown on this chart was taken from "Engineer's ACI Handbook" by Robicon Corporation, April, 1977.


System Head (Feet)
250
200
150
100
50
0
0
1000
2000
3000
4000
5000
6000
7000
System Capacity (GPM)


System Head (Feet)
250
200
150
100
50
0
"Base Flow" Pumps
Two Pumps
0
1000
2000
3000
4000
5000
6000
7000
System Capacity (GPM)

WATER TRANSMISSION PIPELINE SIZE OPTIMIZATION

Steven Charles Ainsworth P.E.
James M. Montgomery, Consulting Engineers, Inc.

SUMMARY AND CONCLUSION

A computerized method for determining total life cycle costs of 24" through 84" diameter water transmission pipelines is presented. The procedure developed takes into account construction costs and detailed operational expenses, including escalations due to inflation and pipeline deterioration in addition to velocity and head loss limit considerations. This study concludes that traditional methods for transmission pipeline size selection do not necessarily yield the economically optimum pipeline diameter. The most critical design parameters, according to the sensitivity study, include design flow, energy cost rate of increase with respect to inflation, and pipeline life span.

DISCUSSION

It is common practice to establish the diameter of water transmission pipelines by limiting either the maximum velocity or the maximum head loss over the entire length of the pipeline. These methods of sizing, although useful and practical from an operational point of view, do not necessarily result in the selection of the optimum size for the pipeline. The optimum pipeline size to be determined herein is defined as the pipeline diameter that is most economical when meeting all operational criteria. In this study, a transmission pipeline is defined as a pipeline that furnishes pumped water to the water system at a daily continuous rate. This study is not intended for use in sizing transmission pipelines for gravity supplied water systems.

When investigating the economics of pipelines, life cycle costs, which include the cost of materials, installation, pumping and maintenance for the expected life span of the pipeline, must be considered. Because of the numerous intricate calculations required, the method presented herein was not tractable until the development of the digital computer. The variables used to determine life cycle costs used in this study are listed in Figure 1 and are defined as follows:

1. ENERGY COST; RATE OF INCREASE (% PER YEAR)

This is assumed to be constant over the life span of the pipeline. The graph shown in Figure 4 clearly illustrates that this is a valid assumption since a 10% variation in the total energy cost rate of increase has little effect (3.6%) on the total present worth of 1000 feet of pipeline.

2. INFLATION RATE (% PER YEAR)

This is assumed to be constant with respect to the energy cost rate of increase for the same reasons given above.

3. CURRENT ELECTRIC ENERGY COST (DOLLARS PER KILOWATT-HOUR)

This is the energy cost today and is fixed for a given area.

4. PIPELINE LIFE SPAN (YEARS)

Mortar-lined and coated steel transmission pipelines in Las Vegas, Nevada, are installed with cathodic protection anodes, where needed, which are designed to last 25 years. If after that time the anodes are not replaced, the mortar coating is designed to protect the pipeline an additional 25 years. Therefore, the life span of such pipelines is expected to be 50 years and could be much longer.

Asbestos cement pipelines have been used in Las Vegas since the early 1950's, and portions of these pipelines that have been exposed after 25 years of service show no objectionable deterioration. Therefore, a life span exceeding 50 years is expected for asbestos cement pipelines.

5. HAZEN-WILLIAMS (H-W) FRICTION COEFFICIENT

The friction coefficient for new pipelines is advertised to be 130 for centrifugal mortar-lined steel mains and up to 140 for new, asbestos cement pipelines. Studies indicate these to be reasonable for properly installed pipelines.[1]

6. RATE OF CHANGE OF H-W FRICTION COEFFICIENT (% PER YEAR)

Due to deposits on the inner walls of the pipeline and the effects of erosion, the friction coefficient decreases as the pipeline becomes older. A rate of change of the H-W friction coefficient of -0.2% per year will results in a coefficient of 120 in 50 years. Studies[2] show

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[1]William D. Hudson, "Loss of Water Main Capacity," Journal of American Water Works Association, Vol. 30, No. 1 (April, 1966): 48.

[2]Ibid.

that water treated with alum in the filtration process will leave an aluminum hydroxide deposit on the pipeline interior walls causing the friction factor to decrease much more rapidly. Therefore, a friction coefficient reduced from 130 to 80 in 50 years is to be expected under this condition.

7. PUMPING EFFICIENCY (%)

This is assumed to be constant over the life span of the pipeline. As shown in Figure 7, the actual pumping efficiency within a range of 65% to 85% was found to have little effect on the selection of the pipeline.

8. DESIGN FLOW (CUBIC FEET PER SECOND)

The design flow is the maximum flow through the pipeline in the design year; that is, the flow on the maximum day when all potential development in the area served by the pipeline has been completed. The flow on any particular day of any given year is a function of the design flow, the initial flow, the annual rate of change of flow, the day and the year.[3]

9. INITIAL FLOW (% OF DESIGN FLOW)

The initial flow, which is the maximum flow in a pipeline during the first year of service, is a percentage of the maximum flow in the design year. Figure 8 shows how the total life cycle cost of 1000 feet of pipe varies with initial flow.

10. RATE OF INCREASE OF FLOW (% PER YEAR)

The rate at which the maximum flow increased to the design flow is shown in Figure 1. It is assumed that transmission pipelines reach the design capacity within approximately 25 years. As long as the maximum flow reaches the design flow in 20 to 30 years, the pipeline size selection is stable, Figure 9.

11. ANNUAL FLOW CURVE

The flow curve, Figure 2, is described by a trigonometric

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[3]The actual flow variation throughout the year and throughout the life of the pipeline is used to compute the pumping costs. Since these costs are proportional to Q EXP 1.851, to use the average flow over the life of the pipe (see Thomas R. Camp, "Economic Pipe sizes for Water Distribution Systems," American Society of Civil Engineers Transactions, Paper No. 2019, December 1937) does not necessarily yield the correct result.

approximation of the average monthly flow for the design year. The coefficients were determined by analysis of the total system water production of the Las Vegas Valley Water District for three years: 1974, 1975, 1976.

The curve is used to determine the most likely flow for a given day of the year. The curve is illustrated in Figure 2. Since we are concerned only with transmission pipelines, the flow during one day is assumed to be constant.

12. PIPELINE INSTALLATION COSTS

The figures listed herein are for the average cost of installation of various-sized pipelines. The figures are for 1000 feet of pipe and include a control valve every half mile. The absolute value of these costs is not as significant as the difference in cost from one pipe size to the next. The installation costs are fixed for a particular area and are usually obtainable from previous construction work. The estimated 1979 pipe installation cost per 1000 feet in Las Vegas, Nevada, is as follows:

DIAMETER	COST	DIAMETER	COST
24"	$ 62,900	60"	$225,000
30"	$ 85,900	66"	$262,000
36"	$108,900	72"	$298,900
42"	$133,100	78"	$334,000
48"	$160,300	84"	$370,300
54"	$191,200		

Maintenance costs per 1000 feet of pipeline are assumed to be approximately equal for adjacent pipe sizes.

Figure 1 illustrates the relationship of design flow to the life cycle cost for various-sized pipelines for the given design conditions. When the actual determination of the optimum size of a pipeline is made, consideration must be given to the total allowable head loss; however, the maximum velocity limit of 5 feet per second is met by any pipeline selected from the chart, since with the given design conditions, the chart limits the water velocity to approximately 3.8 feet per second.

A pipeline selection example that demonstrates the use of Figure 1 follows. As noted previously, the selection tables always provide a pipeline with a design velocity of less than 5 feet per second, and a head loss of less than 5 feet per 1000 feet of pipeline for H-W friction coefficients as low as 100.

SAMPLE DESIGN PROBLEM

Assume a transmission pipeline is to serve an area with a projected design flow of 32 cubic feet per second (CFS) and a current demand of 8 CFS. The proposed pipeline is to be 5,000 feet long with a maximum head loss of 25 feet. The maximum allowable water velocity is set at 5 feet per second (FPS).

Step 1. Figure 1, reading vertically from 32 CFS, indicates that a 42" diameter pipeline is the optimum size.

Step 2. Check the water velocity in the pipeline to insure that it does not exceed 5 FPS:

32 CFS/9.6 SQF (42" pipeline) = 3.3 FPS.

Step 3. Check the head loss over 5,000 feet of pipeline. The H-W friction coefficient for the design year is used, C = 120:

Using the H-W formula, Head loss = 4.6 feet.

Therefore, a 42" diameter pipeline is satisfactory.

If the pipeline were to be selected by the standard application of the velocity and head loss limits, the size selection would proceed as follows:

Step 1. Determine the minimum pipeline diameter such that at 32 CFS, the total head loss does not exceed 25 feet (C = 120):

PIPE DIAMETER	HEAD LOSS
24"	69.6 feet
30"	23.5 feet

A 30" diameter pipeline is sufficient.

Step 2. Determine the minimum pipeline diameter such that at a flow of 32 CFS, the water velocity does not exceed 5 FPS:

PIPE DIAMETER	WATER VELOCITY
30"	6.5 F/S
36"	4.5 F/S

The velocity limit is exceeded by using a 30" diameter pipe, therefore the minimum diameter pipe that does not exceed either the velocity or head loss limit is 36".

From Figure 1, the present worth of the total life cycle dollar savings incurred by using a 42" instead of a 36" diameter pipeline is approximately $17,000 per 1000 feet of pipeline, which is approximately 13% of the $133,100 installation cost per 1000 feet of pipeline.

The following graphically presented sensitivity study illutrates the effects of small variations of the separate parameters on the total life cycle cost. The arrow indicates the design point assumed for this evaluation.

FIGURE 1

OPTIMUM PIPELINE SIZE SELECTION

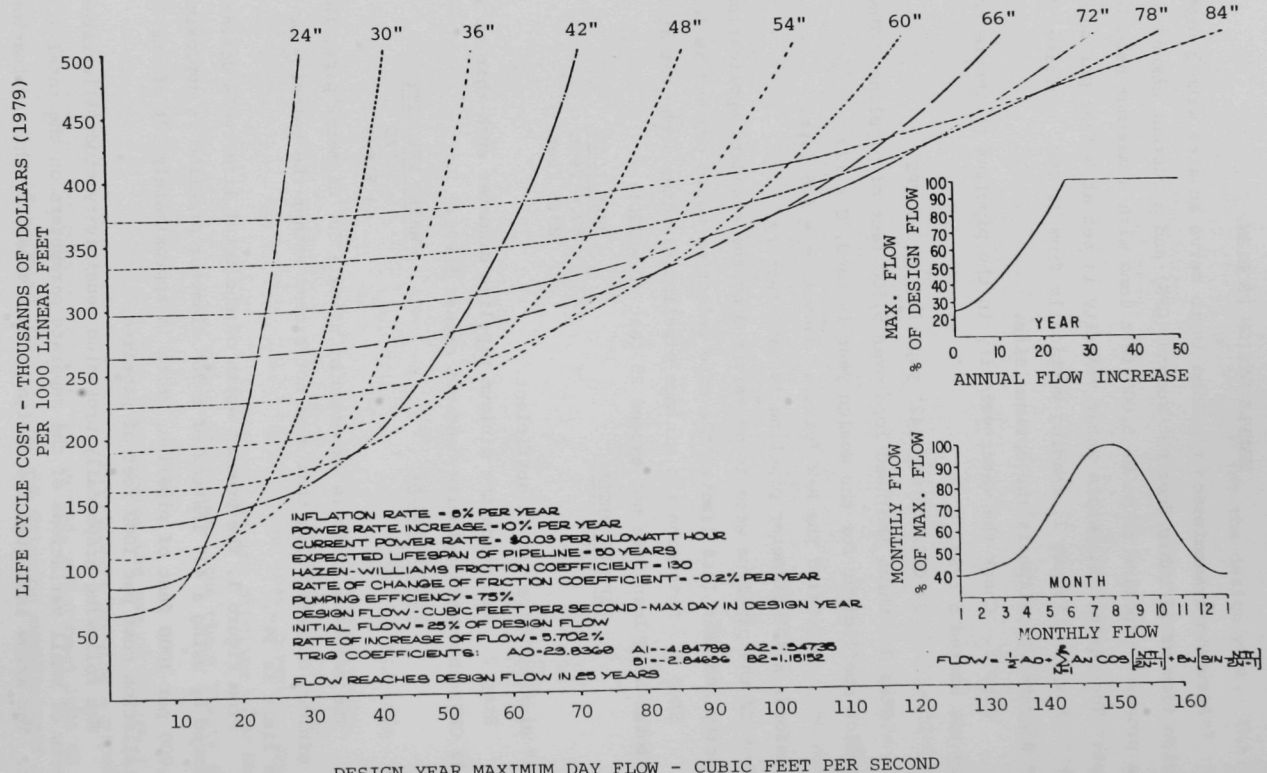


FIGURE 2

ANNUAL FLOW CURVE

FOR LAS VEGAS, NEVADA

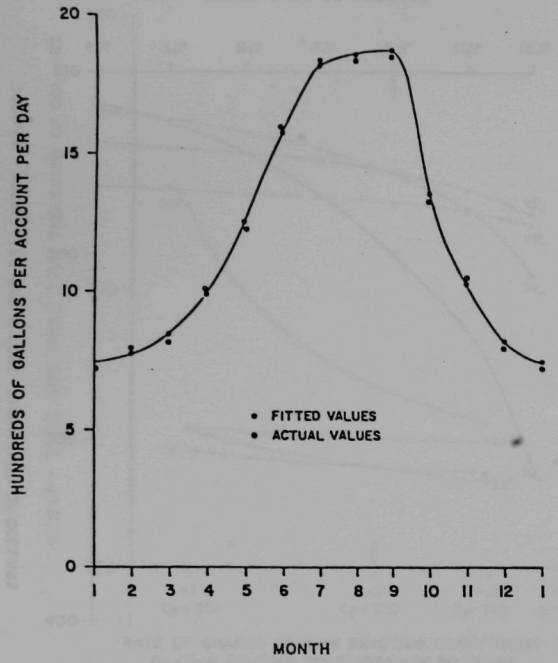


FIGURE 3

EFFECT OF DESIGN FLOW

ASSUMPTION

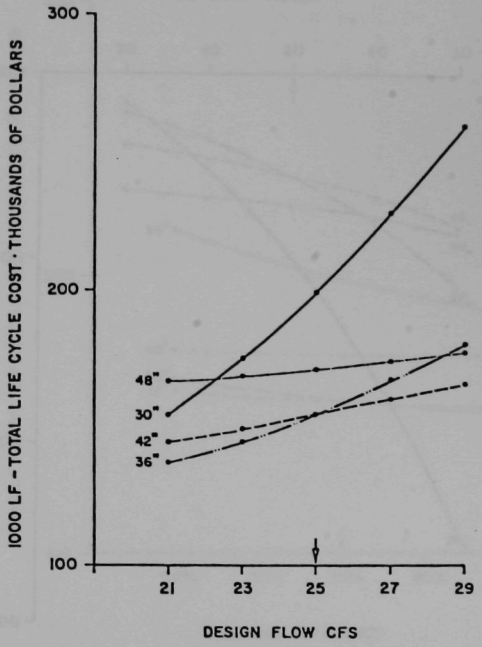


FIGURE 4

EFFECT OF POWER COST
RATE OF INCREASE

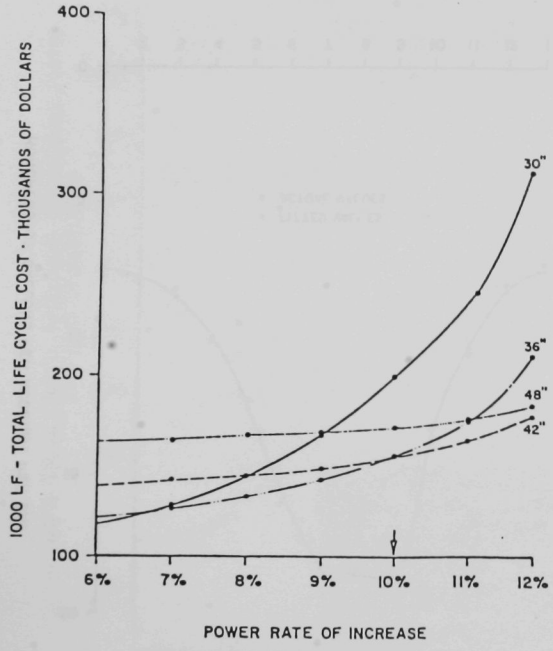


FIGURE 5

EFFECT OF PIPELINE
LIFE SPAN ASSUMPTION

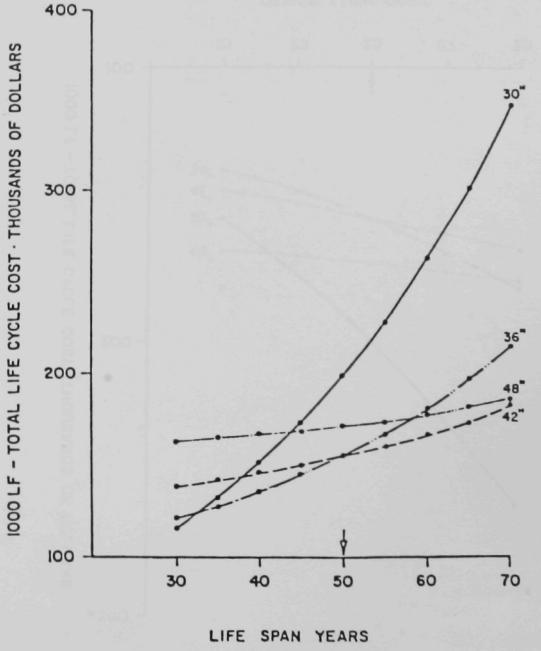


FIGURE 6

EFFECT OF PIPELINE ROUGHNESS AND DETERIORATION

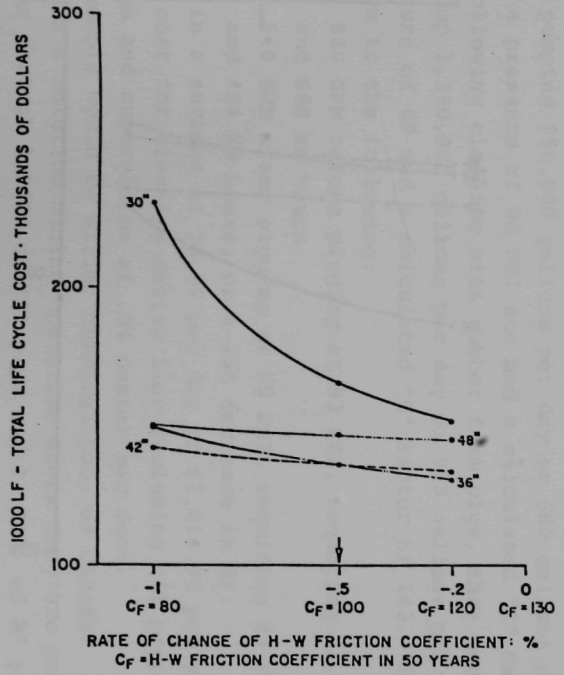


FIGURE 7

EFFECT OF PUMPING EFFICIENCY

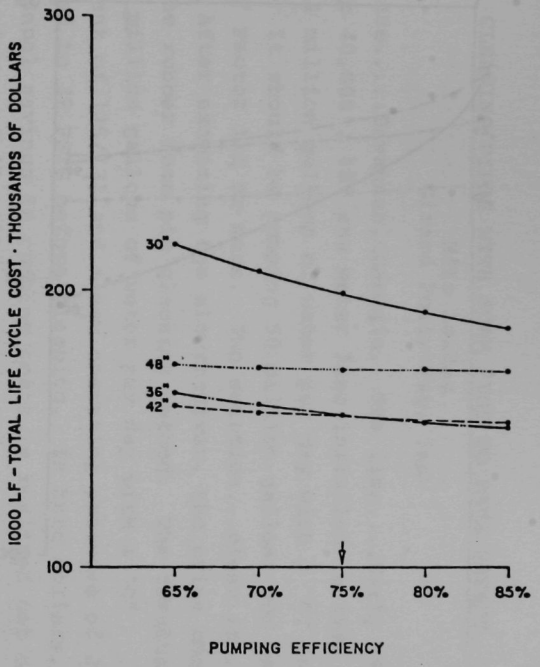


FIGURE 8

EFFECT OF INITIAL

FLOW RATE

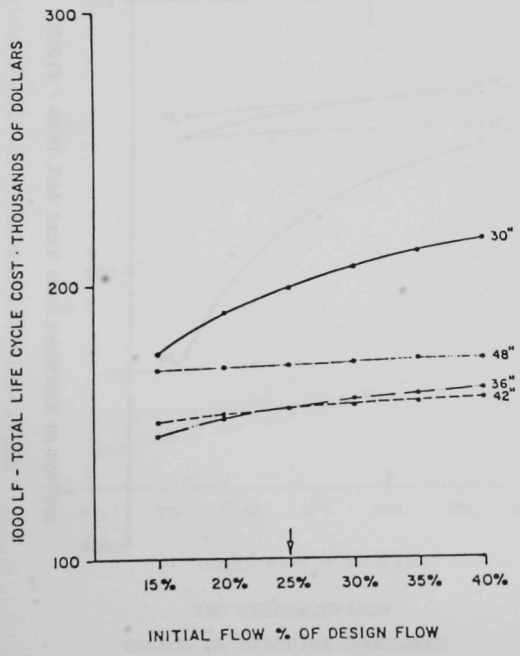
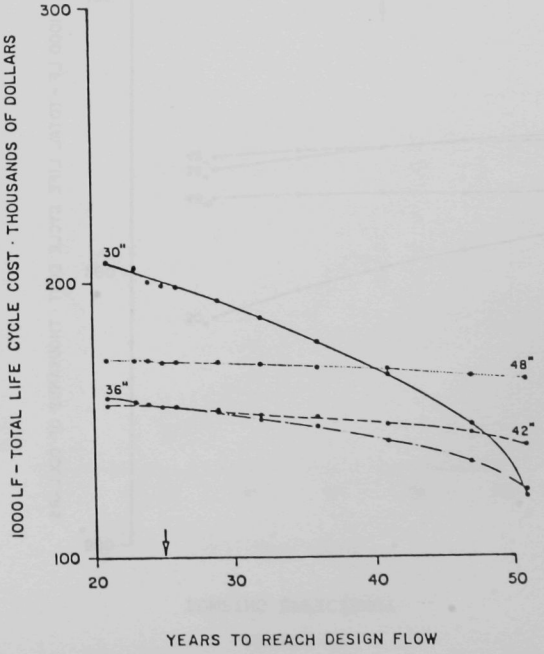


FIGURE 9

EFFECT OF ANNUAL

FLOW INCREASE



CLEANING PIPES WITH FOAM PIGS TO SAVE ENERGY

Mike Landes
Girard Polly-Pig, Inc.

The place.....Savannah, Georgia. The time....Early 1978. The city's 40,000', 48" raw water line installed in 1946 is pumping 44 million gallons of water per day with a "C" Factor of 78/83. It should be pumping 50 million gallons per day with a "C" Factor 115 or more. The solution...clean it... but how? After examining the alternatives, the prime contractor chooses the rubber foam pig cleaning method. The results..... fifty-six million gallons of water per day with a "C" co-efficient of 125/131 and a new operating pressure of 27 PSIG as compared to 32 PSIG before cleaning. In hard dollars, a $45,000 annual savings in pumping costs is realized not to mention a pickup of 12 million gallons of water per day.

The City of Marshall, Mo., was pumping treated water through 10 miles of 12" cast iron cement lined pipe with a very slight deposit of soft mineral approximately 1/16" thick which could be wiped from the interior walls of the pipe by hand. They were pumping 950,000 gallons per day or 660 gallons per minute at a pressure of 83 PSI and had a calculated "C" Factor of 92. Following cleaning with rubber foam pigs, they were able to pump 1,390,000 gallons per day or 965 gallons per minute at a pressure of 60 and a calculated "C" Factor of 143. This breaks down to the following:

660 GPM before pigging at 83 PSI, requiring 31 HP and 668 KW hours

660 GPM after pigging at 60 PSI, requiring 23 HP and 484 KW hours, or a 25% decrease in HP,

Resulting in a savings of $5.52 per day or $2,014.00 per year. Estimated cost for cleaning entire line including all preparation, labor, pigs and supervision of .04 center per foot.

Another example would be Allied Chemical Co., of Canada. Allied Chemical has a solution mining operation where they are pumping river water through 1,000' of 12" pipe and 11,000' of 8" pipe

into various wells to dissolve salt and then flow the brine back through another pipe to a storage reservoir. At this point there is a 175 HP pump which takes the brine and transfers it to the plant for processing which is 4 1/2 miles away. Due to increasing pressures and decreasing flows, Allied had increased their injection pump motor from 200 HP to 300 HP approximately 5 years ago and presently had on order another 400 HP motor. Their pump had been changed from a 5 stage to a 7 stage. All of this just to pump the same amount of volume as the initial installation of a 200 HP 5 stage pump was able to handle. Upon inspection of the line and through line flow characteristics and data gathered by one of Allied's engineers, it was determined that the tuberculation, mud and silt had accumulated to the point that the 12" and 8" pipe was reduced to approximately 5 1/2" I.D. Again the rubber foam cleaning method was used. After cleaning these lines, the following data and results were tabulated:

Field Operating Pressure Prior to Pigging	-305 PSI
After Pigging	-177 PSI
A Reduction of 42%	
Hydraulic HP required for 1020 GPM prior to Pigging	-180 HP
After Pigging	-105 HP
A Reduction of 42% in HP Usage	
Kilowatt Hours Required Prior to Pigging	-3791 KW/hr
After Pigging	-2211 KW/hr
A reduction of 1580 KW/hr per day	

Assuming an average of $0.03 per KW/hr., the resulting savings are $47.40 per day or $17,301.00 per year.

These three examples demonstrate several things that clean pipes produce:

1. Lower Operating Pressures
2. Higher "C" Factors
3. Increased Flow
4. Lower Pumping Costs

All of which add up to energy savings.

The last decade has brought many new techniques to the field of pipeline cleaning. Mechanical pigging, hydroblasting, chemical cleaning and power rodding to name a few. None of these new techniques have been more revolutionary than the rubber foam pig cleaning system. Rubber foam pigs which are sometimes referred to in the trade as polly pigs come in a variety of shapes and foam densities. Basically, this cleaning unit is manufactured of open cell polyurethane rubber foam in the shape of a bullet. These pigs can be put in 3 categories:

Bare, Coated, and Scraping Types

The bare type pig comes in 2 densities. The 2 lb. density pig which is sometimes referred to as a swab, has several uses. It is usually the first unit that passes through the system for the purpose of proving the direction of the flow. It is also used behind undersized cleaning pigs in order that the seal be maintained, and finally, it is often used for sweeping the final debris out of a pipe following cleaning.

A second model of the bare unit is a 6 lb.per cubic foot model often referred to in the trade as a bare squeegee. Its main function is to prove the exact inside diameter of the pipe to be cleaned. This unit will actually shave itself down, and its diameter upon exiting is usually indicative of the true inside diamater of the pipe to be cleaned. The next category of rubber foam pigs is what is referred to as a coated or cleaning pig. This unit has the same foam body as the bare squeegee. However, on the outside is a tough coating of polyurethane synthetic rubber. This rubber is applied in a criss-cross pattern. When pressure is applied to the rear of the unit, these bands actually expand and form a wedge inside the pipeline. A significant feature is the hole found in the nose of the pig. This hole provides some jetting action through the pig keeping some of the debris in suspension out in front of the unit. Most types of buildup and growth found inside pipelines can be cleaned with this tool, however, there are those that are too hard and require more scraping action

for removal, and this brings us to the third category of rubber foam pigs.....the scraper type. This cleaning unit is again built on the 6 lb. per cubic foot squeegee foam body. It too has a tough coating of polyurethane synthetic rubber. But in addition to this, there are spirals of flame hardened steel wire brushes. These brushes are facing forward. They are self-sharpening and tend to claw away at the harder scales. It has been said that one of the greatest sources of water supply that we have in this country is sitting in the pipeline or the distribution system. Of course, that was an exaggeration, but its not far from being right. Many times the water is there and we can't pump it out.... and one of the phenomenons of friction loss is that it doesn't always do a lot of good to add another pump.

Anytime water flow is changed from laminar flow to turbulent flow, pumping costs go up and so do water pressures, and "C" Factors come down. Let me say right here that rubber foam pigs are not a cure-all. By the same token, neither is any other cleaning method. But the last decade has shown that it can be, and often is, a very effective tool. The key to their success is in knowing how to apply them. An annual meeting for the last five years with our service companies from all over the world....and by-the-way, that includes 8 service companies from Japan alone... have produced a single technique known as The Progressive Cleaning Method. Basically, The Progressive Cleaning Method means that we introduce small cleaning pigs that will just fit the opening in the pipe and progress up to the original inside diameter. The rules for progressive pigging are as follows:

1. Isolate the line to be cleaned.
2. Check to make sure that all valves are fully opened.
3. Turn on the water or gas to double check the direction of flow.
4. Run a full sized bare swab to prove the direction of flow.

5. Run a bare squeegee unit. Measure its diameter upon exiting and introduce a criss-cross type unit into the line that will just fit the "true" opening. Run a full sized bare swab behind the criss-cross unit to assure a tight seal. Continue this prcess until a unit is discharged from the line in reusable condition.
6. Increase the size of the criss-cross pigs in one inch increments until units which measure the same as the pipe I.D. are being used. For pipes with a buildup of hard scale, such as iron oxide, criss-cross wire brush pigs can be applied on the final pass.
7. Run a full sized bare swab to sweep out any loose debris.

SPECIAL NOTES FOR BEST CLEANING RESULTS:

1. To assure against excessive abrasion, do not apply more than two wire brush pigs on the final pass.
2. Launching can be accomplished using fire hydrants for lines of 8 inches or smaller, or with concentric reducers, pipe couplings, spools, eccentric reducers, in-line launchers, or by hand.
3. Ideal pigging speed is between 440 and 1,320 feet per minute.
4. For lines of less than 4 inches diameter, keep pig runs under 300 feet.

Gentlemen, these rules that I have just read to you were all learned the hard way. There are stuck pigs, there are destroyed pigs, there are blown up streets and there are pigs still lost that I'm sure will never be found. What I am saying here is that it is not enough to have a good product that works. It must be coupled with the know-how to use it. Cleaning pipelines with rubber foam pigs does not require a genius. I'm certainly living proof of that; however, a little technical assistance that first time around will be or could be the cheapest insurance that you can every buy. Therefore, I urge you to have someone with pigging experience lend you a

hand the first time you use rubber foam pigs in your system.

RESIDENTIAL PRESSURE SEWERS USING GRINDER PUMPS REDUCE ENERGY CONSUMPTION DURING CONSTRUCTION AND OPERATION

R. Paul Farrell, P.E.
Technical Director
Environment|One Corporation

1. INTRODUCTION

Wastewater transport has traditionally been accomplished by means of gravity sewer systems. These are basically man-made, well vented, underground, drainage systems. Hydraulically they operate under open channel flow conditions. The pipelines must be constructed on a precise grade and acurately aligned to insure satisfactory operation.

Historically, gravity sewers were first installed in urban areas, and usually confined to a single drainage basin. Slopes required for proper operation usually roughly paralleled surface contours so that excessive trench depths were seldom required. Until the system expanded to include more than a single drainage basin, there was little if any need for pumping. Except for the occasional line repair and cleaning, the system essentially used no energy during operation. In those days, long before the current emphasis on energy conservation, this gravity sewer system approach was an extremely appropriate low energy choice for the circumstances then existing. However, as cities grew and expanded into the suburbs, these so called "gravity" systems became far more extensive, and more often then not spanned a number of drainage basins. As a result deeper and deeper trenches were employed and extensive pumping stations and force mains were added. Thus a simple system, appropriate for its time, gradually evolved into something quite different from the standpoint of energy efficiency during both construction and operation.

In today's world, it is this evolved form of so-called "gravity sewers" which we are usually considering when comparisons are made to newer, alternative systems. It is the object of this paper to compare low pressure wastewater collection systems to these conventional "gravity" approaches from the stand-point of energy usage during construction and operation. Pressure sewer systems have been developed and introduced as a viable alternative during the past 15 years. They are intended for use in those situations where the conventional system is either technically or economically undesirable because of conditions such as: waterfront location, steep topography, high water tables, rock, and similar difficult construction conditions.

In a pressure sewer system the wastewater is prepared for transport by grinding in a compact household unit known as a grinder pump. The solids in the wastewater are reduced to a fine slurry, so it can be transported through small diameter pressure pipe. Since the flow is under pressure, these small pipelines can run over hill and dale at a constant depth just below the frost line. No deep cuts are needed and lines can easily serve more than one drainage basin.

At first glance one might think of pressure sewers as a rather energy-intensive method for collection of wastewater. Investigation of the details, however will show that, in many cases, pressure sewer systems are much more energy-efficient than their older counterpart, the so-called gravity approach.

2. PIPING MATERIAL USAGE

A typical operating pressure sewer system can be found at Weatherby Lake, Missouri. That system was constructed in 1974 after extensive engineering studies of the alternatives available. (1) Table 1 compares the polyvinyl chloride (PVC) pipe material which would have been required, to construct the proposed gravity system to the PVC material actually used, to construct the pressure sewer system.

TABLE 1

SEWER PIPE MATERIAL USAGE COMPARISON
Location - Missouri Constructed - 1974

I. PROPOSED GRAVITY SYSTEM
Material: PVC Sewer Pipe DR35

Size	Linear Ft.	#/Ft.	Weight	
4"	37,100	0.984	36,506	
8"	59,000	3.991	235,469	
10"	2,000	6.261	12,522	
		TOTAL	284,497	# PVC PIPE

II. "AS BUILT" PRESSURE SYSTEM
Material: PVC Pressure Pipe SDR-26

Size	Linear Ft.	#/Ft.	Weight	
1-1/4"	37,100	0.25	9,275	
2"	2,800	0.447	1,252	
2-1/2"	6,700	0.644	4,315	
3"	8,600	0.951	8,179	
4"	14,500	1.545	22,403	
6"	9,100	3.383	30,785	
		TOTAL	76,209	# PVC PIPE

These data show that the pressure sewer system used barely one-fourth as much PVC as the gravity system would have used. It is significant to note in passing that PVC is a petroleum based material. It is also interesting to note that until recently, gravity sewers have been constructed of far heavier materials, such as cast iron, vitreous clay, or asbestos cement.

3. ENERGY REQUIREMENT FOR TRENCH EXCAVATION

Table 2 shows typical widths and depths involved in excavation of gravity and pressure sewer trenches. In gravity systems the depth varies with respect to the surface as the construction progresses since the pipeline must be constructed on a precise grade referenced to a datum elevation. Gravity street sewers are seldom shallower than 8 feet, since they must normally serve the basement elevation of residences, and depths of 30 feet are not at all uncommon. Table 2 also shows that a gravity trench 12 feet deep and 3 feet wide will require 23,760 foot pounds of work for each linear foot of trench excavated. In comparison a pressure trench 4 feet deep and 6 inches wide will require only 440 foot pounds of work for each linear foot of trench excavated. The ratio of work required in this example is 54 to 1.

TABLE 2
RELATIVE EARTH REMOVAL REQUIRED
IN GRAVITY AND PRESSURE SEWER EXCAVATION
Earth Density = 110 lb/ft.^{3}

I. TYPICAL GRAVITY EXCAVATION

Depth, ft.	12.0*
Width, ft.	3.0
Volume per linear ft., ft.^{3}	36.0
Average depth, ft.	6.0
Weight moved lb/lin. ft.	3960
Work done - ft.lb/lin. ft.	23,760

* Shoring and bracing also required (for safety)

II. TYPICAL PRESSURE EXCAVATION

Depth, ft.	4.0
Width, ft.	0.5
Volume per linear ft., ft.^{3}	2.0
Average depth, ft.	2.0
Weight moved lb/lin. ft.	220
Work done - ft. lb/lin. ft.	440

The energy requirement for back-filling the two trenches has not been calculated but would be in approximate proportion to the volumes of earth moved, or 18 to 1.

Actual data from a pressure sewer system constructed in Florida was recently discussed at a national wastewater conference in Ann Arbor, Michigan. This information, presented in Table 3, shows that the relative work required per foot of trench constructed was about 70 times greater for the gravity system than for a pressure system.

TABLE 3

RELATIVE WORK REQUIRED IN CONSTRUCTION OF GRAVITY AND PRESSURE SEWERS (2)
Location - Florida Constructed 1977

	Gravity	Pressure
Trench Depth, Ft.	15-20	3 to 3-1/2
Trench Width	NA	NA
Crew Size, Persons	12	3
Trench Construction Rate, Ft/Day	300	5280
Relative Work Required per ft. of Trench Constructed	70	1

4. PUMPING ENERGY REQUIREMENT

Extensive demonstrations conducted on grinder pumps and pressure sewer systems during the early 70's established the detailed operating perameters for this equipment.(3) The Environment|One grinder pump was shown to run about 18 times per day for approximately 60 seconds per operation. At an average load of 800 watts, this amounts to 88 kilowatt hours of electric energy per year. Subsequent to the demonstration, a continuously energized heater for condensation control was added which consumes 100 kilowatt hours annually. This brings the total to 188 kilowatt hours per year. As shown on Table 4 this energy usage falls somewhere between a coffee maker and a black and white television set.

TABLE 4

ELECTRIC ENERGY CONSUMPTION OF
SELECTED HOME APPLIANCES

ITEM	ANNUAL ELECTRIC ENERGY USAGE - KILOWATT HOURS
Air Conditioner	2000
Clothes Dryer	1200
Refrigerator (12 cu. ft. auto def.)	750
Black & White Television	400
E|ONE GRINDER PUMP	200
Coffee Maker	100
Vacuum Cleaner	45
Clock	17

Data from: U.S. Government Publication, Citizen Action Guide to Energy Conservation and report No. EPA R2-72-091, A Pressure Sewer System Demonstration.

One of the outstanding findings from the demonstration project was the reduced wastewater volume which a pressure sewer system produces compared to conventional gravity systems. It was shown that the average per capita wastewater flow in the pressure sewer system demonstration was only 37 gallons per day. Traditional gravity systems are normally known to produce flows averaging 125 gallons per capita per day. This > 3:1 reduction in wastewater volume is reflected throughout the collection and treatment system. Additional capital and operating energy savings are realized in the treatment works, since all treatment units need be designed for only 1/3 the traditional hydraulic loading.

Pressure sewers are normally not employed unless the topography requires pumping. Therefore, in a comparison of pumping energy required between a gravity and pressure sewer system it is appropriate to assume that the relative elevation change or lift is the same with either system. The principal factors which determine overall energy consumption then become the volumes of liquid to be pumped, and the efficiency of a few large pumps vs many small ones. The conclusion as shown is Table 5, is that the pressure system has an advantage of lower volume which more than offsets the higher efficiency of

"gravity" system pumps. Thus the relative pumping energy requirement for the gravity system is about 77 percent higher than the corresponding pressure system.

TABLE 5

RELATIVE PUMPING ENERGY REQUIRED
FOR GRAVITY AND PRESSURE
SEWER SYSTEMS

	Gravity	Pressure
Per Capita Flow, gpd (1)	125	37
Relative Volume	3.38	1.0
Relative Lift (2)	1.0	1.0
Overall Pumping Efficiency (3)	0.665	0.35
Relative Pumping Energy Required (4)	1.77	1.0

(1) From "A Pressure Sewer System Demonstration" -EPA-R2-72-091.

(2) Assume that in either case, the total flow is pumped through the same elevation change or lift.

(3) Gravity system will use a small number of relatively large pumps so efficiency will be higher. Assumed efficiency; pump 70 and 50, motor 95 and 70 for gravity and pressure respectively.

(4) $\text{Energy} = K \frac{\text{Volume} \times \text{Lift}}{\text{Efficiency}}$

5. CONCLUSIONS

From the above preliminary, but basic, evaluations it is evident that pressure sewers when compared to conventional systems employing gravity piping and large pumping stations, are usually more energy efficient in the following ways:

1. Smaller pipe sizes mean less material of construction (usually PVC) in the piping system.

2. Shallow, narrower trenches require smaller, lighter, construction machinery, and the earth-moving involved in excavation is dramatically reduced.

3. The water volume transported is less in a pressure sewer system because infiltration/inflow is eliminated, and even though the small local grinder pumps are less efficient, the overall pumping energy required is significantly reduced.

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ENERGY OPTIMIZATION - ENVIRECON PRESSURE SEWER SYSTEM

Jack Weinberger, PE
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INTRODUCTION

Never before, has the economics of energy use in the treatment of municipal and industrial wastewater been so critical as it is now -- and for the foreseable future.

Not only have recent world events made this apparent, but significantly, the rising costs of power are escalating in multiple of dollars already drained by inflation to a point of unpredictability.

The need for an ecologically sound and energy conservative means of sewage waste treatment is extremely necessary in America now. Specifically, an area most directly affected is that of the small communities, towns, villages and cities that are not, or will not, be built up to maintain large, expansive and costly sewage treatment plants. Those same communities or towns also lack the major industrial plants who contribute their large sums of user charge fees. This results in an unique "Catch 22" situation for the individual homeowner, small plant, motel or institution operator who can only sit and watch his money follow his waste discharge --down the drain.

Our focus herein is on both factors because it takes a lot of money to generate a lot of energy to operate any wastewater system today. So in essence, we are concerned with energy conservation as an economic measure and simultaneously as a natural resources management conservation system. We are concerned with it as an entirety for a whole system, as well as a singular savings item within that system.

To this point our attention is directed at the use of an innovative system for processing wastewater in suburban areas, small communities, industrial-commercial parks, that is 3.5 times less costly in overall consideration than other conventional systems.

To start at the beginning --

During the 1960's, a leading Florida community developer was faced with a serious problem. Due to poor soil conditions, seasonal high water tables and, in many cases, faulty installation by independent contractors, more and more difficulties were being encountered with septic tank/drainfield systems.

The cost of providing conventional gravity sewers to a large number of sparsely populated, widely scattered communities was prohibitive.

This situation led to the design and installation of a uniquely new pressure sewer system in the area of Port Charlotte in 1970. These original units were so successful that in 1972 the Gulf Cove area of Port Charlotte was designed as a test area for expansion of the Pressure Sewer System. There are presently over thirty-three (33) units in operation there and approximately fifteen (15) in the Lakeshore Circle area.

The original installations have now experienced more than nine years of successful operation. With the completion of an ongoing demonstration program being conducted under the perusal of the State of Florida Department of Environmental Regulation there will be an additional 230 units in operation by the end of 1979 to yield a total of 465 units in the Port Charlotte and St. Lucie communities together.

Originally conceived and developed by Harold Schmidt, PE, and identified as "Suburbanaer," the system has since been expanded to reflect its broad current application and is now identified and registered as the ENVIRECON SYSTEM.

Evolution of the need for a more economical means of processing sewage arose through recognition of the financial difficulty of installing expensive gravity sewers during early stages of community growth. This need becomes more evident when one considers that during early stages of growth there are no means for a private or public agency to collect money for the more expensive gravity sewers. Indeed, in some suburban areas it may take 25 years or more before 100% density is reached. During this interval it is intuitively obvious that a fully installed conventional gravity sewer system during initial development will be high energy use, intensive and underutilized.

Ten years ago most of the technology, components, and computer simulation techniques necessary to design an alternative to a gravity sewage collection system did not exist. However, within the past ten years, perfection of submersible pumps, improved plastic pipe, and 6 years of operational experience have enabled the Envirecon Sewerage Processing System to meet these needs.

The area used for illustrative purposes in this report consists of 1517 single family lots. Most of these lots were originally sold for use as homesites for present or future occupancy by the owners. As a result, the actual growth of the area residential units will be slower than a typical housing development.

The balance of this report will illustrate how the Envirecon system will process wastewater from source through central treatment plant in a more efficient - energy saving manner than conventional gravity designs.

EXECUTIVE SUMMARY

The Envirecon's practicability of use has been proven herein through system design and simulation of normal and stressed dynamic loading upon this design.

The Envirecon is more ecologically efficient than a conventional system since input to the sewage treatment plant is lower in 5-day biochemical oxygen demand up to 68.5% and lower in suspended solids by as much as 63.5%.

The Envirecon system effectively reduces the huge capital investment requirements which would accompany installation of a gravity system. The Envirecon system will eliminate infiltration resulting in smaller central treatment plant requirements as well as saving groundwater from ecological insult. Unlike the gravity system, major portions of the capital investment required for the Envirecon system can be deferred until homes are actually built or actual need is developed. As a result through 1987, $802,775 less capital expenditures will be required for line installation alone in our existing illustrative facility.

Since the Envirecon system eliminates any possibility of groundwater and rain water infiltration, any given sewage treatment plant can service more than twice as many Envirecon connections than a conventional gravity system. Using conventional gravity standard design criteria, the 1517 lots will utilize $481,650 of investment in sewage treatment plant capacity. The installation of the Envirecon system for the same 1517 lots will utilize only $216,740 of investment in sewage treatment plant capacity and is thus a more cost-effective system, requiring some $264,910 fewer capital investment dollars.
By 1987 when all lines have been installed under both systems, the conventional gravity system will require a capital investment of $1,699,310 while the Envirecon system will require a capital investment of only $631,652, a savings of $1,067,658 or $703.79 per lot for the 1517 lots. However, this is only a part of the savings. Assuming that financing is by 7.75% serial redemption bonds, the total cost for the gravity system is $3,538,028 while the total cost for the Envirecon system is only $1,322,526, a $2,215,502 differential as shown in the following table.

Energy and operating savings are very impressive. At 100% density, the elimination of infiltration will reduce treatment plant operating costs by $43,000 per year. Savings will also accrue through a reduction in lift station operating costs of some $1,640 per year.

ENVIRECON SYSTEM
REQUIRED FACILITIES INVESTMENT AND FINANCING COSTS

	Gravity Sewerage System					Envirecon System				
Year	Plant Costs	Line Costs	Cumulative Costs	Financing Costs Principal	Financing Costs Interest	Plant Costs	Line & Tank Costs	Cumulative Costs	Financing Costs Principal	Financing Costs Interest
1977	$ 481,650	$124,037	$ 605,687	-	$ 46,940	$ 216,740	$ 70,874	$ 287,614	-	$ 22,290
1978		122,366	728,053	-	56,424		8,655	296,269	-	22,960
1979		55,156	783,209	-	60,698		11,951	308,220	-	23,887
1980		119,161	902,370	-	69,933		21,663	329,883	-	25,566
1981		98,680	1,001,050	-	77,581		20,934	350,817	-	27,188
1982		84,500	1,085,550	-	84,130		22,818	373,635	-	28,957
1983		94,620	1,180,170	-	91,463		26,173	399,808	-	30,985
1984		55,608	1,235,778	-	95,772		27,594	427,402	-	33,123
1985		78,098	1,313,876	-	101,825		29,090	456,492	-	35,378
1986		80,319	1,394,195	-	108,050		42,847	499,339	-	38,699
1987		305,115	1,699,310	$ 113,333	122,962		132,313	631,652	$ 42,667	46,293
1988 Through 2002				1,586,667	922,250				597,333	347,200
TOTAL				$1,700,000	$1,838,028				$640,000	$682,526

NOTE 1 - Financing by serial redemption bonds of $1,700,000 and $640,000 at 7.75% redeemed 1987-2002 with interest up to 1987 allocated in proportion to amount of cumulative investment required for this project.

1. AN ECOLOGICALLY-ENERGY SAVING SYSTEM

The need for an energy conservative means of treatment of sewage wastes that is ecologically sound is a vital necessity that is self-evident today. Indeed, the Water Pollution Control Act of 1972 mandated by federal law that thousands of cities and towns will have to install sanitary sewers by 1977. The ironic fact is that this means putting in gravity sewer systems whose designs date back to the ancient Roman days and which have a cost today that few cities or their inhabitants can afford. A well documented example is the village of Walton, New York. Here a village of 3,744 inhabitants with an annual budget of $575,137 has been required to outlay $9,000,000 or more than $2,400 per person or $6,000 per connection for a sanitary sewer system to replace septic tanks. Incredible!

The Walton example accents the need for a universal type system to replace the old gravity sewer concept where excessive costs due to inefficient energy and environmental utilization make such gravity sewer installations prohibitively expensive to the public. The Envirecon system was specifically developed to meet these needs.

Envirecon Method of Operation:

Interceptor Tank:

Wastewater as it leaves the home through the standard home gravity drain discharges into a 1050 gallon interceptor tank. This tank is of a size that will allow the normal anoxic (anaerobic) action common in a septic tank to take place. Its size can be the normal capacity of the septic tanks presently in use. Based upon the flows experienced, the normal water flows through the interceptor tank would be on a five to seven-day period. The anoxic decomposition starts the energy saving process since a substantial reduction in biochemical oxygen demand is removed here through natural physical, chemical, and biological processes rather than at an aerobic, high energy requiring sewage treatment plant. As a result, the central sewage treatment plant can be designed for smaller operating capacity which will increase its efficiency at less energy consumption and made more ecologically efficient and energy saving.

Sump Pump Chamber:

Subsequent to anoxic decomposition in the interceptor tanks, water discharges into a sump pump chamber where storage capacity of up to 90 gallons exists. When the water level reaches 40 gallons, a pump is activated and the pump proceeds to empty the sump pump chamber into

sealed pressure lines thus eliminating groundwater contamination. The remaining 50 gallons of capacity provides reserve capacity in the event of power failure or surge loads where inputs will exceed pumped outputs. The soluble organic matter is pumped at a minimum of electrical costs since the settling action within the interceptor tank eliminates the need for a grinder type pump and allows for the use of the highly efficient submersible pump.

Sealed Pressure Lines:

Subsequent effluent transportation takes place within sealed pressure lines resulting in no water infiltration thus utilizing less central sewage treatment plant capacity. When the effluent from these pressure lines is received at the plant, the remaining material is all in a soluble state and easily oxidized.

It should be remembered that substantial energy savings can be attributed to the fact that a substantial portion of the BOD is removed by natural anoxic processes within the interceptor tank. Infiltration is eliminated, thus requiring much smaller amounts of direct outside source energy for the plant operation.

2. IMPROVED ECOLOGICAL EFFICIENCY

This part shall briefly highlight the numerous benefits to the environment which will result by employment of the Envirecon Sewerage Processing System. The best method to illustrate this improved ecological efficiency is to track the improvement at stages through each system as shown below:

Comparison of Collection and Treatment Systems
Conventional Sewers vs. Envirecon System
Based on a Typical 1500 Connection Installation With 2.5 People/Connection

Description	Conventional Sewer (.17 #/capita)	Envirecon System
1. Household effluent	BOD_5 = 445 ppm SS = 350 ppm Q = 112.5 gal.	BOD_5 = 445 ppm (.17#/capita) SS = 350 ppm Q = 112.5 gal.
2. Interceptor Tank	Not Applicable	BOD_5 = 140 ppm (.05#/capita) SS = 130 ppm Q = 112.5 gal.

Observation: The interceptor tank effectively reduces BOD_5 by 70% and suspended solids by 63%. Where 0.17 pounds of BOD_5 is contained in the conventional sewer effluent per capita, the Envirecon System effluent is reduced to 0.05 pounds per person.

		(.17 #/capita)	
3.	Groundwater and rainfall infiltration and other contributions	BOD_5 = 200 ppm SS = 160 ppm Q_{Inf} = 137.5 gal. Q_2 = 250 gal. (100 gallons/day/capita)	Not applicable (sealed system)

Observation: Since standard design criteria is 250 gpd for conventional sewer systems while the Envirecon effluent amounts to 112.5 gpd, 208,590 fewer gallons of effluent will require treatment.

4.	Extended Aeration Process Organic Loading	Required Capacity $\frac{625.5 \text{ \#BOD}}{12.5\text{\#}/1000 \text{ ft.}^3}$ = 50,040 $ft.^3$	Required Capacity $\frac{197.0 \text{ \#BOD}}{12.5\text{\#}/1000 \text{ ft.}^3}$ = 15,760 $ft.^3$

Observation: 34,280 fewer cubic feet are required under the Envirecon system for aeration capacity at the treatment plant.

Description		Conventional Sewer	Envirecon System
5.	Extended Aeration Process Oxygen	$\frac{625.5 \text{ \# BOD}}{2.4\text{\#}/\text{\#BOD}}$ = 260.6 pounds of oxygen per day	$\frac{197.0 \text{ \# BOD}}{2.4\text{\#}/\text{\#BOD}}$ = 82.1 pounds of oxygen per day

Observation: 178.5 fewer pounds of oxygen are required under the Envirecon system for aeration process at the treatment plant.

6.	Secondary Settlement Basin Surface Loading Rate	Required Capacity $\frac{375{,}000 \text{ gallons}}{600 \text{ gal./ft}^2\text{/8 ft.}}$ = 5000 $ft.^3$	Required Capacity $\frac{168{,}750 \text{ gallons}}{600 \text{ gal./ft.}^2\text{/8 ft.}}$ = 2,250 $ft.^3$
7.	Chlorination Basin	Required Capacity $\frac{15\text{x}375{,}000 \text{ gallons})}{(60 \text{ x } 24 \text{ x } 7.481)}$ = 522 $ft.^3$	Required Capacity $\frac{15\text{x}375{,}000 \text{ gallons})}{(60 \text{ x } 24 \text{ x } 7.481)}$ = 235 $ft.^3$
8.	Plant Effluent	90% Removal Through Plant BOD = 20 ppm SS = 16 ppm Q = 250 gpd/conn.	90% Removal Through Plant BOD = 14 ppm SS = 13 ppm Q = 112.5 gpd/conn.

Observation: Plant capacity requirements are significantly reduced by utilization of the Envirecon system. A reduction in the size of the secondary settlement basin by 2,750 cu.ft., a reduction in size of the required chlorination basin by 287 cubic feet, and a reduction in output of 137.5 gallons per connection.

The merits of the Envirecon system in improving ecological efficiency have been clearly demonstrated in the above process description. More importantly as shown in Section 8 above, final output to the environment is improved by a 30% reduction in BOD_5 and a 19% reduction in suspended solids. This is reflected in the whole process energy use consideration by the major decrease in energy source demand load for normal operation vs. conventional systems.

3. THE ENVIRECON IS ENERGY SAVING

A substantial portion of the installation and operating savings attributed to the Envirecon system will be in the form of a direct reduction of energy input to install, operate, and maintain the system.

Energy Saved at Installation:

Installation of conventional sewage lines requires minimum slopes to assure required flow rates in turn required to prevent a build up and eventual blockage of the pipes by solids. As a result, the depth of the trenches run from 4 - 15 feet requiring substantial energy expenditures for initial installation and later for dewatering and excavation. Installation of Envirecon sewerage lines requires no slope and can be laid at a depth of 3.0 feet with no dewatering required.

Installation of conventional sewerage lines must be in straight lines, again due to their transportation of solids, with manholes every 400 feet. The Envirecon lines can be installed on the side of the road and can bypass obstacles. Probable cost associated with excavation, repairing, and patching of roads can be avoided.

The resulting average energy installation requirements for the conventional systems are 3.5 times as great as requirements for the Envirecon system.

Energy Saved During Operation:

As previously explained, since gravity is employed to collect the sewage, expensive lift stations must physically lift the sewage and pump it to the treatment plant in the conventional system. Within the design area of a

1,500 lot development, four such lift stations would be required under the conventional gravity design. Only one lift station will be used under the Envirecon design. Significantly, the average energy use cost of operating a lift station is $547.00 per year. Thus, the Envirecon system will save $1,640 per year, a 3 to 1 advantage!

As previously detailed, the Envirecon system will utilize signifcantly less treatment plant capacity and thus save energy costs associated with processing some 208,590 fewer gallons of sewage per day. Sewage treatment plant operating costs in the South Port Charlotte installation amount to $0.56 per 1,000 gallons and out of a total of $242,088 in operating costs, $40,228 or 16% was power costs. Thus, of the $42,635 annual savings, $6,800 worth of energy will be saved per year at 100% density. This 16% energy use and cost reduction is a major consideration when one considers that the Department of Energy (DOE) guidelines call for 7 - 10% in most industrial and municipal applications.

Energy Requirements of the Envirecon:

The Envirecon system does have energy requirements not associated with the conventional sewerage system. These are associated with the sump pump operation and interceptor tank cleaning. Power costs for pumping will be in the $1.33 per unit per year range when 100% density is achieved. Interceptor tank cleaning will most frequently be done once every 5 years at $30.00 per unit or $6.00 per year.

Energy Requirements Summarized:

Examining then the energy requirements on a per unit basis for the Envirecon as well as the conventional, clearly shows the actual energy savings at $22.11 per unit per year.

Energy Item	Total Yearly Energy Costs Envirecon System	Conventional Gravity System
1. Sump Pump Operating Costs Per Year	$ 2,018	0
2. Interceptor Tank Pump Out Costs	9,102	0
3. Lift Station Operation Costs	547	$ 2,188
4. Treatment Plant Operating Costs	34,492	77,518
Total Costs	$46,159	$79,706
Total Per Unit Cost	$ 30.43	$ 52.54

In summary, it is important to note, first that the Envirecon Pressure Sewer System provides an affordable viable alternative for sewage collection, treatment, and transmission in any type of community. Second, it is a system that allows for the facile conversion of a septic tank/soil absorption to a workable sewer system. Third, Envirecon is an example of an extremely innovative means by which to meet the serious problems of our present and future econenvironomic situation of balance between ecologic stability and energy management. Finally, the very idea of a septic tank effluent pressure sewer system is an indication of what can result from serious application of of engineering talent and the desire to improve conventional systems so routinely accepted by the engineering profession.

Thomas J. Jorling, Environmental Protection Agency Assistant Administrator for water and hazardous materials recognizes this by stating in a recent edition of Environment Reporter that the EPA intends to scrutinize new sewage treatment construction grant applications to ensure that state officials adequately consider alternative and innovative sewage treatment systems. It is encouraging that this observation has been made at such a high level of authority in a federal regulatory agency. It is a ray of hope for those of us who look to alternative systems, such as Envirecon, as a means of continuing the on-going process of improving our way of living.

SESSION 11

ENERGY ALTERNATIVES

Chairpersons: Peregrine White

Barbara Broomfield

TRACE CONTAMINANT REMOVAL FROM SECONDARY DOMESTIC EFFLUENT BY VASCULAR AQUATIC PLANTS

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INTRODUCTION AND OBJECTIVES

Advanced wastewater treatment techniques which are employed for the tertiary treatment of domestic wastewaters are energy intensive, expensive and relatively ineffective for the removals of ammonia, nitrate and total nitrogen; specific heavy metals (including selenium, mercury and boron); and trace organics including phenol (1,2,3). Arsenic, cadmium and PCB are other trace compounds which are of concern. More economical and efficient methods of trace contaminant removal will be necessary if the reuse potential of wastewaters is to be fully realized.

Biological methods for purification of wastewaters are generally considered more energy efficient and cost-effective than physical-chemical methods. Vascular aquatic plants employing solar energy as the principal energy source have been shown capable of absorption, translocation and/or metabolic breakdown of heavy metals and trace organics. Wolverton concluded from lab scale wastewater investigations that water hyacinths can remove a maximum of 0.50 mg of nickel and 0.67 mg of cadmium per gram (dry weight) plant material over a 24-hour period (4). A maximum concentration of 0.176 mg lead and 0.150 mg of mercury per gram dry plant tissue by water hyacinths has also been reported by Wolverton and McDonald (5). During the same study alligator weeds removed a maximum of 0.101 mg of lead per gram of dry plant tissue over twenty-four hours and a minimum of 0.150 mg of mercury per gram over six hours. Wolverton has reported phenol removal potential by water hyacinths at a rate of 12 mg per gram dry plant weight per day (6). The capability of vascular aquatic plants to assimilate nutrients and remove excess nitrates and phosphates from sewage effluents has also been noted (7,8,9,10,11).

Nutrient, heavy metal and trace organic removals, pathogen destruction and usable by-products (harvested plants) may be realized by stocking aquatic plants in polishing ponds subsequent to secondary biological treatment or the

inclusion of such plants in stabilization basins. Such treatment systems may represent the ultimate in energy conservation and optimization. This study was designed to compare the relative efficiency of organic, nutrient and trace contaminant removals from domestic wastewater secondary effluent by selected vascular aquatic plants. Field, batch and continuous flow studies were conducted to screen plant species and to determine the feasibility of incorporating such a treatment system on a full scale basis.

METHODOLOGY

Methodology employed for the field survey, batch screening, and continuous flow studies will be discussed as follows.

Field Survey

The species were selected for study based on a high contaminant removal efficient potential as determined by a preliminary literature evaluation and the experiences of Dr. John T. Barber and Leonard B. Thien, Department of Biology, Tulane University. Plants were divided into floating, submersed, and rooted classifications and included:

Common Name	Scientific Name	Classification
Bulrush	Scirpus Lacustric	Rooted plant
Rush	Juncus spp.	Rooted plant
Arrowhead	Sagittaria L.	Rooted plant
Water hyacinths	Eichhornia crassipes	Floating plant
Duckweed	Lemnaceae spp.	Floating plant
Water-bonnet	Pistia spp.	Floating plant
Elodea	Elodea spp.	Submersed plant
Coontail	Ceratophyllaceae spp.	Submersed plant
Alligator-weed	Alternanthera spp.	Submersed plant

Plant, water, and sediment samples were collected from various water bodies surrounding the New Orleans area and analyzed for pertinent trace contaminant concentrations. Some of these plants were cleaned and stocked in hydroponic solution for the subsequent batch screening study. Analysis for this and the remaining phases of the study were performed in accordance with Standard Methods for the Examination of Water and Wastewater (12) and Methods for Chemical Analysis of Water and Waste, EPA (13). Plant tissue was prepared for analysis by drying at 60°C over a two day period prior to weighing

followed by wet acid digestion. All values are reported on a dry weight basis.

Batch Screening Study

The batch screening study extended for approximately six months and consisted of screening the aquatic vascular plants previously listed for relative trace contaminant removal efficiency. Ninety liter aquaria were filled with secondary effluent from a trickling filter waste treatment facility and stocked with selected acclimatized mature plant species. The effluent water was spiked with quantities of As, B, Cd, Hg, Se, phenol and PCB to yield concentrations of 1,5,1,1,1,1, and 0.03 ppm respectively. Samples were withdrawn over a four week period in accordance with the testing schedule outlined in Table 1. Plants were grown in the Tulane Riverside Research Center greenhouse under constant temperature conditions of 25°C ± 5°C.

Continuous Flow Study

Based on the results of the batch screening study two plants, bulrush (Scirpus spp.) and water hyacinths were selected for the continuous flow study. Baffled, epoxy coated wooden tanks of approximately 900 liter capacity were employed during the 39 day study. Flow rates were adjusted to yield a 15 day retention for the nonrecirculation run and a 7.5 day retention for the 1:1 recirculation run. Secondary effluent was spiked as in the batch study except the boron concentration was reduced to 1 mg/1. The study was conducted in the greenhouse at 25°C ± 5°C. Testing was conducted as per the schedule outline in Table 2.

RESULTS AND CONCLUSIONS

Results of the field survey as presented in Table 3 indicate almost all of the aquatic plants exhibited very high concentration factors $\left(\frac{\mu g/gm \text{ dry plant tissue}}{\mu g/gm \text{ water}}\right)$ for most contaminants evaluated. Selenium, phenol, and mercury generally exhibited the highest concentration factors in the plants observed (48,980; 65,000; and 20,330 µg/gm dry wt. per µg/ml water, respectively). This is of particular significance since these parameters are perhaps the most difficult to remove by secondary and advanced treatment techniques. Another important finding was that the efficiency of trace contaminant removal is plant specific. For examples, duckweed exhibited a concentration for boron of over 7,000 compared to those of bulrush, Juncus spp., arrowhead, water hyacinth, coontail, and alligator weed of approximately 600 to 800.

Table 4 shows the results of the batch screening study. For most

TABLE 1

Testing Schedule-Screening Study

Parameters to be Measured	Sampling Location	
	In Pond	Plant Tissue (root, stem, leaves)
pH*	X	
Temperature*	X	
Evaporation	X	
Solar Radiation*	X	
Biochemical Oxygen Demand (BOD)	X	
Total Organic Carbon (TOC)	X	
Dissolved Oxygen* (DO)	X	
Oxidation Reduction Potential* (ORP)	X	
Phenol	X	X
Polychlorinated Biphenols (PCB)	X	X
Heavy Metals (B, Cd, Hg, As, Se)	X	X
Nitrogen (TKN, NH_3, NO_2, NO_3)	X	X
Phosphate	X	X
Fecal Coliform	X	

*Measurements to be made daily. Other parameters will be monitored daily for the first week, three times during the second, twice during the third, and at the end of the fourth week.

TABLE 2

Testing Schedule-Continuous Flow Study

Parameter to be Measured	Frequency of Analysis	Location in Sampling			Plant Tissue (root, stem, leaves)
		Influent	In Pond	Effluent	
pH	daily	X	X		
Temperature	daily		X		
Flow	daily	X		X	
Evaporation	daily		X		
Solar Radiation	daily		X		
BOD	2/week	X		X	
TOC	daily	X		X	
Total and Volatile Suspended Solids	2/week	X		X	X
D.O.	daily		X		
ORP	daily		X		
PCB	2/week	X		X	X
Phenol	2/week	X		X	X
Heavy Metals, B, Cd, Hg, As, Se	2/week	X	X	X	X
Nitrogen, TKN, NH_3, NO_2, NO_3	2/week	X	X	X	X
Phosphate	2/week	X	X	X	X
Fecal Coliform	2/week	X		X	

TABLE 3 Field Study Results

ROOTED	Bulrush		Juncus spp.		Arrowhead	
Trace Contaminant	Tissue conc. (mg/gm)	A.F.*	Tissue conc. (mg/gm)	A.F.*	Tissue conc. (mg/gm)	A.F.*
As	0.0611	2,180	0.0599	2,140	0.0632	2,260
B	0.1163	690	0.1025	610	0.1069	640
Cd	0.0019	1,900	0.0009	900	0.0014	1,400
Hg	0.0052	1,930	0.0041	1,520	0.0148	5,480
Se	0.8867	29,560	1.1433	38,110	0.8540	28,470
Phenol	0.0025	2,500	0.0035	3,500	0.0055	5,500
Total N	8.9040	2,280	5.8520	1,500	10.0520	2,570
Phosphate	1.1640	1,950	1.4120	2,360	3.9800	6,670
FLOATING	Water hyacinth		Duckweed		Water-bonnet	
As	0.0657	2,350	0.0712	2,540	0.0629	2,250
B	0.1406	840	1.2079	7,210	0.3975	2,370
Cd	0.0008	800	0.0016	1,600	0.0008	800
Hg	0.0058	2,150	0.0049	1,810	0.0549	20,330
Se	1.2390	41,300	1.1270	37,570	1.3300	44,330
Phenol	0.0226	22,600	0.0050	5,000	0.0091	9,100
Total N	11.5360	2,950	8.2810	2,120	11.6256	2,970
Phosphate	4.1880	7,010	4.3520	7,290	3.1200	5,230
SUBMERSED	Elodea		Coontail		Alligator-weed	
As	0.0648	2,310	0.0596	2,130	0.0644	2,300
B	0.4050	2,420	0.1200	720	0.1028	610
Cd	0.0008	800	0.0010	1,000	0.0040	4,000
Hg	0.0297	11,000	0.0053	1,960	0.0219	8,110
Se	1.2087	40,290	1.4694	48,980	1.1118	37,060
Phenol	0.0650	65,000	0.0262	26,200	0.0236	23,600
Total N	17.6848	4,520	13.7368	3,510	12.3368	3,510
Phosphate	5.7600	9,650	12.1040	20,270	2.1580	3,610

*Accumulation Factor $\left(\frac{\mu g/gm \text{ dry weight plant tissue}}{\mu g/ml \text{ (water)}}\right)$

TABLE 4 Batch Study Results

Trace Containment	Plant	% Removal	Concentration in water after 28 days
ROOTED			
As	Bulrush	82.1	0.20
B	Arrowhead	16.5	4.07
Cd	Bulrush	98.9	0.02
Hg	Bulrush	92.8	0.06
Se	Bulrush	94.9	0.08
Total N	Juncus spp.	99.9	0.03
Phosphate	Bulrush	89.6	0.59
PCB's	Bulrush	100.0	0.00
	Juncus spp.	100.0	0.00
	Arrowhead	100.0	0.00
FLOATING			
As	Water hyacinth	12.5	1.03
B	Duckweed	17.8	4.02
Cd	Water hyacinth	68.6	0.43
Hg	Duckweed	70.5	0.24
Se	Duckweed	11.0	1.30
Total N	Duckweed	55.2	5.99
Phosphate	Duckweed	17.9	4.74
PCB's	Water hyacinth	100.0	0.00
	Duckweed	100.0	0.00
SUBMERSED			
As	Elodea	20.7	1.01
B	Elodea	17.5	4.02
Cd	Coontail	91.1	0.13
Hg	Elodea	79.2	0.19
Se	Coontail	28.9	1.02
Total N	Alligator-weed	96.5	0.45
Phosphate	Alligator-weed	38.1	3.23
PCB's	Alligator-weed	100.0	0.00
	Elodea	100.0	0.00
	Coontail	100.0	0.00

contaminants, bulrush was observed to be the most efficient rooted species. Water hyacinth and duckweed appeared the most effective floating species for trace contaminant reduction. Duckweed, however, did not adapt well to the secondary effluent resulting in a significant decrease in productivity. Results of the submersed plants are mixed with elodea and coontail displaying poor acclimation to the secondary effluent. Alligator weed adapted well but was only effective in removing nitrogen and PCB. Based on these results bulrush and water hyacinths were selected for the followup continuous flow study.

Results of the continuous flow study indicated that recirculation enhanced pollutant removals. Results of the 7.5 day retention 1:1 recirculation run are presented in Tables 5 and 6. As indicated both water hyacinth and bulrush systems were excellent in reducing organics (BOD and TOC) and suspended solids to levels expected from a physical-chemical tertiary treatment system. This occurred in spite of the poor quality of the secondary effluent feedwater. Nitrogen removals were also very effective as was heavy metals removal. Water hyacinths were more efficient in the removal of nitrogen, whereas bulrush was much more effective in the removal of arsenic, cadmium, and selenium.

Table 6 shows the accumulation of arsenic, cadmium, and selenium in the plant root, stem, and leaves during the 39 days of study. Accumulation by at least one order of magnitude was observed for most cases. Cadmium was concentrated from 0.4 to 1138.3 µg/gm dry plant tissue by the water hyacinth with very little translocation experienced, compared to bulrush. The other metals were significantly translocated to the stem and leaves.

Preliminary results indicate negligible quantities of phenol and PCB in the effluent from both treatment systems (in most cases non-detectable levels) with feed-water spiked at 1 ppm phenol and 30 ppb PCB. Accumulation in plant tissue was also observed.

Overall results indicate that aquatic vascular plants can effectively reduce organic, nitrogen and trace contaminant content of secondary effluent to with essentially no energy requirement except solar radiation. Residue levels in most cases are less than those achievable from most tertiary physical-chemical treatment systems. Based on these results, it is recommended that full scale testing be implemented particularly in suitable climatological regions for the plant species selected.

TABLE 5

Trace Contaminant Removal by Vascular Aquatic Plants (1:1 Recirculation)

Parameter	Influent Concentration (mg/l)		Effluent Water Concentration (mg/l)			
			Water Hyacinths		Bulrush	
	Mean	Range	Mean	Range	Mean	Range
BOD_5	67.2	42.2 - 92.4	3.2	0.4 - 7.3	3.2	0.4 - 7.1
TOC	21.8	18.8 - 26.8	6.5	5.6 - 7.8	7.1	5.5 - 9.1
TSS	73.3	26.0 -107.0	1.6	0.0 - 7.0	1.1	0.0 - 4.0
TKN	21.4	8.4 - 31.5	2.6	1.6 - 5.5	2.9	0.8 - 5.6
NH_3	13.2	2.9 - 20.5	0.5	0.0 - 1.9	0.9	0.0 - 3.2
As	1.096	1.067-1.188	0.523	0.473-0.643	0.407	0.033-0.297
Cd	1.368	1.111-1.529	0.543	0.368-0.594	0.129	0.033-0.297
Se	1.673	1.567-1.925	0.827	0.682-0.995	0.159	0.055-0.352
Hg	1.842	1.350-2.133	0.061	0.038-0.118	0.041	0.031-0.060

TABLE 6

Heavy Metal Concentration in Plant Tissue (µg/gm Dry Plant Tissue)

1:1 Recirculation Run

Parameter	Roots			Stems			Leaves		
	0 days	15 days	39 days	0 days	15 days	39 days	0 days	15 days	39 days
Water Hyacinths									
As	17.8	95.3	239.4	12.6	75.9	77.0	14.0	59.4	66.0
Cd	0.4	326.7	1138.3	0.4	24.2	70.4	0.4	6.6	11.0
Se	32.2	255.2	585.9	32.2	266.2	271.7	28.4	253.0	281.6
Hg	384.0	1631.3	3078.6	528.8	787.6	495.0	460.0	782.1	453.2
Total N*	13.72	13.52	20.61	13.16	20.93	23.51	13.78	19.82	22.69
Bulrush									
As	23.4	73.7	169.4	13.6	72.6	73.7	12.7	75.9	85.8
Cd	1.2	102.3	451.0	0.4	34.1	121.0	0.4	57.0	137.5
Se	31.6	271.7	279.4	28.2	258.5	291.5	28.4	270.6	284.9
Hg	897.6	608.3	711.7	1206.4	577.2	425.7	1177.6	607.2	595.1
Total N*	8.62	9.77	13.40	11.20	11.93	10.36	12.04	9.25	16.02

*mg/gm Dry Plant Tissue

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ENERGY AND RESOURCE CONSUMPTION IN LAND AND AQUATIC TREATMENT SYSTEMS

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INTRODUCTION

Land and aquatic wastewater treatment systems are alternatives to the use of conventional mechanically intensive treatment facilities. It is the purpose of this paper 1) to assess the consumption of energy and resources in selected land and aquatic treatment systems, and 2) to compare these consumption values to those for conventional treatment facilities. To accomplish this purpose, the material to be presented is organized into sections dealing with: 1) brief descriptions of the treatment systems to be considered, 2) a review of the energy consuming functions associated with each system, 3) a summary of the basic data and information used in the energy and resource analyses, 4) energy and resource consumption estimates for the selected treatment systems, and 5) the conclusions derived from the energy and resource analyses.

ALTERNATIVE TREATMENT SYSTEMS

To assess the consumption of energy and resources in the treatment of wastewater, alternative systems that can be used to achieve essentially the same level of treatment must be compared. For this paper it is assumed that the influent is domestic wastewater with BOD_5 and SS equal to 220 mg/L. The treatment systems for which energy and resource requirements are to be evaluated are listed in Table 1. As shown in Table 1, two commonly used conventional treatment systems are to be compared with land and aquatic systems, which are the special focus of this paper.

Conventional Treatment Systems

The two conventional treatment systems to be used as a basis of comparison are the activated sludge and trickling filter processes. Both of these systems can be used to meet the effluent requirements specified by the Environmental Protection Agency for secondary treatment (BOD_5 = 30 mg/L and SS = 30 mg/L.) Detailed descriptions of these systems may be found in Reference 3.

Land Treatment Systems

Overland flow, rapid infiltration, and slow rate application are the land treatment systems for which energy and resource requirements are to be evaluated. Depending on the system flooding, spraying with solid set sprinklers, and ridge and

Table 1

WASTEWATER TREATMENT SYSTEMS FOR WHICH ENERGY AND RESOURCE REQUIREMENTS ARE TO BE EVALUATED

No	Description	Designation
	Conventional treatment systems	
1.	Conventional activated sludge + chlorination	AS+c
2.	Conventional high rate (rock type) trickling filter + chlorination	TF+c
	Land treatment systems[a]	
3.	Primary + overland flow (flooding) + chlorination	P+OF(f)+c
4.	Primary + rapid infiltration (flooding)	P+RI(f)
5.	Primary + slow rate (solid set sprinklers)	P+SR(sss)
6.	Primary + slow rate (ridge and furrow)	P+SR(r&f)
7.	Facultative pond[b] + overland flow (flooding)	FP+OF(f)
8.	Facultative pond[b] + rapid infiltration (flooding)	FP+RI(f)
	Aquatic treatment systems[a]	
9.	Primary + artificial wetland + chlorination	P+AW+c
10.	Primary + water hyacinths + chlorination	P+WH+c
11.	Facultative pond[b] + artifical wetland	FP+AW
12.	Facultative pond[b] + water hyacinths	FP+WH

[a] As shown, all of the land and aquatic treatment systems are composed of two treatment subsystems.

[b] Includes influent screening to remove coarse solids.

furrow irrigation are the methods used to apply the wastewater. Pretreatment options to be used in conjunction with the land systems include: 1) conventional primary and 2) facultative ponds. Thus each complete land treatment system in Table 1 is composed of two subsystems. The expected effluent quality with either pretreatment option would be classified as advanced secondary (BOD_5 = 10 mg/L, SS = 10 mg/L). Additional details on these processes may be found in References 7, 8, and 9.

Aquatic Treatment Systems

Artificial wetlands (such as marshes or meadows) and water hyacinth covered basins are the aquatic treatment systems for which energy and resource requirements are to be evaluated. An irrigation type distribution system would be used to apply wastewater to the artificial wetlands. Parallel plug flow channels would be used in the water hyacinth systems. As with the land systems, primary and facultative ponds are the pretreatment options for the aquatic systems so that each complete aquatic system is composed of two subsystems. The expected effluent from the wetland systems would be classified as secondary. Additional details on these systems may be found in References 2 and 11.

ENERGY AND RESOURCE CONSUMING FUNCTIONS IN WASTEWATER TREATMENT

Energy and resource consuming functions for the treatment systems or subsystems reported in Table 1 are presented in Table 2. For the land and aquatic treatment systems that are composed of two subsystems, energy and resource consuming functions common to each subsystem, such as building heating and cooling, are reported with the first subsystem. Where appropriate, important factors affecting the estimation of energy and resource consumption such as the total dynamic head and the chlorine dosage are also identified.

BASIC DATA AND INFORMATION USED FOR COMPUTATION OF ENERGY AND RESOURCE CONSUMPTION

In evaluating energy consumption in treatment systems, both primary and secondary energy must be considered. Primary energy consumption is usually defined as the amount of electricity and fuel consumed in the day-to-day operation of the treatment system. Secondary energy consumption is defined as the amount of energy expended in the initial construction of the treatment facilities and in the manufacture of spare parts and supplies and chemical resources such as chlorine. Secondary energy consumption is estimated on the basis of the cost of the parts, supplies, or resources or, where available, on actual data on the energy used in manufacture of these items. Secondary energy consumed in transmitting electricity and transporting fuel has not been considered in this paper. It is assumed that the secondary energy associated with the transporation of parts and supplies and resources is reflected in their cost.

Cost Indexes and Energy Computation Criteria

The cost indexes and energy computation criteria used in this paper are presented in Table 3. As shown, all cost data have been adjusted to an Engineering News Record construction cost (ENRCC) index of 3,000, which corresponds to the value of the index in June 1979. The corresponding EPA sewage treatment plant

Table 2

ENERGY AND RESOURCE CONSUMING FUNCTIONS ASSOCIATED WITH THE CONVENTIONAL TREATMENT SYSTEMS IDENTIFIED IN TABLE I

Function	Value[a,b]
Activated sludge	
Primary treatment (as shown below)	
Aeration, mechanical	1.0 lb O_2/lb BOD_5 removed[2]
Secondary settling	600 gal/ft^2·d
Thickening, gravity[c]	
Chlorination	10 mg/L
Vehicle operation (1 mgd plant size)	1500 gal/y[d]
Trickling filter	
Primary treatment (as shown below)	
High rate rock filters	0.4 gal/ft^2 min TDH = 10 ft. RR = 2:1
Secondary settling	600 gal/ft^2·d
Thickening[c]	
Chlorination	10 mg/L
Vehicle operation (1 mgd plant size)	1500 gal/y[d]
Primary treatment	
Influent pumping	TDH = 12 ft
Screening	
Primary settling	1,000 gal/ft^2·d
Truck hauling of sludge	20 miles round trip
Land spreading of sludge	
Bldg. heating and cooling	
Vehicle operation (1 mgd plant size)	500 gal/y[d]
Misc. lighting etc.	
Facultative pond	
Influent pumping	TDH = 12 ft
Screening	
Bldg. heating, cooling	
Vehicle operation (1 mgd plant size)	1,500 gal/y[d]
Misc., lighting, etc.	

Function	Value[a,b]
Overland flow (flooding)	
Pumping	TDH = 12 ft
Vehicle operation (1 mgd plant size)	1,500 gal/y[d]
Chlorination (following primary only)	5 mg/L
Rapid infiltration (flooding)	
Pumping	TDH = 12 ft
Vehicle operation (1 mgd plant size)	1,500 gal/y[d]
Slow rate irrigation	
Pumping (solid set sprinklers)	TDH = 100 ft
Pumping (ridge and furrow)	TDH = 12 ft
Vehicle operation (1 mgd plant size)	1,500 gal/y[d]
Artificial wetlands	
Pumping	TDH = 12 ft
Vehicle operation (1 mgd plant size)	1,500 gal/y[d]
Chlorination (following primary only)	5 mg/L
Water hyacinths	
Pumping	TDH = 12 ft
Vehicle operation (1 mgd plant size)	1,500 gal/y[d]
Harvesting	
Hauling	20 miles round trip
Landfilling	
Chlorination (following primary only)	5 mg/L

[a] Unless otherwise noted, energy computations for the various functions were based on values given in References 4,5,8, and 12.

[b] TDH = total dynamic head, gal/ft^2·d = gallons per square foot per day, gal/y = gallons per year, RR = recirculation ratio.

[c] Units not included in plants with a capacity less than 1.0 mgd.

[d] Gasoline

Table 3

BASIC DATA AND INFORMATION USED TO ADJUST COST DATA AND FOR ENERGY CONSUMPTION COMPUTATIONS

Item	Value
Cost indexes[a]	
ENRCC Index	3,000[b,c]
EPA STP Index	334.1[d]
EPA O&M Index	2.54[e]
Bases for energy computations	
Mechanical equivalent of heat	3413 Btu/kW•h
Heat rate (6)	$\frac{\text{heat supplied in fuel, Btu}}{\text{energy generated, kW•h}}$
	$\frac{\text{3413 Btu/kW•h}}{\text{conversion efficiency}}$
Heat rate used in report	10,800 Btu/kW•h[f]
Heating value for gasoline	124,000 Btu/gal[g]
Energy required for manufacture of chlorine	42 x 10^6 Btu/ton (12)
Factor used to estimate secondary energy for construction	70,000 Btu/$ in 1963[h]
Factor used to estimate secondary energy for supplies and parts	75,000 Btu/$ in 1963[h]

[a] Reported values are for June 1979

[b] Basis for adjusting cost data given in this paper

[c] 1913 = 100

[d] 1957 - 1959 = 100

[e] 1967 = 1.0

[f] Assumed conversion efficiency = 31.6 percent (10)

[g] To convert the Btu value of gasoline to primary energy in terms of fuel oil, the given value must be multiplied by 1.208 (5)

[h] To use the reported conversion factors, current cost data must be converted to the equivalent cost in 1963. This conversion can be accomplished using the 1963 ENRCC index which was equal to 900.

(STP) construction and operation and maintenance (O&M) cost indexes are also reported for purposes of comparison.

The heat rate used to convert electrical energy to primary energy in terms of fuel oil is 10,800 Btu/kW•h. This heat rate value is obtained by dividing the value for the mechanical equivalent of heat by an assumed efficiency of 31.6 percent. Values for converting the cost of construction and parts and supplies to primary energy intensity are also reported. A factor of 1.208 has been used to convert the Btu value for gasoline to an equivalent Btu value for fuel oil.

Land Requirements and Construction Costs

Land requirements and construction costs for the treatment systems or subsystems listed in Table 2, are given in Table 4. The area requirements for the subsystems are estimated on the basis of the complete treatment flowsheets identified in Table 1. For example, the area requriement for an overland flow system receiving either primary or facultative pond effluent would be 60 acres for a plant size of 1.0 mgd (see Table 4). The area requirements for the land and aquatic subsystems reported in Table 4 will vary with the type of wastewater and pretreatment. The total construction cost for the land and aquatic treatment systems identified in Table 1 would be composed of the combined costs for the individual subsystems.

Labor Requirements and Parts and Supplies Costs

Labor requirements and parts and supplies costs for the treatment systems or subsystems identified in Table 4 are presented in Table 5. The total labor and parts and supplies costs for the land and aquatic systms given in Table 1, are obtained by adding the values for the individual subsystems. In deriving the labor requirements for the land and aquatic treatment subsystems, it has been assumed that some of the labor requirements would be shared between the subsystem comprising the complete treatment system given in Table 1.

ENERGY AND RESOURCE CONSUMPTION IN ALTERNATIVE TREATMENT SYSTEMS

Using the data and information presented previously and the data from the literature (4, 5, 8, and 12), the consumption of energy and resources has been estimated for the complete treatment systems identified in Table 1. The results of these analyses for treatment plants with capacities of 0.1, 0.5, and 1.0 mgd are presented in Tables 6, 7, 8 respectively. As shown, both primary and secondary energy requirements are given for each system. Chlorine (which has both a primary and secondary energy component) is the only chemical resource considered in this report.

To obtain the total amount of energy consumed annually, expressed in Btu/y, the primary electrical energy expressed in kW•h/y was converted using a heat rate of 10,800 Btu/kW•h (see Table 3). Reviewing the total energy data presented in Tables 6, 7, and 8, in can be seen that the energy differential between land and aquatic systems and conventional treatment systems increases with increasing plant size, and that the energy scale factor for the plant sizes used in this analysis is nonlinear. Note also that, depending on the system and based on a return period of 20 years, secondary energy requirements for initial construction vary from about 22 to 54 percent of the total annual energy consumption.

Table 4

LAND REQUIREMENTS AND CONSTRUCTION COSTS FOR TREATMENT SYSTEMS OR SUBSYSTEMS IDENTIFIED IN TABLE 1

System or subsystem	Land reqd., acre			Total constr cost, $\$x10^{-6}$[a]		
	Plant size, mgd			Plant size, mgd		
	0.1	0.5	1.0	0.1	0.5	1.0
Activated sludge[b]	1.0	2.5	4.0	0.710	1.234	1.600
Trickling filter[b]	1.5	3.0	5.0	0.732	1.226	1.700
Primary	0.5	0.8	1.5	0.191	0.252	0.450
Facultative pond	5.0	15.0	30.0	0.320	0.815	1.350
Overland flow (f)	6.0	30.0	60.0	0.210	0.410	0.600
Rapid Infiltration (f)	2.0	10.0	20.0	0.120	0.220	0.340
Slow rate (sss)	16.0	80.0	160.0	0.290	0.650	1.050
Slow rate (r&f)	16.0	80.0	160.0	0.230	0.460	0.680
Artificial wetland	4.0	20.0	40.0	0.170	0.300	0.450
Water hyacinths	2.0	10.0	20.0	0.140	0.270	0.380

[a]The cost of the land is not included.

[b]Reported data are for complete treatment system.

Table 5

LABOR REQUIREMENTS AND PARTS AND SUPPLIES COSTS FOR TREATMENT SYSTEMS OR SUBSYSTEMS IDENTIFIED IN TABLE 1

System or subsystem	Labor reqd., p•h/y[a]			Parts and supplies, $/y		
	Plant size, mgd			Plant size, mgd		
	0.1	0.5	1.0	0.1	0.5	1.0
Activated sludge[b]	1,600	3,600	5,500	8,000	12,000	16,000
Trickling filter[b]	1,200	2,800	4,200	6,000	10,000	14,000
Primary	500	750	1,000	1,500	2,000	3,000
Facultative pond	1,000	2,000	2,500	4,000	6,000	8,000
Overland flow (f)	1,000	2,400	3,200	5,000	7,500	10,000
Rapid infiltration (f)	1,000	2,200	3,000	4,000	7,000	9,000
Slow rate (sss)	1,500	2,400	3,200	5,000	7,500	10,000
Slow rate (r&f)	1,500	2,400	3,200	5,000	7,500	10,000
Artificial wetland	500	1,500	2,000	2,000	2,500	3,000
Water hyacinths	1,500	2,000	3,000	2,000	3,500	5,000

[a]p•h/y = person hours per year

[b]Reported values are for complete treatment systems

Table 6

ENERGY REQUIREMENTS FOR ALTERNATIVE TREATMENT SYSTEMS WITH A PLANT CAPACITY OF 0.1 MGD

System[a]	Primary energy		Secondary energy, (Btu/y)x10^{-6}			Total, (Btu/y)x10^{-6}
	Electricity, (kW•h/y)x10^{-3}[b]	Fuel, (Btu/y)x10^{-6}	Plant constr.[c]	Chemicals, etc.	Parts and Supplies	
AS+c	34	515	746	64	180	1872
TF+c	22	435	769	64	135	1641
P+OF(f)+c	12	384	421	32	146	1112
P+RI(f)	11	384	327		124	953
P+SR(sss)	27	384	505		146	1326
P+SR(r&f)	11	384	327		146	976
FP+OF(f)	6	400	557		202	1223
FP+RI(f)	6	400	462		180	1106
P+AW+c	12	384	379	32	79	1003
P+WH+c	12	434	348	32	79	1022
FP+AW	6	400	515		135	1114
FP+WH	6	450	483		135	1132

[a]See Table 1.

[b]To convert to Btu/y multiply indicated value by 10,800.

[c]Based on a return period of 20 years.

Table 7

ENERGY REQUIREMENTS FOR ALTERNATIVE TREATMENT SYSTEMS WITH A PLANT CAPACITY OF 0.5 MGD

System[a]	Primary energy		Secondary energy, (Btu/y)x10^{-6}			Total, (Btu/y)x10^{-6}
	Electricity, (kW•h/y)x10^{-3}[b]	Fuel, (Btu/y)x10^{-6}	Plant constr.[c]	Chemicals, etc.	Parts and Supplies	
AS+c	134	1700	1296	320	270	5033
TF+c	71	1260	1287	320	225	3859
P+OF(f)+c	40	765	695	160	214	2266
P+RI(f)	40	765	496		203	1896
P+SR(sss)	112	765	947		214	3136
P+SR(r&f)	37	765	748		214	2126
FP+OF(f)	27	500	1286		304	2382
FP+RI(f)	27	500	1087		293	2171
P+AW+c	40	765	580	160	101	2038
P+WH+c	40	1045	548	160	124	2309
FP+AW	27	500	1171		191	2154
FP+WH	27	780	1139		214	2425

[a]See Table 1.

[b]To convert to Btu/y multiply indicated value by 10,800.

[c]Based on a return period of 20 years.

Table 8

ENERGY REQUIREMENTS FOR ALTERNATIVE TREATMENT SYSTEMS WITH A PLANT CAPACITY OF 1.0 MGD

System[a]	Primary energy		Secondary energy, (Btu/y)x10^{-6}			Total, (Btu/y)x10^{-6}
	Electricity, (kW•h/y)x10^{-3}[b]	Fuel, (Btu/y)x10^{-6}	Plant constr.[c]	Chemicals, etc.	Parts and Supplies	
AS+c	237	2240	1680	640	360	7480
TF+c	130	1940	1785	640	315	6084
P+OF(f)+c	74	1280	1103	320	293	3795
P+RI(f)	69	1280	829		270	3124
P+SR(sss)	209	1280	1575		293	5405
P+SR(r&f)	69	1280	1167		293	3485
FP+OF(f)	52	750	2048		405	3765
FP+RI+(f)	52	750	1775		383	3470
P+AW+c	74	1280	945	320	135	3479
P+WH+c	74	1780	871	320	180	3950
FP+AW	52	750	1890		248	3450
FP+WH	52	1250	1817		292	3921

[a] See Table 1.

[b] To convert to Btu/y multiply indicated value by 10,800.

[c] Based on a return period of 20 years.

CONCLUSIONS

From even a cusory review of the data presented in Tables 6, 7, and 8, it is clear that significant savings in the amount of energy and resources consumed for the treatment of wastewater can be achieved using both land and aquatic treatment systems. Based on these results, it would appear that the cost-effectiveness techniques now used to compare treatment alternatives should be reassessed so that, where applicable, land and aquatic treatment systems are not eliminated from consideration because of land and labor costs.

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INNOVATIVE DESIGN IN THE REDUCTION OF ENERGY FOR LAND APPLICATION OF EFFLUENTS

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INTRODUCTION

The EPA has mandated that land application be considered as one of the alternatives in the evaluation of systems to meet water quality standards. When moderately priced suitable land is available not too far distant from the treatment plant, land application will compete favorably with conventional advanced waste treatment methods. Most of the present land treatment systems utilize the sprinkler method of water application. Because of the cost of labor, most of the sprinkler systems are automated systems, usually center-pivot systems. Some disadvantage of sprinkler systems are the initial investment, the cost of producing the pressure needed, the amount of maintenance required, traction problems under some situations, and the possibility of the productions of aerosols containing virus. Conventional surface irrigation has a high labor requirement, higher land leveling costs, an aversion to contact with sewage contacted equipment, and a lack of experience of sanitary engineers with the design of surface irrigation systems. Certain EPA Case Studies at San Angelo, Texas and Bakersfield, California, have indicated that surface irrigation is a low cost alternative that has been satisfactorily used with primary treated effluent.

An automated surface irrigation system which eliminates labor, avoids contact with the effluent, eliminates aerosals, and has a moderate installation and operating cost should find acceptance. The purpose of this paper is to suggest how such an existing system could be adapted to land application of effluents.

DESCRIPTION OF AN AUTOMATED FURROW IRRIGATION SYSTEM

Figures 1 and 2 show a sequence of operations on an existing automated furrow irrigation system.

Figure 1 shows the check dam closed at the end of the first Bay. Water is being discharged through the tubes of that bay.

Figure 2 shows the check dam has been removed from the end of Bay 1.

Bay 1 is shut off and Bay 2 is irrigating. Irrigation continues down the ditch in this manner.

The principles used in the design of the furrow irrigation system will be used in the design of an energy efficient border system for land application of sewage effluents.

Figure 3 is an elevation drawing of automated delivery system using hooded-inlet discharge tubes for a border irrigation system. The drop between bays would be greater than the head on the invest. Each successive bay is irrigated as the check dam of the previous bay is released. This could be done either manually or by time clock depending on the desires of the designer.

Figure 4 shows the relationship of head to discharge for galvanized pipes 2.1 feet long with hooded inlets. A standard 21 foot length of pipe would produce 10 tubes.

DESIGN OF AN AUTOMATED BORDER IRRIGATION SYSTEM

A system of 1 mgd. will be designed using a slow rate system. A crop will be produced in the summer, but sufficient growth will remain to form a grass filter bed during the winter.

Figure 5 shows a drawing of a suggested 1 mgd. system of 160 acres. The city involved should own the land. The first step is to build a security fence around the field. Another feature is a grass buffer strip completely surrounding the application area. An all-weather access road surrounds and crosses the design area adjacent to the concrete ditches. The field consists of 48 dead level border checks 95 x 1170 feet with 1½-foot high borders completely around each border check. The system is designed to apply the flow that arrives every day and to store any rainfall that might be expected to occur. No external storage will be needed. This means that moderately permeable soils with no impermeable layers will be needed. If long periods of drying between irrigations are available, and if a good grass cover is maintained the soil permeability will be maintained at satisfactory levels. If no runoff occurs, no storage pond and return flow system will be needed.

The crop suggested for southern areas is bermuda grass, based on nutrient removal, submergence tolerance, and plant characteristics. The last cutting should occur early enough to permit a good growth of grass before dormancy. Most cities would probably not make money farming, so it is suggested

that the grass should be contracted as a standing crop to accomplish nutrient removal. Experience may show a winter crop of rye-grass or similar to be desirable.

The depth applied per irrigation will depend on the slope of the land in the direction of the head ditch. One million gallons per day equals 3.07 acre-feet. Each border strip contains 2.55 acres. If four borders are irrigated for a day, 3.6 average inches will be applied. With 48 borders available the borders will be irrigated every 12 days. If three borders are irrigated for a day, 4.8 average inches will be applied every 16 days. If two borders are irrigated for a day, 7.2 average inches will be applied every 24 days. The design head will be approximately constant, and the drop will be greater than the head. The drop divided by the distance between check dams will be equal to the average slope in the direction of the ditch. The system should be applicable to a range of slopes.

The application area is 122.5 acres. One mgd. will apply 110 inches per year to this area. The annual contribution to groundwater = 110 inches + rainfall - (crop use + evaporation). The pretreatment required would probably be primary treatment. The underground strata should be such as to accomodate this increase in groundwater. If the groundwater mound reaches to within about six feet of the surface, subsurface drainage would be indicated.

Assume a slope in the direction of the head ditch of 0.3 feet per 100 feet. If three border strips are irrigated at a time the drop is 3 borders x 95 ft. width x $\frac{0.3}{100}$ = 0.855 feet. The drop should be larger than the head, so assume a 0.75 head available. If a peak flow of twice the average daily flow is assumed, the peak would be 1.55 x 2 = 3.1 cfs. From Figure 4, a 3-inch tube with a 0.75 feet head will discharge 0.22 cfs.

$\frac{3.1}{0.22}$ = 14 tubes will be required, use 15 tubes

$\frac{3.1}{15}$ = .207 cfs. The head from Figure 4 = 0.68 ft.

Five tubes per border strip will be needed and a check dam will be located at each third border. The top of the check dam should be set low enough in the ditch so that if excess flows occur they will be passed over the dam to the next border strips.

If monitoring indicates the need, a drain tile could be placed under the borders, with the spacing determined by soil permeability.

ECONOMIC AND ENERGY ANALYSIS

Compare the energy costs with a center pivot system operating at 75 psi

at a pump efficiency of 75 percent. If we assume a 2-foot head in the ditch.

Horsepower saving = $\frac{694 \text{ gpm} \times (173.25 - 2.0)\text{ft.}}{3960 \times 0.75}$ = 40.0 Horspower

Horsepower hours/year = 350,400

KWH at 85% motor efficiency = 307,528

Power Cost at $0.04 per KWH = $12,301

Assume a 20 year life at 10% interest

Amortization factor = 0.1175

Additional investment justified = $\frac{12,301}{0.1175}$ = $104,689

This is about $654 per acre of additional investment above the cost of a center-pivot system which could be justified by the energy saving. Higher values of assumed power costs will result in proportional increases in justified investment. This is in addition to the savings in depresiation, maintenance and labor which an automated surface irrigation system would have. The installation cost should not greatly exceed a sprinkler system. With minimum maintenance, the system should last as long as a concrete ditch, which is usually considered to have a life of 20 years or longer.

SOME COMMENTS

Many of my colleagues do not agree that this is the proper approach to the design of land application systems for sewage effluent. They put more emphasis on the use of the effluents for agricultural production. If the application is limited to crop needs, about 3 to 6 times as much land will be required, depending on the local rainfall. Storage will need to be provided for 4 to 8 months per year or longer. The storage facility will become a significant part of the total cost. If the effluent is applied to sloping fields, a collection pond and a return flow system will be needed for runoff losses.

This system will require less labor than any other system, surface, or sprinkler. Maintenance requirements will be minimal. Clogging problems should be non-existent. Labor skills need not be high. The energy required would be as low as any system. There is no need for personal exposure to the effluent. No aerosols will be produced by this system.

Many questions remain. It is not known how far north the system would function year-around. For more northern climates, bermuda grass would not be adapted. A crop, such as rye-grass, may be desirable in the winter instead of dead bermuda grass. Some form of mosquito control may be needed in

extremely wet years.

I hope this paper has suggested to the reader that there may be better methods of land application, but we will not know unless we try some more innovative systems.

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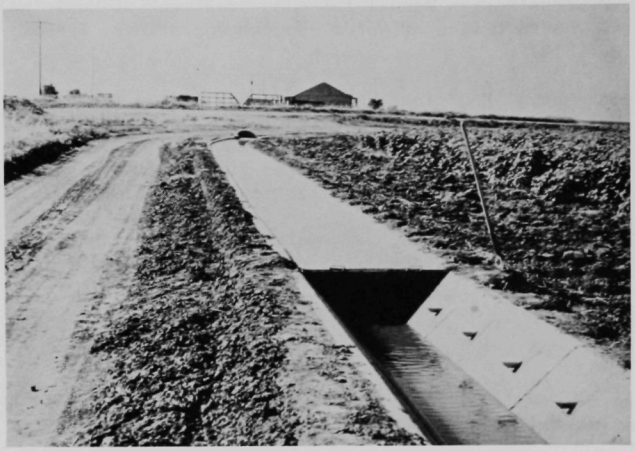

Figure 1. An Automated Furrow Irrigation System installed on the Irrigation Research Station, Altus, Oklahoma. The outlet tubes in this first bay are submerged and water is being delivered to the furrows.

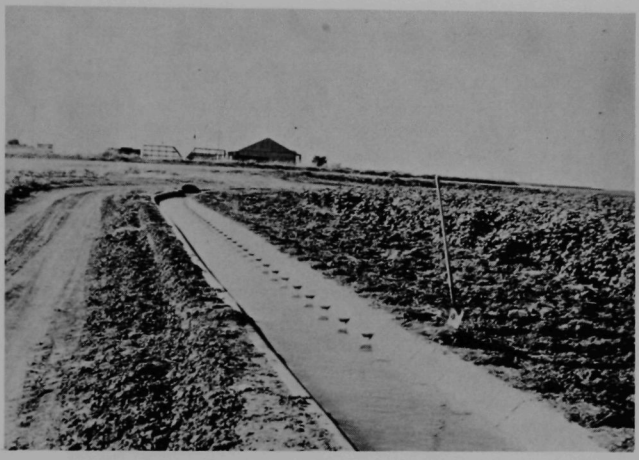

Figure 2. The check dam has been removed from the First Bay and the water level falls below the tubes. This waters Bay 2 and automatically shuts off the flow to Bay 1. The labor required for this 15-acre field is the removal of five check dams.

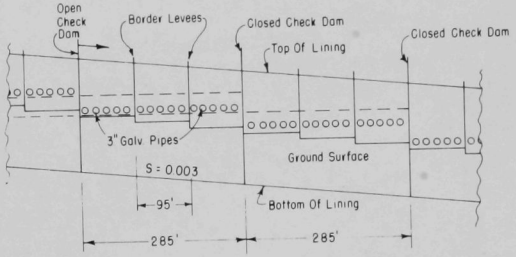

Figure 3. An elevation drawing of a proposed automated border irrigation system for land application of effluents. In this example, three 95 foot borders are being irrigated each day by a check dam. Depending on the slope, the individual borders, or groups of three, could be levelled in the same horizontal plane.

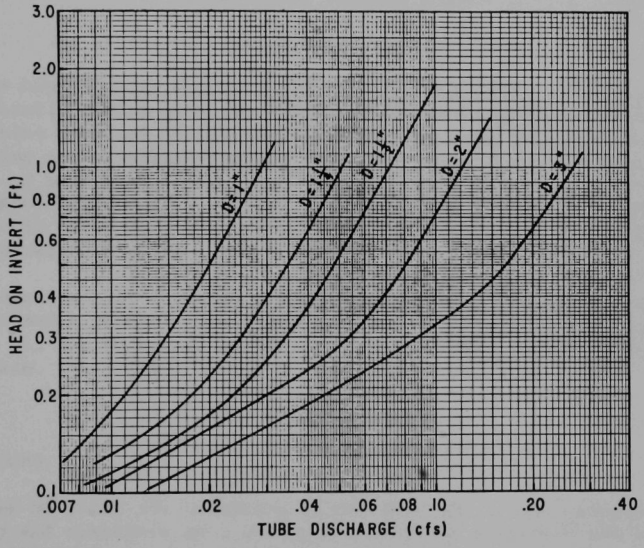

Figure 4. Relationship of head to discharge for galvanized pipes 2.10 feet long with hooded inlets.

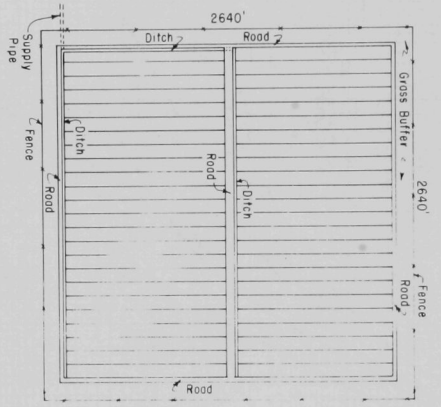

Figure 5. A conceptual design of a 1-mgd. effluent disposal system on a quarter-section of land. Features are, an enclosing privacy fence, a grassed border strip, an all-weather road system, the automated ditch system described, and 48 dead level borders with enclosing levees. This system would operate 24 hours per day, 365 days per year.

UPGRADING ROCK TRICKLING FILTER WITH PLASTIC MEDIA FOR ENERGY OPTIMIZATION

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1.0 INTRODUCTION

Today's high cost of energy along with the availability of highly efficient plastic media, make the trickling filter a viable solution to wastewater treatment problems. It is estimated that several thousand municipal rock trickling filter plants are in use today. Many of these installations are in need of upgrading to increase capacity or improve performance. Replacing rock media with plastic media or adding modern trickling filters containing plastic media in series or parallel are cost and energy effective means for upgrading treatment.

This paper is also directed toward the use of a highly efficient sheet type plastic media recently developed and now being used in depths from 6 ft. to 30 ft. for modern trickling filters, denitrification, and odor removal towers.

2.0 DISCUSSION

2.1 Background

For over 50 years the trickling filter has been widely recognized in this country and elsewhere as a reliable biological treatment process. Subsequently stabilization lagoons, activated sludge in various modes, and more recently rotating biological contactors have become a popular choice for municipal installations.

However, the trickling filter utilizing high efficiency plastic media when compared to other wastewater treatment processes offers features which must be considered.

1) Low energy in terms of capital and operating expenditures.
2) Stable performance.
3) Rapid recovery capability from hydraulic and organic shock loads.
4) Unsophisticated operational procedures.
5) Effective land utilization.
6) Reduced sludge bulking problems.

Plastic media has been utilized in trickling filters for more than 20 years and until recently been available in only two basic forms - random and sheet type.

Random media is characterized by small cylindrical disks manufactured primarily from plastic and randomly dumped in the trickling filter. This media, although providing increased specific surface area and void ratio, is often characterized by many of the adverse features of rock, such as plugging, settling, and channelization. Some of the older types of sheet media were characterized by surface shapes offering little or no lateral redistribution and reduced retention time.

The remainder of the discussion deals with the description and performance of the newly developed plastic sheet type media.

2.2 Description

As shown in Figure 1 the PVC media is fabricated from completely corrugated rigid PVC sheets into modules measuring approximately 2 ft. high x 2 ft. wide x 4 ft. long. Each sheet in a module is cross corrugated with the adjacent sheet. No flat sheets are used. This enables the wastewater moving through the media to be constantly redistributed horizontally creating a turbulent mixing action and increasing the contact time between the wastewater and media biofilm. Figure 2 illustrates further details of construction and sizes (surface areas) available.

The photograph in Figure 5 shows the media being installed. The modules in each layer are oriented 90° to the layer immediately above and below. This not only enhances the internal redistribution, illustrated in Figure 1, but also improves the structural integrity. The modules are easily handled by one man and can be cut in the field to conform to any configuration.

2.3 Performance

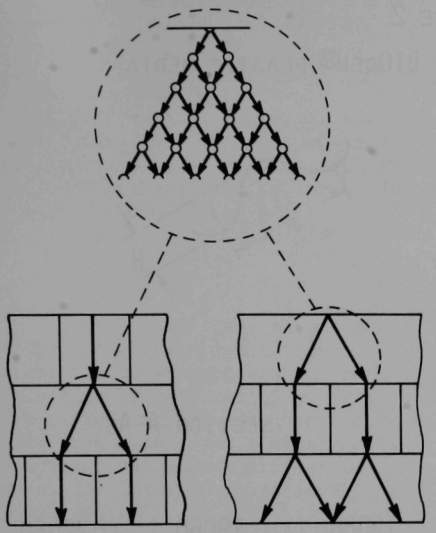
Figure 1 illustrates the lateral path and redistribution of the air and wastewater as they move through this type of media. The numerous redistribution points (over 120 points per cu.ft. of media) provide a turbulent mixing action which increases the liquid film diffusion of the wastewater across the bio-mass.

The removal rate of dissolved organic matter is highly influenced by liquid film diffusion of the soluble organics into the biofilm. The shape of the media is important. A smooth surface without mixing points produces the greatest resistance to liquid film diffusion.

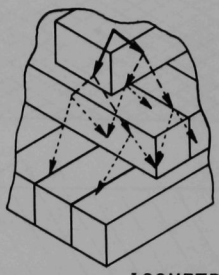
The redistribution characteristic of the media also provides for increased contact time of the wastewater with the media substrate. Figure 3 shows a comparison of retention times between two sheet medias at various hydraulic loadings in a trickling filter as reported in the literature.[1]

FIGURE 1

FLOW CONFIGURATION OF BIODEK® PLASTIC MEDIA

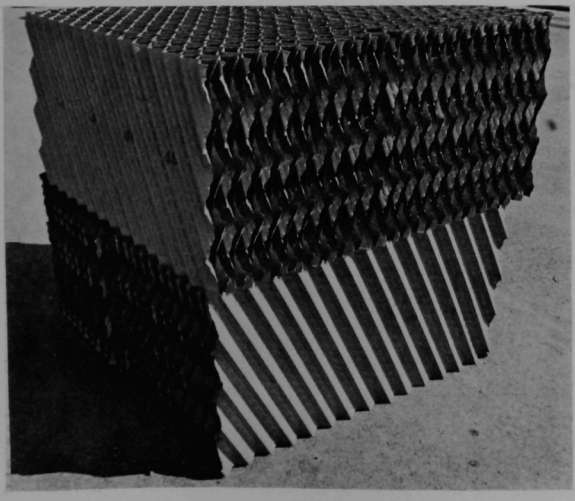


FRONT ELEVATION SIDE ELEVATION



ISOMETRIC VIEW

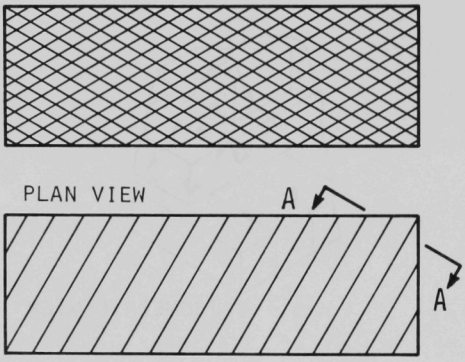
→ = LIQUID PATH

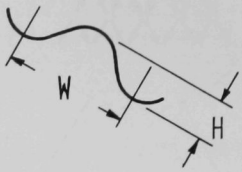


PHOTOGRAPH OF BIODEK® MODULES ORIENTED AS IN DIAGRAM ABOVE

FIGURE 2

REPRESENTATIVE SIZES OF BIODEK® PLASTIC MEDIA





SECTION A-A

ELEVATION VIEW

DESIGNATION	12060	19060	27060
FLUTE HEIGHT H IN	0.46	0.75	1.07
MM	12	19	27
FLUTE WIDTH W IN	1.1	1.9	3.1
MM	28	48	79
SURFACE AREA SQ FT/CU FT	68	42	30
SQ M /CU M	223	138	98
APPROX. VOID VOLUME %	95.4	97.2	98.1
APPROX. MIN. FREE PASSAGE DIA. IN	0.4	0.7	1.1
MM	10	18	28
MAX. LATERAL LIQUID SPREAD IN/FT OF HEIGHT	13.9	13.9	13.9
MM/M OF HEIGHT	1158	1158	1158

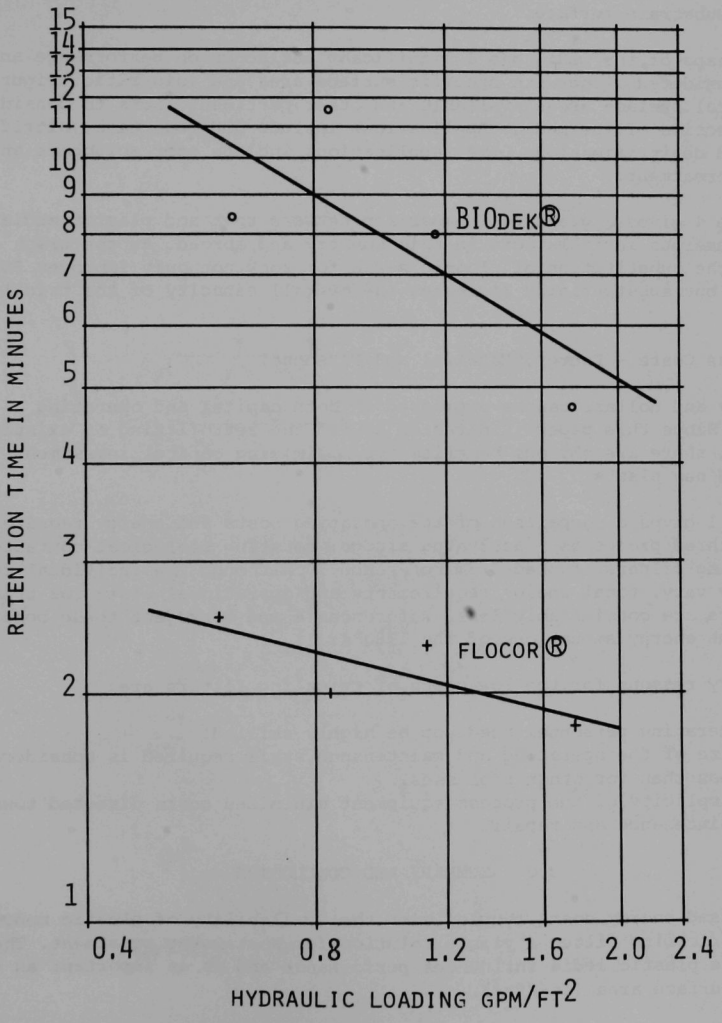


FIGURE 3

COMPARISON OF RETENTION TIME

BETWEEN TWO SHEET TYPE PLASTIC MEDIAS[2]

The test was conducted by introducing a salt solution of a known concentration into the influent. The retention time was calculated as the time when 50% of the influent salt solution concentration could be measured in the effluent. Results show a multifold increase in residence time caused by the shape of the media substrate surface.

The shape of the media has a significant influence on performance and must be considered along with specific surface area and void ratio. Figure 2 shows several surface areas available and other pertinent facts to consider in the selection of the media. Applications include BOD reduction, nitrification, and denitrification. Other applications include odor scrubbers and anaerobic treatment.

Figure 4 gives a graphical comparison between rock and plastic media based on numerous installations in this country and abroad. As the graph indicates the substitution of plastic media for rock not only improves BOD reduction, but substantially increases the overall capacity of the trickling filter.

2.4 Process Costs - Energy, Material and Personnel

Energy and dollars can be expressed in both capital and operating terms 1-5. Since this paper is directed toward the retrofitting of existing facilities, there are obvious benefits for minimizing capital investment in alternative new plants.

Table 1 gives a comparison of the operating costs and energy requirements for three processes - activated sludge; rotating biological contactors and trickling filters adapted from reference 3. Although the individual cost figures may vary, total energy requirements and operational costs for trickling filters are considerably less. References 4 and 5 project these benefits into various energy situations of the 1980's.

Primary reasons for the low costs of trickling filters are:

1) Operating personnel need not be highly skilled.
2) Size of the operating and maintenance staff required is considerably less than for other processes.
3) Simplicity of the process equipment minimizes costs directed toward maintenance and repair.

3.0 SUMMARY AND CONCLUSION

Increased energy costs coupled with the availability of plastic media makes the trickling filter a viable solution for wastewater treatment. The shape of the plastic media influences performance and is as important as the amount of surface area (ft^2/ft^3).

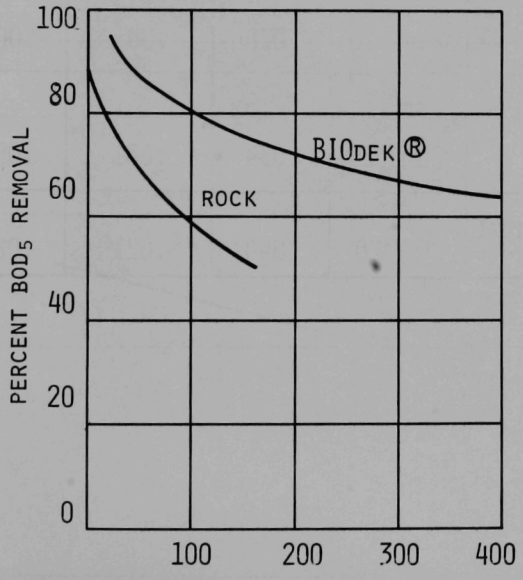


FIGURE 4

RELATIVE EFFICIENCY CURVES

TABLE 1

COMPARISON OF OPERATING COSTS AT 10 MGD[3]

PROCESS	EFFLUENT QUALITY*		ANNUAL OPERATING COSTS MILLIONS OF DOLLARS			
	BOD_5	SS	LABOR	MATERIAL	POWER	TOTAL
TRICKLING FILTER (SINGLE STAGE)	25	20	.020	.001	.009	.030
ACTIVATED SLUDGE CONVENTIONAL/ DIFFUSED AIR	20	20	.034	.031	.055	.120
ROTATING BIOLOGICAL CONTACTORS	20	20	.044	.021	.080	.145

* MG/L

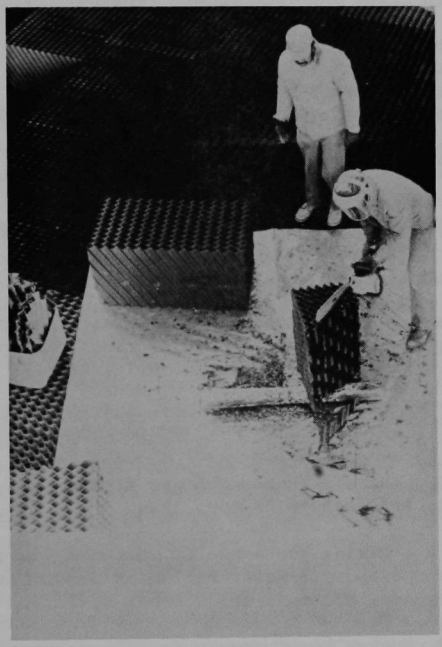


FIGURE 5

INSTALLATION OF BIODEK® PLASTIC MEDIA

The advantages of retrofitting existing rock trickling filters with newly developed plastic media include:

1) Reduced capital investment and low operating costs.
2) Increased capacity by as much as 400%.
3) Improved BOD reduction.
4) Increased surface area, void ratio, and retention time.
5) Uniform distribution of wastewater and air throughout the media with no channelization, or short circuiting.
6) Ponding, plugging and associated odors are eliminated.
7) Problems with flies and insects are reduced.
8) Lower tower height.
9) Reduced pumping head.
10) Will not settle.
11) Self-supporting, exerts no lateral load.
12) Will accommodate maintenance personnel without the need for special walkways.
13) Cost effective light weight tower structures.

With the advent of high efficiency plastic media, the trickling filter will continue to play an important role in wastewater treatment.

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5. Talbot, Richard S. Energy Requirements for Alternative Biological Wastewater Treatment Systems; Water Pollution Control Federation; 52nd Annual Conference; Houston, TX; October 9, 1979.

PLANT UPGRADING/EXPANSION UTILIZING AN ENERGY EFFICIENT FIXED FILM REACTOR

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1. INTRODUCTION

The City of Memphis entered into the business of wastewater treatment in 1975 when the City's first wastewater treatment plant, the T. E. Maxson Plant, was officially placed in operation. Since that time, the City has experienced many difficulties and problems in operating the Maxson Plant and more recently the North Wastewater Treatment Plant. The Maxson Plant receives flow from three interceptor systems draining the south half of the City of Memphis and portions of north Mississippi. Due to the nature of the contributing population and industries, the organic load into the plant is highly variable (hourly, daily, and monthly). This variability has caused operational problems in the past as documented in the Nonconnah Creek 201 Facilities Study as well as studies performed for the Chamber of Commerce by private engineering firms. The Maxson Plant was designed based on surveys and studies performed in 1968. Due to many factors related to the sewer system at the time and the inability of the survey team to obtain an accurate analysis of existing wastewater, much less be able to adequately project the growth within the basin, the design load for the treatment plant was not an accurate projection of future conditions. Compounding the problem at the Maxson Plant was the fact that the plant was originally designed for a life through 1995; however, due to financial constraints and other considerations, the design life was moved back to 1985 which provided a short operational life for the plant. Another operational constraint which was built into the plant was the lack of adequate sludge disposal facilities. Anaerobic holding lagoons were provided as a temporary facility, unfortunately, a permanent facility was never developed.

The Maxson Plant, when it was brought on line in 1975, was at its design organic load as far as pounds per day of BOD (biochemical oxygen demand). Although the plant was at only approximately half of its hydraulic capacity, the high loading of BOD has caused considerable operating problems because it was an unexpected condition and did not fit the pattern of operation as prescribed by the design engineers in the operation and maintenance manual. The numerous problems with the plant lead to the City and EPA entering into a consent agreement in 1977 wherein the City agreed to upgrade and expand the Maxson Treatment Plant to meet existing federal effluent guidelines and to provide an adequate sludge handling

system. One of the factors compounding the plant's operational problems and leading to the consent decree was the fact that the original design was for 85% BOD removal; however, during the construction period, the plant effluent guidelines were changed to read 30 mg/l BOD or 85% removal, whichever is more stringent. This placed on the plant an unobtainable effluent based on design criteria.

In the City's recognition of the problems at the Maxson Plant and the necessity of proceeding in a determined manner, the City prepared a 201 Facilities Plan in 1978 detailing the alternatives available for the upgrading/expansion of the Maxson Plant (liquid treatment plant) and those alternatives available for adequately managing the sludge from the upgraded and expanded facility (sludge management). The recommended system in the 201 Facilities Plan carried with it the recommendation for pilot plant evaluations prior to detail engineering design. Partially because of past experience of the City with large scale design without pilot plant testing, the decision makers were hesitant of proceeding in this manner again. Also, because the recommended system was a drastic modification to the existing biological plant, the sludge generated from that modified plant would be expected to be considerably different in characteristics than the sludge as presently generated at the T. E. Maxson Plant. Accordingly, in October of 1978, the City of Memphis authorized Memphis State University to proceed with pilot plant evaluations of the liquid treatment portion of the upgrading/expansion as per the recommended system in the Nonconnah Creek 201 Facilities Plan.

The basic goal of the MSU studies was to evaluate the recommended liquid treatment system (roughing filters followed by existing contact stabilization plant with the addition of primary sedimentation basins and equalization facilities) as laid out in the Nonconnah Creek 201 Facilities Plan. During the execution of the studies, it became obvious that a slight modification to the recommended plan could be made which would be more cost effective than that recommended in the 201 Facilities Plan.

2. PILOT PLANT EVALUATIONS

A 20 gallon per minute pilot plant was constructed at the T. E. Maxson Wastewater Treatment Plant in order to evaluate the recommendations made in the 201 Facilities Plan for the liquid phase modifications to the Maxson Treatment Plant. The basic recommendation in the 201 Facilities Plan was for the addition of primary clarifiers, equalization basins, and bio-towers (plastic trickling filters) prior to existing contact stabilization of the treatment facility. The recommendations made in the 201 Facilities Plan were based on theoretical considerations with the assumption that it would have to be further evaluated in the field prior to full scale design and operation.

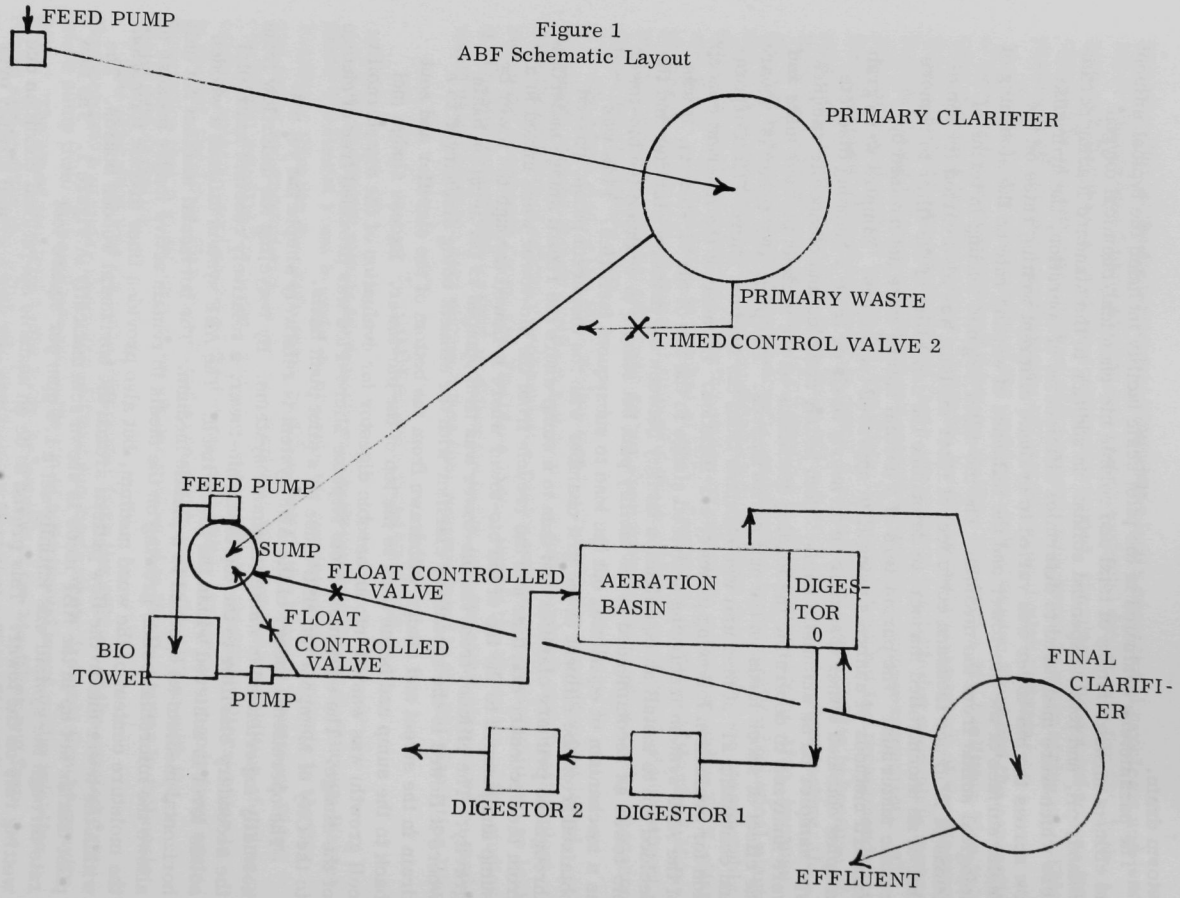
The pilot plant was set up to pump water from the effluent of the aerated grit chamber into a circular primary sedimentation basin where it would flow by gravity into a sump prior to being pumped to the top of a horizontal wood media trickling filter. The sump received a flow from the recycle of the bio-filter and from the return sludge from the bottom of the secondary clarifier. The effluent from the bio-tower was subsequently pumped to a pilot scale aeration basin complete with secondary clarifier from where the clarified effluent was eventually dumped into

a storm drain.

The parameters evaluated in the pilot scale facility included the typical influent and effluent measurements of total and soluble bio-chemical/chemical oxygen demand, pH, and total suspended solids. In addition to the standard loading parameters which were measured to determine efficiencies of operation, the hydraulic flow across the bio-tower was varied to evaluate different wetting rates on the plastic surface of the bio-tower and the influence of wetting rate on the sloughing of biological solids from the medium. The bio-filter organic loading in terms of pounds of BOD per thousand cubic feet of filter medium was also varied (in somewhat of an uncontrolled manner) to determine the capability of the filter to remove organic materials. The parameters of a normal nature were determined by standard methods of analyses using EPA approved techniques. Samples were grab samples obtained either manually on an hourly basis or from a Manning discrete type sampler on an automatic basis. Both hourly samples and composite samples were analyzed to determine the loading conditions on the various process units and the effect of shock loads on the unit performances. The pilot ABF tower (4' square and containing 21' of medium) was obtained from the Neptune Micro-Floc Corporation for evaluation following primary clarification. Because of the inherent concept of the ABF system (returning activated sludge to the top of the bio-tower), it was decided not to install an equalization facility between the primary clarifier and the bio-tower as recommended in the facility plan but simply to evaluate the bio-tower as a mechanism of equalizing out the load to subsequent facilities. Flow was obtained from the effluent of the grit chamber with the diaphram pump, passed through the primary clarifier and then to a sump where the return sludge underflow from the secondary clarifier and the recycle from the bio-tower were mixed in a sump and pumped to the top of the bio-tower where it flowed through the tower by gravity. The effluent from the bio-tower was then pumped to the aeration basin where it flowed to the secondary clarifier with the effluent being discharged to a drain in the street and sludge withdrawn from the bottom of the clarifier and sent back to the sump and hence back to the top of the bio-tower. Excess sludge (net cell growth) was wasted into an aerobic digestor for evaluation of the dewaterability of the sludge. The equipment from Neptune Micro-Floc was provided free of charge to the City of Memphis for MSU's use on a pilot plant basis.

The operational theory of the ABF system is relatively simple but yet not normally experienced in waste treatment systems. By recycling the underflow from the secondary clarifier to the top of the bio-tower, a relatively constant microbial solids level is achieved within the tower itself. The ABF system utilizes a wooden horizontal medium as opposed to a plastic medium. The horizontal medium not only allows the microbial solids growing on the media to remain active longer because of the moisture content of the wood medium, but also provides finer droplet formation within the tower due to the flow pattern around the horizontal wooden boards. The pilot unit as set up in the ABF mode is shown schematically in Figure 1. The flow rate through the system was initially set a 1.5 gpm per square foot wetting rate on the tower. This proved to be an unstable operational condition due to uneven sloughing of solids from the tower. The unit was operated slightly over three weeks in this mode and then the wetting rate increased to 2 gallons per

Figure 1
ABF Schematic Layout



minute per square foot at which point a uniform constant sloughing rate was achieved. At this wetting rate the raw wastewater flow into the tower was 16 gallons per minute with a recycle of return activated sludge of 7 gallons per minute and a recycle from the tower of 9 gallons per minute providing the 2 gallon per minute per square foot wetting rate. Hourly grab samples and 24 hour composite samples were utilized to evaluate the performance of the system. The primary clarifier was evaluated on the basis of percent removal of suspended solids and BOD as well as the sludge production out of the primary unit. The operating parameter in the bio-tower was the loading in terms of pounds of BOD per 1000 cubic feet. The effluent from the tower flowed directly to an aerated basin for further metabolism and stabilization of the organics. The air flow rate and detention time in the aeration basin were varied in this study to evaluate the minimum and maximum values which could be utilized. Also, the loading rate across the system in terms of a system food to microorganism ratio was observed and correlated with percent removal as will be discussed in later paragraphs. The concept of a system F/M ratio is more valid for this type of a biological system than a F/M ratio for the aeration basin.

3. RESULTS

The flow to the top of the bio-filter was a combination of the primary settled raw wastewater, return biological floc from the secondary clarifier, and tower recycle from the bottom of the bio-tower. Thus, the bio-filter became in reality a large part of the total biological treatment system since the biological solids on the tower medium were considerably greater than that in the normal high rate trickling filter. The flow from the bottom of the bio-tower passed to an aeration basin of short term aeration time with the effluent being pumped to the circular secondary clarifier. The clarifier effluent was then discharged to the street gutter. As mentioned previously, the underflow from the clarifier was sent to the bio-filter sump and subsequently the top of the bio-tower. In essence, the bio-tower functions as both a stabilization system for the return sludge and as a biological absorption/ oxidation system for the incoming wastewater. The various unit processes will be described individually with some final conclusions being made at the end. It was decided not to utilize the equalization basin with the ABF system due to the reported ability of the ABF system to absorb shock organic loads better than other types of filter.

primary clarifier

The circular clarifier was equipped with peripheral weir and continuous sludge scraping and draw off. The influent to the primary clarifier varied considerably over the three month study period using the circular clarifier. The removal of solids across the primary unit consistently was around 50%. The BOD removal across the primary clarifier varied over a wider range than did the suspended solids removal due to the soluble BOD variations entering the system. The average removal rate across the primary was approximately 25% of the incoming total BOD. It was possible to control the primary underflow concentration to

approximately 1% solids by controlling the rate of withdraw. It was also possible to concentrate the primary sludge to 3 to 4% before floating solids appeared on the surface of the primary clarifier.

activated bio-filter

The initial operational rate was set at 1.5 gallon per minute per square foot of exposed surface area. This was chosen on the basis of published data by the manufacturer. During the initial phases of operation, uneven sloughing of biological solids occurred from the filter and, upon the advice of the manufacturer, the wetting rate was increased to 2 gallons per minute per square foot with the added flow being made up from tower recycle. This modification achieved a constant sloughing of biological solids from the bio-tower. The effluent from the bio-tower which was not recycled back from the top was sent to a complete mix aeration basin for further stabilization of the wastewater. Because the bio-tower was operated at a relatively constant hydraulic loading, wide fluctuations in the organic loading in terms of pounds per thousand cubic foot per day of BOD were experienced. The bio-tower, because it contained a high population of biological solids on horizontal medium, was also monitored for parameters related to a normal aeration system, i.e. oxygen uptake rates and sludge volume indices. The bio-tower performed extremely well and much better than was originally anticipated when it was installed. The organic loading varied from a high of 452 pounds per day per thousand cubic feet to a low of less than 110 pounds per day per thousand cubic feet. Even with this wide fluctuation in loading, the bio-tower removed a consistent level of soluble BOD. The oxygen uptake rates at the bottom of the bio-tower were very high when compared to the complete mix activated sludge system. The solids settled reasonably well as indicated by the sludge volume index values. Several studies were performed by taking hourly samples of the influent and effluent from the bio-tower to evaluate the ability of the bio-tower to absorb shock organic loadings. The soluble removal across the tower was consistently greater than 80% and often times reached as high as 97%.

aeration basin

The flow from the bottom of the bio-tower was taken initially to a complete mix aeration basin with a detention time of six hours. The hydraulic detention time was later reduced to 4.5 hours to evaluate the effect of detention time on soluble BOD removal. The aeration tank was provided by Neptune Micro-Floc Corporation as part of the pilot plant. The load into the aeration basin consisted primarily of the suspended BOD due to the sloughing of solids from the bio-tower. The soluble BOD going into the aeration basin was relatively low due to the performance of the bio-tower. The amount of aeration which was required to stabilize the underflow from the clarifier from the bio-tower was an unknown entity. A laboratory study was performed on the flow from the tower bottom to determine the optimum aeration time. This study was performed by pulling a sample of the bio-tower bottom flow, aerating it for a prolonged period of time, and pulling samples of the mixed liquor at various time increments. Analyses of the laboratory and field studies indicated that an aeration time of less than 4 hours and probably less than 3 hours would be

adequate to remove most of the carbonaceous BOD. The aeration basin was operated at varying mixed liquor suspended solids levels with the mean cell residence time at about 4 1/2 to 5 days. The mean cell residence time was calculated based on the amount of mixed liquor solids in the aeration basin. An alternate procedure using total solids inventory in the system was not utilized. This was an arbitrary point as far as the study was concerned but it tended to provide a stable system. The oxygen uptake rate in the aeration basin was relatively low (around 44 mg/l per hour). The low oxygen uptake rate was an indicator of the low level of soluble BOD entering the system. With the ABF system of process, it is almost inappropriate to speak of the bio-tower without speaking of the activated sludge portion. A systems analysis approach where the system considers the total load of the bio-tower as the food and the microorganisms in the aeration basin as the amount of microbes was adopted. This results in a system F/M ranging from .4 to better than 1.29. The total system performance was found to be less influenced by the loading on the bio-tower in terms of pounds per day per thousand cubic feet than on the aeration time. Based on the hourly samples collected around the bio-tower (which indicated that the bio-tower consistently removed 80% of the incoming soluble BOD), projections were made as to the actual operating conditions of the system. Based on the 80% removal across the bio-tower, the soluble loading onto the aeration in terms of pounds per thousand foot of aeration volume per day became relatively low. The aeration time proved to be a critical factor in the level of BOD in the effluent. It is normally recognized that total BOD values from a pilot plant are often misleading due to scale-up problems and controlling solids through a small clarifier. The soluble BOD removal across the total system was entirely satisfactory with soluble BOD levels consistently less than 10 mg/l in the effluent. The total BOD in the effluent of the aeration basin was at or above 30 mg/l for most of the test period. It was determined towards the end of the test program that nitrification was occurring in the effluent samples and steps were taken to alter laboratory procedures to compensate for the nitrogenous BOD. When this compensation was made, the pilot plant unit achieved the desired effluent quality of 30 mg/l BOD and suspended solids. The indications from the pilot plant tests are that, with the wastewater in the City of Memphis, the aeration time should be less than 3 hours to achieve the desired soluble BOD removal.

secondary clarifier

The secondary clarifier provided with the pilot plant generally performed in a satisfactory manner; however, certain upset conditions did occur which had a negative effect on the total effluent. The sludge solids coming from the bio-tower/aeration system demonstrated a good settleability and tended to clarify in Imhoff cones with little trouble. In the clarifier, however, difficulties with settleability were encountered during portions of the study as evidenced by pin point floc in the clarifier supernatant. It was observed that when the aeration basin volume was lowered (thus reducing the aeration time) and when the dissolved oxygen was maintained at less than 2 mg/l, the pin point floc tended to disappear. Some reports by other invesitgators have indicated that the sludge from this type system has poor settling characteristics and could present some problems in final

clarifier evaluation and operation. Such was not the case with the system evaluated at the T. E. Maxson plant. The biological solids did settle satisfactorily and provided a reasonably good supernatant.

DISCUSSION RESULTS

The Maxson Plant has long been noted for the variable organic loadings which enter the system. This was documented in the 201 Facilities Plan as well as studies by other consultants. The fluctuations entering the pilot plant were somewhat dampened by the effect of the aerated grit chamber; however, the influent BOD ranged from a high of over 900 mg/l to a low of 215 mg/l. The soluble BOD fluctuations were just as dramatic with soluble levels approaching 400 mg/l on some days and yet below 100 mg/l on others.

The ABF system more properly should be considered on more of a system basis rather than individual unit operations or processes. Due to the recirculation of solids back to the top of the bio-tower, much higher organic loading rates were achievable on the tower than what was experienced with the high rate plastic media trickling filter. The daily fluctuations in bio-tower loading were extreme, ranging from a high of almost 450 pounds per thousand cubic feet per day to a low of 100. Even with this variable influent to the bio-tower, the stability of the system, as measured by the effluent soluble BOD, was very satisfactory. The effluent soluble BOD values very seldom were greater than 5 to 6 mg/l regardless of the tower influent. Using the concept of a system F/M ratio, the food to microorganism ratio varied considerably based upon the tower loading with values ranging from a high of 1.29 pounds per day of BOD per pound of mixed liquor suspended solids to a low of .4 pounds per pound per day. The ABF system pilot plant was able to absorb shock loading or variable organic loadings and still produce a high quality effluent. The ABF system did show a quick response when upsets did occur with the mixed liquor suspended solids usually recovering within three or four days. Another interesting observation with the study was the variable loading on the aeration basin of soluble BOD and the overall plant performance. The soluble BOD load on the aeration basin (assuming an 80% removal of soluble BOD across the bio-tower) ranged from a low value of 10 pounds per day per thousand cubic foot of aeration volume to in many cases over 40 pounds per day per thousand cubic foot.

The aeration portion of the studies indicated that the solids and BOD coming from the bottom of the bio-tower were not necessarily similar to a normal BOD coming from a raw wastewater. The aeration basin was loaded at relatively low soluble BOD levels causing the oxygen uptake rate to be consistently below 60 mg/l per hour and averaging approximately 44 mg/l per hour. The performance of the aeration basin also indicated that the desired detention time following the ABF tower with this particular wastewater should be less than 3 hours. The longer the detention time in the aeration basin with this wastewater is not justified on the basis of additional soluble BOD removal. With short detention times, nitrification in the effluent BOD tests were a problem.

Various alternatives have been studies over the past several years for upgrading and expanding the T. E. Maxson facility. These alternatives have included most known wastewater management practices ranging from spray irrigation (application on land), conversion to pure oxygen activated sludge system, and conversion to conventional diffused air activated sludge. In all of the alternatives which have been considered, the capital costs as well as the operational costs were key factors. The energy consumption of each alternative became a key factor in the evaluation of the alternatives; it considerably influences the operation cost and is an ever increasing item. One of the objectives of the pilot plant studies performed by Memphis State University was to evaluate the applicability of an energy efficient system at the Maxson facility to provide a technically adequate system for producing an acceptable effluent as well as lowering or stabilizing the energy input into the plant. At the present time, operating in the contact stabilization activated sludge mode, the energy cost at the T. E. Maxson Plant approaches 1.5 million dollars per year. Of this total energy cost, 69% is attributable to the cost of running the blowers for the aeration basins. Any system which could substantially reduce this cost would be an attractive alternative for the upgrading and expanding of the Maxson facility.

Analyses of the results of the pilot plant studies indicates that the ABF mode of operation would achieve desired effluent parameters using reasonable design criteria. The application of this concept to the Maxson plant was based on economic as well as technical factors. According to the City's design consultants, the most cost effective system for the upgrading and modification of the Maxson facility (next to ABF system, that is) would be a split flow system with 63% of the flow going through the existing basins (converted to complete mix activated sludge) and 37% going to a new pure oxygen system. As shown in Table 1, the ABF alternative is not only lower in capital cost but also in operations and maintenance costs when compared with the split flow activated sludge system. This saving is enough to quality the ABF system in this location as innovative/alternative technology. The magnitude of the savings is further illustrated by the data in Table 2. The energy saving is in the power required for the aeration basins. Due to the large amount of BOD removed in the tower, the energy requirements are considerably lower than the alternative. The additional power to supply the tower is not substantial enough to negate the savings.

CONCLUSIONS

Based on the data presented in this paper and the observations which were made during the execution of this project, it can be concluded that the modifications to the Maxson Treatment Plant should take the form of the activated biofilter concept. This concept has several advantages over other alternatives including approximately equal installation costs and significantly less system operating costs. The ABF system, based on estimated power savings alone would result in a significant monetary saving to the City. As illustrated by the following data, when the system is changed to the ABF mode, a 31% energy savings will be

realized. At the present cost of energy, this equates to about $450,000 per year savings in energy due to the reduction in blower usage.

	Existing Plant	ABF system @ 60 MCD
Energy Use, kWh/mth		
Lift Pumps	780,000	780,000
Blowers	4,140,000	885,000
ABF Lift Pumps	-	1,021,000
Intermediate Pump	-	367,875
Other	1,080,000	1,080,000
	6,000,000	4,133,875

TABLE 1

T. E. Maxson Upgrading/Expansion
Alternative Cost Analysis (1)

	ABF mode	Air Activated/Pure Oxygen	Savings
Construction Cost	17,027,000	17,100,000	-
Annual Power Cost	837,000	1,263,000	36%
Annual O&M Cost (2)	949,000	1,483,000	36%

(1) Secondary processes only
(2) EPA 430/9-78-009

TABLE 2

T. E. Maxson Upgrading/Expansion
Energy Estimates

	ABF Mode	air/pure oxygen	Savings
Power Requirements, kWh/mth			
Main Lift Pumps	1,040,000	1,040,000	-
Intermediate Pumps	490,500	490,500	-
Blowers/Oxygen	1,180,000	4,250,000	72%
ABF Lift Pumps	1,362,000	-	-
Other	1,080,000	1,080,000	-
	5,152,500	6,866,500	25%

SELF-PROPELLED AERATORS FOR LAGOONS

Boris M. Khudenko
Wayne State University

INTRODUCTION

Aerated lagoons offer several advantages over other types of ponds used for wastewater treatment in small communities and at some industrial plants. Mixing and forced aeration permits a two to six fold increase in pond depth and produces overall treatment rates at least two to four times greater than in non-aerated lagoons. The major disadvantage of aerated lagoons is their relatively high energy consumption, which is about five to ten times greater, per kg of BOD removed, than other biological processes (the conventional activated sludge process requiring approximately 1 kWh/kg of BOD removed). The high energy requirement is primarily due to the stationary nature of presently employed aeration equipment, including both anchored mechanical and diffused air aerators. Since the effectiveness of the aeration process depends on the amount of mixing in the lagoons, thereby suspending the activated sludge, either four to ten times the number of stationary aeration units needed to satisfy the oxygen requirement are installed, or additional mixing devices are employed. Either of these approaches causes additional, and perhaps unnecessary, energy consumption.

An alternative approach would be the use of mobile, and particularly self-propelled, aerators. Such devices can provide a more balanced energy usage between aeration and mixing, thus reducing energy consumption and operation costs for wastewater treatment in lagoons. The major parameters of self-propelled aerators are the linear velocity of their movement upon the surface of the lagoon, their period of rotation, the service area, and oxygenation capacity. The determination of these parameters for mechanical and jet aerators is discussed in the following sections.

SELF-PROPELLED MECHANICAL AERATORS

A self-propelled mechanical aerator is illustrated in Fig. 1A. This aerator consists of an open-bladed-rotor and motor-gear mounted on a platform supported by floats. A hinged arm links the floating structure to the pile. The aerator is free to rotate around the pile. The hinges compensate for water level fluctuations in the basin.

The Linear Velocity and Period of Revolution of the aerator can be determined by equating the tractive, F, and drag, D, forces:

$$\text{i.e.,} \quad F = D \tag{1}$$

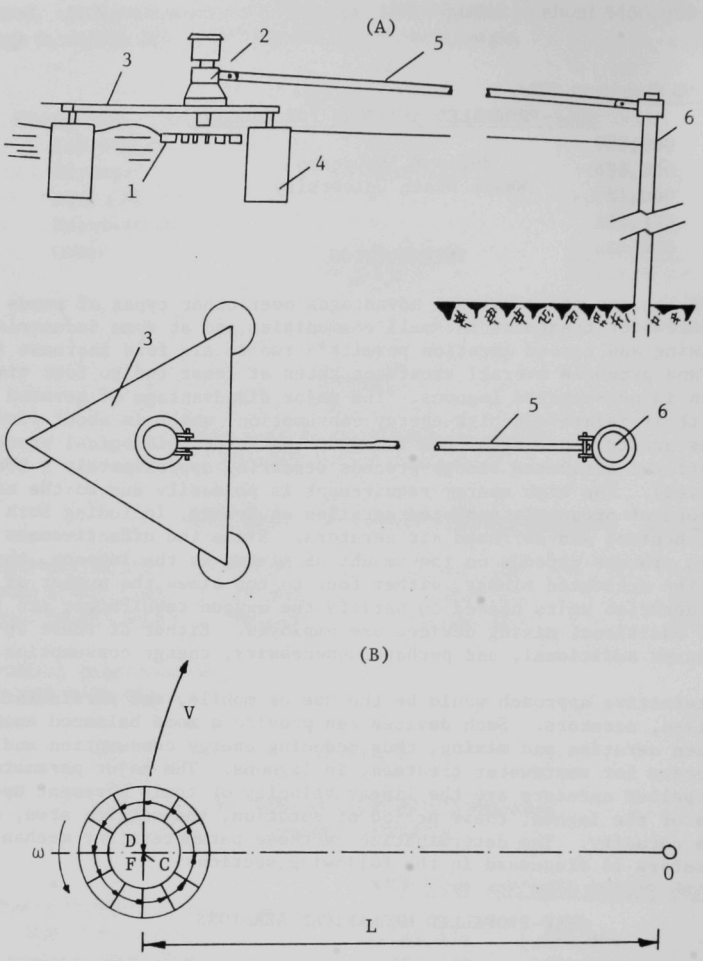


Figure 1. (A) Schematic of a mechanical self-propelled aerator (1) rotor, (2), motor, (3) platform, (4) floats, (5) arm, (6) pile.

(B) Sketch of forces acting upon chemical self-propelled aerator.

The tractive force depends on the torque M_o and the arm length L, or,

$$F = M_o/L \tag{2}$$

Because the forces applied to rotor blades form a closed polygon (Fig. 1B), their moment is the same for any reference point located in the horizontal plane, hence,

$$M_o = \ldots = M_c = P/\omega \tag{3}$$

Where M_c = torque of the rotor ($M_c = P/\omega$),
P = the aerator net power,
ω = rotor angular velocity ($\omega = 2v_o/d$),
d = rotor diameter,
v_o = rotor linear velocity (on the rim)

The drag force D has two components: water resistance and wind sail effect. The latter alternately propells and retards the motion of the aerator, and for moderate winds, is negligible. The water resistance can be estimated from the equation:

$$D = C_d \gamma V^2 A/2g \tag{4}$$

Where C_d = drag coefficient,
γ = water specific weight,
V = aerator linear velocity,
A = area of a rectangle which circumscribes the submerged part of floats-rotor assembly in the vertical plane perpendicular to the direction of motion.

Combining Eqs. 1 through 4, the velocity of a mechanical self-propelled aerator can be found as:

$$V = (2Pg/C_d \gamma \omega LA)^{1/2} \tag{5}$$

A conservative value of C_d, based on experimental data (Ref. 6), is from two to three.

The period of one revolution can be found by means of the formula:

$$\tau = 2\pi L/60V \tag{6}$$

The Service Area of the self-propelled aerator rotating around the pile is four times larger than that for a stationary aerator, if the arm length (from the pile axis to the rotor center) equals "the radius of influence" for the same aerator anchored at a certain point.

The radius of influence for stationary aerators can be estimated on the basis of the design procedure suggested by Karelin Ja. A., et al. (Ref. 4). In accordance with this theory, the water flow in the tank can be represented as a vortex flow with the following distribution of surface velocity:

$$V_x = K_1 \sqrt{Qv_a/\pi dr} \tag{7}$$

Where K_1 = empirical constant, which is a specific value for a given type of aerator (for open-bladed rotors $K_1 = 1.41$),

Q = water flow through the aerator,

d = diameter of aerator,

r = distance from the rotor center to the point x on the water surface,

a = absolute velocity of water outgoing from the aerator, which can be found as:

$$v_a = \sqrt{v_o^2 + v_r^2} \tag{8}$$

Where v_r = radial velocity of the water outgoing from the rotor:

$$v_r = Q/\pi hd \tag{9}$$

Where h = height of rotor blades.

The Q value can be found by the use of the following expression:

$$Q = K' \frac{\pi^2 d^2 v_1}{2Z} \left(1 - \frac{\Gamma_{out} - \Gamma_{in}}{d}\right) \tag{10}$$

Where K' = coefficient which can be determined from the formula suggested by Prascura for the design of pumps:

$$K' = [1 - 3.6 \ (1 - \Gamma_{in}^2/\Gamma_{out}^2)]^{-1}, \tag{11}$$

Z = number of blades,

Γ_{in} and Γ_{out} are the respective diameters of circles inscribed in and sircumscribed around the blades,

v_1 = velocity of the upstream flow under the aerator.

$$v_1 = \sqrt{2g \ (h_s + h)} \tag{12}$$

h_s = submergency of aerator.

The vertical distribution of velocities can be approximated on the basis of Altshule's formula for the velocity distribution in box channels at their bends:

$$u/v_x = (h/H)^y \tag{13}$$

Where u = velocity of flow at the point submerged to the depth h,

H = total depth of the reservoir,

V_x = surface velocity at a given point, which is determined by relationship (7),

y = empirical value: $y = 7.45/C$,

C = Chezy coefficient determined by Manning's formula: $C = (1/n)H^{1/6}$.

The Oxygenation Efficiency of self-propelled aerators can be found on

the basis of manufacturers data or computed in the same manner as for stationary aerators (see, for example, Ref. 6).

SELF-PROPELLED JET AERATOR

The self-propelled jet aerator (Fig. 2A) consists of a float-supported box containing an orifice or weir and a low pressure pump. This structure is linked to the pile by means of an arm similar to the previously described mechanical aerator.

The Linear Velocity and Period of Revolution for the jet-aerator can also be found on the basis of equating tractive and drag forces. The tractive force for this aerator can be determined as:

$$F = P/V_o$$

Where P = jet power ($P = \gamma QH'$),
H' = waterhead over the weir crest or orifice center line,
Q = jet flow rate (fraction of flow outgoing from spillway or orifice in the direction opposite to the direction of the aerator motion.
V_o = velocity of jet discharge:
$V_o = \phi\ 2gH'$ for orifice, or $V_o = Q/B \cdot h$ for a weir,
h = water depth over the weir ($h \simeq 2/3H'$),
ϕ = velocity coefficient.

The drag force is as above.

The linear velocity for the self-propelled jet aerator can be found from Eqs. 1, 5, and 8:

$$V = (2gQH'/C_d V_o A)^{\frac{1}{2}} \qquad (14)$$

Formula (6) can be used for the computation of the period of revolution.

The service area for the jet aerator can be approximately evaluated on the basis of the formulae developed for the estimation of the washout cone for a console waterfall (References 1, 2). Fig. 3A represents the schematic of a jet and the washout cone it produces. The total length of the jet impact zone can be found as:

$$L' = L_f + L_c + L_s + L_o + L_a \qquad (15)$$

Where L_f = length of free fall for the jet, L_c = distance from the entrance point of the jet into the lower water to the bottom of washout cone, L_s = distance from the bottom of the cone to the virgin ground level along the jet path, L_o = the length of the jet reflected from the bottom of the washout cone and penetrating across the water in the pond, L_a = aftereddy zone.

The radius of influence for jet-aerator with water discharge at both sides of the aerator equals double the length of jet impact zone. The length of free fall for a jet which is horizontal at the exit point equals:

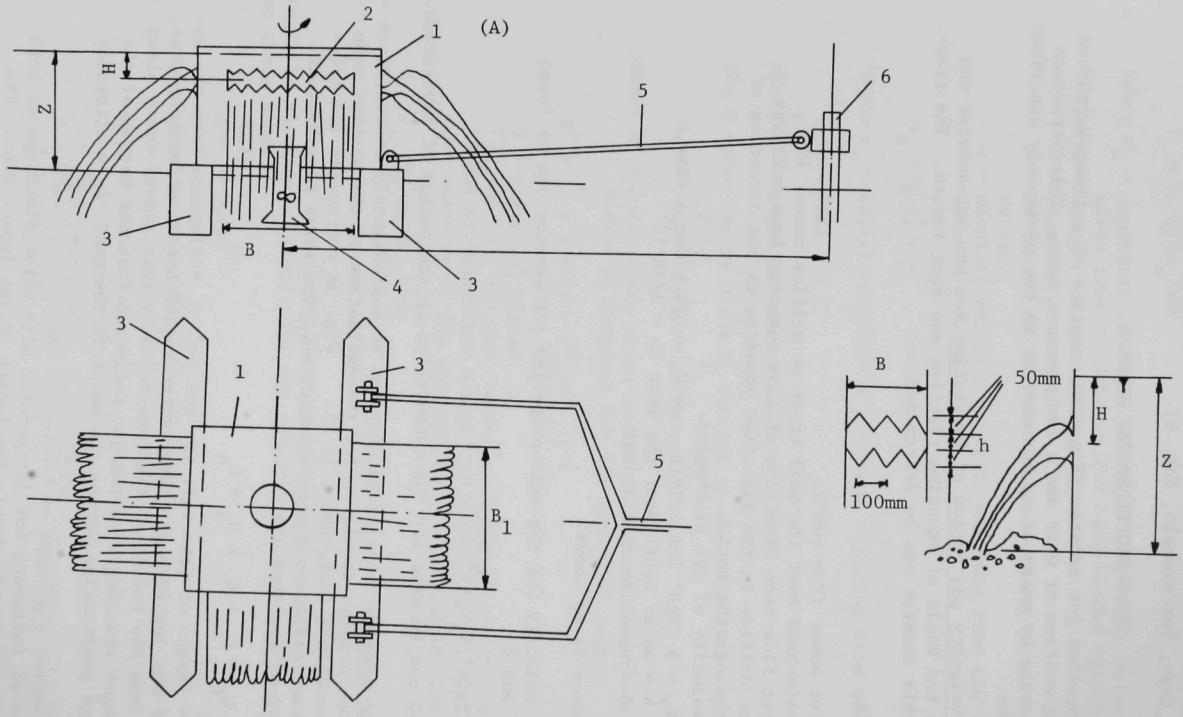
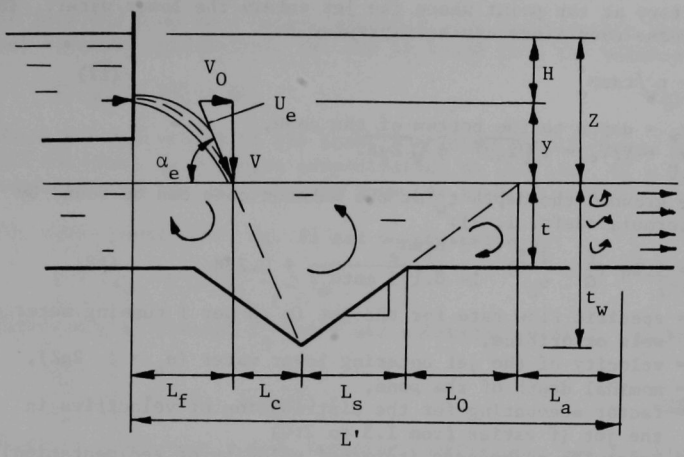


Figure 2. (A) Schematic of a jet self-propelled aerator
(1) box, (2) orifice or weir, (3) floats, (4) propeller pump, (5) arm, (6) pile.
(B) Sketch of a tooth-shaped orifice.

(A)



(B)

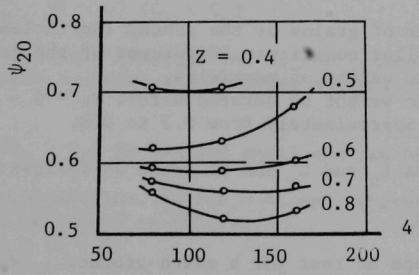


Figure 3. (A) Schematic of a washout cone and "zone of influence" for jet aerator.

(B) Deficits ratio ψ_{20} as function of flow rate of water per running meter of orifice width and function of difference between upper and lower water levels (tooth-shaped orifice, h = 50mm).

$$L_f = V_o\sqrt{2y/g} \tag{16}$$

Upon entrance into the lower water, the trajectory of the jet is a straight line, the poistion of which can be found by drawing a tangent to the jet trajectory at the point where the jet enters the lower water. The L_c value for these conditions can be expressed as:

$$L_c = t_w/\tan\alpha_e \tag{17}$$

Where t_w = depth to the bottom of the cone,
$\tan\alpha_e = V/V_o = \phi\sqrt{2gy}/\ \phi\sqrt{2gH}$.

For loose ground, the depth t_w at the wahsout cone can be found by means of the formula (Refs. 1, 2):

$$t_w = 2.4q\ (\frac{f}{G} - \frac{2.5}{u_e})\ \frac{\sin\alpha_e}{1 - 0.175\ \cot\alpha_e} + 0.25H \tag{18}$$

Where q = specific flow rate for the jet (m^3/s per 1 running meter of weir or orifice,
u_e = velocity of the jet entering lower water ($u_e = \phi\ 2gZ$),
H = nominal depth of the pond,
f = factor accounting for the distribution of velocities in the jet (f varies from 1.5 to 2.0)
G = hydraulic coarseness (terminal velocity of sedimentation) for the particles of the ground.

The value of G can be estimated by the use of the formula:

$$G = \sqrt{2g\ (\gamma_g - \gamma_o)d/1.75\gamma_o} \tag{19}$$

Where d = diameter of grains of the ground (particles of this size and smaller constitute 90 percent of the ground by weight),
γ_g = specific weight of particles,
γ_o = specific weight of aerated water: $\gamma_o = 1 - \beta$, where β varies approximately from 0.5 to 0.8.

The sum of lengths L_s and L_o can be found as follows:

$$L_s + L_o = mt_w \tag{20}$$

Where m = the slope at rest for a given ground.

The length of the aftereddy zone can be estimated by the relationship:

$$L_a \simeq aL_o \tag{21}$$

Where a = empirical coefficient which varies in the range from 1.0 to 3.0.

The Efficiency of Oxygenation for jet aerators can be found on the basis of data on water oxygenation at hydraulic structures. This data is often given in terms of a deficit ratio ψ or its reciprocal R (Refs. 3, 5, and 6):

$$\psi = (C_s - C_t)/(C_s - C_o) \qquad (22)$$

Where C_s = concentration of oxygen saturation, C_o = initial concentration of oxygen in the water, C_t = concentration of oxygen in the water after jet aerator.

If jets are used in a batch reservoir, the relationship between ψ and the mass transfer coefficient, Ka, can be found from the balance:

$$W \frac{dC_t}{dt} = QC'_t - QC_t \qquad (23)$$

Where W = the volume of the tank, Q = total jet discharge rate for all aerators, C_t = oxygen concentration in the tank, C'_t = oxygen concentration in the jet outgoing from the zone of aeration.

For these conditions, Eq. 22 can be presented as:

$$\psi = (C_s - C'_t)/(C_s - C_t) \qquad (24)$$

Expressing C'_t through C_t and ψ and substituting it in the Eq. 23 yields:

$$\frac{C_s - C_t}{C_s - C_o} = \exp - \left(\frac{1 - \psi}{T}\right) t \qquad (25)$$

Where $(1- \psi)/T$ = Ka = mass transfer coefficient, and T = W/Q = time of one turnover of water in the basin due to pumping.

Oxygen balance for a complete mix flow-through lagoon can be presented by the equation:

$$W \frac{dC_t}{dt} = QC'_t - Wr - Q' (C_o - C_t) \qquad (26)$$

Where r = rate of oxygen consumption, C_o and C_t are concentrations of oxygen in the water incoming and outgoing respectively, Q' = volume of water treated.

The value of $Q'(C_o - C_t)$ is usually small and can be neglected.

For steady-state conditions, $dC_t/dt = o$, and because $C'_t = C_s - \psi (C_s - C_t)$, Eq. 26 can be solved as follows:

$$T = \frac{(C_s - C_t)(1 - \psi)}{r} \qquad (27)$$

A tooth-shaped orifice of 50 mm height (Fig. 2B) can be recommended for self-propelled aerators, because it requires a substantial waterhead H' for the discharge of the optimal flow rate q (about 160 l/s·m). This results in a comparatively strong jet outgoing from the orifice ($V_o \simeq 2.6$ m/s) and, therefore, sufficient tractive force. The waterhead H' for this orifice can be calculated from the formula (Ref. 1):

$$q = C\bar{h}\sqrt{2gH'} \qquad (28)$$

Where C = discharge coefficient (C = 0.59, Ref. 6),

$\bar{h}$= average height of orifice (for tooth-shaped orifice $\bar{h} = h_{or} + h_{tooth}$)

The ψ value depends on (a) the difference in water levels in the aerator box and in the basin, Z, and (b) flow rate per running meter of the orifice, q (Fig. 3B). Temperature and water quality correction factors can be taken into account by the relationship

$$\psi = \psi_{20}^{n\alpha} \qquad (29)$$

Where n = temperature correction, $n = 1 + 0.02\ (t° - 20)$, α = water quality correction (Ref. 6).

SUMMARY AND CONCLUSIONS

The major disadvantage of aerated lagoons is inadequate aeration equipment, which requires more energy for mixing of the liquid in lagoons than for oxygen supply. Self-propelled aerators are a feasible alternative for aerated lagoons. The use of self-propelled aerators reduces energy consumption and operation costs. Methods for computing design parameters of self-propelled mechanical and jet aerators have been presented. These parameters include 1) the velocity of the aerators movement on the pond surface, 2) the period of rotation for the aerator, and 3) the area and volume of the pond served.

Comparisons of self-propelled and stationary aerators have shown significant potential energy and cost savings.

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SESSION 12

PUBLIC AND PRIVATE COOPERATION

Chairpersons: Caroline Brooks

William Owen

PRIVATE-PUBLIC COOPERATION FOR EFFECTIVE ENERGY CONSERVATION

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INTRODUCTION

Increasing energy prices have impacted all sectors of American society including such vital services as water supply and wastewater treatment. These increases have been particularly pronounced since 1972. The cost for electricity has risen anywhere from 10 to over 20 percent per year, far more steeply than the general inflationary trend. Electricity cost data for the past decade, shown on Figure 1, emphasize this point.[1,2] The unremitting instability in world petroleum markets forewarns us of continued escalation at an even higher level .

Strong incentives are thus present to encourage energy conservation, but institutional constraints often mitigate against this desirable goal. For example, some wastewater treatment plants have had generation capacity and fuel supplies in excess of their offpeak demands. The attitudes and policies of local utilities and regulatory agencies have until recently discouraged the use of such generation facilities as a supplement to the power grid. Treatment plant managers and operating staffs favor the basic philosophy of maximizing the use of scarce and costly resources. However they must evaluate conservation prospects in terms of cost savings and effect on operating performance before implementing any program.

This paper will discuss some general approaches to energy conservation that apply to water and wastewater management. We will then present several case histories showing how such approaches may be implemented. Finally we will point out areas where additional improvement is possible but changes in methods or attitudes are needed for project success.

CONSERVATION THROUGH COOPERATIVE PROGRAMS

Energy conservation, defined here broadly to include altering the period of peak electrical load, reducing overall energy use, switching to alternative fuel sources, and cogenerating both heat and electricity, may benefit public and private utilities, municipal agencies, and private industry. Load management can reduce peak system demand and allow a utility to forego construction of expensive peak generating capacity or perhaps eliminate it completely. Energy use reduction can similarly reduce capacity requirements and perhaps allow better utilization of the most efficient system components or those that use cheap fuel sources such as hydropower. Steam

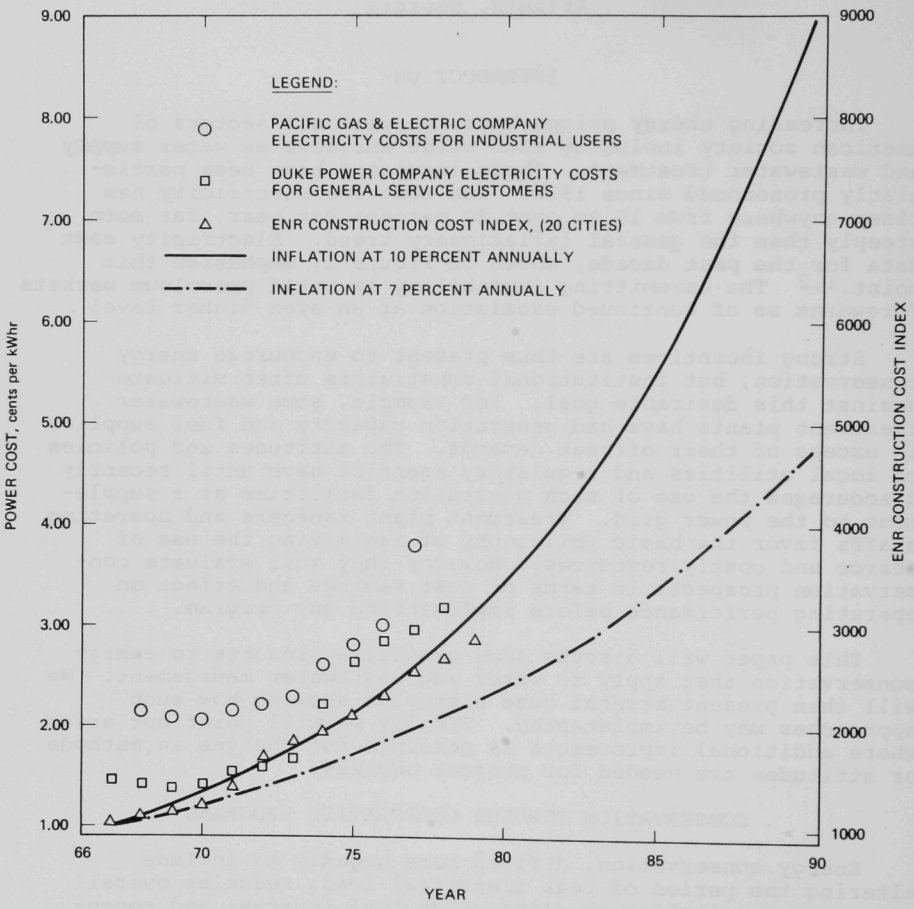

Fig. 1 Cost Increases for the Past Decade and Effects of Future Scenarios

generated by burning municipal refuse is just as effective for industrial applications as steam produced from imported oil. Refuse may also be a cheaper source of heat, but industries must work with local government to gain access to the refuse. The government may in turn benefit if industry will finance the refuse-to-energy facility. Cogeneration requires similar joint efforts by the public and private sectors for successful implementation. Some approaches to energy conservation that make use of such cooperation will now be discussed.

Education is a primary tool used by utilities in their recent conservation efforts. Their staffs have produced fliers and leaflets explaining bills, rate structures, and the potential of various conservation opportunities such as thermostat adjustments and lighting reductions. Utilities have sponsored seminars or workshops for major power users, particularly municipalities and industries. Utility staffs conduct these meetings, often assisted by consultant specialists. Utility policies, billing rates, and especially demand charges are typically explained and case histories of successful conservation efforts are discussed.

Utilities have also run free surveys or preliminary energy audits of client facilities. Such surveys are intended to review billing records and identify demand related charges; surveyors also try to spot apparent energy conservation opportunities (ECO's) and decide whether more detailed study and perhaps facilities modifications will benefit the owner. Utility personnel or private consultants under contract to the utility carry out this work. In some cases the utilities have reviewed private consultant qualifications and established lists of firms capable of detailed studies and the design of modifications. These references are available to their customers on request.

Low interest or no interest loans for equipment or modifications needed to implement ECO's are another approach adopted by some utilities. The dollar limit on such loans (e.g., $100,000) generally makes them most attractive to small users. Automatic load control on interruptable equipment such as air conditioners is also proving itself very valuable, especially for electrical cooperatives which must pay significant demand charges for power they buy.

Cogeneration, a technique for recovering both electrical energy and process heat (usually as steam or hot water) from fuel combustion, has been used in many industrial applications to improve overall process efficiency. Facilities for cogeneration have traditionally been owned by utilities or have not been connected to utility power grids because of potential legal entanglements resulting from state or Federal power regulations. Until the Public Utility Regulatory Policies Act (PURPA) of 1978, any generator of electrical power which entered a grid with interstate connections was considered a utility and thus subject to all Federal Power Act (FPA) regulations for utilities.

Related state laws often added further constraints. Under PURPA, small power generators (less than 30 MW) can be exempted from the FPA, the Federal Public Utilities Holding Company Act, and all related state laws and regulations. Cogenerators of any size can similarly be exempted. The Federal Energy Regulatory Commission is revising regulations required by PURPA. These regulations contain many intricate provisions and should be reviewed by anyone interested in cogeneration. Of most importance for potential cogeneration projects involving water and wastewater facilities, they recognize the fuel value of digester gas and municipal refuse. The regulations will require that utilities buy power from cogenerators at a fair and reasonable price and provide cogenerators with standby power without punitive standby rates. Overall, PURPA makes it advantageous for utilities and public or private customers to cooperate on efficient power production opportunities. Some states have already taken more definitive action on cogeneration. The State of California Public Utilities Commission passed a resolution in January 1978 directing all state utilities to augment cogeneration projects (see Case History 1 for an example results).

CASE HISTORIES

The case histories that follow illustrate examples of cooperation between the private sector (e.g., utilities, equipment manufacturers, private industries, and private consultants) and public agencies (e.g. public treatment works, publicly financed utilities, and regulatory agencies) which have or soon will result in energy conservation. These examples are by no means inclusive but they demonstrate that approaches such as cogeneration can be made to work. They also point out some innovative ways to achieve conservation or more efficient resource utilization.

CASE HISTORY 1 - COGENERATION FUELED WITH DIGESTER GAS. A conventional activated sludge system near Laguna Niguel, California, operated by the Aliso Water Management Agency (AWMA) was designed by Brown and Caldwell to treat domestic and commercial wastewater as well as sludge from other treatment plants. The AWMA plant has an initial hydraulic capacity of 2.7 mgd. Total ultimate sludge capacity will be equivalent to that from a 20-mgd secondary treatment facility. The sludge will be digested anaerobically, dewatered on belt filter presses, and hauled to a sanitary landfill for disposal. Since anaerobic digestion generates a by-product gas that is typically 50 to 70 percent methane, Brown and Caldwell suggested powering engine generator sets with the gas to generate electricity and produce the heat needed to assure efficient, controlled sludge digestion. The U.S. Environmental Protection Agency also requires a standby or backup power source at wastewater treatment plants to prevent complete system failure during electrical outages; the generator sets would also serve this vital function.

Our analysis of power load requirements and gas availability determined that digester gas would initially be available to power two 400-kilowatt (kW) generators during off peak times and three 400-kW generators on peak. At ultimate sludge capacity, an additional 400-kW set will be added. Power demand analysis (see Figure 2) predicted a net surplus generation capacity of 100 kW from 4 p.m. to 8 a.m. during initial plant operations. The excess capacity will increase as the plant reaches organic and hydraulic capacity, especially when more gas producing sludge is added from other plants. The projected power surplus prompted AWMA and BC to explore the possible sale of power to the local utility, Southern California Edison (SCE), under provisions of federal and state regulations governing cogeneration.[3]

AWMA and SCE subsequently entered into discussions and negotiations regarding the system and fiscal implications of cogeneration. System provisions include such items as switching gear and relays to protect both the treatment plant and the power from improper parallel operation grid. To pay for power returned to the grid, SCE proposed a time-of-use schedule. AWMA and BC suggested a flat rate of 80 percent of AWMA's average cost for power purchased from SCE. The latter approach has been adopted. This level is probably slightly below the maximum allowable rate in California which the State Public Utilities Commission (PUC) has set at the cost of power generated with newly added capacity. Negotiations also covered the purchase of power from SCE for on peak operation and the provision for standby capacity should the digesters fail to produce gas.

The engine generators will also be connected to the local natural gas line. The natural gas will provide about five percent of the total energy in the digester gas. The natural gas will be blended with digester gas to assure uniform combustion in the engines. In the event of a grid power outage when digester gas production is low, one 400-kW generator can be operated solely on natural gas, providing additional backup to the plant systems.

The AWMA cogeneration system will initially provide over 50 percent of the total electricity requirements for wastewater treatment, feed about 1600 kWhr per day back into the SCE grid, and shave almost 40 percent from the AWMA's peak daily power demand. At ultimate capacity, the savings should be even greater. Efficient use of a byproduct from waste treatment may also qualify AWMA for supplemental funding from U.S. EPA for the implementation of alternative technology.

CASE HISTORY 2 - UTILITY PROMOTED ENERGY CONSERVATION. As discussed previously many utilities have undertaken programs to promote energy conservation by their customers. The programs differ depending on the utilities' needs and the response of their customers. This case history documents the conservation efforts of the Tennessee Valley Authority (TVA).[4] Commercial and industrial (C&I) consumers use two-thirds of all power sold by

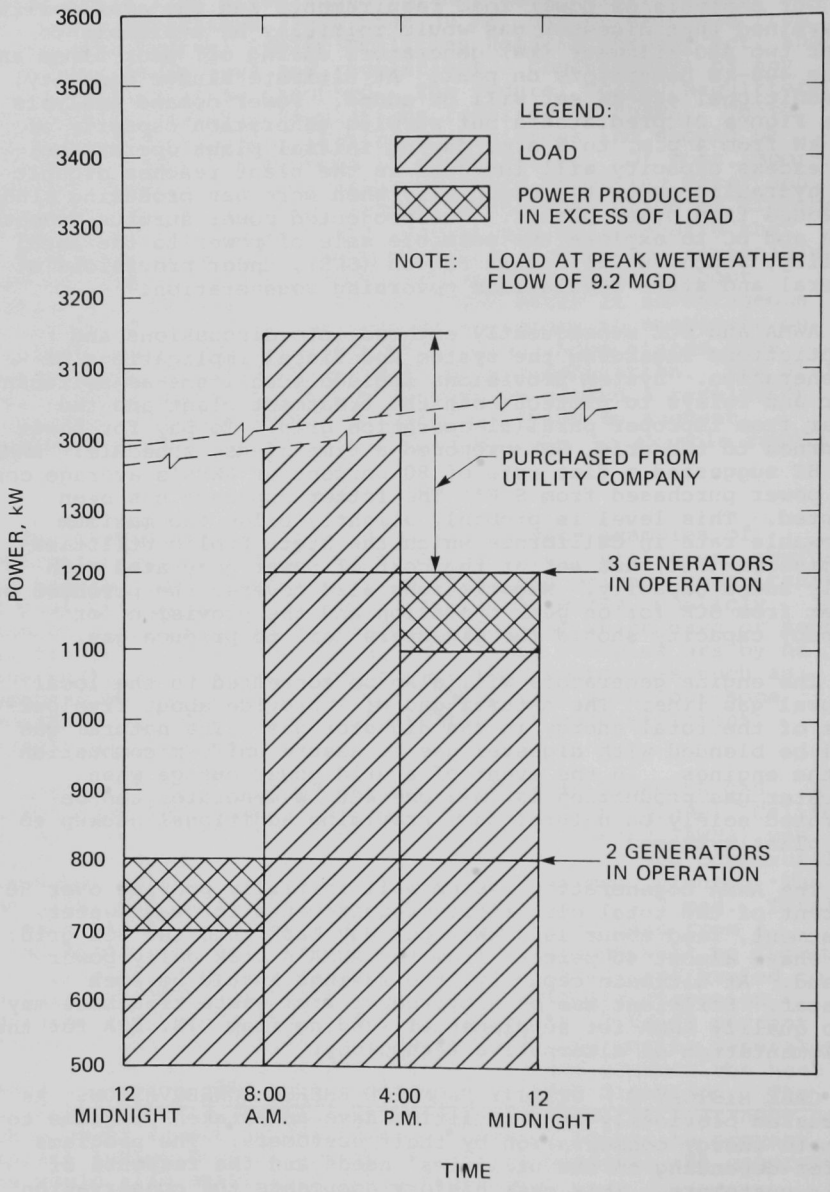

Fig. 2 Projected Power Demand at AWMA Facilities, Laguna Niguel, California

TVA, about 76 billion kWhr in 1978. While TVA's rates are still low by national standards, the power cost for these users has increased at more than 20 percent per year for the past decade. In an attempt to hold down future increases, TVA has launched an aggressive energy conservation program designed to reduce peak demand in 1988 by 1000 megawatts and annual energy use by 5.6 billion kWhr. The program includes the following features:

- Energy surveys.
- Computer energy analysis.
- Technical assistance.
- Workshops.
- Advice on new construction.
- Financial assistance.
- Cogeneration.

The energy surveys, provided free to C&I customers, are conducted by TVA personnel or private consultants whose qualifications have been reviewed and approved by TVA. Since January, 1979, over 460 surveys have been completed. The surveys examine all forms of energy use and all energy-using systems to determine where, how, and how much energy is used. The output from the surveys is a prioritized list of ECO's and estimates of potential energy savings and an estimate of the cost of implementing conservation and each ECO's payback period.

The computer energy analysis program provides C&I customers with access to computer programs for use in studying energy applications and financial impacts for new and existing systems. Technical assistance is also made available to power distributors who buy power from TVA. The workshops and advice on new construction are part of an information service designed to educate specific customers of energy conservation measures and to allow TVA engineers to work with the planning staffs on new projects to assure the efficient use of energy

TVA's loan program can provide a customer with $1,000 to $100,000 to implement electrical ECO's. Repayment is by monthly installment for up to 10 years, with an interest rate one percent above TVA's current rate for borrowed money. Some collateral for the loans is required and private firms must implement ECO's with payback periods of less than three years to qualify for loans for the ECO's with repayment periods exceeding three years.

TVA also has a parallel program designed to provide energy surveys and no interest loans for homeowners. Over 140,000 homes have been checked already. The authority estimates it is

already saving 160 million kWhr per year from those homes.[5]

CASE HISTORY 3 - ENERGY CONSERVATION THROUGH DESIGN AND PROCUREMENT. Working with a California utility, Brown and Caldwell has recently implemented an innovative bid procedure for publicly funded wastewater treatment facilities that focuses on energy conservation as well as capital costs. This approach uses specifications that require a manufacturer's quoted level of equipment efficiency as well as overall materials and construction quality. The bid documents call for installation of aeration equipment needed as part of a facilities expansion.

The following table summarizes bid results. Under traditional evaluation procedures, Bid 1 would have received the award based on the lowest total capital cost. Bid 2, with the third highest capital cost, has the lowest overall cost, however, because of its energy efficiency. The projected power use for Bid 2 is 45 percent below that for Bid 1, the largest energy user. For purposes of evaluation, all systems were assumed to have equivalent maintenance costs and equipment lives. Other assumptions are shown on the table.

To assure overall performance, the contractor must post a bond equal to 60 percent of his bid price. This bond will be returned when the equipment satisfactorily meets (or exceeds) his bid performance estimate at acceptance testing upon completion of construction. The initial specifications describe the testing procedure in detail. If the system fails the test, the district retains both the system and the bond.

Competitive Bids for Energy Efficient Aeration Equipment

Bid number	Equipment type	Bid prices, million dollars		
		Power[a]	Capital[b]	Total
1	A	2.29	2.04	4.33
2	B	1.27	2.18	3.45
3	C	1.86	2.14	4.00
4	C	1.86	2.45	4.31
5	B	1.27	2.28	3.55

[a]Assuming power use over 20 years, a base power cost of 2.9¢/kWhr, and annual inflation at 6.875 percent.

[b]1979 dollars.

This bidding approach has potential advantages for the procurement of large equipment items that are capital and energy intensive. There is also the possibility of further indirect energy savings. The combination of capital and energy costs places a burden on the contractor and his equipment suppliers to pick an optimum system that uses just enough equipment to successfully treat a waste stream. Thus the manufacturer who knows the capabilities of his equipment benefits if he specifies the minimum amount of equipment to do the job. Less equipment means less first cost of energy. This approach differs from one where the equipment manufacturer supplies design data to the engineer. Then the manufacturer may benefit by selling the most equipment.

Disadvantages also accompany this approach. The crucial evaluation is a performance test, completed long after the contract award. There is an inherent risk in getting a system you did not expect since specific technology is not required. Finally, writing specifications will require emphasis that the engineer previously had concentrated on design drawings. In fact, the contractor rather than the engineer provides the final drawings of the as-constructed system.

CASE HISTORY 4 - CODISPOSAL OF SLUDGE AND MUNICIPAL REFUSE. Brown and Caldwell has recently completed analysis and demonstration testing of refuse derived fuel (RDF) as a fuel source for multiple hearth furnaces. This work was done to develop a means for minimizing purchased energy requirements at the 40-mgd water reclamation plant of the Central Contra Costa Sanitary District (CCCSD), Concord, California. This facility uses a combination of physical, chemical, and biological processes to reclaim up to 20 mgd of sewage and make it suitable for use as industrial process water. Such high level treatment requires large energy inputs for plant processes and also produces chemical and biological sludges for reuse or disposal.

When fuel costs began to rise rapidly in the early 1970's, the District recognized that energy was available in the more than 700 tons per day of mixed municipal refuse (MMR) buried in a landfill adjacent to the plant site. They authorized first, an engineering study and later, pilot scale and demonstration testing of a hybrid energy recovery/sludge disposal concept. The basic flow scheme for codisposal of sludge and refuse, shown in Fig. 3, includes processing of MMR in a front-end or materials recovery plant to separate out recyclable metals, RDF, and an unsalvageable fraction, followed by the feed of RDF and wastewater sludges to a multiple hearth furnace. The RDF-sludge mixture is combusted in the furnace and heat recovered as steam from the furnace afterburner is used directly for process energy in the wastewater treatment plant and converted to electricity for inplant system use.

Testing of this codisposal process was done in two modes - incineration and starved air combustion (SAC) - at wet feed rates

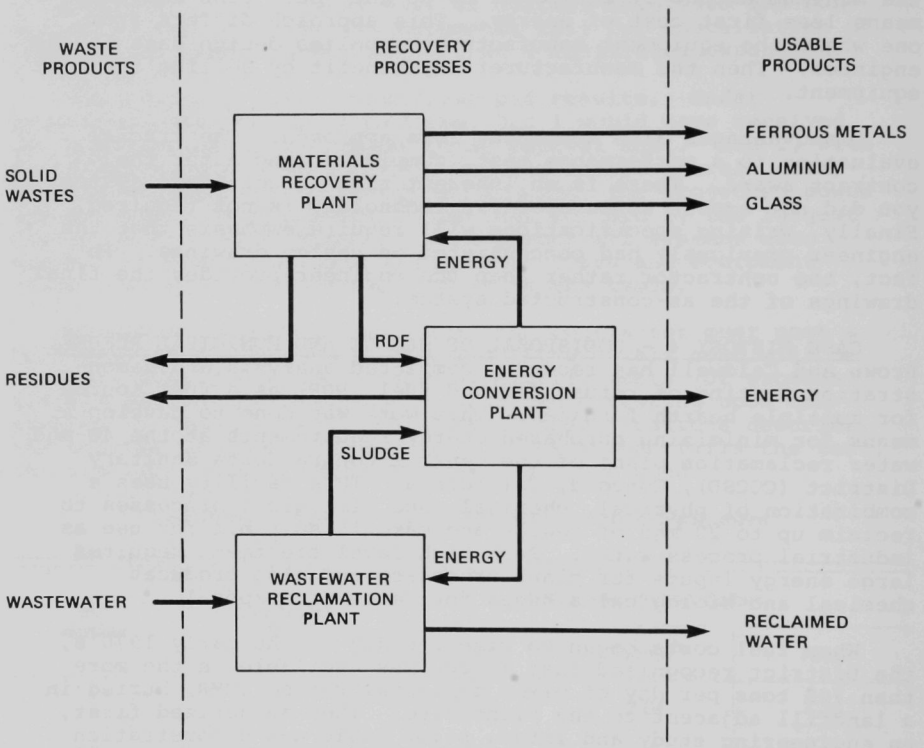

Fig. 3 Resource Recovery Concept Applied at Central Contra Costa Sanitary District

of up to 10 tons per hour. Furnace feeds included sludge alone, various sludge/RDF mixtures and RDF alone. For incineration, excess air is added to release all heat from the fuel and completely combust all volatile elements. For SAC, the combustible elements in the feed material are gasified by heat in the presence of a controlled amount of oxygen. We recommended SAC as an optimum operational mode because of its potential for better fuel efficiency, higher feed capacity to the furnaces, and less air pollutant production.

In the incineration mode for feeds of sludge and RDF, 70 to 100 percent excess air was used to ensure complete combustion. Autogeneous combustion could be maintained in both incineration and SAC with an RDF to sludge ratio of 1:2 and a sludge feed of only 16 percent solids. Off-gases from the multiple hearth furnace in the SAC mode were combusted in the afterburner at temperatures of 2200 F with the addition of air. No supplemental fuel was required. This temperature is more than adequate to destroy all volatile organics carried over from the furnace.[10]

Autogeneous SAC (no supplemental fuel addition and an afterburner temperature of 1400 F) was achieved for 24 percent sludge solids feed; the sludge had a heating value of 9,000 Btu per pound dry solids and a volatile solids content of 75 percent. The SAC combustion off-gas had a heating value of 90 Btu per standard dry cubic foot (scf) with a sludge only feed and a mean of 136 Btu per scf for sludge/RDF mixtures.

An analysis of our CCCSD test results and data on the availability of refuse at a nearby landfill site showed enough refuse is available from the local community to supply RDF for all of the purchased energy needs in the CCCSD wastewater reclamation plant and the materials recovery plant. Analyses indicated that there would be sufficient power available for satisfying all peak local demands. Economic analysis showed the use of RDF with sludge in a multiple hearth furnace to be more cost-effective than using conventional fuels.[6]

The basic concept of codisposal of MMR and wastewater sludges is being considered or undergoing implementation in several major U.S. metropolitan areas. Any project of this kind requires substantial cooperation between the agency responsible for wastewater treatment and the agency or contractor responsible for solid waste collection and disposal. Since cogeneration is a distinct possibility, power utilities and steam used may also have mutual interests.

SUMMARY

In this paper, we have attempted to show some of the ways the private sector and public agencies can cooperate to achieve energy conservation. Our examples have results that are on the whole positive. Unfortunately they probably do not represent uniform success throughout the country. Achieving cooperation

requires substantial effort by all parties involved. Vested interests, for example jurisdictional disputes between wastewater and solid waste disposal agencies, must be put aside in favor of the common good. Attitudes must change. There is nothing inherently wrong with an industry generating its own power or even selling power to a utility if the operation is economically viable and environmentally sound. Siting a waste-to-energy facility or a power plant near an urban center may lead to the best overall use of resources. The continuing energy crisis is a powerful stimulus for positive action. We must cooperate now to achieve energy conservation.

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THE CODISPOSAL OPTION-KREFELD'S METHOD OF REFUSE AND SLUDGE DISPOSAL WITH CONCOMITANT ELECTRICAL POWER GENERATION*

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INTRODUCTION

The European codisposal facilities employ predominantly two successful technologies which can be classified as co-composting and co-incineration. While composting is considered by most as a simple and inexpensive approach, it has demonstrated persistent marketing problems with its product materials. In most recent times, due to the progressive sewering of industrial and commercial neighborhoods, it has suffered greatly from the uncertainties which surround the fate of the heavy metals. Its critics have charged that through the agricultrual use of compost produced by codisposal, heavy metals may enter the food chain and endanger human health.

Unlike composting systems, large incineration systems, on the other hand, have proliferated in Europe in spite of the fact that they are both capital and energy intensive and, unless properly designed, can have major environmental impacts (Reference 1).

TOP CONTENDER FOR LARGE URBAN AREAS

The European experience indicates that "Codisposal - Refuse Power Plants" or C-RPP's are most responsive to the real need of large urban areas for nearby and centrally located facilities with high daily processing capacities. In these C-RPP's, sludge is dried by direct contact with hot flue gases to the point of its conversion to a fuel which is also called "Sludge Derived Fuel", or SDF, which at a solids concentration of 95%, or more, and with a heating value of 6,000 to 10,000 Btu/Lb resembles low quality coal. This SDF is then fired in suspension in a zone downstream of, and separate from, the grate-fired refuse. This method of direct drying followed by suspension firing is also known as the DD-SF method.

Energy is efficiently recovered in the form of superheated steam, if electrical power generation and/or district heating is comtemplated. Ferrous metal is also recovered, either from the raw refuse itself, or from the residue resulting from combustion. This aspect further enhances the resource recovery capabilities of these facilities. It is important to recognize the main distinction which separates C-RPP's from other codisposal/co-incineration approaches: thermal and/or electrical energy <u>is</u> exported for sale to external customers in significant quantities.

Table 1 lists the more important C-RPP's of the DD-SF type and indicates their technology status. All of the plants listed serve large populations either by their proximity to major cities, or by their integration into regional plans.

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In terms of processing capacities, these C-RPP's range from 150 to 1,500 STPD (short tons per day) for refuse and from 50 to 1,800 WSTPD (wet short tons per day) for sludge. Electrical output, if expressed as installed generating capacity, covers the entire 4 to 250 MW span.

Three well-known European companies need to be recognized for their outstanding contribution to the development and application of the DD-SF type codisposal technology: Vereinigte Kesselwerke (VKW) of the FRG, Widmer + Ernst and Von Roll, both of Switzerland. Of these three, as can be gleaned from Table 1, VKW clearly emerges as the leader, having provided such technology since 1961 for a variety of large scale projects. Its two main competitors, Widmer + Ernst and Von Roll, are catching up fast however. During its past development efforts, VKW was privileged to draw on the combined resources of its parent, Deutsche Babcock, and its subsidiary, Keller-Peukert.

These same three companies are active now in the emerging U.S. market for codisposal, and their technology can be provided to interested municipalities either through American licensees or by direct representation. During recent conferences with this author, all three companies voiced a definite concern for the proper transfer of their technology, because any attempt at modification might jeopardize ultimate success during application of this technology in the U.S.

With regard to the DD-SF technology, two recent developments deserve to be mentioned in particular.

One was the award by the City of Munich in the FRG to build a large C-RPP of the DD-SF type. Munich, (with a population of 1.4 million) is a modern, large and highly commercial city which opted to reduce its various composting operations in favor of electrical power generation. As in the case of two large and plain RPP's previously built in Munich, this new C-RPP will be provided again by the team of VKW and Martin.

The other highly significant event will be the commissioning of the EBS Vienna codisposal plant by the Von Roll/INOR team early in 1980. In this case, about half of all industrial waste generated in the country of Austria is to be codisposed with the sewage sludge from the city's 1.6 million inhabitants. As a result of such waste processing, energy is recovered in the form of electrical power and district heat. Approximately 9.2 MW will be generated, the majority of which will be sold to the adjacent sewage treatment plant. The district heat will be transported through hot water loops to residential neighborhoods.

ENVIRONMENTAL CONCERNS ASSOCIATED WITH REFUSE POWER

Several characteristics of major environmental significance must be associated with the operation of Refuse Power Plants in general, and with those of the C-RPP type in particular, regardless of the origin of their technology:

- -safety and health hazards
- -liquid effluents
- -gaseous emissions
- -residue stability

Of these four, the nature of gaseous emissions and their control has received major attention from the regulatory agencies in Europe and especially from those in the FRG.

Apparently, most U.S. municipalities preferred to shut down their incinerators rather than upgrade them for compliance with new pollution control regulations; whereas just the opposite is true for the FRG, where during the last decade and a half a great deal of new capacity has been added. (See References 2 & 3). As a result, during 1976, the FRG could boast of approximately 35,000 STPD installed capacity versus some 45,000 STPD in the U.S.

While particulates are well controlled by the application of high performance electrostatic precipitators, the same cannot be said for HCl and SO_x, because precipitators have little effect on gaseous pollutants.

The FRG Federal EPA took due notice of these adverse developments and responded by rewriting the emission standards for Refuse Power Plants. The resulting T.A. Luft 1974 is clearly more stringent than any standard presently in force in the U.S., or for that matter, anywhere else in the world. (Reference 4)

The regulations for C-RPP's are even tougher because of the unique physical, chemical and biological characteristics which distinguish sludge from refuse. The first of these is the resistance of sludge to complete burnout, even under the most favorable conditions of the three T's (temperature, time and turbulence).

Other problems could arise in areas where raw, unprocessed sludge is stored or transferred, due to biological decomposition with accompanying odor and/or explosion hazards.

Therefore, T.A. Luft, 1974 contains special requirements which deal with the unique problems of C-RPP's. An example is the storage and transfer of sludge which must occur under negative air pressure in a manner which permits combustion of any evolving gases and vapors at temperatures in excess of 800° C ($1,472^{\circ}$ F). (Reference 4)

KREFELD C-RPP: THE MODEL FOR MODERN CODISPOSAL

Of the plants listed, the Krefeld C-RPP was selected for an in-depth review as part of this paper mostly because its air pollution control system (APC) is the most advanced installed in any plant anywhere. Furthermore, Krefeld has just completed its first four years of continuous operations, thus providing a first opportunity to analyze development trends.

The Krefeld C-RPP is located on the west bank of the Rhine River in Krefeld, FRG, adjacent to the municipal sewage treatment plant. (See Figure 1) This plant, built by VKW and started up in 1975, is considered the most modern example of the codisposal type of plant because:

- it practices cogeneration with concomitant export of both electricity and district heat;
- it has the first full-scale combined precipitator/wet scrubber air pollution control systems (including desulfurization);
- it survived the tough challenges from the state licensing board in a heavily industrialized locality with non-attainment stature.

OWNERSHIP, STAFFING AND INVESTMENT COSTS

The Krefeld C-RPP is owned by the City of Krefeld and operated by its

Department of Sanitation in unison with the adjacent sewage treatment plant. The present operating staff consists of 85 people, 15 of whom perform administrative functions and 15 of whom tend to the needs of the sewage treatment plant, thus leaving 55 people for operating the C-RPP itself. In order to prepare for the pending plant expansion, the C-RPP operating staff will be expanded late in 1980 to 80 people for a final complement of 110 people.

Investment costs in 1975, at the completion of Phase I construction, amounted to DM 58.5 million ($23.8 million at average 1975 conversion rates). Phase II construction during 1980, in terms of a third mechanical system and certain associated structural changes is estimated at DM 40.0 million ($21.6 million at 1979 conversion rates). A new condensation turbine complete with generator and switch gear will add in 1980 an additional DM 3.0 million ($1.6 million at 1979 conversion rates). Upon completion in 1980, the Krefeld C-RPP will represent a total investment value of roughly DM 115.0 million, or $62.2 million at 1979 conversion rates.

Of these investment costs, 30% was provided in the form of non-repayable state grants, whereas the remaining 70% was borrowed at an interest rate of about 6.5% on the commerical mortgage market. Most importantly, it should be pointed out that the equipment dedicated to codisposal added only about DM 4 million, or $1.6 million to the 1975 capital requirement for the basic RPP. In other words, by changing the waste disposal concept from RPP to C-RPP, a capital growth increment of only 8% resulted. By comparison, a separate sludge combustion facility equipped with fluidized bed reactors or multiple hearth furnaces would have cost approximately DM 12.0 million ($4.9 million at average 1975 conversion rates) thus indicating substantial savings accrued to the City of Krefeld because of its decision to go codisposal.

An even more convincing case for codisposal can be made in a subsequent section dealing with operating costs.

PHYSICAL PLANT DESCRIPTION

The sectional view contained in Figure 2 illustrates the working elements of the Krefeld C-RPP for the reader who seeks a more detailed understanding. For reasons of brevity, only three key aspects are discussed in this section by means of simplified block diagrams: sludge processing, flue gas purification and energy recovery.

Figure 3A shows the three basic steps which comprise the basic sludge processing subsystem: initial dewatering by centrifuging, subsequent drying by direct heat transfer and final destruction by combustion.

Figure 3B summarizes the three basic steps which are used in the flue gas purification subsystem: particulate removal by filtering, gaseous pollutant reduction by wet scrubbing and exhaust plume suppression by reheating.

Figure 4 displays the intricate relationships as to how the energy released by the burning of refuse and sludge can be harnessed for the autogenous operation of both the C-RPP and the adjacent STP. Because of the cogeneration principle employed, surplus energy can be exported in electrical and/or thermal form to industrial customers.

PLANT PERFORMANCE

The City of Krefeld has no operational landfill left. Therefore, it is completely dependent on the C-RPP for its waste disposal needs. Since its startup in 1975, the C-RPP has met this obligation without any interruption. From Table 2 it can be seen that the annual tonnages of waste processed showed a steady increase. Plant utilization, as expressed by a consistent growth in the grate capacity factor, is an excellent indicator for the maturing of plant operations at Krefeld. Collectively speaking, these gains were brought about by the staff's learning experience, changes in operating strategies and last but not least, by equipment modifications.

In 1976, during its first full year of operations the Krefeld C-RPP processed 112,834 ST of refuse which was mostly contributed to by the three municipalities of Krefeld (230,000 people), Rheinberg (13,000 people) and Lintorf (5,000 people). During 1978 the processed tonnage increased to 138,899 ST with most of the increase (about 10,600 ST) coming as overflow tonnage from the nearby Oberhausen RPP (20 miles to the Northeast). Earlier in 1979, the City of Ratingen joined the Krefeld system with another 52,000 people, a move which should add another 26,000 ST (at the prevailing wasting rate of about 2.7 Lb/capita-day) to the annual tonnage to be processed. Consequently, at the end of 1979, upwards of 322,000 people will be served by the Krefeld C-RPP, a number which is expected to grow to 480,000 when the third line will begin operations and additional neighboring communities will have signed up.

Unlike refuse processing, the sludge processing rates have shown only a very modest increase during the first four years of operations, i.e. only a growth of approximately 10%. As the dewatering capacity factor (centrifuges) indicates in Table 2, the plant's sludge processing subsystem is still greatly underutilized. This situation is bound to change drastically once the sewage treatment plant upgrades the quality of its treatment and expands the quantity of its flow volume. The present rate of 7 tons of refuse per ton of sludge cake is expected to decrease in the future as additional sludge quantities become available.

Ash production, as a function of wastes processed, is about 6% higher for the Krefeld C-RPP than for RPP's which fire only refuse, a fact which can be readily explained by referring to the sludge component at Krefeld. For example, the Krefeld C-RPP averaged 39% compared to 33% at the nearby Duesseldorf RPP. A private contractor hauls the Krefeld ash to his own site for ferrous scrap removal and for the screening out of construction aggregates. The latter are then barged down the Rhine River to be used in Holland for dike construction.

ENERGY RECOVERY

While the boilers are designed for a specific steaming rate of 3.33 (44 stph of steam output per 13.2 stph of refuse input) this number is hardly ever reached in practice. This is mainly due to two reasons.

First, annual average heating values (LHV) for refuse tend to remain below design values. For example, during 1978 the average heating value at Krefeld was only approximately 3,240 Btu/Lb which is well below the 4,500 Btu/Lb used for boiler design.

Second, the actual heating values of refuse loads delivered fluctuate on an hourly, daily and monthly basis, and unlike the operator in a coal-fired power plant, the refuse power plant operator has little if any control over the quality of his fuel. As a result, steaming rates fluctuate widely and even with the best in operating strategies, annual averages are always bound to stay below design values.

For 1978 an average gross steaming rate of 2.08 tons of steam per ton of refuse fired was established. From this about 10% must be subtracted for in-plant use, leaving a net steaming rate of 1.87 for electrical power production.

The two turbo-generator sets are identical back pressure machines of the AEG type G16. Each generator has a rated output of 1.41 MW for a steam consumption of 19.5 stph of superheated steam at 290 psig and 700° F. With zero equipment outage, an adequate supply of steam and a constant steaming rate, an annual output of 2 X 1.41 X 8,760 = 24,760 MWh would be possible.

In reality, due to the aforementioned lack of control over the heating value of refuse, this output must be derated by an estimated 20%, thus leaving only 19,808 MWh. Actual steam supply was limited to an average flow of (138,899 X 1.87) ÷ 8,760 = 29.7 stph. This deficiency in steam supply further reduced the output potential to an estimated:
(19,808 X 29.7) ÷ (2 X 19.5) = 15,085 MWh and the true electrical output was recorded as:

Turbogenerator Set I:	9,189 MWh
Turbogenerator Set II:	6,074 MWh
Annual Plant Total =	15,263 MWH

This same electrical output can also be expressed in terms of electrical energy yield per ton of refuse fired: 15,263,000KWh ÷ 138,899 ST = 110 KWh/ST

An upswing of electrical output for 1979 is expected because equipment availability and refuse throughput continue to increase, thus permitting projection of a higher total. Due to further growth in the heating value of refuse, an increase in the specific steaming rate is expected as well.

The long term outlook is even better because a new 11.5 MW condensation turbine will be installed shortly and additional quantities of refuse can be processed once the third boiler is installed.

Since the Krefeld C-RPP is also set up for cogeneration, i.e. the use of the exhaust heat from the turbines for district heating, one needs to combine electrical exports with thermal exports to understand the full impact of energy conservation at Krefeld.

At present, the major thermal customer is a railroad car manufacturer located about 1.3 miles from the C-RPP. He is supplied with space and process heat by means of a hot water loop. Energy in this loop is replenished by condensing the back pressure steam from the turbines in condensing type heat exchangers. Because this service is of the non-interruptible type, the Krefeld C-RPP maintains two oil fired hot water boilers as back up equipment.

The original contract in 1975 called for providing a heat demand of 48 MM Btu/hr. In 1976 this contract was expanded to 104 MM Btu/hr and a current option calls for 136 MM Btu/hr by 1981. Actual deliveries in 1978 ranged from

6 MM Btu/hr during the summer to 28 MM Btu/hr in the winter for an annual average of approximately 16 MM Btu/hr. This latter figure can be converted to an electrical equivalent of 41,058 MWh/yr. Assuming that essentially all of this thermal energy was refuse derived, the total energy output by the C-RPP for 1978 can be estimated at:

$E_{tot} = E_{electrical} + E_{thermal} = 15,263 + 41,058 = 56,321$ MWh.

Considering the heat release of 138,899 ST of refuse fired at an average lower heating value of 3,240 Btu/Lb, a net thermal output of 900 X 10^9 Btu/hr would result. This latter number can be converted to the equivalent of 263,600 MWh/yr thus permitting an estimate of the overall energy utilization at (56,321 ÷ 263,640) X 100% = 21.4%.

APC TESTING

As the Krefeld C-RPP is undergoing modifications and expansion, several corrections were made in the scrubber part of the Air Pollution Control System (APC). As a result, APC testing is incomplete and a cohesive data base has not been established as yet.

However, from various preliminary tests performed during the past three years, several trends became apparent. By referring to Table 3 "Particulate Emissions" the following observations can be made:

1. When firing sludge in addition to refuse, volumetric gas flow rates will increase by a factor of 1.2 to 1.5 depending on the relative feed rates of refuse and sludge.

2. The release of particulates will approximately double due to the firing of sludge.

3. Particulate loads during periods of soot blowing will also double, regardless of whether refuse is fired alone or in combination with sludge.

4. Under all sets of operating conditions, that is refuse firing only, refuse and sludge combination firing and soot blowing, the Krefeld C-RPP will meet the legal requirement not to exceed 100 mg/Nm^3 in particulate emissions. Performance of the electrostatic precipitators must be judged as excellent under all circumstances.

Table 4 "Gaseous Emissions" deals with the flue gas pollutants which for the better part, can only be removed by effective scrubbing. The following observations can be made:

1. HCl concentrations average about 1,100 to 1,200 mg/Nm^3 on the inlet side of the scrubber system and they are not affected in any obvious way by the presence of sludge derived flue gases.

2. HF concentrations appear to double at the scrubber inlet as a result of sludge firing, but the data base is by far too small to attach any significance to this observation at the present time.

3. SO_2 concentrations in the scrubber inflow are raised by a factor of approximately 1.24 due to sludge firing; this does not come as a surprise, since raw sludge contains hydrogen sulfide.

4. The influent loads for HCl, HF and SO_2 are high enough to justify the installation of a scrubber system, if one accepts the validity of the T.A. Luft 1974 criteria.

5. The scrubber demonstrated an efficiency of about 99.7% in removing HCl pollutants under most operating conditions except when certain instabilities occurred during the firing of sludge.

6. HF removal is again highly efficient with about 97.4% removal efficiency, except when instabilities occurred during sludge firing.

7. SO_2 removal is excellent, with an efficiency of about 97% when firing refuse only. During sludge firing, according to these first data, the efficiency drops off rather rapidly as instability occurs.

8. Effluent concentrations of NO_x and C (organic) are low in all cases and do not seem to be affected by the firing of sludge.

9. The scrubber system should be able to meet regulatory requirements for HCl and HF removal in all cases, including during the firing of sludge, as long as flow stability can be retained in the APC system.

10. The causes for flow instability in the scrubber during sludge firing are not fully understood at present and are the subject of continued investigations.

11. At this writing, it appears that regulatory SO_2 removal requirements cannot readily be met with confidence and some additional equipment modifications may be required. However, this statement needs to be verified by additional testing.

12. Generally, scrubber performance at the Krefeld C-RPP equals or exceeds the results previously published for the Kiel RPP, where the first scrubber was installed in conformance with T.A. Luft 1974 requirements (Reference 5).

Preliminary ash analyses were performed both for flyash and bottom ash to check on the degree of burnout achieved. Dust from the precipitator had the following composition: 94.6% ash, 5.3% combustibles and 01.% moisture for refuse firing only, versus 94.2% ash, 5.7% combustibles and 0.1% moisture for sludge fired in combination with refuse. Bottom ash showed concentrations of only 0.2% fermentables and 6% combustibles during normal sludge and refuse firing. During peak sludge disposal periods, when side injectors were used for the direct injection of sludge cake, these values increased slightly to 0.7% and 6.5% respectively.

OPERATING COSTS

Accurate operating costs are difficult to come by, because of the many changes which have and still are affecting plant operation. After all new construction is completed in 1981, it will probably take an additional 3 to 4 years of full scale operations before operational equilibrium will be reached.

However, sludge incineration costs for the year 1978 can be approximated by the following calculations:

Amortization of equipment (6.5% of DM 4 X 10^6)	DM 260,000
Interest for equipment (6.5% of DM 4 X 10^6)	DM 260,000
Personnel (5 men @ 40,000 each)	DM 200,000
Maintenance, repairs and miscellaneous	DM 280,000
	DM 1,000,000

Using a conversion rate of 1 DM = $0.500 for October 1978, annual costs of $500,000 result which can also be expressed as:

$\frac{\$500,000}{18,843 \text{ ST @ } 26\% \text{ TS}} = \$26.54/\text{WST}$, or $\frac{\$500,000}{4,900 \text{ ST @ } 100\% \text{ TS}} = \$102.04/\text{DST}$.

By comparison, the separate incineration plant previously mentioned under "investment costs" would have resulted in $74.31/WST for the cake and $285.71/DST for the powder, due to significant increases in all of the items listed above plus auxiliary fuel and electricity which would have to be purchased in addition.

Regardless of the type of technology to be used for sludge incineration or composting, initial dewatering by mechanical means is required to raise the solids content of raw sludge from 4-5% TS to 26-30% TS. During 1978 this added $36.77/WST or $141.43/DST. As pointed out during the discussion of Table 2 before, the sludge processing subsystem with a capacity factor of only 0.15 is highly underutilized, so that unit costs will be greatly reduced in the future when sludge throughput is increased.

As far as refuse disposal is concerned, the basic disposal fee of about $27/ST has remained essentially constant for the 1977 to 1979 period. This was possible because improved plant operations resulted in savings which balanced out inflationary increases.

Unfortunately, it was not possible to discuss within the context of this paper the entire operating budget and to include offsets achieved by the sale of energy to electrical and thermal customers. Ash is currently hauled away by a private contractor at no cost to the C-RPP, and therefore, does not enter into the analysis at all. Obviously, no credit can be taken for the scrap sales accomplished by the private contractor.

DOE GRANT FOR RESEARCH AT KREFELD C-RPP

Recognizing the limited data base presently available on the emissions from C-RPP's employing the DD-SF method, and being keenly aware of the need to provide a complete and well documented data base before any large scale applications of this technology can be accomplished in the U.S., the US-DOE granted a research program in October, 1978 with the following technical objectives (Reference 6):

- prepare typical thermal and mass balances for simultaneous refuse and sludge firing;
- determine gross and net energy production, thermal and electrical; and
- evaluate quality of energy production as shown by typical 24-hour, weekly and monthly profiles for steam and electrical outputs.

A combination of grab sampling, wet chemistry analysis and on-line monitoring will be utilized to provide a complete data base which will include:

- simultaneous measurements of particulates HCl, HF, SO_2 and CO before the electrostatic precipitator, after the electrostatic precipitator and after scrubbers;
- simultaneous measurement of water quality in the scrubber effluent and the water treatment systems, plus in the quench tank effluent;
- micro-pollutant analysis, especially of heavy metals, with samples of raw garbage from feed chutes, sludge coming from the HGS mill, flyash and bottom ash respectively, boiler tube deposits, ash quench water, scrubber water, and stack effluents.

Critical importance will be attached to the determination of the fate of the heavy metals because of the concern expressed in many sectors of the United States environmental field with regard to the alternative approaches to sludge disposal. Krefeld is most suitable for the pursuit of this aspect because of the presence of significant amounts of sludge derived from industrial waste waters.

CONCLUSION

The urgent need to identify alternative approaches to dumping and landfilling carries with it the obligation to develop new approaches which reflect a more prudent use of energy. The European experience, through technology transfer, can provide such an alternative at acceptable risks and in a cost-effective manner.

The codisposal of sewage sludge with municipal refuse has been successfully practiced in Europe for some time, and its associated technology can be considered proven both in terms of significant scale and longevity of operations.

Among competing technologies, the DD-SF method of thermal sludge processing as practiced in Krefeld has proven superior. This holds especially true for the very large plants serving metropolitan areas.

By utilizing refuse as an alternative fuel in place of oil and gas, the operation of C-RPP's has resulted in a meaningful reduction in energy consumption, which benefits both the local government and the national economy.

Based on this experience, many communities in the United States are now investigating the establishment of codisposal facilities. They are being hampered in this effort, however, by the lack of adequate documentation. In the absence of any prolonged testing, no extensive data base exists, today, on two key aspects: energy efficiency and environmental impact, particularly in terms of heavy metals emissions. Since the availability and use of such data is essential to project development in the United States, it is hoped that the DOE program will provide the long range answers to most, or perhaps even all of these questions. It is regrettable, however, that this program will not yield tangible results in time to affect the implementation of compliance programs presently being planned by a number of large Eastern municipalities traditionally involved with ocean dumping.

It would be prudent for the U.S. EPA to consider permit extensions for municipalities which are seriously committed to finding long term solutions to the sludge problem, such as C-RPP's. The latter, by their very nature, may require a 3 to 4 year planning and construction cycle.

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COMBINED INCINERATION OF MUNICIPAL SOLID WASTES AND MUNICIPAL WASTEWATER SLUDGE

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INTRODUCTION

Each year in the United States, roughly 150 million tons of municipal solid waste and 8 million dry tons of municipal sewage sludge are generated. Disposal of the daily generation of these waste solid materials can be very costly from both an energy and a dollars standpoint. Increasingly, due to pressures on alternative disposal techniques, such as land disposal, some type of combustion operation is adopted for disposal of these materials.

Figure 1 describes a typical steam generating solid waste incinerator. The unit described is a mass burner, that is, the feed material is unprocessed solid waste although similar units exist that produce energy from processed refuse, or refuse derived fuel (RDF). Figure 2 describes a typical flowsheet for an RDF plant. In either case, combustion of the fuel is used to produce steam or electricity. In combustion operations such as that described in Figure 1, electrostatic precipitators have been shown to be an effective air pollution control device.

Figure 3 describes a multiple hearth incinerator. This is the most common unit used to combust municipal sewage sludge. Fluid bed units, as described in Figure 4, are also used extensively. Sewage sludge incinerators are constructed so as to allow for a prolonged residence time for drying and combustion. Most often, sewage sludge is mechanically dewatered prior to introduction into the combustor. Still, even combustion of dewatered sludge normally requires introduction of a great amount of auxiliary fuel in the form of natural gas or oil.

As the availability of natural gas started to decrease and the price of oil started to increase from 13 cents a gallon in the early 1970's, municipalities started looking at ways to recover some of the energy from solid wastes and they also started looking at ways to reduce the amount of energy used to incinerate municipal wastewater sludge.

One way to recover the energy from solid wastes was to use it to replace the oil or gas used to incinerate sludge. Since the solid wastes incineration used one set of technology for incineration and sludge used another,

there were two general methods to develop: modify the sludge incineration to fit the solid waste technology or modify the solid waste incineration to fit the sludge technology.

SOME CASES

Several full-scale tests have been made co-incinerating solid wastes and sludge. Contra Costa, California[1], made a full-scale test adding refuse-derived fuel to the sludge in a multiple hearth furnace. This was a modification of the sludge technology. The furnace was operated for about 60 days and was successful at using the refuse-derived fuel to replace natural gas or fossil fuels. The furnace operated in both the incineration and the starved air modes of operation. Based on this test, Contra Costa completed plans for a co-incineration unit which is designed to process about 1200 tons per day (TPD) of solid waste into 600 TPD of refuse-derived fuel. This will be used as the fuel for a starved air combustion (pyrolysis) furnace operation which will incinerate 96 dry TPD of sludge.

Another full-scale demonstration was made at Minneapolis-St. Paul, Minnesota[2]. The original tests were planned to use coal, wood chips, or refuse-derived fuel to replace fossil fuels in a multiple hearth incinerator. Coal was originally hand fed and later, mechanically fed to the furnace. The tests proved that the use of coal resulted in lower costs for fuel than did the use of oil. Wood chips were also used to partially replace the oil used for incineration. Refuse was not used because the tests at Contra Costa had been completed and had demonstrated the economics of using refuse-derived fuel. The conclusions drawn from these tests were that coal or wood chips could be used to replace at least part of the oil normally used for sludge incineration and this would reduce the cost of fuel for the furnace. However, it was also concluded that modifying equipment to dewater sludge so that it contained 28 percent solids was more economical than using coal, wood chips or refuse-derived fuel to replace oil.

There is also an example of solid waste technology that has been modified to accept sludge at Harrisburg, Pennsylvania[3,4]. Normally sludge is not incinerated with solid wastes because when 15 percent solids sludge is incinerated, the water forms steam and creates a large volume of gas through the incinerator. This large volume of gas causes high velocities and substantial particulate carryover. This results in increased air pollution or requires that the air pollution equipment be very large to clean the gas. The inciner-

ator equipment must also be large to handle the steam volume. To eliminate this problem at Harrisburg, the sludge is dried to 85 percent solids in a Porcupine* [6] dryer. This dryer has a steam heated jacket and steam heated blades which mix, heat, and dry the sludge until it is 85 percent solids. The system is enclosed so the steam can be condensed and odors can be trapped with very little extra equipment. The resulting 85 percent solids sludge is then added to the solid wastes and incinerated. This makes it possible to use a solid wastes incinerator system for co-incineration with a minimum amount of modifications. The incinerator at Harrisburg is a water-wall which generates steam. The steam is used to dry the sludge in the Porcupine and there is surplus steam which can be used elsewhere.

The Eco-Rock system at Philadelphia[5] is also an example of solid waste incineration which adapts sludge to a solid wastes technology. This system will use burned-out solid wastes which have been landfilled as the source of energy. The landfill material is recovered and it has a heating value of about 1000 BTU/lb. Sludge, about 15 percent solids, will be added to the fill material and it will be incinerated in a rotating kiln. The residue from this incineration, which consists of ash, metal and glass, will be melted to form a top grade slag which can be used for road construction in the Philadelphia area. The system will also be capable of using the solid wastes as received if the landfill material is not available.

At Duluth, Minnesota[3,4], a hybrid technology of solid wastes and sludge incineration is taking place. It is a combined plant which incinerates the sludge using a fluidized bed system. After startup incineration using oil, the fuel is switched to refuse-derived fuel, also generated at the plant. About 340 TPD of sludge will be burned using 160 TPD of refuse-derived fuel. There is an excess of refuse-derived fuel generated at the plant which could be sold along with metals and glass separated from the solid wastes.

The Purox*[7] process (starved oxygen incineration) has been tested at South Charleston, West Virginia. The Purox system incinerates solid wastes using 90 percent oxygen and no air. It produces a gas having about 300 BTU/ft^3 heating value and the bottom residue is a slag. Sludge has been added to the process for a demonstration with no adverse effect on the process. However, the use of sludge dewatered to 40 or 50 percent solids content was recommended

*Trade name. Use of trade names does not indicate EPA endorsement.

for future operations. The water (steam) from sludges having lesser solids contents caused high gas velocities and higher than desired particulate carryover. To limit carryover, throughput had to be reduced, which of course reduced the overall capacity of the unit.

The Purox process was the beginning of the return to the gasifier systems that existed in the 1800's[8]. They could convert coal or any cellulose material into a gas usable for internal gas engines or heating. Millions of portable units were used in Europe and Japan during World War II. Inexpensive natural gas, fuel oil, and gasoline after World War II caused them to be abandoned and the technology was lost.

The Department of Civil Engineering, University of California, Davis,[9] has built and tested a 100 lb per hour gasifier which used refuse-derived fuel and sludge. The unit is described in Figure 5. It is a packed-bed down-draft unit which uses refuse-derived briquettes for gasification. As the briquettes are made up, sludge is added as a binder and/or coolant. This gasifier produces a gas having a BTU content of 140-160 BTU.

One other process has been started which will co-dispose of solid wastes and sludge in a pyrolytic type method. This the liquefaction of sludge and solid wastes to form oil, asphalt and coke. This process is being developed by the City and County of Honolulu, using Battelle Northwest[10] as the contractor. Although the process has not been proven, it is very likely to be successful. Refineries and major olefin gas producers have "cracked" oil into olefins, carbon and other oils for many years. Sludge contains about 40 percent volatiles which can also be cracked. Caustic was added to peat moss and the mixture heated and mixed to form oil in 1940[11]. Combining those two processes, sludge and/or solid wastes will be converted into oil, asphalt and carbon[12]. The carbon and part of the oil will be used for heating the process. The asphalt and ash will be used for road construction. This process was also tested to make a producer gas in the early 1970's, but was considered a failure because it produced oil, asphalt and char and not enough gas at an economical price.

CONSTRAINTS

Although co-disposal of solid wastes and sludge is technically feasible, there are some problems involved. Institutional problems have and will be a constraint. Solid wastes and municipal wastewater sludge are often handled by different government or political entities. Immediate changes in these

established patterns are not realistic, but with encouragement from EPA and a realization of the cost of fuel for separate sludge and solid waste disposal, in time, coordination among the various waste handling entities should result in more unified movement toward co-disposal.

Air pollution problems are not yet solved. The Duluth plant uses water scrubbers and the Harrisburg plant uses electrostatic precipitators for particulate removal. There are data for the release of heavy metals, sulfur compounds and nitrogen oxides into the atmosphere from solid waste burning facilities and there are similar data for solid waste incinerators. However, there are no data for co-incineration. A facility at Glen Cove, New York, has had construction delayed for a year because of questions on air pollution.

At Research Triangle Park, North Carolina, the EPA has started work on determining the air pollution from gasifiers using coal[13]. This work has discovered hazardous materials, polynuclear aromatics (PNA's), that are created when coal is used for gasification. Work on co-incineration could indicate that these substances are not present when solid wastes and sludge are used for gasifiers. At the present time, gas made from coal is cheaper than natural gas in some areas[14]. These economics should be better if solid wastes and sludge are substituted for coal.

The energy savings from co-disposal can be substantial. Duluth will save about three million gallons of oil per year. Harrisburg will save 8.4 million gallons of oil per year. Assuming these two plants are average for the nation and co-incineration is done at 200 locations in the United States, it can save one billion gallons of oil per year. Duluth will have refuse-derived fuel for sale and Harrisburg has steam for sale. Also, the savings in land used for landfill is tremendous. Duluth will save 1000 acres in 20 years. Harrisburg, which was already incinerating solid wastes, will save 20 acres in 20 years. Both plants will have scrap metal for sale. Co-incineration is a proven technology, and the outlook is that the technology will be commercialized to convert our wastes into usable energy or other useful products and, at the same time, the cost of disposal will be less than is experienced at the present time.

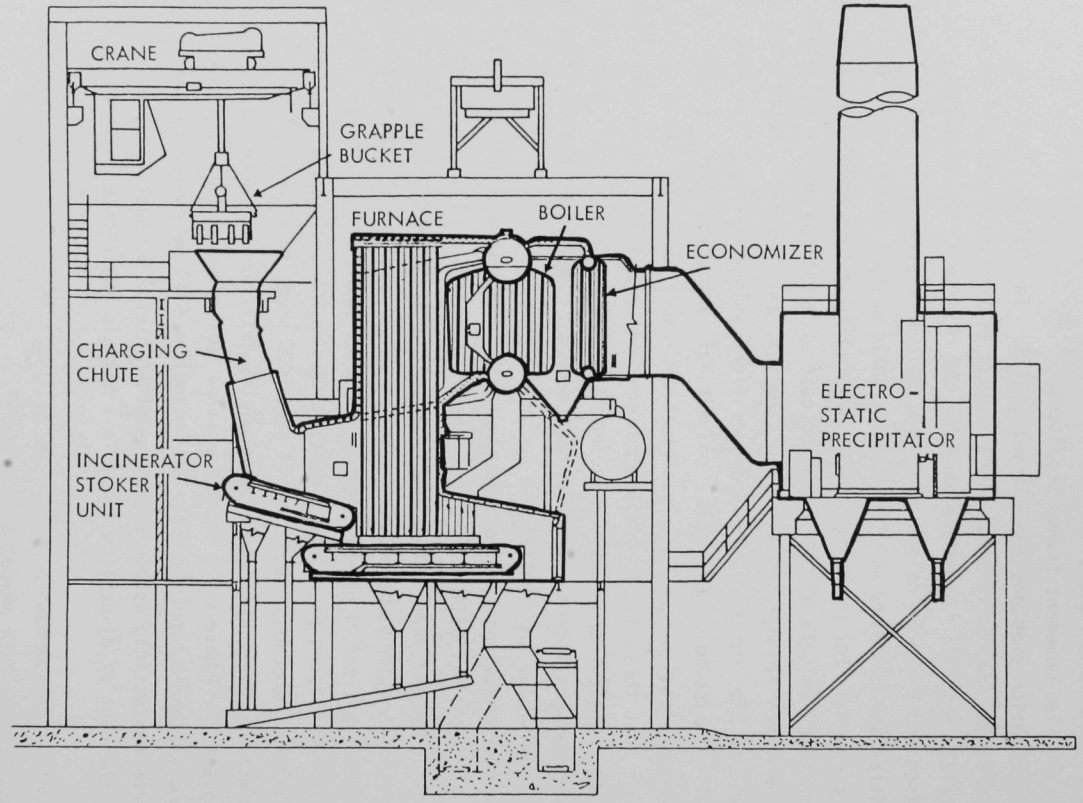

Figure 1. TYPICAL STEAM GENERATING SOLID WASTE INCINERATOR

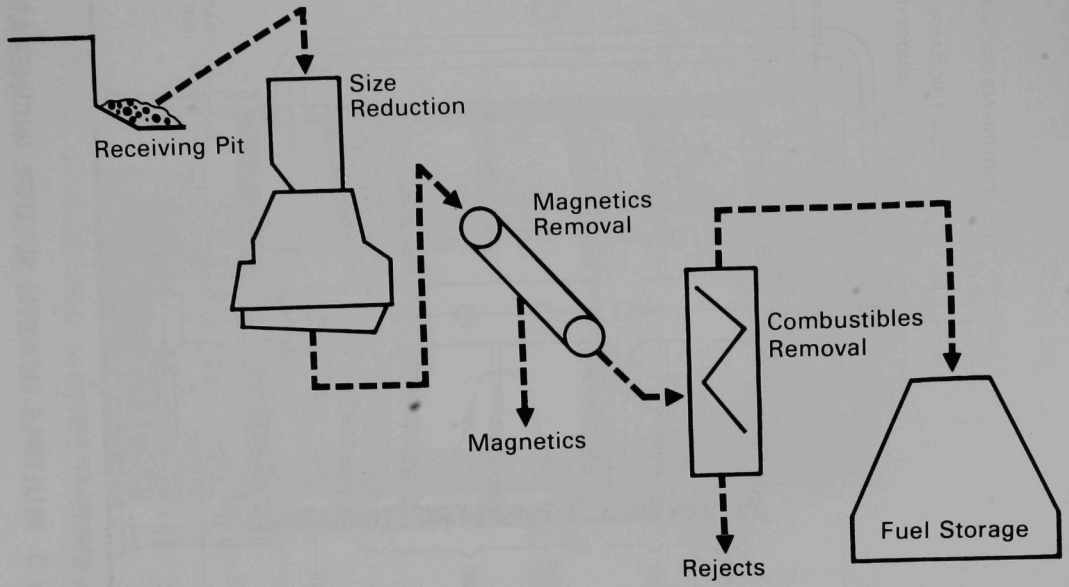

Figure 2. PROCESSED WASTE FUEL PRODUCTION FROM M.S.W.

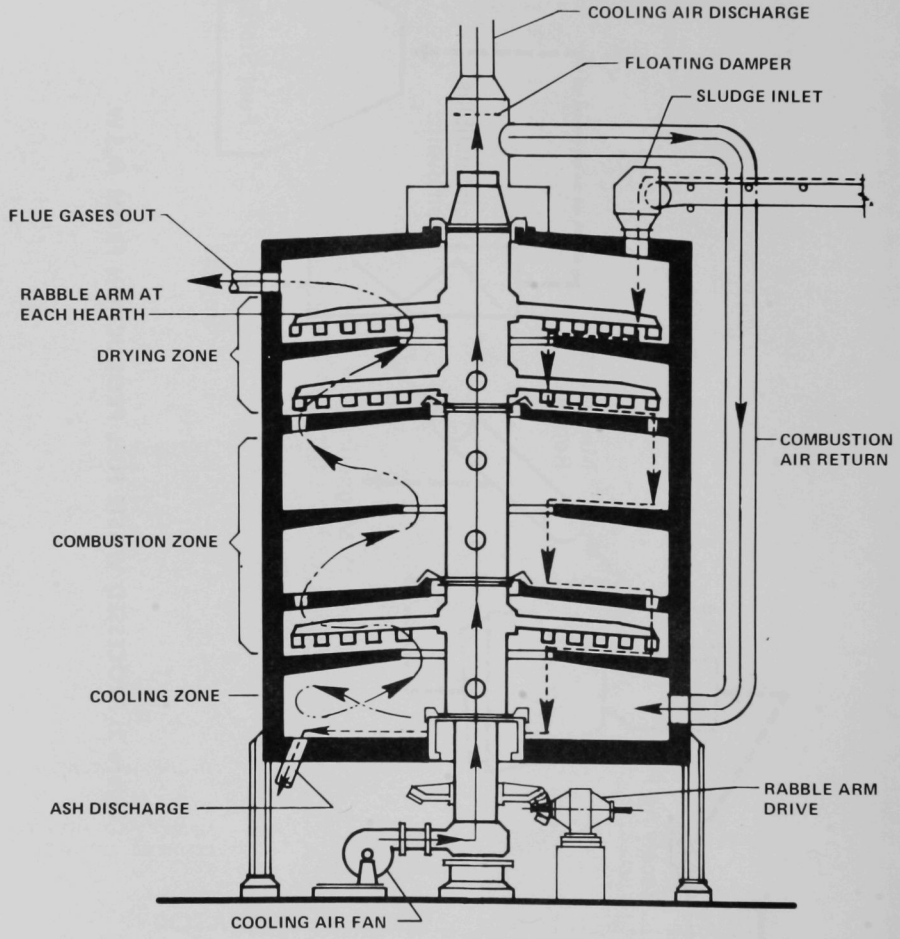

Figure 3. MULTIPLE-HEARTH SLUDGE INCINERATOR

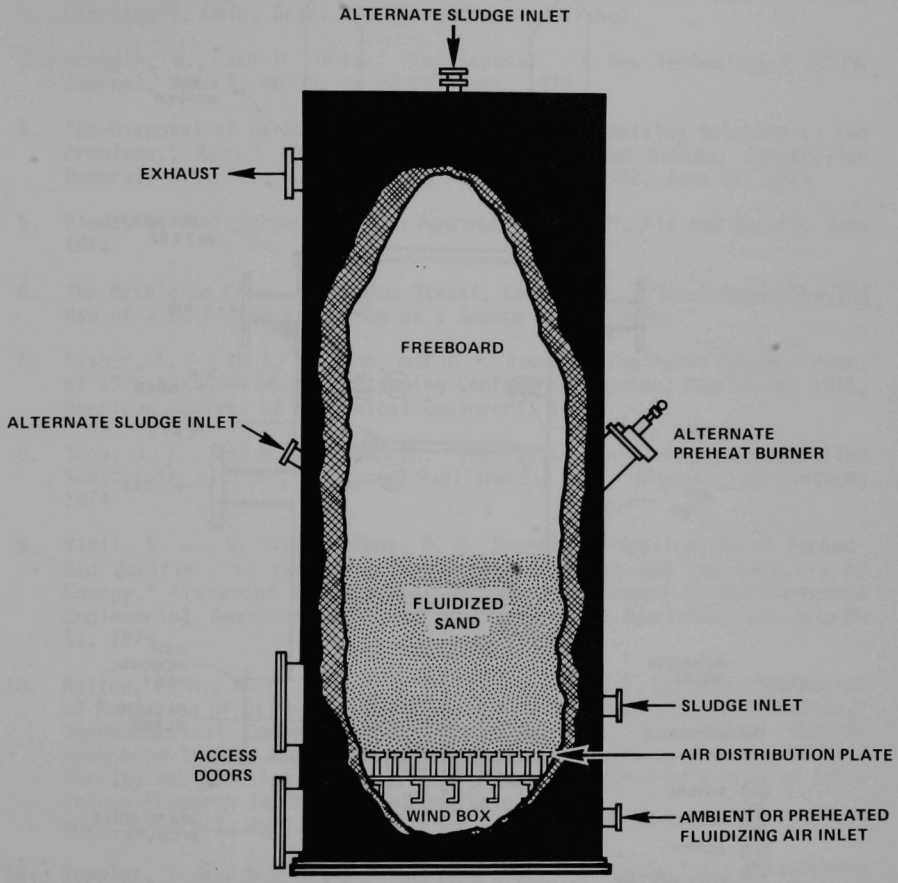

Figure 4. FLUID-BED SLUDGE INCINERATOR

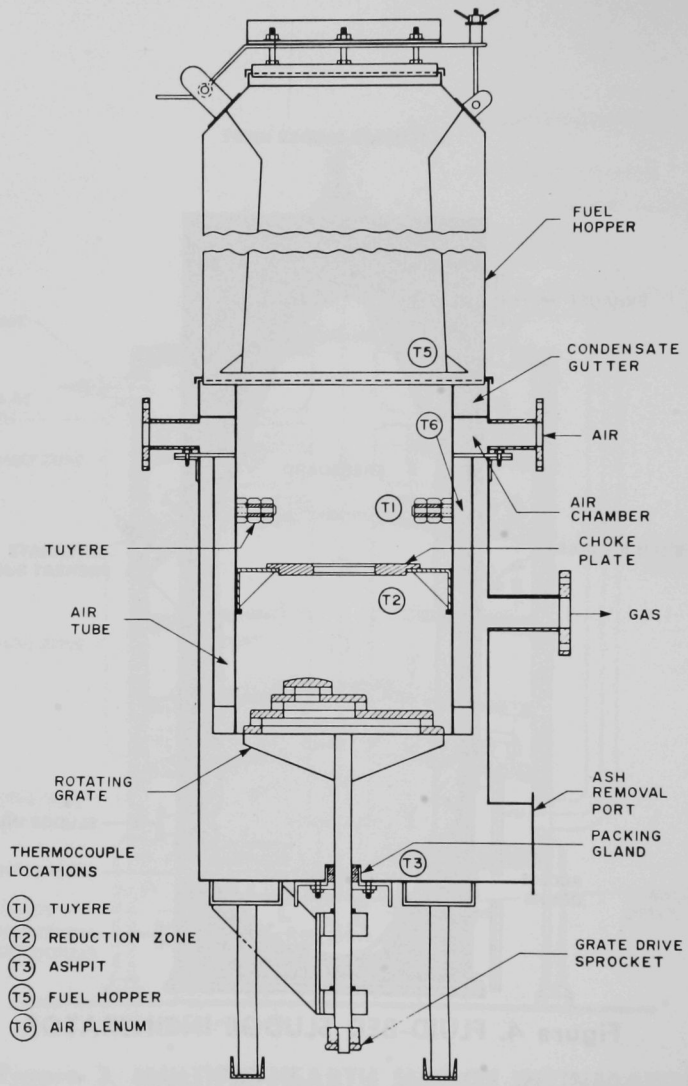

Figure 5. DOWNDRAFT GASIFIER

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ENERGY CONVERSION AND RECOVERY THROUGH SLUDGE/REFUSE CO-DISPOSAL

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1. INTRODUCTION

The City of Memphis began evaluating the economic and technical feasibility of the conversion of refuse to energy with materials recovery in 1970. Various plans or systems have been considered with many rejected for economic, technical, and implementability factors. Energy conversion was initially considered as a supplement for coal in a Tennessee Valley Authority power plant. Materials recovery was limited to ferrous metal recovery. This concept was eventually rejected for all three factors; however, during the exhaustive analysis which was performed, the magnitude of the intrinsic energy value of the City's refuse and the potential for solving other problems in the City became apparent. An example of this was a detailed analysis of the use of energy in the City's refuse to supply steam and chilled water to downtown Memphis businesses and industries. At the time of the evaluation (1975-78), the energy available from the community's refuse appeared to be the logical, economic replacement to the City's dwindling supply of natural gas available for non-residential use. In the final analysis, however, markets for the energy disappeared with the national de-regulation of natural gas. This, coupled with the results of a detailed economic analysis, inactivated the energy conversion system.

During the long evaluation/planning process which the City has experienced, energy conversion systems were oriented around disposal of refuse with the accompanying recovery of energy. One system which also had potential for solving several environmental problems for the Memphis community was extensively analyzed from 1977 to 1979. This system entailed the complete combustion of wastewater treatment plant sludge and industrial/flammable wastes with the use of energy from refuse in an environmentally sound technology. This co-disposal system has as its primary function the proper environmental management of the community's three waste materials - refuse, sludge, flammable industrial wastes - and, as by-products, ferrous metals and steam for industrial use. The detailed planning and evaluation of the economic and technical attributes were part of a U. S. EPA funded facilities planning effort. The co-disposal system was accepted for funding as an innovative process by the U. S. EPA as part of the overall recommendations of the facilities study. This concept offers an ideal utilization of

the energy available in the refuse for the maximum beneficial use of the community.

MEMPHIS CO-DISPOSAL SYSTEM

The City of Memphis forms the population and commerce nucleus for an urban community of almost one million people which has environmental problems not unlike many other urban centers. Refuse and wastewater treatment plant sludge originate from the populus making up the urban area while industry contributes not only these two but also flammable wastes. In the development of the recommended co-disposal system, the City approached the system's evaluation considering the environmental/cost effective management of all three problem wastes. This attitude of total waste management was fostered by the high cost of environmental control, allocation of limited community resources to the most important needs, and the spiraling energy crisis. Within this philosophy, three basic alternatives were available for refuse management, each with varying costs and benefits to the community. Intermeshed with these three were the considerations for sludge and flammable industrial wastes. Diagrametrically, the alternatives are shown in Figure 1. The various combinations range in technical extremities from landfill to co-combustion, in recovery extremities of zero to maximum proven capabilities, and in energy extremities of energy users to energy exporters.

The recommended co-disposal/co-combustion plan is shown in Figure 2. Refuse will be processed at two locations where a refuse derived fuel (rdf), ferrous metals, and a reject stream will be produced. The rdf will be utilized in an energy recovery center (erc) where de-watered sludge, rdf, and flammable wastes will be co-combusted to generate steam for industrial use. The erc makes use of the intrinsic energy of the rdf to dispose of the wastewater treatment plant sludge. In addition, excess rdf over and above that required to combust the sludge will be fired in spreader stoker waterwall incinerators with coal to provide backup and peaking steam requirements. The available market for process steam on a non-interruptable basis plus the City's obligation to dispose of all the refuse dictated the coal/rdf furnaces.

System Optimization: The entire system was designed around optimizing the handling of solid waste in the City of Memphis. Material processing centers were located to minimize the haul of refuse in either collection vehicles or compaction-transfer equipment. Although the savings attributable to this optimization are substantial, they were not included in the cost effectiveness analysis portion of analyzing alternatives. The erc was similarly located based on providing distribution to the industrial steam consumers. Available markets and the various cost penalties associated with various alternatives were a significant factor in the final location of the erc.

Materials Recovery Center: Two materials recovery centers are proposed to produce rdf and a saleable ferrous metal product. Initially the southern materials recovery center (mrc) will be twice the size of the northern to compensate for the larger volumes of refuse in the southern portion of the City. Refuse will be deposited on a tipping floor by both collection vehicles and compaction-transfer vehicles. Front-end loaders will push the refuse onto

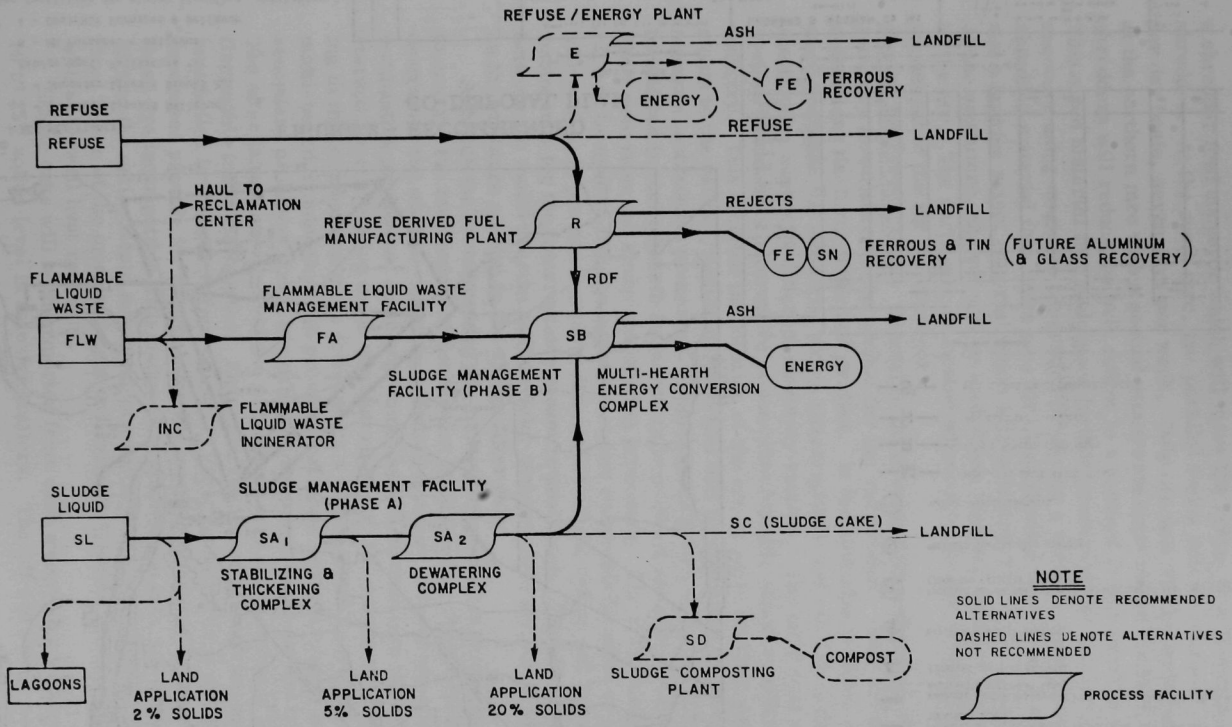
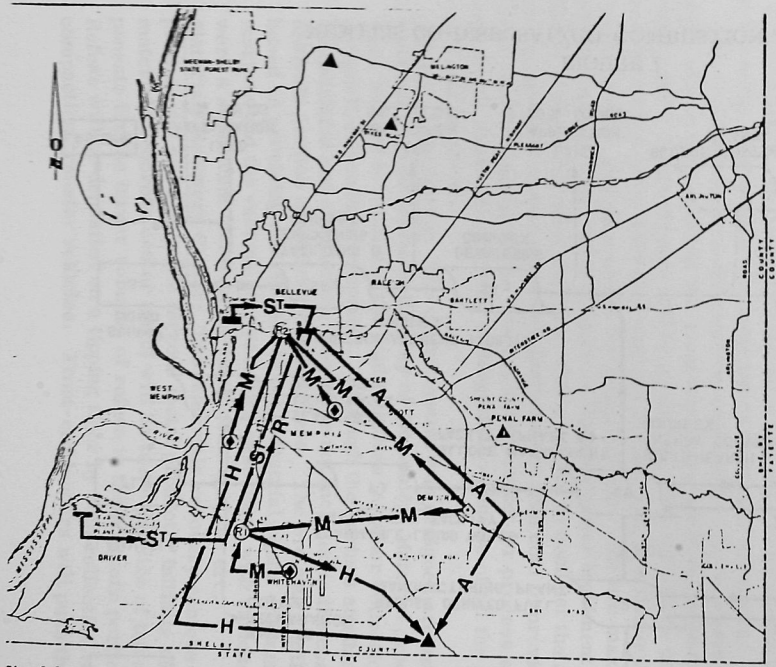


FIGURE 1
MEMPHIS CO-DISPOSAL/CO-COMBUSTION ALTERNATIVES



LEGEND

- PROPOSED MULTI HEARTH SLUDGE DISPOSAL ENERGY CONVERSION FACILITY AT ALLENTE
- PROPOSED SLUDGE MANAGEMENT FACILITY AT SOUTH WASTEWATER TREATMENT PLANT
- PROPOSED SLUDGE MANAGEMENT FACILITY AT SOUTH T.E. MAXSON WASTEWATER TREATMENT PLANT
- EXISTING SANITARY LANDFILL
- EXISTING TRANSFER STATION
- (R1) PROPOSED REFUSE DERIVED FUEL MANUFACTURING (RDF MFG) PLAN (2400 TPD FACILITY 7 DAYS BASIS)
- PROPOSED NEW TRANSFER STATION
- NONRISE REFUSE CENTROID
- —M— MUNICIPAL SOLID WASTE (MSW)
- —R— REFUSE DERIVED FUEL (RDF)
- —H— AIR CLASSIFIED WASTE (AC)
- —S'— SEMI-TRUCK TRANSFER (ST)

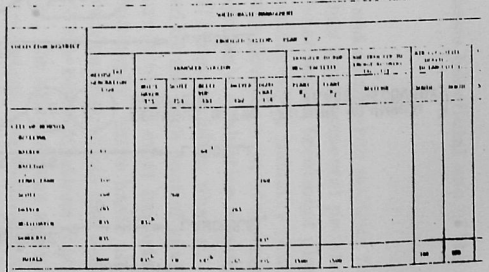
Plan 9-2

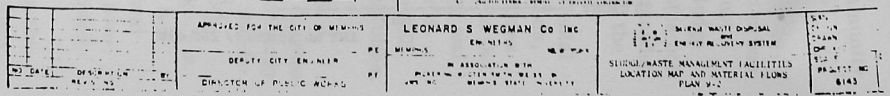
2 - RDF Mfg Plants
2 - Shredder Lines @ Bellevue
2 - Shredder Lines @ Brooks Rd.
Sludge Mgmt. Facilities *
8 - MH Furnaces @ Bellevue
4 - Coal/RDF Furnaces @ Bellevue

* Includes facilities for sludge digesting, thickening & dewatering by vacuum filtration

FIGURE 2 - RECOMMENDED CO-DISPOSAL PLAN

SCALE IN MILES 0 1 2 3 4 5





a shredder feed conveyor which will move the refuse to a 75-ton per hour shredder. At the south mrc, two parallel trains will be operated on a 10-hour per day basis, seven days per week. Only one unit will be installed initially at the northern mrc with a second processing train slated for 1986. The 75 tph shredders will reduce the bulk refuse to a nominal 3-inch particle size or less. This sized material then undergoes further processing by ferrous metal recovery using magnetic separation producing a saleable byproduct. The shredded material then undergoes air classification to produce a rdf product and a heavies reject material which will be transported to a sanitary landfill. At the south mrc, the rdf will be loaded into transfer vehicles and conveyed to the erc. The north mrc is located adjacent to the erc which allows the movement of the rdf via conveyor.

Energy Recovery Center: Crucial to the development and design of an energy recovery installation is the market for the end product. This market also determines the form of the end product. In the Memphis system, markets were available for industrial steam (interruptable and non-interruptable) and electricity, each with different monetary returns. The most cost effective system included the alternative of non-interruptable steam supplied to industry. This dictated the location of the erc so as to feasibly supply the steam to the users. Refuse-derived fuel will be converted to energy in two separate combustion processes. Approximately 90% of the rdf (1800 tpd) will be utilized in multi-hearth incinerators to combust the almost 1200 tpd of filter cake from the sludge dewatering processes. The remaining 10% of the rdf will be used to fire the spreader stoker waterwall furnaces in conjunction with coal to 1) provide complete disposal of all the refuse and 2) provide redundancy for the interruptable steam requirement. The exact balance of rdf between the multi-hearths and spreader stokers will vary day to day depending on the moisture content in the sludge filter cake.

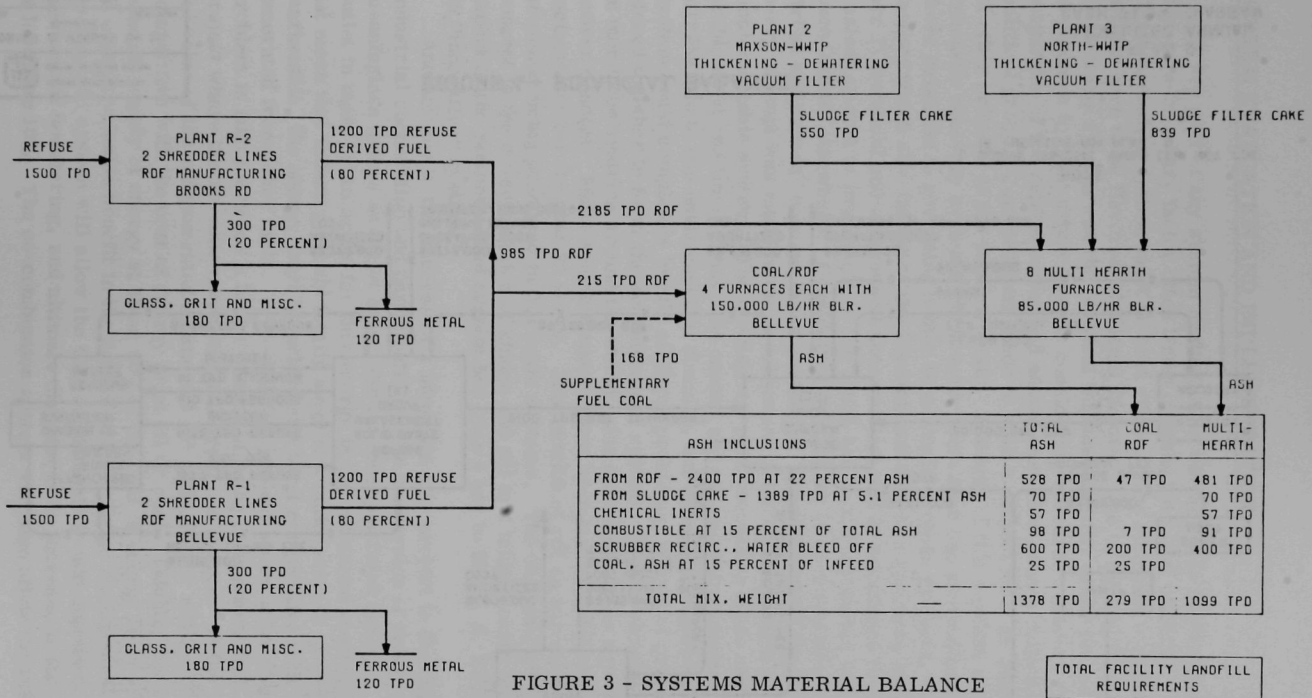
Although the recommended alternative accomplishes conversion of refuse to energy, the mechanism of conversion is dictated by the necessity of destroying the sludge. After an exhaustive survey of the state-of-the-art of combustion processes, the use of multi-hearth incinerators operating in the starved air combustion mode was determined to be most promising. Functioning in this combustion mode followed by a higher temperature after burner, the multi-hearth combustion unit provides complete waste combustion with minimum secondary problems due to air pollution from hydrocarbons and heavy metals. The after burner also provides a convenient area for injection of the pumpable flammable industrial wastes. Energy is recovered through the use of a waste heat boiler in the afterburner section. Other incineration (combustion) processes provide for a more efficient transfer of energy refuse; however, these are incompatible with sludge cake combustion and, accordingly, do not fit into the entire waste management requirements. The excess rdf will be fired in one of the more efficient and reliable systems in use, spreader stoker with waterwall heat recovery.

Sludge cake will be transported from each wastewater treatment plant at a 20 to 25% solids level in transfer trailers. The rdf and sludge cake will be

added to the upper hearths of the multi-hearth units with the sludge on the top (#1) hearth and the rdf on the #4 hearth. Any non-pumpable, flammable industrial waste will be added with the rdf on hearth #4. Temperatures within the 12-hearth furnace will be maintained below 1100°F by controlling the excess air. Gaseous products of incomplete combustion pass upward through the top most hearths drying the sludge. From the top hearth, these gases enter the afterburner chamber where complete combustion at 2400°F takes place. Hot exhaust gases pass through a waste heat boiler and then through a 60-inch headless venturi scrubber prior to exhaust to the atmosphere. Six multi-hearth furnaces are designed at the erc to process the sludge quantities in the initial year of operation (1982). Five units are projected to be used initially, all operating 24 hours per day, 7 days per week. The other furnace is to be used for standby service and future capacity. Each furnace is equipped with an 85,000 lb. per hour, 650 psig waste heat boiler which will supply approximately 440,000 lbs. per hour of steam. To assure sufficient capacity to supply the non-interruptable service and peak demands, three coal/rdf spreader stokers have been designed into the system. These units are each fitted with a 150,000 lb. per hour waterwall boiler or a total capacity of 450,000 lbs. per hour.

The conversion of refuse to energy is primarily with steam as the end product ; however, co-generation of steam and electricity is also a possibility. The steam demand will not be constant from day to day due to variations in industrial processes and other non-controllable factors. To provide a load factor closer to 1.0, steam driven electricity generators can be installed and operated when steam is available, either on weekends, during normal industrial slow down or during other periods of industrial inactivity. The City of Memphis receives its electrical power as part of the Tennessee Valley Authority system. Local generation with equivalent swap-out of power is a viable alternative. Thus, not only can refuse be used in a co-disposal system for environmental management, but also as part of co-generation of energy.

The recommended system is relatively complex and capital intensive. As indicated by the data presented in Figure 3, the co-disposal/co-generation system provides for substantial energy and materials recovery from refuse while utilizing the intrinsic refuse energy to solve other community environmental problems. While not providing for the most efficient conversion of energy nor the maximum materials recovery, the recommended system is the most cost effective in solving the City's three environmental problems with proven technology and systems. Twenty-two separate systems covering the range of land disposal to full energy conversion were evaluated on a cost effective basis using the U. S. EPA methodology. A synopsis of the system cost is shown in Figure 4. In such a co-disposal system, other sources of revenue are available in addition to energy revenues. This aspect is discussed in greater detail in the following paragraphs.



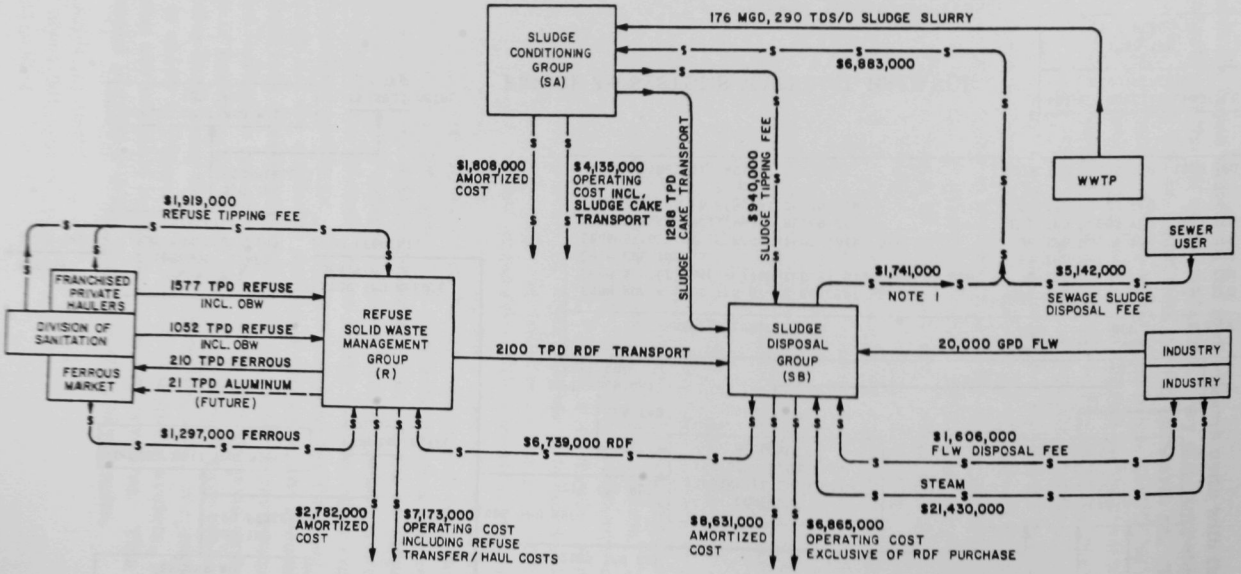
ASH INCLUSIONS	TOTAL ASH	COAL RDF	MULTI-HEARTH
FROM RDF - 2400 TPD AT 22 PERCENT ASH	528 TPD	47 TPD	481 TPD
FROM SLUDGE CAKE - 1389 TPD AT 5.1 PERCENT ASH	70 TPD		70 TPD
CHEMICAL INERTS	57 TPD		57 TPD
COMBUSTIBLE AT 15 PERCENT OF TOTAL ASH	98 TPD	7 TPD	91 TPD
SCRUBBER RECIRC., WATER BLEED OFF	600 TPD	200 TPD	400 TPD
COAL, ASH AT 15 PERCENT OF INFEED	25 TPD	25 TPD	
TOTAL MIX, WEIGHT	1378 TPD	279 TPD	1099 TPD

FIGURE 3 - SYSTEMS MATERIAL BALANCE

TOTAL FACILITY LANDFILL REQUIREMENTS
1738 TPD 15.4 ACRES/YR

MATERIAL FLOW DIAGRAM (1998)

LEONARD S. WEGMAN CO. INC. ENGINEERS MEMPHIS NEW YORK	CITY OF MEMPHIS, TN. SLUDGE/WASTE DISPOSAL AND ENERGY RECOVERY SYSTEM	PROJECT 614
IN ASSOCIATION WITH PICKERING-WOOTEN-SMITH-WEISS, INC. JWS, INC. MEMPHIS STATE UNIVERSITY	SOLID WASTE MANAGEMENT STUDY PLAN B-2	FIGURE X 3.9-



NOTE

1. SLUDGE DISPOSAL GROUP (SB) NET COST IS NEGATIVE FOR PLAN 9

FIGURE 4 - FINANCIAL BALANCE

SLUDGE/WASTE DISPOSAL AND ENERGY RECOVERY SYSTEM

LEONARD S. WEGMAN CO. INC.
ENGINEERS
NEW YORK

IN ASSOCIATION WITH
PICKERING-WOOTEN-SMITH-WEISS, INC.
MEMPHIS STATE UNIVERSITY

PLAN 9
LEVELIZED ANNUAL
CASH FLOW DIAGRAM
FIG. NO. X 3.9-4

SYSTEM EVALUATION AND IMPLEMENTATION

A key part of any system evaluation is whether it is implementable or not. This non-monetary factor often times overshadows cost viability. Truly, to be implemented, a system must be cost viable. The elements composing evaluation matrix have to be flavored with local factors. One of the key factors in the City of Memphis system was the political implementability of the project. This factor is common to most urban centers considering resource recovery systems. The magnitude of the capital outlay and annual cash flow causes concern to any political group. Commitments made to such a project must be made at the expense of other projects in the public domain. Because of this concern and reality, resource recovery systems which solve more than one environmental problem are more palatable to governing bodies than single purpose projects. Two interrelated factors which also influence the implementability of a resource recovery system are 1) technical and economical uncertainties and 2) resistance to change. The easiest system to implement is one which has been constructed and operated successfully elsewhere. Considering the infancy of co-disposal systems, this "tried and proven" concept is not available. Considering all the above factors, every attempt was made in the development of the City's system to provide an implementable system. The co-disposal plan reconciles three environmental problems at one time. Thus, the capital and net operating expenses are not attributable to one environmental concern, but three. Recognizing that no existing system could completely handle all three problems, pieces of proven systems were pieced together to form the recommended system. Refuse shredders are operating in numerous systems around the world. Air classification is a perfected and proven concept. Multi-hearth furnaces have been successfully used for sludge incineration for the past 20 years. Incineration of rdf on spreader stokers is presently being practiced in full scale systems. The only untried part of the concept is the co-combustion of sludge and rdf in a multi-hearth furnace. This aspect of the recommended system has been a key to the U. S. EPA designation as an "innovative" system.

Another key to the success of a Memphis type project is the degree of industrial cooperation and participation. The conversion of refuse to energy is meaningless unless an outlet exists for the thus generated energy. The obvious outlet in most urban areas for this energy is the industrial community. This does not mean that industry should totally support a multi purpose project. To be marketable, the energy must be priced at a level competetive with energy generated from other fuels. A co-disposal project where more than one community problem is addressed has a better chance of being competitive than a single purpose project where the revenue must originate from one source. In the Memphis system, the value of the rdf generated steam was set at $3.78 per million BTU's which is competitive with the cost of energy from oil or coal. Industry not only obtains a constant supply of energy at about 80% of the cost of energy from oil but shares in an equally valuable benefit in the cost of proper sludge management. The co-disposal system will allow the City of Memphis to add digestion, thickening, mechanical dewatering, and ultimate disposal at an increase in the sewer use fee of less than 10%. The co-combustion system will also allow the refuse tipping

fee or dump fee to remain at approximately $2.00 per ton. Industrial involvement is also vital in the flammable waste management aspect of the proposed system. With a disposal outlet available at a nominal cost, generators of such waste which is incompatible with normal sanitary landfill operation must take advantage of this outlet for the environmental problem to be adequately addressed. Cooperative participation on industry's part will be far more successful than enforced participation.

A co-disposal project within an urban area offers the opportunity for other governmental entities to take advantage of the large system at a cost comparable to normal landfill costs. Communities surrounding an urban area face the same disposal problems as the larger community but do not have sufficient volumes of refuse to consider resource recovery. Aside from the non-monetary factors such as materials conservation and alternate energy sources, most small communities or political entities will primarily rely on cost of disposal as the primary decision criteria.

In the City of Memphis system, many factors other than costs were utilized in evaluating the twenty-two different alternatives. All too often in the past, decision makers tended to overlook such factors as environmental effects, net energy consumption (production), capital intensiveness, and employment opportunities. Evaluated on a comparative basis with the other options, the recommended system is not only most cost effective but also most desirable based on non-monetary factors.

Present federal energy and environmental programs in the United States provide capital funds for certain categories of projects. The co-disposal attributes of the Memphis project not only resulted in the innovative classification from the U. S. EPA but also qualified the project for water pollution control funds for sludge management under the Clean Water Act of 1977 as a multi-purpose project. This project is the first so funded in the United States. The co-generation potential of the project has drawn the interest of the U. S. Department of Energy. Between the EPA and DOE, the total federal contribution to the capital cost is anticipated to range between a confirmed 50 million dollars and a potential of 90 million dollars.

The total project is anticipated to be on line in 1982. Portions of the sludge handling (dewatering) system will be operational in 1981 through the use of fast track design, prepurchase of equipment, and multiple construction contracts. When completed, the project will be an outstanding example of environmental management through refuse conversion to energy and material recovery.

MINIMUM ENERGY DEMAND BY INTEGRATED AND OPTIMIZED PLANT DESIGN (MEDIOP)

Dr. Werner Lengyel
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Pentech/Houdaille

1. GENERAL

In the year 1963, 10 years before the first worldwide energy crisis, the first Austrian almost energy autarkic sewage treatment plant went into operation. The plant was designed for activated sludge treatment and anaerobic sludge digestion.

Briefly, the design concept for this type of treatment plant is based on the use of the digestion gas as fuel. The gas, which formerly was simply flared, is now used for fuel to feed both the sludge digester heating furnace and the combustion engines driving either generators or blowers to produce the air for aeration. In order to achieve the best possible efficiency ratio, the thermic energy resulting from the combustion process (exhaust and cooling water) was mainly used to heat the sludge for digestion, thus reducing the gas consumption for the furnace and providing more gas for the engines.

During the past few years, 12 new sewage treatment plants based on MEDIOP for approximately 1.7 million population equivalents have been installed or are under construction.

The recent problems encountered inside the oil producing countries and in dealing with them made everybody energy-minded. Even good concepts were again examined for improvements.

The second-largest sewage treatment plant in Austria improved its energy balance by the installation of a jet directional-mix system for aeration and the additional use of heat pumps for recovery of thermic energy.

The development of improvements to reduce the external energy demand for the sewage treatment process will be shown in the following development of design data for the LINZ plant.

2. ALTERNATIVES FOR AERATION

The sewage treatment plant for LINZ was designed based on the results of model tests for aeration. The design figure which has been used for the discussion of all following alternatives and possibilities is the oxygen demand for the biological treatment:

$$OD = \underline{2{,}125 \text{ kg } O_2/h}$$

All of the following alternatives considered the use of digestion gas (bio-gas) to drive gas-engines to produce the energy required for aeration:

1 a) Surface Aeration (Brush)
1 b) Surface Aeration (Rotor)
2 Fine Bubble Aeration
3 Jet Aeration

The design criteria for the plant and the various aeration systems are shown in Table 1.

3. GENERAL LAYOUT OF THE ENERGY TRANSFER SYSTEMS

The energy transfer system is based on the concept of using the digestion gas (bio-gas) not only for the furnace to heat the sludge, but also as fuel for gas driven motors for aeration blowers and generators.

3.1 SURFACE AERATORS

The bio-gas is used to drive a generator unit. The generated power is transformed to the required voltage for the aeration equipment and the return sludge pump.

The thermic energy of the bio-gas engine is used to heat the sludge by two heat exchangers in-line. The cooling water, after leaving the radiator through the first heat exchanger, flows around the exhaust for further heat recovery. During the return flow, the water cools as it heats the digester feed sludge in the second heat exchanger. This sludge is either excess sludge pumped from the secondary settling tank or circulated sludge from the digester.

The only equipment which can feasibly be operated by the internal power generation are those units which can be combined for switching and control. All other motors must be hooked to the external power system.

3.2 AERATION BY COMPRESSED AIR

3.2.1 Fine Bubble Aeration

In general, the same layout for the bio-gas engine and the heat exchangers as described above, is considered for fine bubble aeration. The only difference is that aeration is carried out by compressed air; and the blower is directly coupled to the bio-gas engine. The efficiency of this aeration system is considerably better than the surface aeration efficiency since the amount of air can be controlled by adjustment of the rotational speed of the engine. The engines can be designed at a lower rating. All electric equipment is hooked to the external power system.

3.2.2 Jet Stream Aeration (Figure 1)

Bio-gas is used to drive both a generator unit and the aeration blower. The generated power is transformed to the required voltage to drive the jet motive pumps, which double as sludge return pumps. This combination has the overall highest efficiency of all alternatives considered. The amount of air can be controlled by adjustment of the rotational speed of the air blower engine; and, the sludge return and jet pumping rate can be controlled by adjusting the pump speed or impeller pitch. Percent return sludge is easily controlled by varying the mixture of return sludge and mixed liquor which is fed by gravity to the pump.

3.3 BASIS OF COMPARISON

It should be noted that the above comparisons are based on demonstrated

clean water transfer efficiencies. There is substantial evidence in the literature (1, 2) that a comparison on the more important mixed liquor or process oxygen basis favors even more strongly the jet stream aeration approach. Though this was not considered as a basis for selection, the reported higher alpha factors for jet stream aeration provide some conservatism in design and generate considerable anticipation as to the ultimate efficiency attainable.

4. THE SEWAGE TREATMENT PLANT LINZ (FIGURE 2)

4.1 ANALYSIS OF THE AERATION SYSTEMS

On the basis of the energy concept, the various systems for the oxygen supply were studied. Finally two systems were taken into consideration:

4.1.1 Aeration Basin With Plug Flow

This type of aeration basin is of rectangular shape with a width-length ratio of 1 : 5 to 1 : 10. The mechanically treated sewage and the return sludge are fed at the front of the basin. Because of the high initial rate of BOD reduction and the considerable oxygen demand of the return sludge, considerable differences in the oxygen concentration exist along the direction of flow. Assuming constant oxygen supply along the basin, there will be an oxygen deficit in the influent side of the basin with oxygen concentration increasing in the direction of the outlet end.

In order to avoid this problem, the inflow of the raw sewage has been changed: a longitudinal distribution channel up to the half-point of the basin length provides better mixing of the raw sewage. However, if the concentration of the sewage varies or if toxic shock loads occur, this channel does not give fully satisfactory results.

Using the fine bubble aeration system, only this type of basin and flow scheme can be used since the bubbles cannot trigger horizontal flow (and thus turbulent mixing) in the basin. Therefore, the influent raw sewage remains generally unmixed.

4.1.2 Aeration Basin With Mixed Flow (Continuous looped reactor)

Using the jet stream aeration which triggers a turbulent horizontal flow of the sewage, it becomes possible to design a different basin with unique operation: the continuous looped reactor or more specifically the jet aeration channel (JAC). The rotating volume of liquid is much larger than the inflow, therefore complete mixing of the activated sludge and the raw influent can be achieved. A much greater buffering capacity is provided against varying inflow concentration and toxic shock loads.

For reasons of soil mechanics, the basin was designed with a water depth of 7.5 meters, which was advantageous to the energy efficiencies of the jet aeration system. The specific oxygen transfer efficiency, considering the whole system (gas-engines, blower, generator, aeration tank, secondary settling tank and return sludge pumps,) can be expected to be:

$$\frac{OC}{N^r} = 2.65 \quad \frac{kg\ O_2}{kWh}$$

Therefore this aeration system was selected for design.

The details of the JAC are shown in Figure 3. Influent enters the JAC at one end of the basin. Effluent exits approximately three fourths of the way around the channel. Longitudinal dispersion and transverse mixing occur along the channel preventing short circuiting yet allowing maximum resistance to shock loading. Return sludge flows by gravity to the pump sump in the center island. A controlled amount of channel mixed liquor is also allowed to flow to the sump as make-up fluid for the jet motive pump. Pumped liquid feeds a main liquid header connected to six directional-mix jet aerators positioned transversely in the channel. The directional mix jet aerators are of a hydrodynamic tear-drop cross section to minimize hydraulic drag. The jets and fluid ratios are specially designed to maximize momentum transfer and oxygen transfer to the channel.

Internal sludge recirculation through the directional mix units accomplishes three functions:

1. Sludge is uniformally returned throughout the JAC reactor, maintaining uniform active solids and D.O. profiles.
2. Return sludge is subject to the most favorable aerobic environment by direct air injection.
3. JAC mixed liquor transfer efficiencies are maximized and can theoretically exceed clean water efficiencies due to the oxygen demand of the sludge inducing reaction enhancement.

4.2 IMPROVED ENERGY CONCEPT

The energy layout was improved as shown on Figure 1:

Bio-gas is used only to feed two gas engines; one of them drives a generator to produce electric power, the other one drives a blower directly. The combined efficiency factor of the two units is in the range of $\eta s = 0.88$. The return sludge is pumped into the same stream as the aeration (jet directional mix: JAC).

The heat exchangers are not only fed by the cooling water. One additional heat exchanger is fed by a heat pump which uses the sewage of the secondary settling tank as the constant temperature medium. Thus the furnace can be eliminated and no external energy will be required.

The digestion gas drives the gas engines with the known capacity of 17.7 GJ/h. The losses of the gas engine amounting to 5.84 GJ/h are to be partly recovered by heat pumps which are operated with 2.2 GJ/h of electric power; 4.0 GJ/h of the electric power is used for the motors.

The heating capacity needed for sludge digestion is 5.66 GJ/h originating from the bio-gas engines and 4.34 GJ/h produced by the heat pumps.

5. CONCLUSION

New sewage treatment plants should not be designed without thorough consideration of energy conservation. Existing plants should be improved where feasible.

Minimum energy demand through integrated and optimized plant design (MEDIOP) is a challenge for the engineer.

Figure 4 shows a diagram representing all characteristic data for MEDIOP up to 1,000,000 population equivalents. All figures relevant for the energy demand of the biological plant, efficiency factors for oxygen transfer and characteristic values for the bio-gas engines, the sludge and the gas production are contained in this graph.

Primarily this graph serves for the sizing of the bio-gas motors and the thermic energy recovery system (Example 1).

The determination of the required thickening of the raw sludge in relation to the energy yield of the digestion gas is also possible (Example 2).

The innovative features incorporated into the LINZ design promise to provide a completely energy self-sufficient treatment plant. The high mixed liquor energy efficiency of the JAC in conjunction with the improved energy recovery concepts offer a real potential for a new level of performance.

Table 1: Design Data of LINZ Sewage Treatment Plant

Description	Unit	Surface	Aeration Fine Bubbles	Jet-Stream
Oxygen demand	$\frac{kg\ O_2}{h}$	2,125	2,125	2,125
$\frac{OC}{N^T}$ Oxygen Tranf. Eff.	$\frac{kg\ O_2}{kWh}$	2.0	2.33	2.65
ηs Eff. Fact. of Syst.	-	0.78	0.98	0.88
Power requ'd f. O_2	kW	1.363	930	911
Misc. Plant	kW	100	100	100
Total Power Req.'d	$\frac{GJ}{h}$	5.15	3.71	3.64

$\frac{OC}{N^T}$ .... Oxygen Transfer Efficiency is the Oxygen demand in relation to all equipment for biological treatment (aeration equipment and return sludge pumps)

ηs .... Efficiency Factor of the Aeration System:

System 1 a + 1 b - Engine + Generator + Transformer + Aerator
System 2 - Engine + Blower
System 3 - Engine + Blower
Engine + Generator + Transformer + Jet Stream Pumps

Example 1

Design Data for Sewage Treatment Plant

Layout	600.000 E.l
Specific Load	40 g BOD_5/E.l x d
OC-load	2,0 kg O_2/kg BOD_5
Efficiency of System	= 1.6
Specific Energy Input	11.500 kJ/kWh
Specific Thermic Recovery	4.600 kJ/kWh

Result: Energy Demand $\frac{30 \cdot 1000}{24}$ = 1250 kW

Thermic Energy Recovery: 135 GJ/d

Example 2 a

Assumption: Suspended solids 48 t/d

Solids concentration 3%

Result: Required Thermic Energy: 220 GJ/d

Example 2 b

Assumption: Total Gas Yield 325 l/kg

Caloric Value 23.000 kJ/m3

Result: Yield of Thermic Energy: 350 GJ/d

Example 2 c

from Example 1: Thermic Energy Recovery 135 GJ/d

How much thickening is required to guarantee gas yield

Result: Summer: 3.8%

Winter: 5.8%

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2. LeCompte, A.R., "A Pasveer Ditch, American Jet Style", Proceedings of the 29th Purdue Industrial Waste Conference, May, 1974.

CRUSHED COAL AND REFUSE DERIVED FUEL AS SUPPLEMENTAL FUEL FOR THE INCINERATION OF SEWAGE SLUDGE

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INTRODUCTION

The City of Indianapolis operates two wastewater treatment plants, each of which provides screening, grit removal, primary sedimentation, secondary activated sludge treatment, and chlorination of wastewater. Plant Number 1 has a secondary treatment capacity of 120 million gallons per day and Plant Number 2 has 56 million gallons per day of secondary treatment capacity.

The primary and waste activated sludges produced at Plant Number 2 are pumped to Plant Number 1 and disposed with the sludges from Plant Number 1. The sludge disposal facilities at the Number 1 plant include: twelve dissolved air flotation units;four 500,000 gallon gravity thickening tanks; 22 rotary drum vacuum filters; and 8 Nichols multiple hearth incinerators.

The following modes of operation are used in the sludge disposal process at Indianapolis. The flow diagram of the process is shown in Figure 1.

1. The waste activated sludge from Plant Number 1 and both primary and waste activated sludges from Plant Number 2 are thickened by dissolved air flotation.
2. The primary sludge from Plant Number 1 is pumped to one of the gravity thickeners and is blended with the ash slurry from the incinerators. The ash and sludge settle together while the overflow is returned to primary treatment.
3. The primary sludge and ash mixture is withdrawn from the gravity thickener and is blended with the floated sludge in one of two gravity thickeners that are used as storage tanks.

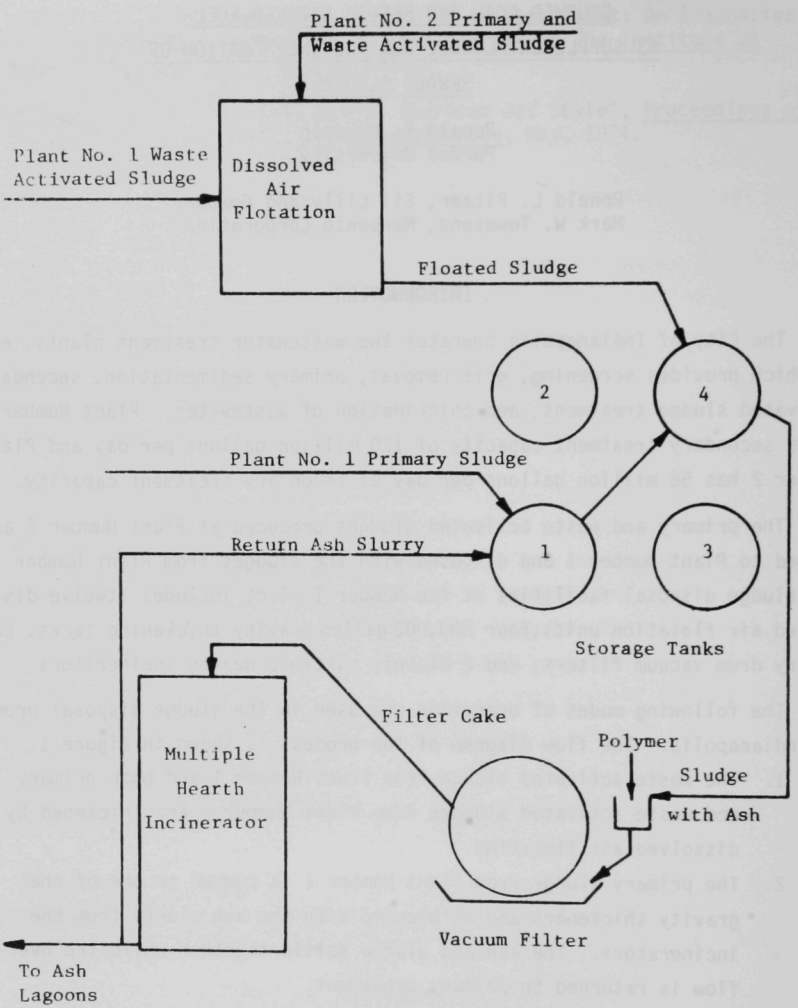

Figure 1

Sludge Disposal Process Diagram

4. The mixture of sludges and ash is withdrawn from the storage tank and pumped to the sludge wet well. From this point, the sludge is pumped to the individual vacuum filters.
5. A cationic polyelectrolyte is added to the sludge prior to vacuum filtration. The ash serves as a conditioning agent to enhance filtration of the sludge and the polymer aids in flocculating the ash and sludge solids.
6. The sludge cake from the vacuum filters is incinerated using fuel oil to supplement combustion. The ash from the incineration of the sludge is quenched with water and the resulting ash slurry is either mixed with primary sludge or is dewatered in lagoons and landfilled.

Even when sludge is dewatered to a moisture content of 65 percent, the heat evolved from the combustion of the sludge solids is not sufficient to evaporate the water entrained in the sludge and auxiliary fuel is necessary.

To obtain reasonable vacuum filtration of sludge at Indianapolis, incinerator ash is added to the sludge at a ratio of approximately 0.89 to 1.20 parts of ash to one part of dry sludge solids. This additional inert material creates even more of an auxiliary fuel demand.

The auxiliary fuel normally used is Number 2 fuel oil, which is rapidly becoming more expensive. In 1978, the combustion of sludge required 5200 gallons of fuel oil per day. This accounted for the largest single fuel expense of the City, even greater than the fuel costs of operating the Indianapolis Police fleet.

OBJECTIVES

The consumption of auxiliary fuel is becoming the most crucial component in the cost of incinerating municipal sewage sludges. Therefore, a material was sought to fulfill two criteria:

* Provide fuel value to the sludge to reduce or eliminate the consumption of auxiliary fuel.
* Serve as a conditioning agent for the sludge to reduce or eliminate the need for recycling the incinerator ash through the system.

More specifically, the objectives were:

A. Laboratory Scale Testing
   1. To determine if coal could perform as a sludge conditioning agent and, if so, the necessary coal particle size.
   2. To determine if RDF alone can act as a sludge conditioning agent.
   3. To determine the ash to coal or RDF to dry sludge solids ratio that will produce a filter cake that will allow autonomous combustion and reduce supplemental fuel costs.

B. Full Scale Testing
   1. To verify that coal or RDF can be used as a sludge conditioning agent in a full scale test.
   2. To determine if autonomous combustion can be achieved in a multiple hearth furnace using coal or RDF is a supplemental fuel.

METHODS

Research into the use of coal or RDF as supplemental fuel for the incineration of sewage sludge was conducted in two phases. The first consisted of laboratory studies using Buchner funnel tests. The rate and the degree of dewatering, along with the manner in which the filter cake released from the filter media were all evaluated in determining filterability. Then bomb calorimetry was used to determine the thermal value of the filter cakes. The second phase evaluated the coal or RDF in full scale tests at the Indianapolis Belmont Sewate Treatment Plant. One Eimco continuous-belt vacuum filter with a filter cloth area of 432 square feet and one Nichols 8-hearth incinerator with a rated capacity of 168 wet tons of filter cake per day were used for these evaluations. Each full scale test lasted approximately 24 hours.

CONCLUSIONS

From these studies on the use of coal or RDF as a conditioning agent for vacuum filtration and as a supplemental fuel for filter cake incineration, the following conclusions are drawn:

I. Laboratory Scale Testing

A. Coal

1. Crushed coal, with particle sizes in the 30 to 60 mesh range, displayed sludge conditioning characteristics similar to sludge ash. Any finer grind, such as conventionaly used in suspension fired boilers, tended to blind the filter cloth.
2. Crushed coal and sludge ash were successful when used together as filter aids for the filtration of sewage sludge. The optimum dose was 0.27 part coal to 0.81 part sludge ash to 1.00 part dry sludge solids. Producing a cake total solids concentration of about 30 percent at a polymer dose of 50 pounds per ton of dry sludge solids. The thermal value of the cake was 4500 Btu's per pound of dry cake solids which would supply as much heat as is presently provided from the fuel oil.

B. Refuse Derived Fuel (RDF)

1. Refuse-derived-fuel alone does not act as a sludge conditioning agent for vacuum filtration.
2. Refuse-derived-fuel and sludge ash can be used together, however, the RDF does not act as a filter aid, hence the full amount of ash is still required for satisfactory filtration.
3. RDF and ash together as conditioning agents yield no improvement in filter cake moisture content. Thus additional water is carried into the incinerator by the RDF, requiring more fuel for incineration.
4. It is best to vacuum filter the sludge and ash alone, and add the RDF to the filter cake.

II. Full Scale Testing

A. Coal

1. Coal and sludge ash as sludge conditioners yielded good cakes from the vacuum filter. Using these components at

coal to ash to dry sludge solids ratios of 0.22:0.54:1.00 up to 0.50:0.54:1.00 filter cakes of 26 to 36 percent total solids were produced at polymer doses of 40 to 60 pounds per ton of dry sludge solids.

2. The coal-conditioned filter cake burned autonomously in a multiple hearth incinerator but this condition was not sustained for more than two hours. However, the sludge cake burned very well at an oil usage rate of about 2 gallons per hour (compared to 30 gallons per hour without coal).
3. A cake thermal value of at least 4000 Btu's per pound of dry cake solids was necessary to obtain a low oil usage rate.
4. The sludge burned uniformily throughout the main burning hearths as opposed to the uneven burning zones imposed by the oil burners.

B. Refuse Derived Fuel (RDF)

1. Ratios of 0.3 to 0.5 parts of RDF to 1.1 part ash to 1.0 part dry sludge solids provided smooth incinerator operation and required minimal amounts of fuel oil.
2. The filter cake-RDF mixture burned autonomously in the multiple hearth furnace for ten consecutive hours, requiring fuel oil only when the filter cake-RDF mixture thermal value dropped below 3200 Btu's per dry pound.
3. The incinerator hearth temperatures remained more uniform with RDF as a supplemental fuel than when oil is used.

Final List of Attendees
Energy Optimization of Water and Wastewater Management for
Municipal and Industrial Applications
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New Orleans, Louisiana
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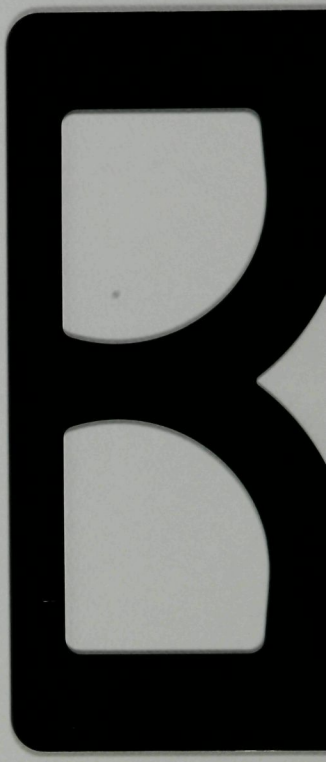
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