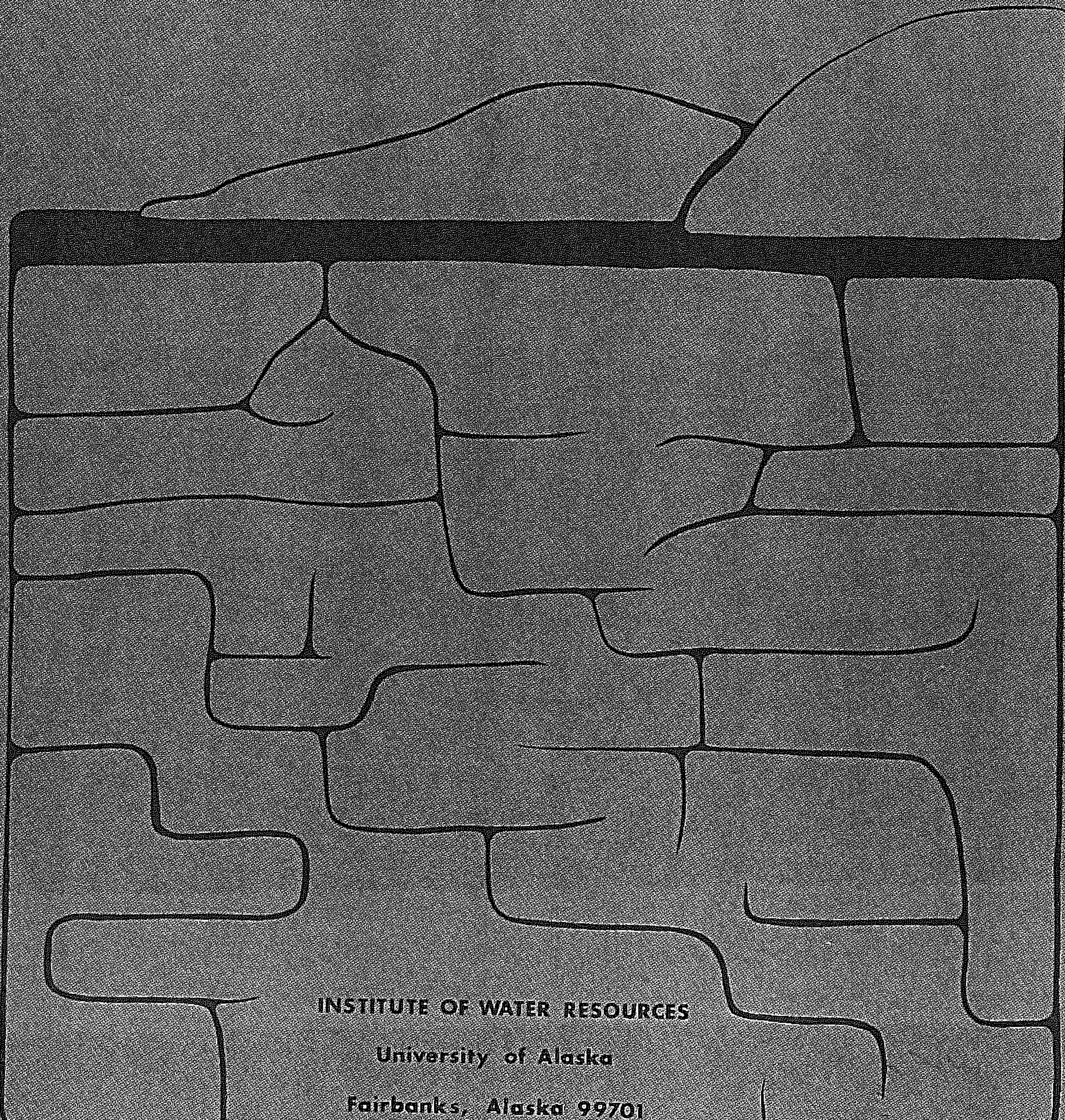
ALASKA WASTEWATER TREATMENT TECHNOLOGY



INSTITUTE OF WATER RESOURCES

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ALASKA WASTEWATER TREATMENT TECHNOLOGY

Completion Report

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Alaska wastewater treatment technology
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A	surface area
ADEC	Alaska Department of Environmental Conservation
AS	activated sludge
AVDP	Alaska Village Demonstration Project
BOD_5	five-day biochemical oxygen demand
c	dissolved oxygen mass concentration
c_s	dissolved oxygen mass concentration at saturation
COD	chemical oxygen demand
CRREL	U. S. Army Cold Regions Research and Engineering Laboratory
d	particle diameter
DO	dissolved oxygen
EAFB	Eielson Air Force Base
ET	evapotranspiration
g	gram
"g"	acceleration due to gravity
gpcd	gallons per capita per day
gpd	gallons per day
$gal/ft^2/day$	gallons per foot squared per day
hp	horsepower
K	thousand
K_1	substrate removal rate coefficient
Kwhr	kilowatt-hour
l	liter
lbm/day	pounds mass per day
lbmpc	pounds mass per capita
M	million
mg/l	thousandths of grams per liter
ml	one thousandth of a liter
MLSS	mixed liquor suspended solids
MLVSS	mixed liquor volatile suspended solids
mm	one thousandth of a meter
n	number of aeration chambers in series
NSF	National Sanitation Foundation
Q	flow rate

LIST OF ABBREVIATIONS (continued)

SOR	surface overflow rate
SS	suspended solids
SVI	sludge volume index
TOC	total organic carbon
U	food to microorganism ratio
USEPA	U. S. Environmental Protection Agency
USPHS	U. S. Public Health Service
V	volume
v_s	settling velocity
VSW	Village Safe Water

INTRODUCTION

This report is intended to be an assessment of wastewater treatment technology in Alaska today. It is not a study of the politics of environmentalists vs. industry, the environmental laws now existing, nor of the design of utilidors in the Arctic. These and other important topics have been dealt with elsewhere. The study is subdivided into three major areas: 1) individual home treatment systems, 2) municipal and military systems, and 3) industrial wastewater treatment. With each category, the existing situation in Alaska is summarized and examples of technology currently being used are presented. Advantages and disadvantages of various methods are discussed with suggestions made for methodologies particularly appropriate to Alaska. Although the bulk of the report is drawn from the "Alaskan experience," results obtained in other parts of the world are cited where appropriate.

An excellent overview of water pollution control in Alaska can be found in a report just released by the State of Alaska (Hammond and Mueller, 1977). This document was motivated by Section 305(b) of the Federal Water Pollution Control Amendments of 1972 (PL 92-500) requiring each state to prepare annual reports on the status of water quality. As emphasized in this report, with three documented exceptions, Alaska's coastal waters have been degraded only minimally. This can be attributed to small populations and limited industry. Far more serious threats are posed by oil spills. Interior Alaska has been essentially unaffected by pollution resulting from wastewater discharges. Exceptions include the Fairbanks area and localized problems related mainly to construction activities or resource exploitation. Significant waste discharges in the coastal areas are mainly confined to the seafood processors at Kodiak and Dutch Harbor and pulp mills at Ward Cove and Silver Bay. Conditions have improved recently at all these locations as a result of updating wastewater treatment facilities. Largely because of the tremendous dilution powers of these tidal waters, it is the position of the state of Alaska that additional treatment is not required at these locations.

Locations of some of the various communities, military facilities, and industries in Alaska producing wastewater are shown on Figure 1. Many of the pipeline camps shown are no longer in existence, although all of the pump stations remain. By far, the largest wastewater dischargers are the three largest municipalities of Anchorage, Fairbanks, and Juneau, and the industries discussed above. Among Alaskan villages, 200 or so have treatment ranging from none (privies and honey buckets) to activated-sludge systems. The harsh weather and remoteness of many of the villages make wastewater treatment very costly. Instead of treatment costs of a few cents per thousand gallons commonly found in the lower 48, one can find costs of a few cents per gallon (Alyeska, 1975). Hence, economics is often the overriding factor in deciding whether a community should have any wastewater treatment at all.

Many individual homes in Alaska utilize septic systems or other individual treatment systems to treat their wastewaters. Because of the poor soils for adsorption fields in much of Alaska, aerobic package plants, composting units, and incinerating toilets are gaining in popularity. With any two identical systems in a given setting, the difference in performance can be like night and day depending on the care given to the system by the homeowners or maintenance personnel.

Because of the long, cold winters common over much of Alaska, special care must be taken to prevent sanitation systems from being destroyed by freezing. In permafrost areas, there is a danger of structural failure caused by thawing of the underlying permafrost. Alter (1969) presents a good discussion of these points. For those systems in which the sewage is treated at or close to ambient temperatures (lagoons, land disposal), provision must be made for the essentially negligible treatment during the winter months. This may include storage of the effluent for subsequent release in the spring. For encapsulated-heated systems, the important phenomena are the same as in warmer climates.

For large-scale systems, the conventional way to classify wastewater treatment processes has been to separate the processes into primary, secondary, and tertiary stages. Although there is not always a clear line of demarcation between one stage and another, the following generalizations may be made.

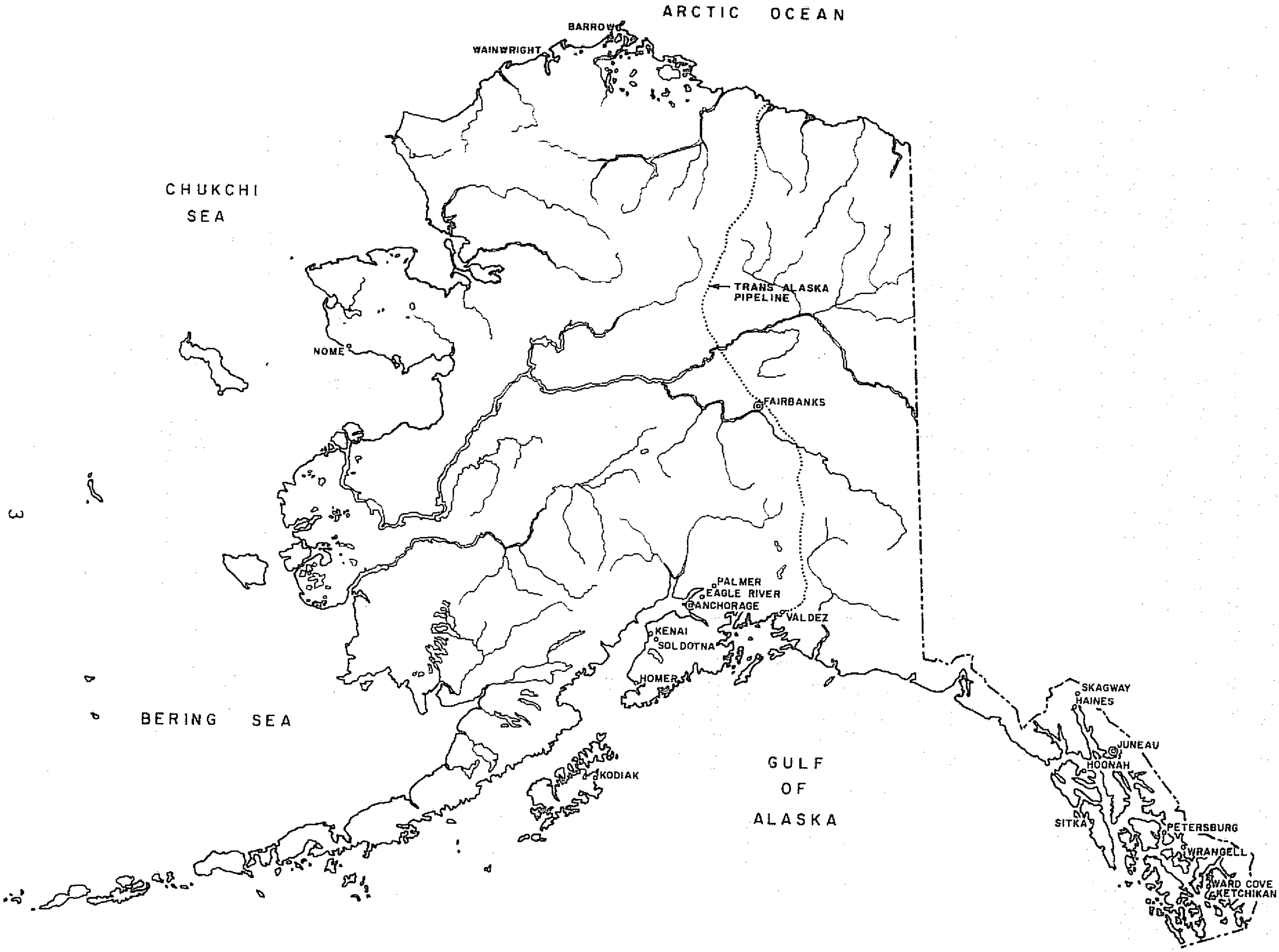


Fig. 1: LOCATIONS OF SOME WASTEWATER TREATMENT FACILITIES IN ALASKA.

Primary treatment (Figure 2) consists of screening, grit removal, flocculation, and settling (Metcalf and Eddy, 1972). In this stage the intent is to clarify wastewater by removing turbidity, sediment, and floating material. Devices that may be used include screens, settling and flotation basins, centrifugal separators, and filters. Sometimes mechanical agitation may be employed to enhance the flocculation process. The resulting increase in particle size improves suspended solids removal by increasing the particle settling or rising rate. Sometimes coagulants are added to enhance flocculation. In some cases, wastewater is passed through fine strainers or other porous materials such as sand to remove suspended solids mechanically. With an efficient primary treatment process, up to 90% of the suspended solids (SS) and 80% of the five-day biochemical oxygen demand (BOD_5) can be removed from the wastewater (Fair and Geyer, 1968). But, average removals are probably more like 50% and 20% (Metcalf and Eddy, 1972). Here biochemical oxygen demand is defined as the amount of oxygen required to oxidize biodegradable matter by biochemical action under specified conditions over a certain time period. The subscript 5 denotes a five-day time period. Suspended solids are those solids removable by filtration through a .45 micron filter.

Secondary treatment is normally considered to be biological treatment whereby soluble organic compounds are converted into bacterial cells and inorganic compounds. Thus, the biological assimilation and degradation processes that occur in nature are carried out in a controlled manner. Basic methods available include a trickling filter, activated sludge (AS), aerated lagoon, spray irrigation, and anaerobic and aerobic digestion processes. Aeration is used to enhance the rate of biological oxidation. Up to 95% of the suspended solids and oxygen demand and 98% of the bacteria can be removed by primary and secondary treatment (Fair and Geyer, 1968) with average removals around 85% (Metcalf and Eddy, 1972). The sludge that is produced in the secondary process is normally concentrated and stabilized before final disposal. Methods of accomplishing this include heat drying, vacuum filtration, and incineration. Land disposal has recently become more popular.

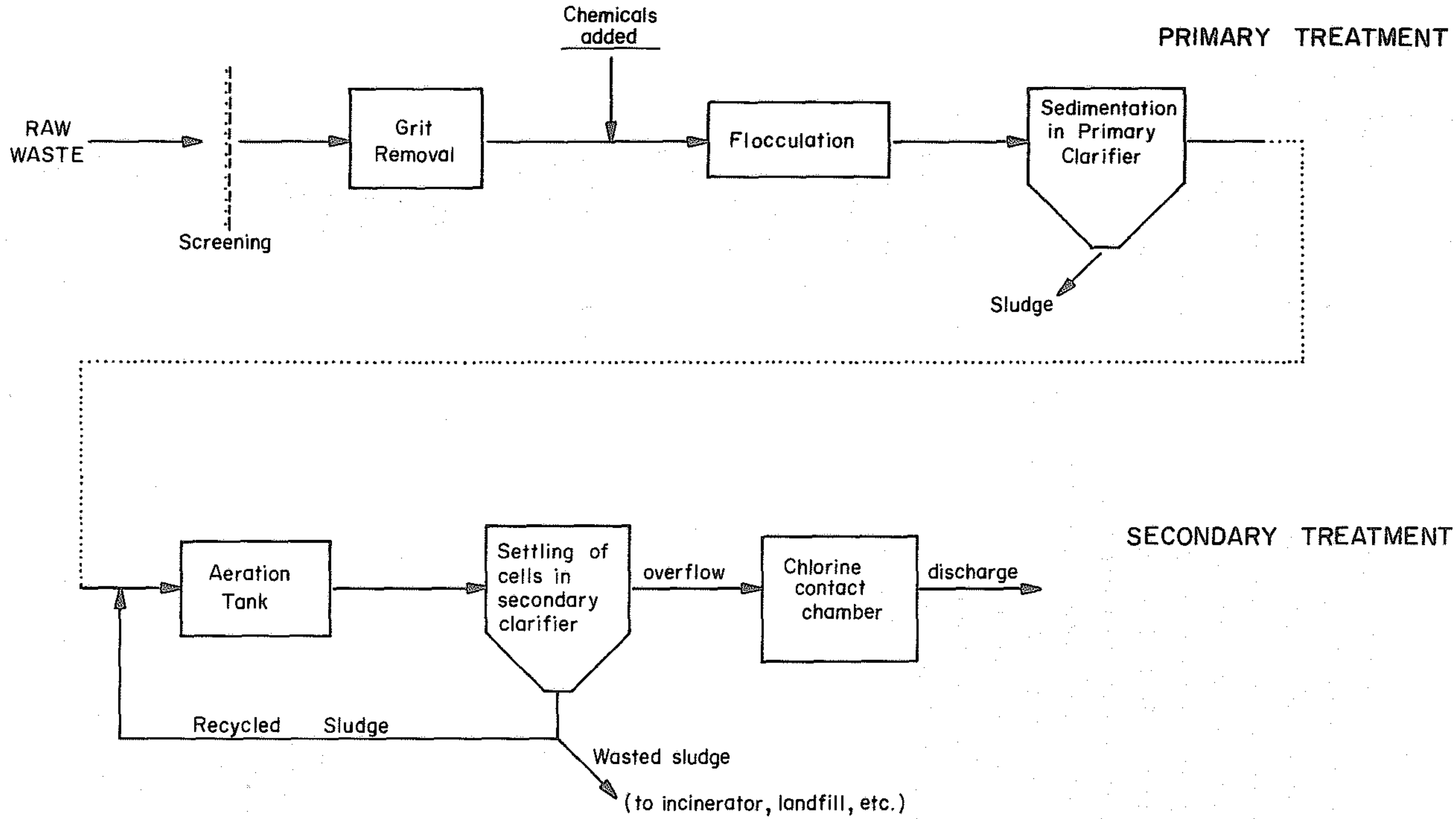


Fig. 2: SEWAGE TREATMENT PLANT

Although it is impossible to describe all the possible treatment methods in these few lines, the disinfection of water and wastewaters should be mentioned. This destruction of water-borne pathogens is accomplished by physical and chemical means. The former include disinfection by heat and light while the latter may involve the use of oxidizing chemicals such as chlorine and surface-active chemicals such as detergents, alkalis, and acids (Fair and Geyer, 1968).

The aforementioned conventional treatment techniques do not result in a product fit for human consumption or many types of industrial use. Since there is a growing need for water reuse, increasing emphasis is being devoted today to the development of so-called tertiary or advanced-treatment techniques (American Water Works Association, Inc., 1971; Fair and Geyer, 1968; Porter, 1968; Brian, 1968; Kirkham, 1968; Walters, 1968; Probstein, 1972; Cookson, 1970). These processes are effective in removing dissolved organic or inorganic solids from the influent stream. Activated carbon has been used successfully for years to treat odorous wastewater (Fair and Geyer, 1968). Activated carbon beds are used to filter raw water and remove taste and odor-causing substances by adsorbing organics and some specific chemical pollutants on the surface of the carbon. After the carbon is exhausted it is regenerated by thermal treatment (Cookson, 1970).

Other advanced-treatment methods--freezing distillation, reverse osmosis, electrodialysis, and ion exchange--may be classified as desalting processes because of their use in treating saline and brackish waters. A typical wastewater from a municipal source shows an increase of 300 mg/l in total dissolved solids above that of the community water supply, and water reuse ultimately demands the removal of dissolved substances (American Water Works Association, Inc., 1971). The first two of these processes involve phase changes, the next two require membranes, and the last two depend on the salt to be removed being ionic in form. In the freezing process (Kirkham, 1968; Probstein, 1972) the saline water is frozen to yield salt-free ice crystals which are separated from the brine and melted to provide potable water. In this discussion, salt can be taken to mean any dissolved organic or inorganic solid and brine is the concentrated liquid. Two freezing techniques currently under investigation are an indirect method in which a volatile liquid (such as freon) is used as the refrigerant and a direct method in which water acts as its own refrigerant via vacuum flashing.

In distillation (Porter, 1968; Probstein, 1972), the saline water is partially vaporized and the pure vapor condensed to form the product. For large-scale seawater conversion, multistage flash distillation is most often used while vapor compression is sometimes used at smaller plants. With the former, the water is heated under pressure and then allowed to expand suddenly or flash into a chamber. In the latter, pure water vapor, which had previously been evaporated, is mechanically compressed to raise its temperature and pressure for use in vaporizing more water. A drawback inherent in all evaporative processes is the problem of carryover of the volatiles. The cost to produce a 500-mg/l product in a 1-Mgpd plant in the lower 48 is on the order of 90¢/K gals for both a multistage flash and conventional freezing process (Probstein, 1972). The cost is essentially independent of feed salinity because the energy requirements for these systems are proportional to overall driving temperature, the change in which (with salinity) is small compared with other temperature losses.

In reverse osmosis (Walters, 1968; Probstein, 1972), the water is passed through a membrane permeable to water but not to salt, leaving a higher salt concentration in the reject stream. The cost of purifying water by reverse osmosis increases with salinity (Probstein, 1972) because the osmotic pressure difference increases with salinity. Reverse osmosis is generally uneconomical at feed salinities in excess of 10K mg/l.

The electrodialysis process (Kirkham, 1968; Probstein, 1972) utilizes two different types of ion selective membranes with one being permeable to positive ions and one to negative ions. The positive and negative ions are driven through these membranes by electric forces leaving behind water which is lower in salinity. These membranes are constructed out of ion exchange materials. Since the cost of treatment increases with the number of salt molecules removed, the cost increases with salinity and becomes uneconomical with salinities in excess of 6K mg/l.

The ion exchange process (American Water Works Association, Inc., 1971), relies on ion exchange resins which exchange ions with those present in the saline water. One method involves exchanging a Na^+ ion for a H^+ ion in the cation resin and a Cl^- ion in the anion resin. Thus NaCl is removed from the water and replaced with H_2O. When "exhausted," these resins have to be regenerated. Conventional ion-exchange methods have proven to be economical only for water having a low dissolved solids content--less than 5K mg/l. Like electrodialysis, the cost goes up with each molecule of salt removed.

INDIVIDUAL HOUSEHOLD SYSTEMS

Background

Often it is most cost effective to treat waste products from homes on an individual basis. These wastes can consist of urine; fecal matter; garbage; and laundry, shower, and dishwashing wastewaters. For a modern American home with running water, about 40% of the average wastewater flow of 60 gallons per capita per day (gpcd) is used for flushing toilets and 40% for showers and washing (Chanlett, 1973). The toilet and garbage disposal wastes contribute about three fourths of the chemical oxygen demand (COD) and SS. A typical composition is shown in Table 1. Note that a sizable fraction of the dissolved solids are fixed (i.e. inorganic). These cannot be removed by biological treatment and present one limitation on renovation and reuse of blackwater or greywater. Unless one is able to afford a relatively expensive advanced treatment system, the increment in inorganic dissolved solids of around 300 mg/l per use limit the number of times domestic wastewaters can be recycled. Even when the recycled water is used purely as carriage water, precipitates will form when the solubility limits of the salts are exceeded. This build up of precipitates will limit even this reuse application.

About one third of the homes in the United States (20 million) rely on individual treatment (Otis, 1977). The types most commonly used are septic systems with aerobic treatment units becoming more popular. Almost all the rest are connected to municipal sewer systems. In contrast, a vast majority of the people in the world practice indiscriminate ground- and water-surface disposal (Chanlett, 1973). Even though a large majority of the households in Alaska are treated via individual or community systems, dumping of excreta on the land or into the sea is still practiced in some arctic communities (Johnson and Dreyer, 1977). For rural residents living at a subsistence level, economic realities sometimes dictate this as the only alternative. For these kinds of rural settings, the wastewater volumes are much less than those quoted for a modern home. For some native villages where the water is individually hauled, the usage may be as low as 2 gpcd and the resulting waste products are much more concentrated than those described in Table 1.

TABLE 1: TYPICAL COMPOSITION OF DOMESTIC SEWAGE[1]

Constituent	Concentration Strong	Medium	Weak
Solids, total	1,200	700	350
Dissolved, total	850	500	250
Fixed	525	300	145
Volatile	325	200	105
Suspended, total	350	200	100
Fixed	75	50	30
Volatile	275	150	70
Settleable solids (ml/liter)	20	10	5
Biochemical oxygen demand, 5-day, 20°C (BOD_5-20°)	300	200	100
Total organic carbon (TOC)	300	200	100
Chemical oxygen demand (COD)	1,000	500	250
Nitrogen (total as N)	85	40	20
Organic	35	15	8
Free ammonia	50	25	12
Nitrites	0	0	0
Nitrates	0	0	0
Phosphorus (total as P)	20	10	6
Organic	5	3	2
Inorganic	15	7	4
Chlorides[2]	100	50	30
Alkalinity (as $CaCO_3$)[2]	200	100	50
Grease	150	100	50

[1]All values except settleable solids are expressed in mg/l.
[2]Values should be increased by amount in carriage water.
SOURCE: Metcalf and Eddy, 1972.

Privies, Honey Buckets, and Cesspools

Diagrams of a privy, honey bucket, and cesspool appear in Figure 3. The latter, sometimes used in Alaska, is simply a sump approximately five feet in diameter sometimes filled with stones. Although intended to serve both as a septic tank and percolation system, it fails as the latter because of the limited surface area for adsorption and subsequent clogging of the pores. The use of privies is more widespread in the state especially for recreational dwellings. In the village of Point Hope, one pit, 4'x4' by 3' deep, could theoretically hold all the human wastes from the school for three years (Johnson and Dreyer, 1977). With at least 1 pint pcd of human excreta being produced and over fifty children in the school, it is obvious that the children utilize honey buckets at home for much of their bathroom activities. We assume here that little degradation of the waste matter occurs but that the liquid drains into the porous soil in the summer. Point Hope has an approximate population of 400. Honey buckets are still widely used in the villages. They can be thought of as privies with removable inserts in order that the waste products may be removed periodically. This results in a transferal of the waste matter from various locations within a village to a central site, typically the dump.

In none of these cases is any appreciable treatment of the waste products occurring. With the below freezing temperatures found in large portions of Alaska for much of the year, no biodegredation is occurring much of the time although some degredation will occur in the summer. Although some research has been done on the decay of pathogens in a cold aquatic environment (Davenport et al., 1976), little is known about such processes for land disposal of wastes. Statements are made that organic matter degrades slowly and pathogens survive for long periods of time in cold environments.

Murphy and Greenwood (1971) state that pathogens survive almost indefinitely in the frozen state. However, the superpermafrost layer is not frozen all the time, and bacteria do decay to some extent during each thaw period. A two-year study (Heinke and Prasad, 1977) on the decay of human wastes in permafrost was recently carried out. They found that little biological activity occurs at -5°C, with small changes

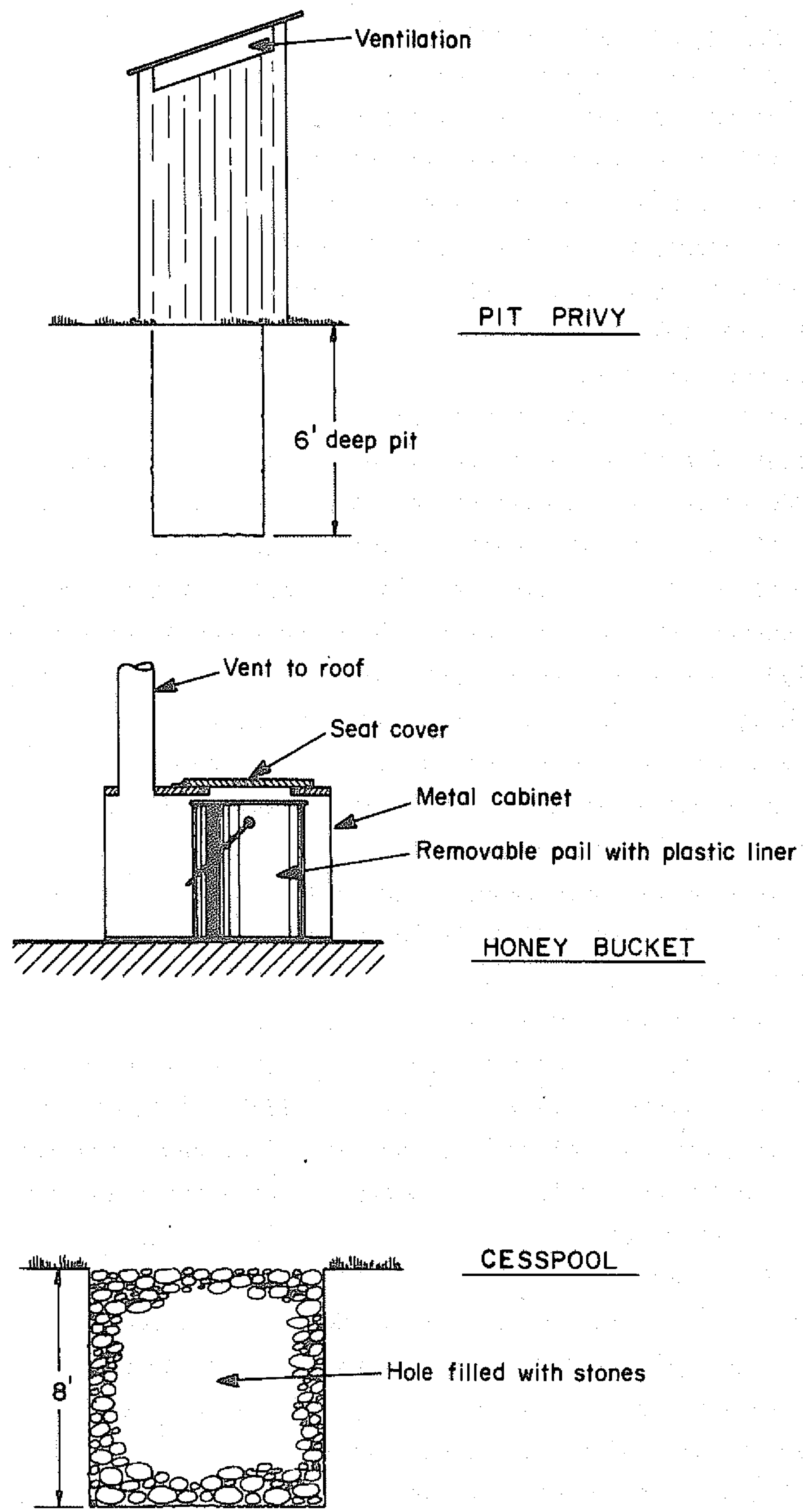


Fig. 3: PRIMITIVE WASTE DISPOSAL FACILITIES

being observed at +5°C. Although bacterial counts were reduced by 99.9% over a two-year period, appreciable numbers (100K per ml) still remained. Several studies (Dyer et al., 1945; Chanlett, 1973) performed in a warmer environment revealed that coliform bacteria move less than 35 feet, even when a waste disposal pit was in direct contact with groundwater. The soils were sand loams with some clay, and the groundwater flow was 3 to 10 feet per day. With no groundwater flow, coliform bacteria were never found more than 5 feet from a pit. Tests to determine the movement of these and other organisms should be performed in permafrost soils. To know the distance at which the water supply would be safe from sewage effluent or solid waste leachate is vital information for planning facilities.

The disinfection of honey bucket and other domestic wastes in the Arctic by the addition of lime is being studied (Mitchell, 1977). Lime has been shown to be an effective disinfectant because of the high pH conditions created. Liming to a pH of 12 has been shown to disinfect at even 1°C (Morrison et al., 1973). Field work needs to be done on the North Slope to see if such an idea is viable for individual homes. If so, the pathogen discharge to the tundra from honey-bucket wastes can be greatly reduced.

There are odor and fly problems associated with both honey buckets and privies. These items are also objectionable from a visual point of view. One must be careful that the location of the privy of the honey bucket disposal site is far removed from the water supply. Sargent et al. (1976) indicate that about half the rural villages in Alaska are in severe danger of contaminating natural waters with untreated sewage. One advantage of honey buckets is the potential for disposing of them far from the village which is very desirable healthwise. Mitchell (1977) notes that the health of the residents of Fort Yukon improved immensely when sound sanitation practices were combined with the installation of an infiltration gallery instead of drawing water directly from the Yukon River. One of these sanitation practices was to ensure that human wastes not be stored near the village. Grainge (1977) reports that the incidence of gastrointestinal disease was highest in northern Canada in the late winter. He attributes this to the spillage and subsequent freezing and thawing of wastewater from honey buckets outside individual homes coupled with the utilization of these ice patches as play areas by children.

A real problem with the use of bucket toilets with plastic bags is the storage of the full bags until they are picked up. Storage inside the house has its obvious aesthetic drawbacks. Outside storage can lead to bags freezing to the ground or being torn apart by the weather or animals. One idea used successfully in Greenland is to store the small toilet bags in large disposal garbage bags (Grainge et al., 1971). Especially in permafrost areas, there are logistical problems involved in removal of the plastic bags or honey buckets. If motorized vehicles are used, gravel driveways are essential to avoid destruction of the vegetation with the resultant erosion problems. But, these problems are secondary to the sanitation problems created by not removing the honey buckets.

Septic Tank Systems

The most popular individual household treatment system used in the United States is the septic tank and absorption field combination. The former is typically constructed out of concrete, steel, or fiberglass. The minimum allowable volume for Alaska is 1K gallons. It is highly desirable to have cleaning ports accessible for maintenance as shown in Figure 4. Note the partitions near the outlet to prevent scum accumulation from clogging it. Although some anaerobic degradation takes place in the septic tank, its primary function is to allow the suspended solids to be removed to avoid rapidly clogging the pores in the absorption field.

Most of the biological stabilization takes place in the absorption field which is commonly a seepage pit in the Fairbanks area and a leaching field in the lower 48. The latter typically consists of looped or lateral trenches around two feet wide and at least 18 inches deep (Figure 5). Uniform distribution of wastewater is achieved via drain tile or perforated pipe in an envelope of gravel. Organics are decomposed in the aerobic-facultative environment of the bed while water seeps downward. For a four-bedroom home, the percolation area may range from 300

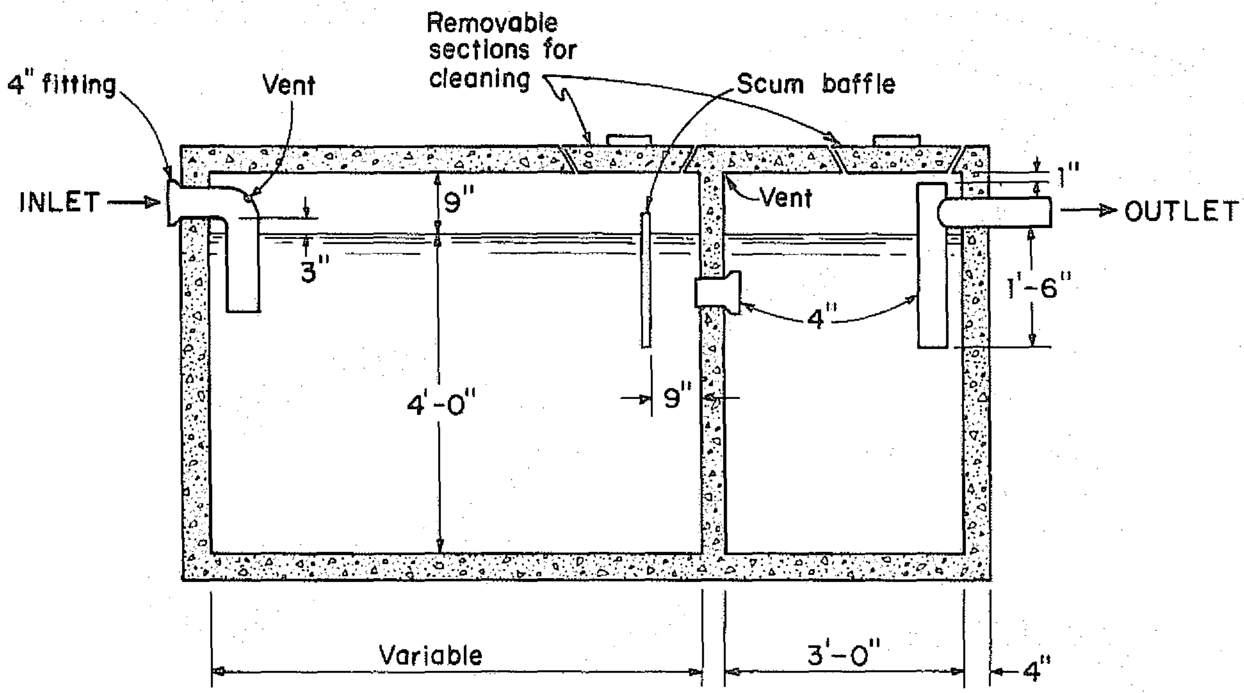
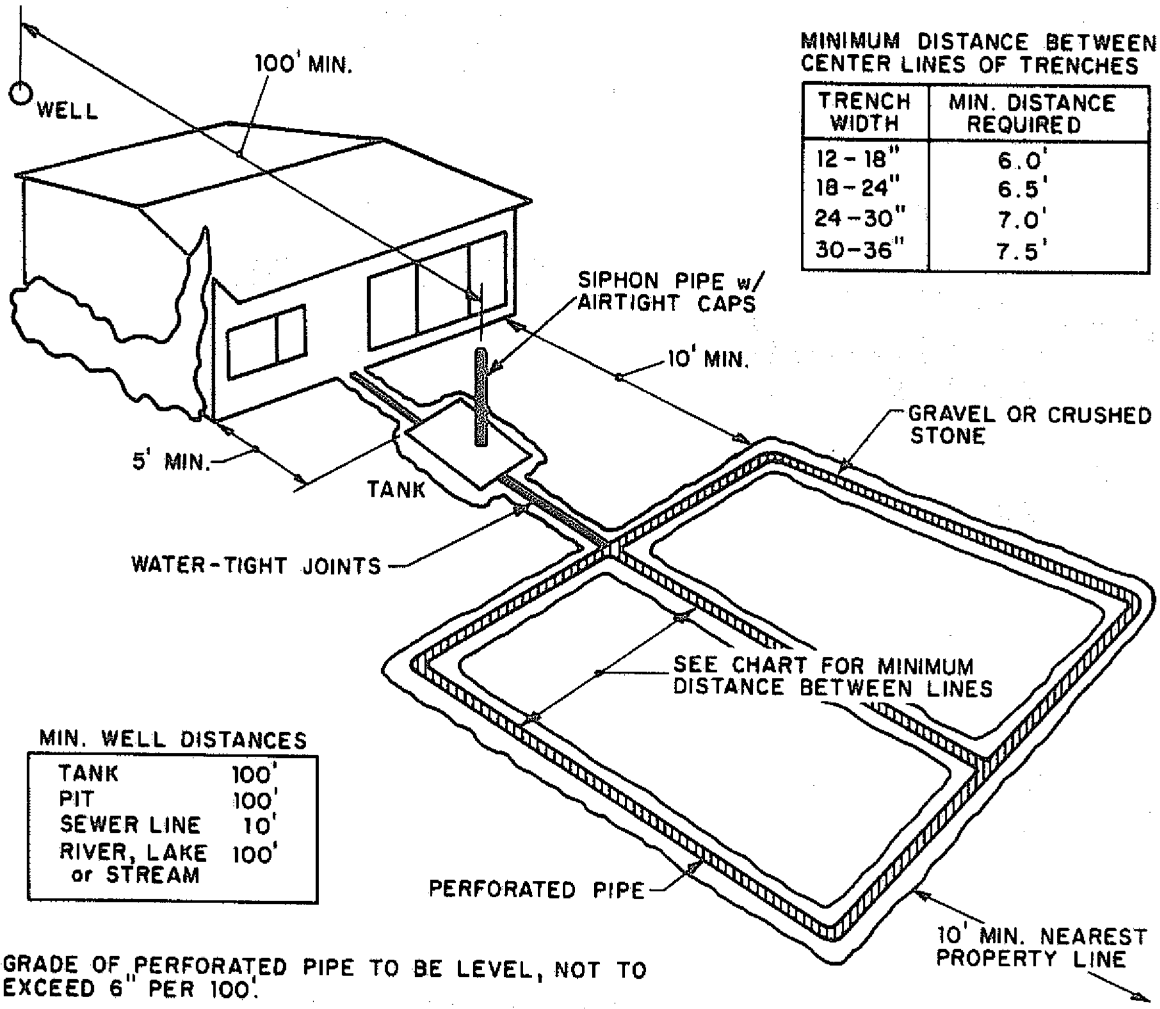


Fig. 4: TYPICAL SEPTIC TANK
(Metcalf and Eddy, 1972)



MINIMUM DISTANCE BETWEEN CENTER LINES OF TRENCHES

TRENCH WIDTH	MIN. DISTANCE REQUIRED
12 - 18"	6.0'
18 - 24"	6.5'
24 - 30"	7.0'
30 - 36"	7.5'

MIN. WELL DISTANCES

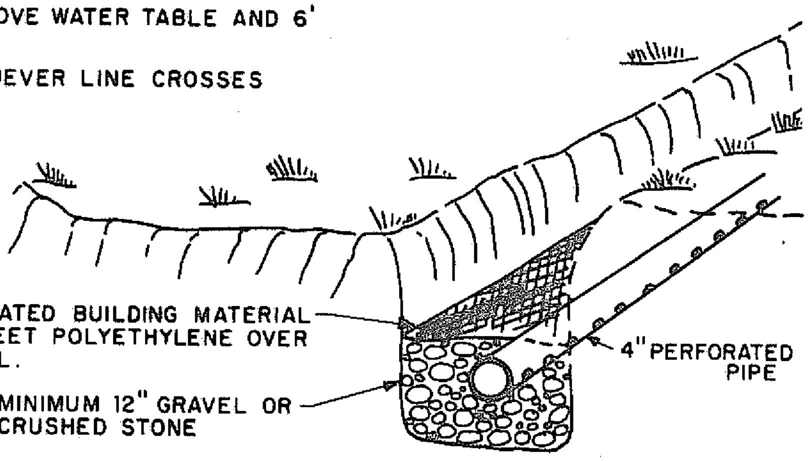
TANK	100'
PIT	100'
SEWER LINE	10'
RIVER, LAKE or STREAM	100'

GRADE OF PERFORATED PIPE TO BE LEVEL, NOT TO EXCEED 6" PER 100'.

INLET AND OUTLET OF SEPTIC TANK WATER TIGHT.

DRAINFIELD 4' MINIMUM ABOVE WATER TABLE AND 6' MINIMUM ABOVE BEDROCK.

CAST IRON REQUIRED WHENEVER LINE CROSSES UNDER DRIVEWAY.



DETAIL - ABSORPTION TRENCH

Fig. 5: ALASKA DEPT. OF ENVIRONMENTAL CONSERVATION REQUIREMENTS FOR TYPICAL DRAINFIELD ON LEVEL TERRAIN. (Alaska Dept. of Environmental Conservation, 1975)

to 1300 square feet depending on the soil permeability (Hammer, 1975). Application rates range from about .5 to 1.5 gal/ft^2/day (Metcalf and Eddy, 1972). For a family of five producing 100 gpcd of sewage, 500 square feet are required at an application rate of 1 gal/ft^2/day.

Sometimes wide trenches are used instead of narrow trenches but these suffer from a lack of sidewall area which provides a better infiltrative surface than bottom area. Either of these will fail if the surface area is rendered impermeable by scraping, driving heavy equipment over the area, etc. If this is not the case and the ground temperature is not too low (less than 10°C), both will work in the biologically active zone via straining, sedimentation, adsorption, and decay. Sand mounds can also be constructed to serve as percolation zones by spreading a three-foot deep bed on the surface. The nutrient uptake capacity of an infiltration zone can be enhanced by allowing some of the dissolved solids to be incorporated into vegetation via evapotranspiration. This assumes this effect is not counteracted by the soils being clogged by roots. For a surface covered with grass, the capacity in warm climates may be 10^6 gal/acre/year (McGauhey, 1975).

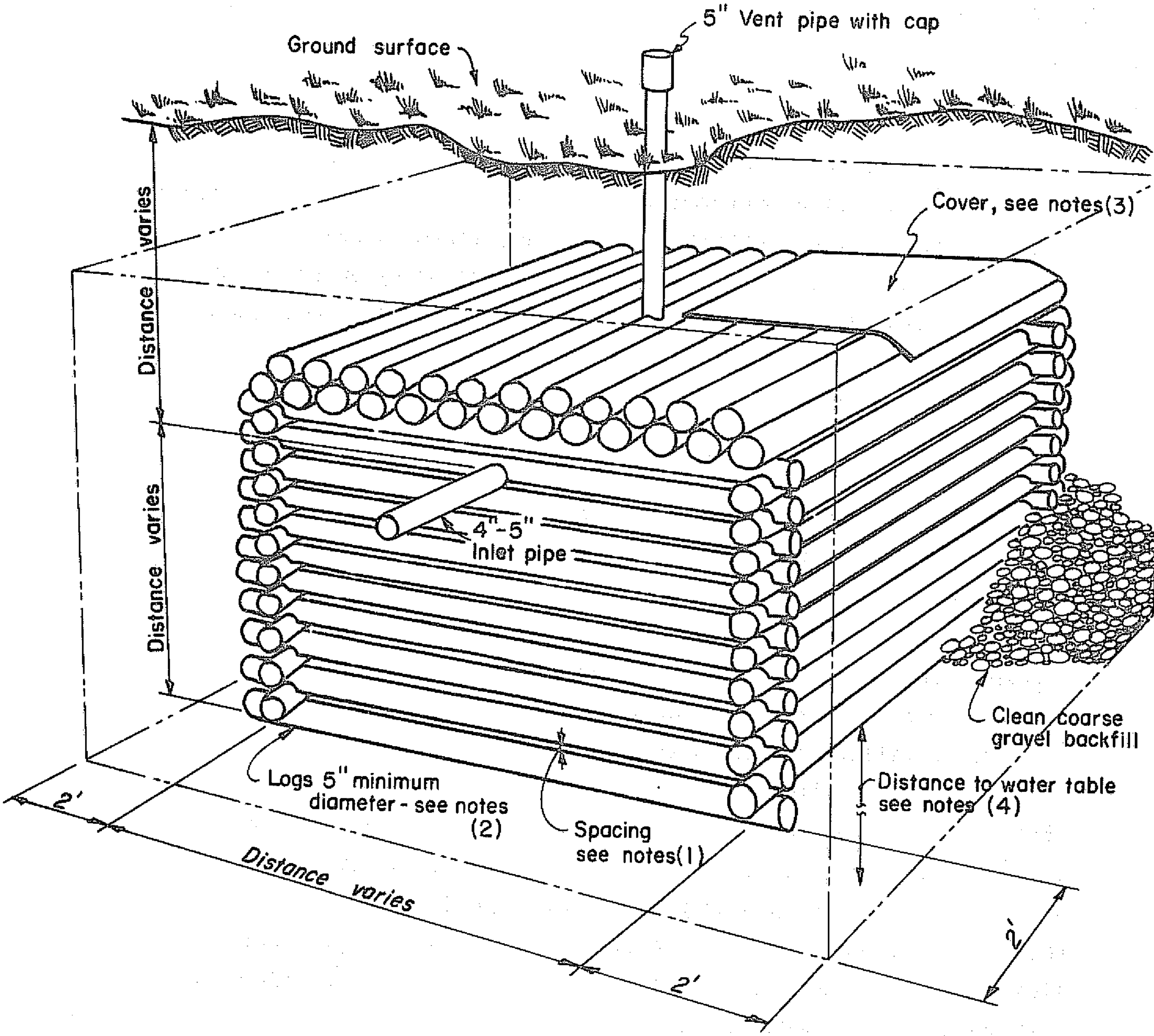
Septic tank systems fail most often not because the tank fails but because the pores of the soil clog. Two reasons for this phenomenon are 1) sludge and scum carryover and 2) the constant application of liquid to the soil. The former can be prevented by annual inspections and by removing scum and sludge when the latter is thicker than 1.5 feet. Some sludge should be permitted to remain as it is needed to serve as a seed for the fresh sewage. The second cause of clogging can be prevented by periodically allowing the soil to drain and the pore spaces to fill with air. Alternatively, hydrogen peroxide can be periodically injected to oxidize the clogging material (Otis et al., 1977). This is necessary because slimes and precipitates will form if anaerobic conditions persist in the soil. Even if the soil were continuously inundated with pure water, the same clogging would soon occur when the aerobic bacteria initially present in the soil became deprived of oxygen. Specific biological clogging mechanisms include growth of organic slimes because of anaerobic bacteria and precipitation of ferric sulfide (McGauhey, 1975). One study (Laak, 1970) found 90% of the clogging material was bacterial cells. Since one cannot dissolve enough oxygen in water to maintain aerobic conditions in inundated soils, aerated septic tanks are not the answer. One other recommended practice to prevent clogging is to avoid abrupt changes in particle size between the trench fill material and the surface of the soil.

Since septic tank systems in Alaska fail for many of the same reasons as in the lower 48, let us review the results of a study conducted in Fairfax County, Virginia (Clayton, 1975). After many system failures in the early 1950s, a new sanitation ordinance was adopted including:

1. the use of percolation tables for sizing drainfields and seepage pit systems
2. the prohibition of construction in areas with high water tables
3. the requirement of two inspections of all systems.

Since the adoption of this ordinance, the health department found that systems properly designed and installed lasted 20 years. In fact, 94% of the 23 systems installed in 1952 were still functioning in 1972. Amendments adopted in 1972 permit the installation of drainfield laterals on 6-foot centers to conserve absorption area and require two separate absorption systems to allow one to be rested while the other is used. By helping to prevent clogging, it is thought that this requirement will allow system lifetimes of 50 years.

The seepage pit is the most commonly used absorption system in Alaska (Figure 6). It is typically built out of logs or rough-cut lumber. Because the upper 6 feet or so of the ground is frozen or very cold for much of the year in Alaska, a surface percolation system is not practical. Hence, the deeper pits are constructed with absorption occurring beneath the biologically active zone. Suggestions for the operation of septic tank-seepage pit systems and regulations for private waste disposal systems from the Alaska Department of Environmental Conservation appear in the appendix. According to Bateman (1977), surfacing sewage is the primary complaint about septic tank systems in the Fairbanks area. The septic tanks themselves, built typically out of steel or fiberglass, appear to hold up satisfactorily.



NOTES:

1. Log spacing: Maximum 3", Minimum 1"
2. Use green logs throughout. Nail log joints.
3. Cover with 2 layers of logs and cover with impervious material such as sheet plastic or sheet polyethylene.
4. Bottom of pit 4' from water table - 6' from bedrock.

Fig. 6 SEEPAGE PIT
(Alaska Dept. of Environmental Conservation, 1975)

Problems are encountered in the vicinity of the city of North Pole due to the high-water table and permafrost and in the foothills around Fairbanks because of a thin overburden. Where there is gravelly ground in the Fairbanks area, systems appear to be performing adequately with annual pumping. But there has not been a recent study on system survival in Fairbanks. The average survival time of septic tank seepage pit systems in the Anchorage area was ten years in 1961 (Hickey and Duncan, 1966). The same study indicated the average survival time in the Fairbanks area to be around six years. In each case, the main source of trouble was in the absorption systems. It was also found that the homes experiencing no failures had greater septic tank and absorption field capacity per person than those where the system failed. But, the differences were small.

It is interesting to note that 17% of the failures reported in the Fairbanks area before 1960 were caused by collapse of the seepage pits. This was prior to the 1960 Federal Housing Authority requirement that the pits be filled or lined with coarse stone. In observations on 136 seepage pits in California (Robert A. Taft Sanitary Engineering Center, 1963), it was found that 18% of the pits not backfilled with gravel collapsed while only 1% of the backfilled pits collapsed. An alternate pit design suggested by Hickey and Duncan (1966) is to dig a trench of the same depth and sidewall area thus eliminating a large amount of almost useless floor area. This trench could be around 1 foot wide. This would require much less stone.

Laboratory studies conducted at Washington State University (1969) found that septic tanks operating at low temperatures (1°C to 15°C) had a BOD_5 removal efficiency of about 50%. Three quarters of this was attributable to biological activity and one quarter to settling. The Hickey and Duncan study concludes that the heat input into the septic tank from the home wastewater was a crucial factor in maintaining the system at operable temperatures. So, part of the operational costs are being provided by waste heat from the home.

A recent study (Neale et al., 1974) indicates the average lifetime of septic systems in the Anchorage area to be less than five years. Here, as in the above 1966 study, tank failures, broken pipes, and structural failures were omitted with only those failures attributable to nonabsorption in the seepage pit consided. The area chosen for this survey was known to have poorer soils than that used for the 1966 survey. Two other factors contributing to the high failure rates were poor-quality backfill material and undersized systems. Since this report, the Borough of Anchorage has tightened its regulations to restrict backfill material to gravel between ½" and 2½" in diameter and to require inspections of all on-site systems. The latter allows the inspector to verify that the excavation is large enough and that the soils are sufficiently permeable. A minimum lot size of 1¼ acres is also required. These regulations are similar to those adopted in Fairfax County, Virginia.

The annual cost for a septic tank-seepage pit system in the Anchorage area was estimated to be about $1050 (Neale et al., 1974). This assumes pit replacement every five years, a tank life of 25 years, 7% interest, no yearly maintenance, and an initial cost of $6,000.

In Fairbanks, Ron Bless of ABC Service and Lamar Wood of Northern Precast (Wells, 1977) have pointed out that homeowners have sometimes constructed marginal absorption systems to minimize capital costs. Such items as wooden boxes and junked automobile bodies have been used as seepage pits. Bud Hilton of Bud Hilton Pumping Company (Wells, 1977) has recommended flushing warm water through the system during periods of low water use to maintain reasonable temperatures in the septic tank. To conserve both energy and groundwater, it would be preferable to insulate the septic tank initially. Joiner (1977) found that at least one Fairbanks resident had not pumped out his septic tank for about 10 years without noticing any ponding. Under these conditions, however, clogging would occur in all but the most porous soils.

Aerobic Package Plants

It is becoming increasingly apparent in the United States today that it is often cost-effective in nonurban applications to utilize individual home treatment systems instead of a centralized treatment plant. Recent studies (Otis, 1977) have shown that 65% to 75% of the total annual cost for municipal wastewater collection and treatment is for amortization and maintenance of the collection system. For those situations where the combination of soil type, lot size, topography, climate, and geology are suitable, the individual homeowner can resort to septic systems. Otherwise, there are a variety of options available in which modern modular systems can be utilized. These range from self-contained toilet units such as chemical and incinerating toilets to small-scale aerobic treatment plants of the activated sludge type. In deciding whether or not such systems are suitable for Alaska, one must be very careful about extrapolating data obtained in the lower 48, especially under laboratory conditions.

In those soils having percolation rates corresponding to times greater than 15 minutes per inch, the Borough of Anchorage will not allow a septic tank system to be used regardless of lot size (Neale et al., 1974). Assuming municipal sewage hookups are not available, the borough has allowed the use of National Sanitation Foundation Testing Laboratory (NSF) approved aerobic treatment plants instead of a septic tank (Strickland, 1977). Furthermore, an earlier borough study (Neale et al., 1974) indicated that such a system in Anchorage would cost the homeowner less than hooking up to the municipal sewer. Air is introduced into such systems either by a draft created by an impellor located at the base of an air tube, or by means of a blower-diffuser combination. This air allows the aerobic bacteria to degrade the waste materials much more efficiently than the anaerobic bacteria could in a septic system. This difference in efficiencies becomes especially pronounced as the temperature drops (Eckenfelder and Englande, 1970). Providing an adequate settling compartment is provided, the resultant clarified effluent will be of much higher quality than that produced by a septic tank. Hence, for equivalent soils, clogging problems should be reduced greatly.

The NSF has conducted evaluations of many individual household aerated wastewater treatment plants. As of 1974, there were twelve models listed by NSF as meeting the standards for Class I and II treatment plants (Neale et al., 1974). The former requires less than 20 mg/l BOD_5 and 40 mg/l SS in the effluent. These standards are slightly higher than those for Class II because the Class I units have a filter to help remove solids whereas the Class II units have only sedimentation chambers.

To illustrate the problems that can arise in actual field usage, consider the aerobic package plant manufactured by the Multi-Flo Corporation in Dayton, Ohio (Fig. 7). It is listed by the NSF as meeting Class I standards. Table 2 presents a summary of the performance data. This was also verified by tests conducted at Alyeska, Alaska (Neale et al., 1974). However, as the subsequent discussion will show, good performance characteristics obtained under carefully controlled conditions in an NSF laboratory do not guarantee satisfactory performance in the field. Similar conclusions were drawn by Smith and Given (1976) for larger extended-aeration plants.

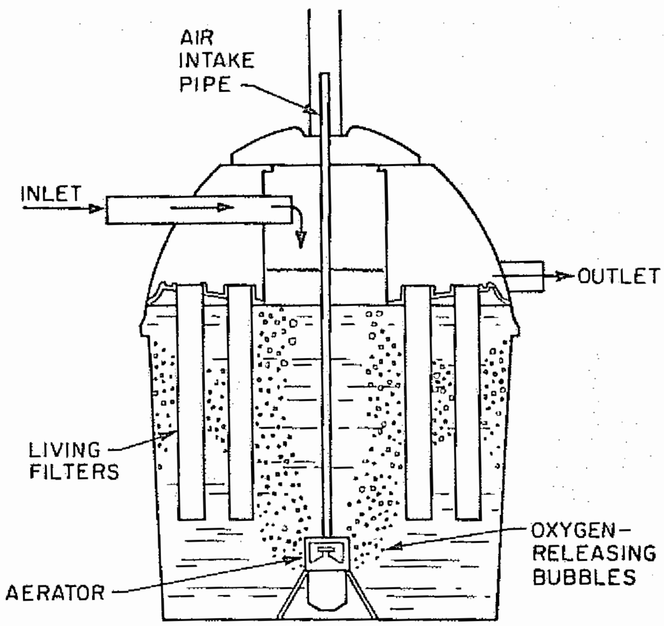


FIG. 7: MULTI-FLO AEROBIC UNIT
(Multi-Flo Inc., Dayton, Ohio)

TABLE 2: SUMMARY OF PERFORMANCE DATA FOR MULTI-FLO UNIT: 16 JUL 73 - 8 FEB 74

		Median[1]	Minimum	Maximum	Interquartile Range[2]
Dissolved Oxygen mg/l	Mixed liquor	5.2	1.0	8.0	5.1- 6.1
12 noon	Effluent	1.4	0.5	4.6	1.0- 2.0
Temperature °C	Influent	19	11	23	13 - 20
12 noon	Mixed liquor	25	12	35	15 - 26
	Effluent	25	9	29	12 - 26
pH	Influent	7.3	7.1	7.6	7.2- 7.3
	Mixed liquor	7.1	6.6	7.7	7.0- 7.3
	Effluent	7.4	6.9	7.9	7.3- 7.5
Biochemical Oxygen Demand	Influent	168	92	739	142 - 205
(5 day) mg/l	Effluent	2	1	19	1 - 4
Suspended Solids (mg/l)	Influent	203	68	1145	146 - 294
	Mixed liquor	4160	25	9085	2160 -5180
	Effluent	5	1	36	3 - 7
Volatile Suspended Solids	Influent	74	34	95	67 - 81
(%)	Mixed liquor	69	36	99	66 - 75
	Effluent	60	0	100	44 - 71
Settleable Solids ml/30 min	Mixed liquor	310	100	890	110 - 450

[1]50% of the values are equal to or less than this value
[2]The range of variability about the median which is sufficient to contain 50% of the observations; it lies between the upper and lower 25% of the observations.
SOURCE: Multi-Flow Inc., Dayton, Ohio, 1974

Air is introduced into the aeration chamber through the air intake pipe with the driving force the suction action created by the impellor at the base of the intake pipe. The impellor also acts to keep the contents of the aeration cell completely mixed. The mixed liquor is forced to flow through cylindrical polyester filters before leaving the unit to drain into the surrounding soils. Even with the system working as planned, the filters must be cleaned periodically to remove the cellular and other suspended matter which has accumulated on the walls of the filter. The unit is constructed out of fiberglass and has a volume of 500 gallons.

Experience gained in the last two years in Anchorage indicates that this Multi-Flo unit is not performing as planned (Strickland, 1977; Feldman, 1977). According to Strickland, the manufacturer claimed the unit would perform adequately in the Anchorage area. Instead, out of roughly twenty units installed in the past two years, none has performed well. It appears that slime and other solids from the mixed liquor rapidly clog the filters such that their permeability is drastically reduced. Since sewage is still flowing into the system, the resulting pressure buildup forces the filters through the mounting holes and into the upper compartment. This happens even with circular wire reinforcements positioned inside the filters. The liquor then short-circuits into the upper compartment thus defeating the purpose of the filters. Eventually hydrostatic forces cause the mixed liquor to flow out the effluent pipe and into the drainfield. Since these systems are used in location with poor soils, the resultant carryover of solids will quickly clog the drainfield.

Even though Multi-Flo claims the motors will last five years, almost every owner has replaced his motor at least once. One owner had to replace seven motors in one year. At $250 for a new motor, this is an expensive proposition as the guarantee expires after two years. Each time the motor is replaced, so are the filter socks at $150 per set. To this $400 is added about $50 for labor. The latter must be replaced because they become plugged with waxes and greases which cannot be washed out with detergents. Instead, acetone must be used. Meanwhile, the manufacturer has claimed the socks will last ten years. According to Feldman (1977), there have been three maintenance contractors for

Multi-Flo in the past two years. As discussed in Neale et al. (1974), it is essential to have a good maintenance contractor and quarterly inspections for aerobic treatment plants. Periodic solids removal is essential and inflow must be stopped if the unit malfunctions. It is clear that a rigorous adherence to a maintenance schedule is essential for the Multi-Flo units in Alaska.

In all fairness to the basic Multi-Flo design appearing in Figure 7, it must be stated that this design has been altered for the Anchorage applications. A cylindrical insert 4 feet long has been added to the upper compartment so the tank will be below the frost line and still have the air intake above ground. At the same time, the small diameter air intake tube was not enlarged or lengthened. Because of viscous losses and a greater spacing between the upper end of the intake tube and the air inlet, not as much air will be transferred into the aeration chamber per unit time. This may tend to make the mixed liquor go septic which would greatly reduce the rate at which organic substrate material is converted into end products and cellular material. But, preliminary calculations indicate these viscous losses to be unimportant. The anaerobic slimes could be largely responsible for clogging the socks. However, since no detailed measurements of the effluent or of the mixed liquor have been taken, these conclusions must be regarded as speculative of the present time. According to Strickland (1977), the temperature of the mixed liquor was measured to be 20°C. Hence, the cold is not directly responsible for system failure. But, it can be the indirect causal factor in terms of requiring a modification of the system design in order to keep the reactor buried below the maximum depth of frost penetration.

There are two other reasons why the Multi-Flo unit is not performing well. First, a rubber coupling on the outflow pipe that hooks onto the drainfield inlet occasionally becomes disconnected. This will result in a localized surge of effluent and subsequent soil clogging. Secondly, along with the idea of adequate maintenance is that of homeowner care on day-to-day operation. He should avoid dumping grease into the sink and flushing solid objects such as string down the toilet that may tend to clog the system. The potential problems arising from such practices should be pointed out carefully to the homeowner before he purchases a package unit.

According to a study conducted by the Alaska Department of Environmental Conservation (ADEC) (1975), of the three 500 gpd Multi-Flo units in operation in Alaska at that time, none was operating in a desirable manner. The first had never had its aerator turned on; a second did not have the socks fitted tightly so mixed liquor passed directly through to the effluent side; and the third was grossly underloaded during the week while overloaded on weekends. Effluent solids were high for both the former units. For the two units operating aerobically, BOD_5 removals of 90% were estimated if (a) influent BOD_5 values of 300 mg/l were assumed and (b) filtered effluent values were used. But, in actuality, at least two of the three units were not achieving good BOD_5 removal. However, as in the case of those installed recently in Anchorage, the available evidence does not justify placing the blame for this on the manufacturer. Improper installation and maintenance is at least partially, if not completely, the reason for poor performance.

An aerobic package plant that has been successfully used in the last two years in Anchorage is the Jet Aeration unit shown in Figure 8.

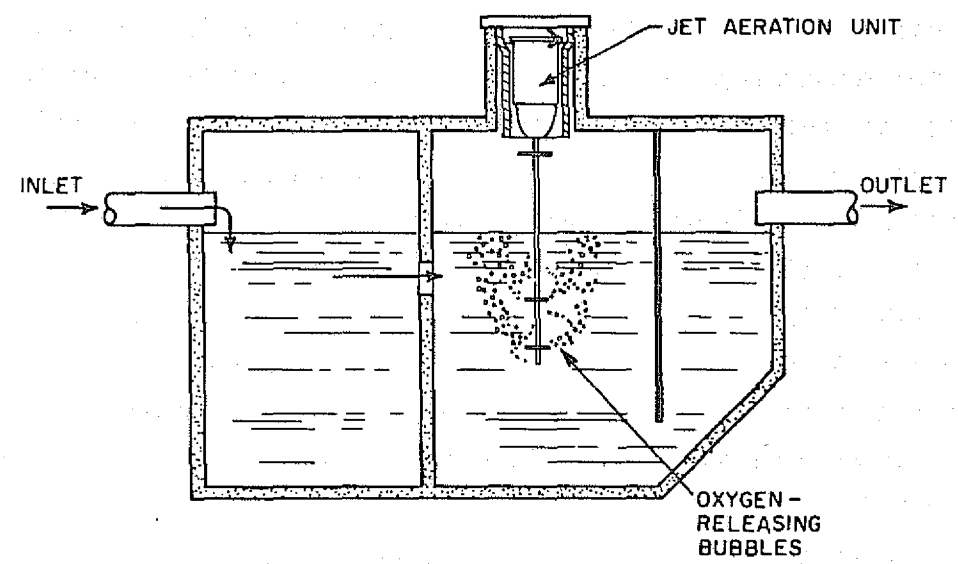


FIG. 8: JET AERATION UNIT
(Jet Aeration Co., Cleveland, Ohio, 1966)

Its volume is 1250 gallons and it is constructed out of concrete. As with the Multi-Flo unit, it is buried at a depth of 4' to 8' in order to protect against freezing. There are currently about twenty in the Anchorage area. Even though no detailed performance data have been collected at the present time, all units appear to be operating satisfactorily. Sludge build-up rates have not been measured for the units in operation in Anchorage.

The oxygen transfer is accomplished by mechanical aspiration tubes with the suction created by the vortex flow around the propeller. The flow patterns are as shown with the sludge settling out in two settling tanks sliding back into the aeration tank. The motor is set by a timer to operate only intermittently. For additional effluent polishing, a sand filter is optional. From Figure 8, it is clear that this is a very simple system. Simplicity of design and operation is a key to getting individual home systems operating reliably.

A Fairbanks resident, Merrit Mitchell, has used one version of a Jet Aeration system since 1973. As long as he periodically cleans out materials like string that tend to block off the air lines, the unit performs reliably. He has it insulated with 4 inches of styrofoam and has it positioned next to the cellar wall to minimize heat losses. As a result, the temperature in the aeration tank is about 20°C. No performance data has been obtained and the sludge was not removed until August, 1977--after four years of use. At that time, Mitchell noticed the soil around the seepage pit was becoming clogged.

Short-term laboratory and field tests (four months) on an Aqua-Reuse system in Fairbanks indicated a potential for adequate treatment using an aerobic package plant (Reid, 1971). As shown in Figure 9, aeration occurs in both primary and seconday aeration chambers, sedimentation is accomplished using plate settlers, and finally chlorine is added. Air is provided by a piston-type air pump. Most of the sludge is recirculated back into the primary chamber by means of an airlift. The recycled cells help provide adequate bacterial concentrations to degrade the organic matter. Settling occurs in the second compartment via both plate settlers and the lower settling zone. A chlorination chamber is provided in case the effluent is discharged to the surface. With discharge to a leaching field, it is not needed.

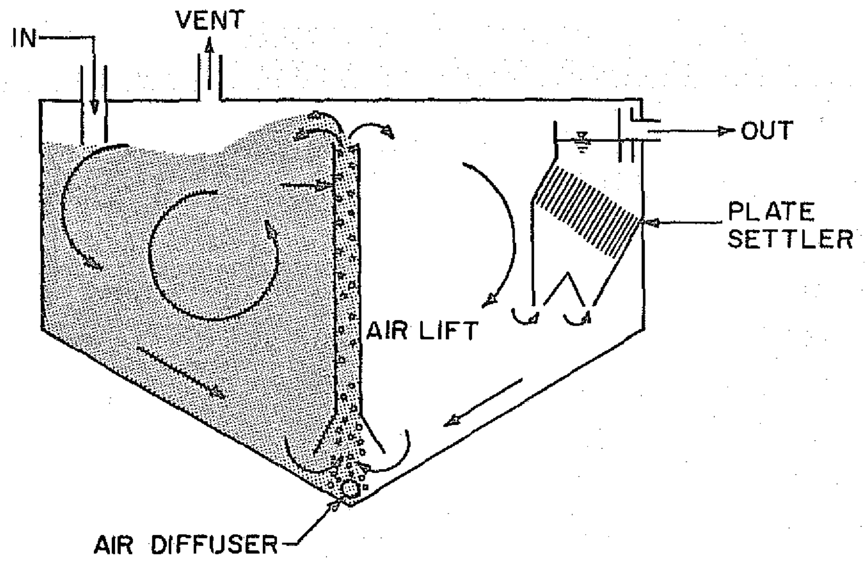


Fig. 9 SECTION OF ORIGINAL AQUA-REUSE SYSTEM SHOWING NORMAL OPERATION. (Reid, 1971)

The plant operates in three cycles with aeration and mixing occurring during the first. Then the air is shut off and the sludge allowed to settle. Finally, supernatant is sprayed over the surface of the effluent chamber both to clean the 60° plate settlers and to break up sludge particles on the surface. Each portion of this cycle lasts less than one hour. A manually operated valve is supplied for sludge wasting. This must be utilized about every six months.

The removal efficiencies for a 10-gpd laboratory unit are shown in Table 3. The influent wastewater averaged 212 mg/l BOD_5 and 280 mg/l SS. One can see that this laboratory unit operating under controlled conditions at room temperature performed well. One year later, a 300-gpd full-scale plant was operated for three months using Fort Wainwright wastewater. Although no data was presented for its BOD removal efficiency, solids removal was said to be 96% complete. Although the 60° plate settlers were supposed to be self-cleaning, the sludge had to be removed from them by backwashing.

TABLE 3: RESULTS OF OPERATIONAL TESTS ON THE MODEL HOUSEHOLD AEROBIC WASTEWATER TREATMENT SYSTEM

Date 1969	Plant Effluent BOD_5 (mg/l)	Plant Effluent SS (mg/l)	Activated Sludge MLSS (mg/l)	Activated Sludge SVI
January 24	16	41	1760	37
January 24	--	24	1580	47
January 29	9	12	2000	61
February 4	12	9	2080	60
February 12	6	6	2710	66
March 3	--	--	2010	70
April 3	27	20	1010[1]	67
April 10	9	4	1180	51
April 16	12	9	1270	51
April 23	10	2	2750	36
April 30	11	7	--	--

[1]Sludge was removed on March 15 so that leaks could be repaired.
SOURCE: Reid, 1971.

Power costs for a family of six were estimated to be no more than \$44/yr in 1971. This is for a ¼-hp air pump operating twelve hours per day. Although the pilot system performed adequately under carefully controlled conditions over a three-month period, this is no guarantee of long-term success in the field. Even during this break-in period, problems developed with respect to hair plugging the backwash nozzles and sludge being carried over into the effluent when wastewater was introduced into the plant during the settling phase. In a real-life application with its accompanying lack of regular maintenance, one would expect other operational problems to develop. For instance, solid objects could block the plate settlers and grease could also build up on the plates. To avoid the resulting compromise in system performance, the homeowner must be conscientious in his periodic inspection and maintenance.

If the soil is such that the only option is surface discharge, one must provide for additional treatment of effluent from a septic tank or aerobic package plant. One promising option is intermittent sand filtration followed by disinfection (Sauer, 1977). At the University of Wisconsin, laboratory and field studies were undertaken by applying septic tank and aerobic unit effluent to sand columns approximately 2 feet high. Hydraulic loading rates ranged from 2 to 10 gal/ft^2/day. The field filters were placed below ground level to avoid freezing problems.

As shown in Figure 10a, the infiltration rate for the sand loaded with septic tank effluent gradually declined over a six-month period. This decline is caused by a clogging of the sand pores by products of microbial growth. To restore its infiltration capacity, the top 2 to 4 inches of sand must be removed and replaced with clean sand. In addition, a resting period is necessary to allow the lower portion of the sand bed to aerate and regenerate. This period, which depends on the characteristics of the sand, may range from one to five months. Effluent quality before and after chlorination is shown on Table 4. It is seen that the chlorinated effluent is of extremely high quality.

TABLE 4: SEPTIC TANK-SAND FILTER EFFLUENT QUALITY DATA[1]

	Septic Tank Effluent	Sand Filter Effluent	Chlorinated Effluent
BOD_5 (mg/l)	123	9	3
SS (mg/l)	48	6-9	6
Ammonia-N (mg/l)	19.2	0.8-1.1	1.6
Nitrate-N (mg/l)	0.3	19.6-20.4	18.9
Orthophosphate (mg/l)	8.7	6.7-7.1	7.9
Fecal Coliforms (#/100 ml)	5.9×10^5	$(0.5\text{-}0.8) \times 10^3$	2
Total Coliforms (#/100 ml)	9.0×10^5	1.3×10^3	3

[1]Loading rate average: 5 gal/ft^2/day (0.2 m/day). Numbers listed are mean values.
SOURCE: Sauer, 1977.

Similar results for aerobic tank effluent appear in Figure 10b and Table 5. Now, even with continuous ponding after a year's time or so, the ultimate infiltration rate was greater than 3 gal/ft^2/day. In contrast to the septic tank effluent, the level of soluble organic matter is sufficiently low that little biological decomposition occurs below the sand surface. Hence, although the accumulated solids must be removed from the surface, there is no need for resting the sand beds between filter runs. A comparison of Tables 4 and 5 shows little difference in sand filter effluent quality between the septic and aerobic wastewaters. But the latter can be applied at a higher hydraulic loading rate and filter maintenance requirements are lessened. The only required maintenance is removal of the top inch of sand every six months.

TABLE 5: AEROBIC UNIT-SAND FILTER EFFLUENT QUALITY DATA[1]

	Aerobic Unit Effluent	Sand Filter Effluent	Chlorinated Effluent
BOD_5 (mg/l)	26	2-4	4
SS (mg/l)	48	9-11	7
Ammonia-N (mg/l)	0.4	0.3	0.4
Nitrate-N (mg/l)	33.8	36.8	37.6
Orthophosphate (mg/l)	1.9×10^4	1.3×10^4	35

[1]Loading rate average: 3.8 gal/ft^2/day (0.15 m/day). Numbers listed are mean values.
SOURCE: Sauer, 1977.

A summary of the capital and maintenance costs appears in Table 6. Assumptions include a family size of five with a wastewater production of 50 gpcd. Sand with an effective size of .4 mm and uniformity coefficient of 3.5 is assumed to be readily available. For an Alaskan application, the sand filters could readily be positioned under a protective layer to keep them from freezing. But, the outlet line would have to be constructed carefully to keep the effluent in it from freezing before it exits. Obviously, in Fairbanks, the effluent would glaciate during the winter. But, its high quality may eliminate pollution problems during spring runoff.

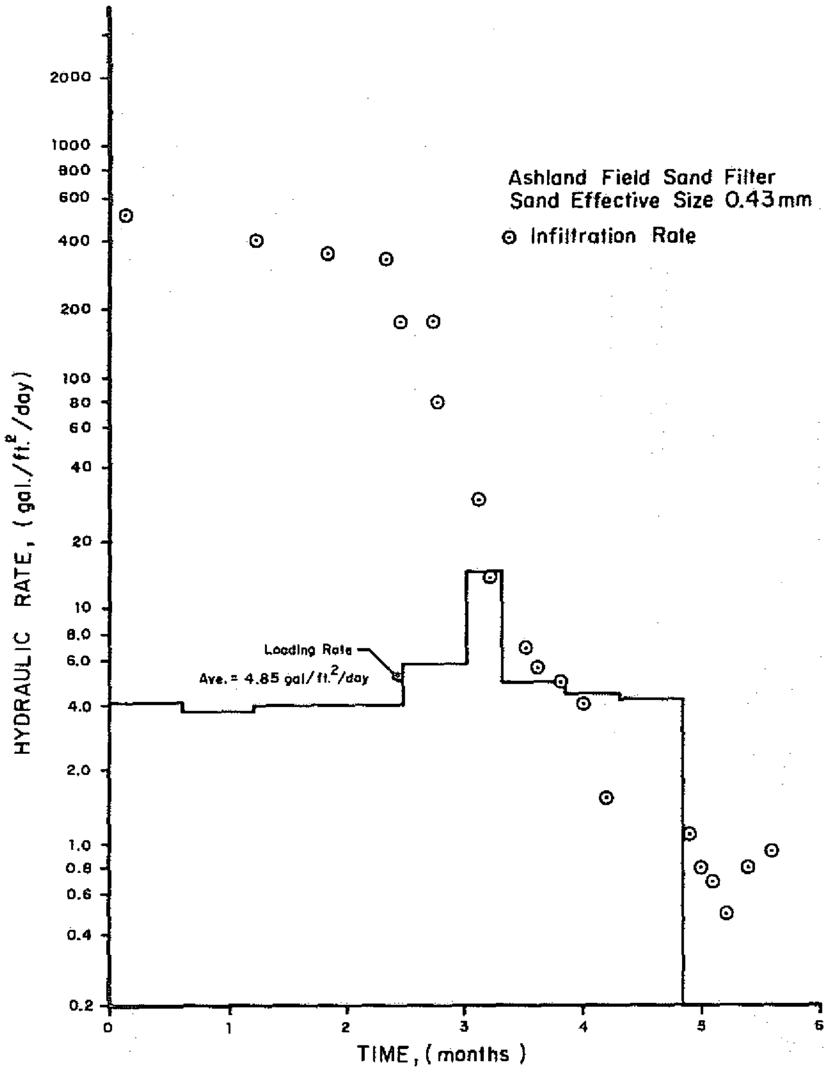


FIG. 10a INFILTRATION RATE DECLINE OF SAND LOADED WITH SEPTIC TANK EFFLUENT. (Sauer, 1977)

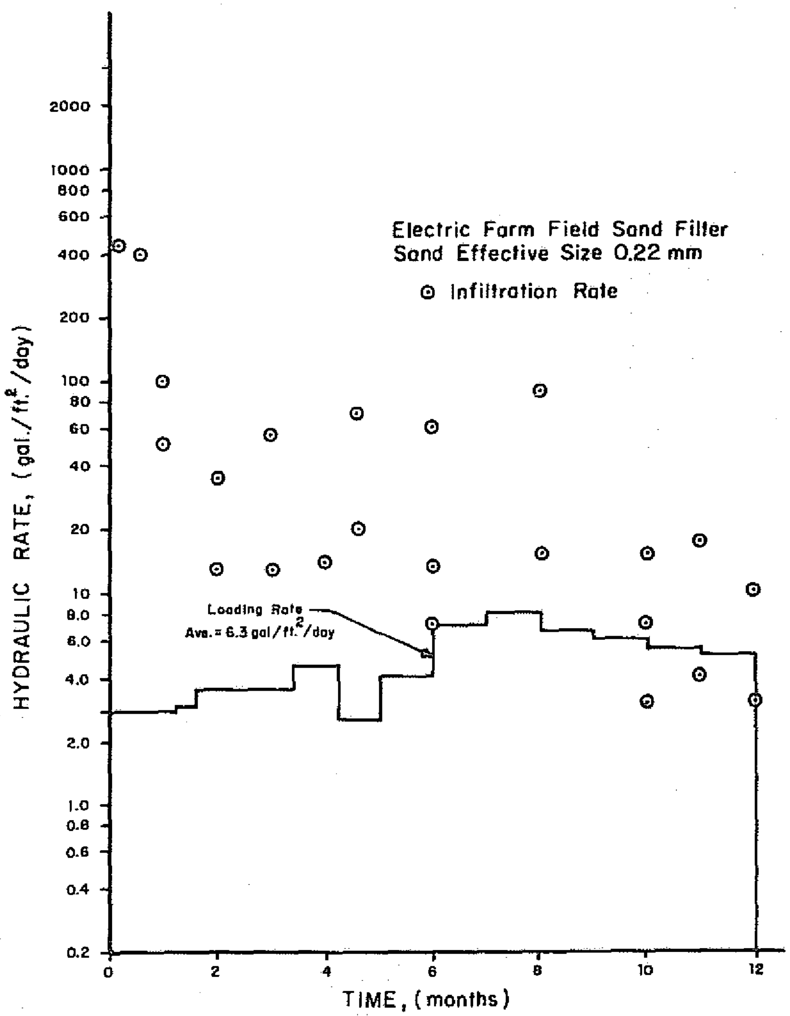


FIG. 10b INFILTRATION RATE DECLINE OF SAND LOADED WITH AEROBIC UNIT EFFLUENT. (Sauer, 1977)

TABLE 6: INITIAL CAPITAL COSTS AND ANNUAL OPERATION AND MAINTENANCE COSTS

Unit	Cost, in Dollars
Septic tank (1000 gal)	
Equipment and Installation Cost	350-450
Maintenance cost	10/yr
Aerobic treatment unit	
Equipment and installation cost	1300-1000
Maintenance cost	35/yr
Operation cost (4 kwhr/day at 4¢/kwhr)	60/yr
Wet well pumping chamber	
Equipment and installation cost	250-350
Operation cost[1] (.75 kwhr/day at 4¢/kwhr)	10/yr
Sand filter	
Equipment and installation cost	10-15
Maintenance cost	1
Chlorination and settling chamber	
Equipment and installation cost	700-1000
Operation cost[1] (chemical)	40/yr
Ultraviolet radiation unit	
Equipment and installation cost	1100-1500
Operation cost[1] (1.5 kwhr/day at 4¢/kwhr)	20/yr
Maintenance cost, cleaning and lamp replacement	Undetermined

[1]Does not include pump relacement.
SOURCE: Adapted from Sauer, 1977.

At the University of Wisconsin in 1972, both laboratory and field studies of five household treatment units were performed (Otis et al., 1975). Summaries of influent and effluent characteristics for the laboratory studies are shown in Table 7. Although results similar to those from laboratory tests for BOD_5 removal were found for the field tests, the aerobic units in the field were not significantly different from the septic tanks with respect to SS removal. This is attributed to periodic discharges of solids because of various instabilities for the field aerobic units. These included buildup of excess solids, hydraulic surges, and bulking sludge. Although the BOD_5 removals are similar, multiple-compartment septic tanks show much better SS removals than single-compartment tanks. The much lower variability in effluent quality for the laboratory units is attributed to the better maintenance they

TABLE 7: SUMMARY OF EFFLUENT QUALITY DATA FROM LABORATORY UNITS

Characteristic (mg/l)	Influent values	Septic Tank		Aeration		Biological Disk
		Single Chamber	Multi-Chamber	Extended	Batch	
COD						
Mean	470	335	220	65	35	65
95% Confidence Interval		290-385	195-250	55-75	25-45	55-70
# of Data Points		24	26	25	24	25
BOD						
Mean	235	140	95	50	15	50
95% Confidence Interval		100-195	75-120	40-70	10-25	40-60
# of Data Points		22	24	21	23	21
SUSPENDED SOLIDS						
Mean	275	135	85	35	17	20
95% Confidence Interval		110-165	70-100	30-50	13-22	18-26
# of Data Points		20	22	20	22	23

SOURCE: Otis et al., 1975.

receive. Economic calculations of Otis et al. (1975) indicate that aerobic units are competitive with septic tanks only if surface discharge is allowed. But, these calculations probably do not take into account the reduced life of the absorption field with septic tank rather than aeration tank effluents. For instance, Laak (1970) found that decreasing the sum of SS plus BOD_5 by 50% increases the service time by 50% for domestic wastewaters. Wells's (1977) calculations (Table 8) indicate a lower annual cost for an aerobic system if the installation of such a system allows the absorption field to last indefinitely. The truth probably lies somewhere between these two extremes.

According to a paper of Otis and Boyle (1976), the performance of aerobic units would be enhanced by: 1) installing a septic tank or trash trap ahead of the aeration unit to remove grease and large debris, 2) using these as flow equalization tanks, 3) utilizing a heat source to maintain mixed liquor temperatures above 15°C, 4) requiring regular maintenance with inspections at least every two months, and 5) constructing effluent weirs at least 24 inches long. The typical weirs are only 6 inches long and are grossly overloaded during surges that occur with discharges from washing machines and baths. This carries solids over the weirs. With the exception of the construction of effluent weirs, incorporation of these suggestions would lead to increased costs.

Table 8 taken from Wells compares the costs of four alternate disposal systems over a 25-year period. The septic tank cost data was provided by Hank Humphreys of Lemeta Pumping and Thawing and Bud Hilton and the aeration data by David Greer of Greer Tank and Welding and Otis et al. (1977). Jerry Culp of the Fairbanks City Engineering Department supplied the municipal figures. An absorption field size requirement of 1K ft^2 is assumed. At first glance, the cost data indicates ET systems to be the most cost effective. But this must be tempered by the realization that the ET costs are lower-48 costs. The fact that Bud Hilton has successfully used an ET system for nine years in Fairbanks indicates its potential in cold climates given the right soils. Of course, there will be no treatment via transpiration during the winter months. Then, the liquid can pass only through the unfrozen soil. Note that the public sewer costs are substantially higher than the individual unit costs and

TABLE 8: APPROXIMATE COST IN DOLLARS OF FOUR ALTERNATIVE DISPOSAL METHODS FOR A FOUR-BEDROOM HOME IN ADEQUATE SOILS IN THE FAIRBANKS AREA, 1977.

Item	Septic Tank (1250 gal.)	Aeration Unit	ET[1] System	Public Sewer
Capital cost (material and installation)	3750	6200	3000-5000[2] [3]	20,000-30,000
Annual maintenance	75-150	35[1]	(minimal)[4]	--
Major repairs or work (every 5 yrs.)	--	60	--	--
Seepage pit replacement (10 yrs.)	2000	--	--	--
Annual electricity cost	--	83	--	--
Annual sewer charge	--	--	--	300
Annual cost over 25-yr. period (no interest)	425-500	378	145-205[4]	1100-1500

[1]Evapotranspiration.
[2]Represents 1977 costs in the lower 48 states (Otis et al., 1977).
[3]Does not include cost of shipping material from the lower 48 states.
[4]Includes an annual $25 to cover any necessary maintenance cost (assumed).
SOURCE: Wells, 1977.

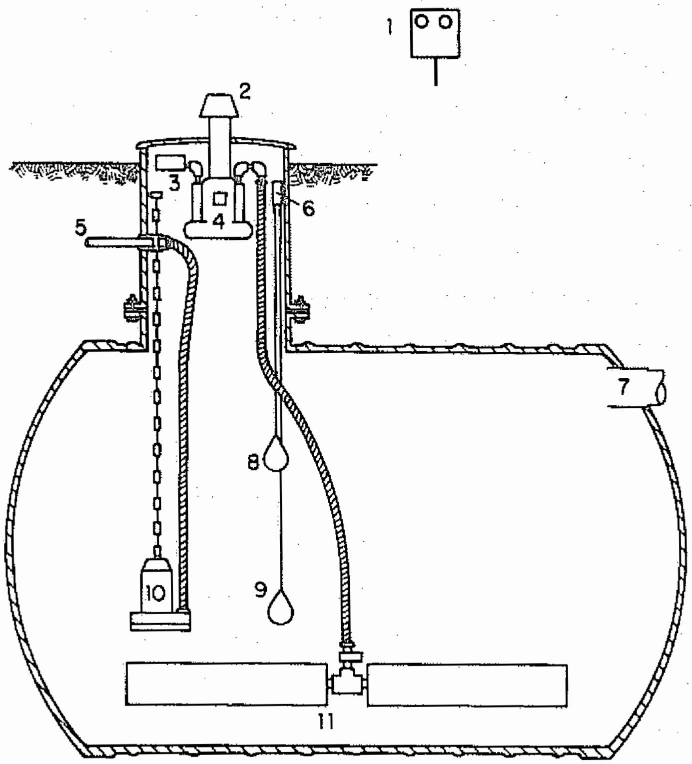
that all annual costs would rise substantially if an interest charge were included in the amortization of the capital costs. Septic tank prices ranged from \$470 to \$825 (Wells, 1977) with an average figure of \$600 assumed in the table. Maintenance includes annual pumping for the septic tanks.

Of the \$6200 capital cost for the aeration system, \$2700 is for the aeration unit itself and the remainder for the absorption field. One reason why the annual effective costs for the aeration system is less than that for the septic system is that periodic pit replacement is not assumed to be necessary for the former. This is probably not strictly true. According to Wells's figures, the seepage pit used with the aerobic system would probably have to last at least twenty years for the aerobic to be cost effective. The sludge from these units is assumed to require pumping only once every five years. The fact that Mitchell has gone four years without pumping is consistent with this timetable. An electricity requirement of a 4 kwhrs/day translates into a yearly cost of \$83 according to the current Golden Valley Electrical Association rates. The high capital costs for the municipal sewer hookup arise largely from the need to lay up to 400 feet of new sewer line for each large lot common in the subdivisions not now incorporated by Fairbanks. At \$75 per foot, this amounts to \$30,000.

An example of another aerobic system currently being marketed in Alaska is the Mini-Plant built by Eastern Environmental Controls, Inc. As shown on Figure 11, the system sold includes the aeration components and controls with the user supplying the tank. The operation is batch with 20.5 hours of aeration followed by 3.5 hours of settling and discharge. The latter can be made to occur during periods of inactivity such as 2-6 a.m. At a flow of 400 gpd, NSF found the BOD_5 and SS removals to be around 90% (NSF, 1972). These and other performance features coupled with the warantee of a two-year preventative maintenance schedule have gained this system NSF approval. Data is not yet available on its performance in Alaska.

Noninfiltrating Systems

Besides the noninfiltrating systems of the water-closet type in which wastewaters are merely contained, there are also noninfiltrating treatment systems. The former include recirculating chemical toilets



Key:

1. Control panel
2. Air intake
3. Air filter
4. Blower
5. Effluent line
6. Junction box
7. Influent line
8. Alarm sensor
9. Pump shut-off
10. Pump
11. Air diffusers

Fig. 11: TYPICAL FIBERGLASS TANK INSTALLATION
(Eastern Environmental Controls, Inc., Chestertown, Md.)

while the latter include composting and incinerating units. It is these two with which we are concerned in this section. For comparison purposes, cost data are presented for five of these systems in Table 9. The costs for the incinerating unit were obtained from Grainge et al. (1971) and, hence, are not up to date. All systems have a septic tank system for greywater disposal. Several people (Wells, 1977) have indicated that incinerating toilets are characterized by excessive odor problems and frequent mechanical failure. Grainge et al. (1971) report that the high salt-content liquid developed during incineration is very corrosive. Hence, much maintenance is required.

Incinerating toilets are power consumptive and chemical and oil flush units make waste permanently unusable, whereas composting toilets turn waste into a usable form, 60 lbmpc per year of humus. Composting is the aerobic or anaerobic decomposition of solid organic matter by microorganisms. The preferred mode of operation is aerobic with the by-products being CO_2 and water. The humus represents the nondecomposable organics plus cellular matter. For optimum operation, the organic matter should consist of both animal and plant (cellulose) wastes. The latter helps provide the porosity to keep the pile well aerated. Moreover, since pathogens are anaerobic, they don't compete successfully with the aerobic forms in the compost pile. Hence, compost sometimes can be safely applied to gardens.

Prices for the three composting systems reflect current Fairbanks costs as compiled by Wells (1977). The Humus and Eco-let units cost less than the Clivus Multrum because of their smaller volumes. To accomplish the desired waste decomposition processes in these smaller units requires electrical heating coils to maintain the piles at 90°F. Moreover, this required power can add up to $5/month to one's electric bill in New England (Maine Times, 1976). Power is also used to drive a mechanical mixer to keep the pile well agitated. Since decomposition is accelerated in the small units, the ash has to be emptied more often than in the larger units. This is done through use of a door at the base of the toilet. There can be problems with excessive compaction if the unit is not used for a long period. On the other hand, the microorganisms will not die during long periods of cold and can be reactivated by simply turning the system back on. Mitchell (1977) reports that

TABLE 9: APPROXIMATE COST OF DISPOSAL ALTERNATIVES

Item	Chemical toilet[1]	Humus toilet	Eco-let	Incinolet	Clivus multrum
Capital cost	$ 900	$ 944	$ 700	$ 595[4]	$1662
Maintenance (yr)	$1300[2]	(minimal)[3]	(minimal)[3]	12[4]	(minimal)[3]
Shipping and installation	--	100	--	--	1000
Septic tank absorption field cost	3750	3750	3750	3750	3750
Electricity (yr)	--	25	25	3757	--
Heating (yr)	--	180	180	--	90
Annual cost over 25-year period	1486	398[3]	384[3]	410[5]	371

[1]Capacity of 10 users.
[2]Includes weekly pumping costs of $25.
[3]Includes $25 annually, to cover maintenance costs (assumed).
[4]Includes $150 for an air compressor in Canada in 1971.
[5]At 5¢/kwhr.
SOURCE: Adapted from Wells, 1977.

power outages or human neglect caused inactiviation of decomposition processes in humus toilets used in Alaskan villages.

Even though the Clivus Multrum is expensive and difficult to install, it does have the advantage of not requiring any electric motors. Because of its large volume (Figure 12), a large-enough heap can be created such that the internal heat generation can be significant (Maine Times, 1977). This is basically a matter of maintaining a sufficiently large volume per unit surface area since the metabolic heat output of the microorganisms is proportional to the former and heat losses are proportional to the latter. The Clivus Multrum is typical of the large-volume units in that the 30° incline allows the waste matter to slide slowly down the chamber at a rate sufficient to produce odorless, pathogen-free compost by the time the material reaches the access door at the lower end of the unit. It had been installed in 1300 homes in Scandanavia by 1975 (Lindström, 1975).

Even though some manufacturers claim composting toilets are almost maintenance free, the pile must be turned frequently to avoid compaction--weekly for some small units. Clivus Multrum admits it will take about two years for equilibrium to be reached (Maine Times, 1977). After that, it supposedly produces 3 to 10 gal per capita per year of humus. Unlike the product of small-volume units, its compost is supposedly pathogen free because of the long, 3- to 4-year detention time.

One must beware of overstating the savings with respect to power consumption needed by these composting units. It is true that the energy used by a large-volume unit such as the Clivus Multrum is nil in warm climates. But, in Alaska, if such a unit is to operate efficiently during the winter, heat must be provided. It is claimed that the Clivus needs 7 l/sec of air (Maine Times, 1977). If one assumes that one's home heating system must warm this air from -23°C to 20°C, this corresponds to a heat loss of around 30K Btu/day. Adding these losses over the colder months of the year would result in an annual fuel cost of around $50 assuming a cost of 60¢/gal for heating oil. Such considerations have led some people to consider the use of solar panels or stack robbers to provide heat for the Clivus Multrum.

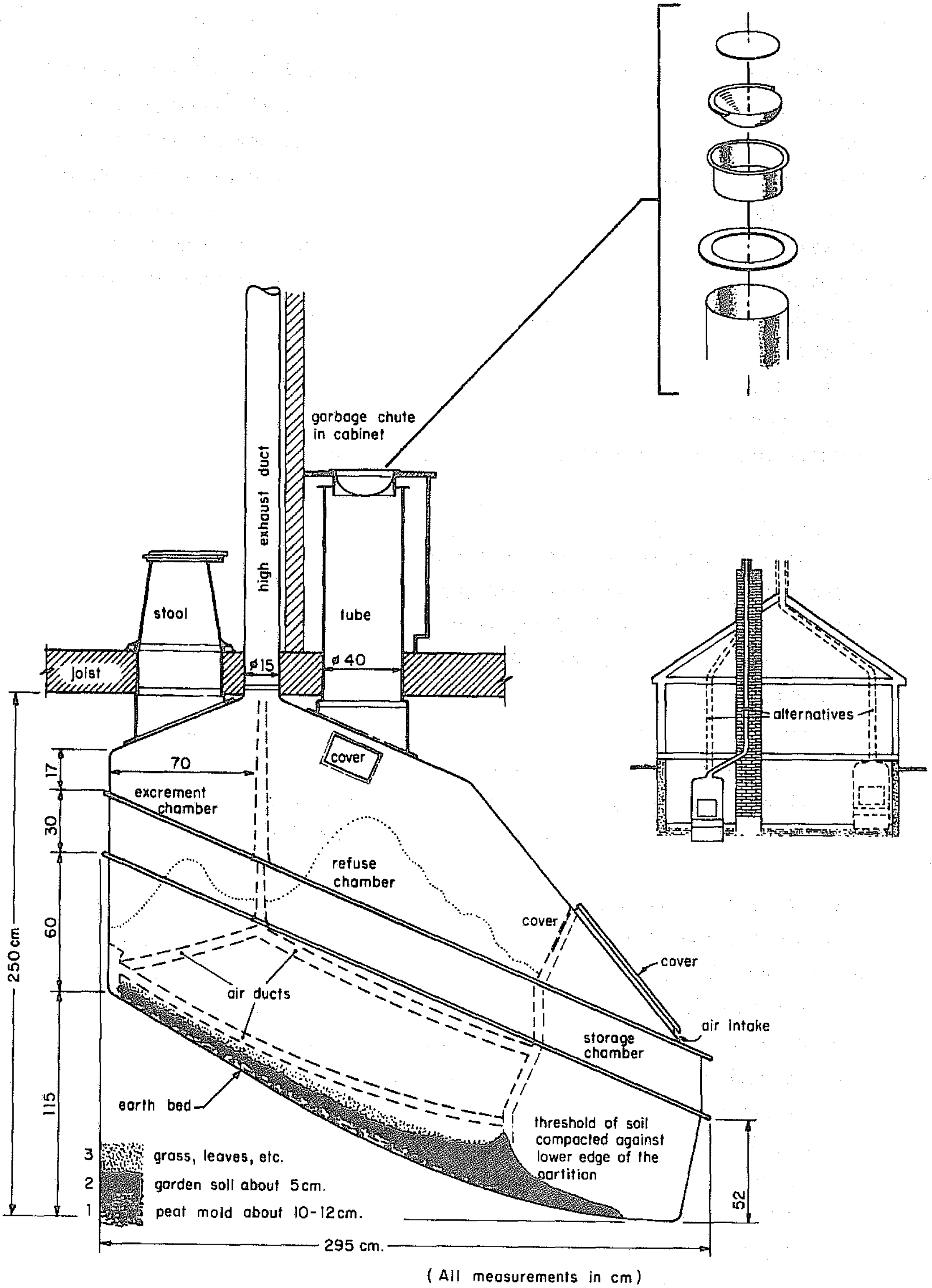


Fig. 12 : CLIVUS MULTRUM UNIT (Lindström, 1975)

Another potential problem with composting toilets is their susceptibility to insect infestations. These are practically inevitable in the initial stages before natural predators such as spiders and beetles keep the undesirables in check. Mites have been known to invade households. Even though the proper draft will keep odors out of the house, it won't stop insects. Furthermore, there have been problems in the north with excessive liquid accumulation because evaporation is retarded at colder temperatures (McKernan and Morgan, 1976). This can cause septic conditions to develop. But these disadvantages must be evaluated in light of the roughly ten gallons of finished compost produced per person per year as opposed to the 10K gallons of clean water which becomes polluted using flush toilets (Lindström, 1975).

Discussion

Both because so many of its inhabitants live in rural areas with large distances between homes and because there is danger of exposed pipes freezing over much of the year, Alaska is an appropriate place for the application of individual household systems. Community systems are prohibitively expensive for homes not in close proximity to one another. To pipe the sewage to the treatment plant would cost far more than the treatment itself. Even in Fairbanks, Wells (1977) found that the costs to hook up to existing sewer lines for a typical new home would exceed $20,000. In nonpermafrost areas, the collection lines must either be heated sufficiently by the household effluents or buried and/or insulated to avoid freezing. In permafrost areas, insulation is essential to avoid thawing of the supporting ground. These and other reasons lead one to avoid extensive sewage-collection networks if possible.

Economic considerations in remote areas have frequently led to indiscriminant disposal of wastes onto the tundra or onto pack ice. As the ice pack breaks up, the wastes are carried out to sea. Privies and honey buckets are the rule rather than the exception in many remote villages in Alaska today. In neither of these cases is appreciable decomposition of the waste products occurring for much of the year. But some decomposition will occur during the summer months. Since the rate of pathogen travel through the soil is typically slow, it is possible that, even with decay rates an order of magnitude less than those applicable for warm climates, the waste disposal areas can be positioned safely moderate distances away from water supplies. The biggest danger would probably occur via surface transport during runoff. More research is needed to quantify these arguments.

Septic-tank systems are the most popular individual household units in the United States today. With the tank serving essentially as a sedimentation chamber, most of the biological treatment occurs in the absorption field. The latter is commonly a seepage pit rather than a leaching field in Alaska. These systems can perform very well providing the climate and soils are suitable. This is not always the case in parts of Alaska where septic systems are being used (Neale et al., 1974; Bateman, 1977). High water tables and/or very fine soils have led to systems' failing even after only a few months. The most frequent cause of failure is the clogging of the absorption field--a situation which often produces ponding and unpleasant odors. This is true both in Alaska and the lower 48. In only a small minority of the cases is system failure due to freezing (Hickey and Duncan, 1966). This problem is aggravated by the homeowner's failure to pump the sludge out of the bottom of the septic tank frequently enough. Depending upon the loading rates, this should be done about once a year.

One possible solution to the clogging problem is to periodically allow the absorption field to rest. This allows the anaerobic slimes and other materials causing the clogging to be removed. Such an idea is being applied to systems being built in Fairfax County, Virginia (Clayton, 1975). Here, systems are being designed with two separate absorption fields so one can be rested while the other is used. It is thought that this plus other sound engineering practices will allow systems to last 50 years. One other idea is to add extra gravel around the seepage pit to enhance the absorptive capacity of the soil.

Another solution, being tried on a moderate scale in the Anchorage area, is to treat the wastewaters aerobically so that the effluent percolating into the absorption field will be more highly stabilized than that leaving a septic tank. The thought is that this will allow less-desirable soils to be used as absorption fields. Laak (1970) concluded from his experiments that cleaner effluents prolonged the life of the absorption field. Such effluents are likely to result from replacing septic systems with aerobic treatment. Hopefully these conclusions apply to soils in the Anchorage area. If they do, the lifetime of absorption fields in Alaska may be extended beyond the six to ten years currently expected (Hickey and Duncan, 1966). Experience in Anchorage (Strickland, 1977) indicates some aerobic systems are far superior to others with respect to reliability. In either case, proper maintenance is essential.

For those homeowners who are unwilling or unable to transport their wastes away using water, there are waterless treatment systems available. Incinerating toilets provide for complete destruction of pathogens with the production of an inert ash. However, they are energy intensive and are easily corroded at the high temperatures encountered. Composting toilets have been used extensively in Scandanavia (Lindström, 1975). With these units, the organic wastes are aerobically decomposed producing carbon dioxide and water as stable end products plus a stablized humus. The tradeoff to be considered in deciding to install a larger composting unit over a smaller one is that the former takes up more space and costs more initially whereas the latter typically requires power to operate heating coils. These coils are needed to maintain proper temperatures in the composting pile. Not widely publicized by the manufacturer is the need to supply an air flow for the microorganisms into either unit. This can lead to a substantial heat loss from the home (in Alaska, on the order of 10^7 Btu/yr). Moreover, the units are not maintenance free, as is sometimes claimed. In the nonmechanized units, the composting mass must be frequently raked to maintain aerobic conditions throughout the volume. But, with adequate attention being devoted to the system by the homeowner, a composting toilet can provide for a complete stabilization of household wastes while simultaneously conserving water.

MUNICIPAL AND MILITARY TREATMENT SYSTEMS

Overview

There are approximately 250 small communities in Alaska included in the 1976 Alaska Sewer Needs Survey conducted by Dames and Moore (Hargesheimer, 1976). For a complete list, see Appendix B. These communities range in size from small villages with populations of less than fifty to as many as 4000. In addition, about twenty larger Alaskan communities have municipal sanitation facilities (Table 10). Besides the vast differences in topography and climate from one community to another, there are significant differences in the ease with which materials, fuel, and qualified personnel can be obtained between bush and urban communities. The largest city, Anchorage, has a population of 161,000. It has only primary treatment for its sewage with the question of whether or not secondary treatment is to be installed to be decided soon. As shown in Table 10, several of the larger municipalities have activated sludge treatment. These include Cordova, Fairbanks, Juneau, Kenai, Haines, Hoonah, North Pole, and Soldotna. All are of the extended aeration type except Fairbanks which uses pure oxygen. In general, these activated sludge systems have been able to operate on a continuous basis. But, most have not consistently produced effluent of secondary quality (Table 11). Several communities including Eagle River, Homer, Palmer, and Valdez utilize aerated lagoons. Kodiak, Girdwood, Wrangell, Petersburg, and Skagway are in the process of completing secondary treatment facilities. These facilities will utilize either rotating biological contactors or active biological filters. According to Jon Scribner (1976) of the Alaska Department of Environmental Conservation (ADEC), Yakatat and Craig have poorly operating secondary treatment plants. The Dillingham plant, installed in 1965, has not operated since 1966 (Ryan, 1977b, and Rogness, 1977).

The communities of Dillingham, King Salmon, Seward, Sitka, Seldovia, Unalaska, Wasilla, and Whittier have Step 1 grants from the United States Environmental Protection Agency (USEPA). This means facilities plans for recommended solutions are being formulated. Eagle River, Ketchikan, and Nome are at the Step 2 level meaning the alternative recommended in Step 1 is being designed in detail. Hence, they

TABLE 10: SANITATION FACILITIES IN LARGER ALASKAN MUNCIPALITIES

City	Population[1]	Treatment (see key, next page)	Flow (MGD)
Anchorage	161,000	1, 3, 14, 33, 34, 36, 39	21
Auke Bay	incl. w/Juneau Borough	1, 3, 5, 14, 30, 36	.01
Cordova	1,915	1, 3, 5, 14, 30, 36	.35
Eagle River		14, 16	.15
Fairbanks	30,000	1, 3, 14, 30, 33, 34, 39	4
Haines	965	1, 3, 5, 14, 30, 33, 35, 36, 39	.2
Homer	1,528	14, 16	.25
Hoonah	843	5	
Juneau (incl. borough)	16,700	1, 5, 33, 36, 30a	2.5
Kenai	4,000	1, 3, 5, 14, 32, 36	.6
Ketchikan	7,527		
Kodiak	4,351		
Mendenhall Valley	incl. w/Juneau Borough	1, 3, 5, 14, 30, 32, 36	.6
Nome	2,500	3, 17, 30	.16
North Pole	288		
Palmer	1,626	16	
Petersburg	2,231	3, 14, 17, 34 U.C.	.4
Sitka	6,073	1, 3, 17, 33, 34, 36, 39	2.5
Skagway	834	1, 3, 14, 17, 33, 34 U.C.	
Soldotna	1,420	1, 3, 5, 14, 32, 36, 39	
Valdez	2,287	14, 16, 17, 36	
Wrangell	2,614	1, 3, 4, 14, 33, 34, 36, U.C.	

[1]United States Census Bureau, Fairbanks Office, as of July, 1975.
SOURCE: Adpated from Hargesheimer, 1976.

KEY FOR TABLE 10

1 - Preliminary treatment
2 - Pumping
3 - Primary sedimentation
4 - Trickling filter
5 - Activated sludge
6 - Filtration
7 - Activated carbon
8 - Two stage tertiary lime
9 - Biological nitrification
10 - Biological dentrification
11 - Ion exchange
12 - Breakpoint chlorination
13 - Ammonia stripping
14 - Disinfection
15 - Land treatment of effluent
16 - Oxidation ponds
17 - Other treatment
30 - Anaerobic digestion
30a- Aerobic
31 - Heat treatment
32 - Air drying
33 - Dewatering
34 - Incineration
35 - Recalcination
36 - Land fill
37 - Land spreading of sludge
38 - Ocean dumping
39 - Other sludge handling
U.C.-Under construction

TABLE 11: PERFORMANCE DATA

Community	BOD_5 Influent	BOD_5 Effluent	SS Influent	SS Effluent	Reference and Date
Anchorage	115	90	147	54	plant log: 3/75, 7/76, 11/76
Cordova	211	28	69	10	EPA: 5/76
Eagle River	170	40	200	100	Christianson (1976): 4/73-3/76
Fairbanks	169	24	168	35	plant log: 4/77-7/77
Haines	49	21	24	12	EPA: 2/76
		44		33	EPA: 1/77-3/77
Homer	260	100	130	70	EPA: 6/8/74, 6/11/74
		25		17	EPA: 1/4/74, 1/7/76[1]
Juneau-Auke Bay		6		10	EPA: 1976
-Douglas	127	5	103	9	plant log: 7/76-6/77
-Mendenhall	146	81	152	52	EPA: 2/76
Kenai	173	26	137	8	plant log: 3/5/76, 3/9/76
	124	80	132	90	EPA: 3/76
Nome	240	170	250	175	Hargesheimer (1976)[2]
Palmer	160	18	160	22	Christianson (1976): 4/73-4/76
Soldotna	164	22	141	6	plant log: 3/5/76, 3/7/76
	62	7	78	5	EPA: 3/75

[1]Data analysis questionable according to Crevensten (1977).
[2]Numbers are estimates.
SOURCE: Crevensten, 1977.

will all presumably have either modifications to existing plants completed or new plants built in the near future. Kenai and Soldotna have plans to modify their existing plants (Kelton, 1977). Most of the remaining communities have no treatment or minimal septic tank treatment. Honey buckets and privies are used extensively in remote locations.

The United States Public Health Service (USHPS), Alaska Native Health Service, has installed sanitation facilities in approximately 93 villages in the last 15 years. These have ranged from just watering points to complete running water and sewage collection and treatment systems. According to Ryan (1977b) and Rogness (1977), of the sixty or so falling into this latter category, less than six have failed. Here failure is defined as a complete shutdown of a system for one reason or another. This can and does include freeze up. Hence, the success rate by these standards is 90%. However, if one were to include a partial shutdown, extensive repairs being made, or discharge requirements not being met in defining "failure," the rate is undoubtedly higher. Partially because USEPA does not monitor waste discharges in the villages, not enough data is available to be quantitative about this kind of failure rate.

USPHS has installed around a dozen package plants and half that many community septic systems. Around thirty of the villages where the USPHS has installed some kind of a facility have watering points only. The other approximately 120 villages in which neither USPHS or ADEC have installed systems are utilizing honey buckets and hauling water from rivers, lakes, etc.

Perhaps 70% of the treatment is via facultative lagoons. The only year-around aerated lagoon is at Metlakatla. The air is supplied via surface aeration. One reason for the success of this lagoon is the fact that the USPHS construction foreman lives nearby. The size of the lagoons is usually determined by using one acre per 100 people as a guide. Generaly speaking, the design population has not been reached, and most lagoons are not loaded as heavily as this criterion would indicate. With a design liquid depth of 11 feet, there is liquid volume even in the winter. There are some lagoons, however, that freeze to the bottom along the outer edges. At least half are nonoverflowing at the present time. Hence, there is no effluent being discharged to surface waters in these cases.

The USPHS program in Alaska was started in 1960 with watering points and privies being constructed and garbage trucks being provided. With the exception of southeast Alaska, the oldest sewage collection and treatment system is seven years old. All the villages on Kodiak Island and many in southeast Alaska have community septic tanks and outfalls. The current plan is to provide secondary treatment within a couple of years for these villages. Of the roughly 120 villages with no sewage treatment at the present time, roughly 100 have asked USPHS to upgrade their facilities.

The Village Safe Water (VSW) Act of 1970 was passed to provide "safe water and hygienic sewage disposal facilities for villages in the state." The VSW Act is administered by the ADEC. The immediate plan is to have sanitation facilities constructed in the first nine villages listed in Table 12 by the end of the 1977 fiscal year (Sargent and Scribner, 1976). Since this report, Council and Tanana have been added to the list. These facilities will include dumping bins for honey buckets to be treated at the facility. At the present time, Chevak, Selawik, Alakanuk, Nulato, Pitkas Point, Koyukuk, and Beaver have sewage treatment facilities completed. These include one lagoon and three extended aeration and three physical/chemical package plants.

Two native villages, Emmonak and Wainwright, have central sanitation facilities funded by USEPA under the Alaska Village Demonstration Project (AVDP). Physical/chemical methods are utilized for wastewater treatment and some water reuse is practiced. Each has had operational difficulties which may be resolved with time.

One significant difference between the VSW and AVDP programs on the one hand and the USPHS programs on the other is that an operating and maintenance budget is provided by the responsible agency in the former cases. USPHS is prevented by its charter from doing the same. According to Sargent (1977), any utility system installed in a remote Alaskan village is doomed to failure unless the village is given financial, technical, and/or management assistance. Hence, provision is made for ensuring that at least one full time, trained, and paid maintenance man attends every VSW facility. Even though USPHS strongly recommends to the villages that they establish ways of financing for adequate maintenance, USPHS cannot provide the funding for this. According to

TABLE 12: VILLAGES RECEIVING VSW FACILITIES

Village	Location in N. Lat.	Location in W. Long.	Population	Mean Jan. Temp. °C.	Mean Annual Temp. °C
Alakanuk	62° 41'	164° 34'	512	-19	-2
Beaver	66° 22'	147° 24'	101	-29	-1
Chevak	61° 32'	165° 35'	447	-17	-2
Council	64° 54'	163° 40'	41	-14[1]	-7[1]
Kongiganak	59° 52'	163° 02'	200	-17	-1
Koyukuk	64° 53'	157° 42'	124	-27	-4
Northway	62° 58'	141° 56'	40	-31	-6
Nulato	64° 43'	158° 06'	330	-27	-4
Pitkas Point	62° 02'	163° 17'	85	-19	-2
Selawik	66° 36'	160° 00'	521	-27	-6
Tanana	65° 10'	152° 04'	349	-24	-4

[1]Weather data unavailable, figures shown are for Nome.
SOURCE: Sargent and Scribner, 1976.

Sargent (1977), the villages are often not able to provide the management and fiscal resources adequate for this task. Hence, periodic system failures will occur. Although the ADEC and the USPHS are currently conducting an inventory of past failures in villages (Capito, 1977), not enough data is available at the present time to be quantitative about this. USPHS maintains that the failure rate (by its definition of failure) is less than 10% with the present system. Only if a detailed inventory is completed, may this issue be resolved. It should be emphasized here that this writer is deliberately omitting the philosophical questions of whether or not the villages should have sanitation systems if they cannot pay for operation and maintenance themselves. In searching for answers for such questions, one must remember that villagers will not utilize a sanitation facility if the price they must pay is excessive. Hence, they may suffer healthwise from inadequate water treatment or waste disposal practices.

Under the VSW Program, ADEC plans to reply more on biological than physical/chemical treatment. The reasons are that less operator attention is required at the former and there is less sludge to be disposed of. ADEC also plans to replace clarifiers with membrane separators similar to those used in the Multi-Flo device (Sargent, 1977).

As shown in Table 13 there are 12 United States Air Force (USAF) facilities in Alaska utilizing aerated lagoons as the basic treatment technique. Their capacities range from 7K gpd to 1 Mgpd. Typically, the air is provided by blowing it through tubular diffusers. Even though the actual flow is typically less than the design flow, most of these systems fail to meet 30/30 standards. Reed (1976) concluded in a U. S. Army Cold Regions Research and Engineering Laboratory (CRREL) study that the main reason effluent standards are not met is that lagoons are being designed for 85% BOD and SS removal and the influent concentrations of these quantities are greater than 200 mg/l. Sites at Campion, Cold Bay, Kotzebue, Fort Yukon, and Murphy Dome all utilize activated sludge processes. Sewage from the U. S. Army facilities at Ft. Wainwright and Ft. Richardson is treated at the Fairbanks treatment plant and Anchorage respectively. At Ft. Greely, a lagoon is utilized. Some of the remote military installations still use septic tanks.

TABLE 13: USAF AERATED LAGOONS IN ALASKA

Location	Design Capacity[1] (gpd)	Actual Flow (gpd)[1] Summer	Winter
Eielson AFB	1,000,000	800,000	800,000
Shemya AFB	332,000	250,000	200,000
Galena Airport	98,000	80,000	64,000
King Salmon Airport	64,000	60,000	50,000
Tatalina AFS	14,000	13,000	10,000
Tin City AFS	13,000	11,000	9,000
Cape Romanzof AFS	13,000	11,000	9,000
Cape Newenham AFS	11,000	11,000	9,000
Cape Lisburne AFS	11,000	11,000	9,000
Indian Mt. AFS	10,000	6,000	5,000
Sparrevohn	7,000	6,000	6,000
Elmendorf AFS	30,000	30,000	30,000

[1]Data provided by AAC 1974.
SOURCE: Reed, 1976.

Activated-Sludge Processes

For the sake of the following discussion, an AS process will be defined as an aerobic biological treatment process in which some sludge is recycled with the remainder being wasted. Typical design parameters for various modifications of the activated sludge process are listed in Table 14. A schematic of a typical process appears in Figure 13. In the aeration chamber, a sizable portion (maybe one half) of the incoming soluble organic matter is converted to low-energy end products such as NO_2^-, $SO_4^=$, CO_2, and H_2O. The remainder is either converted into cellular mass or passes through unaffected. Besides removing as much of the organic matter as possible from the influent stream, it is important that the bacteria form a satisfactory floc so they can be separated from the effluent in the final clarifier. This means the microorganisms must remain in the aeration tank on the average at least several days (Metcalf and Eddy, 1972).

In Table 14, the mean cell residence time θ_c is defined as the average time a cell (microorganism) spends in the aeration tank. As this time increases, the cell yield decreases (Schroeder, 1977). Hence, cell yield can be approximately defined as the fraction of incoming substrate converted into biological cells. The mixed-liquor, volatile suspended solids concentration (MLVSS) provides a measure of the microorganism concentration in the mixed liquor. The food-to-microorganisms ratio (U) indicates how rapidly food is being supplied to the microorganisms. If it is very low, the phenomenon of endogenous respiration will dominate, and there will be a net decrease in cellular mass. The hydraulic residence time $\theta = V/Q$. Here, V is aeration tank volume and Q is flow rate. With no recycling of sludge, $\theta = \theta_c$.

Although there are various modifications of activated sludge processes, we will be concerned here primarily with either the complete mix, oxidation ditch, or extended aeration processes. Although the process schematic is the same for each (Figure 13), the loading parameters and residence times are different as shown in Table 14. In the completely mixed reactor the organic load and oxygen demand are constant throughout the tank as is the microorganism concentration x. Prefabricated package plants used in small treatment plants often utilize an extended-aeration process with its low organic loading and long detention time. Here, the microorganisms are typically in the endogenous

TABLE 14: DESIGN PARAMETERS FOR ACTIVATED-SLUDGE PROCESSES

Process modification	θ_c, days	U, lb BOD_5/lb MLVSS-day	Volumetric loading, lb BOD_5/1,000 cu ft	MLSS, mg/liter	V/Q, hr	Q_r/Q
Conventional	5 -15	0.2 -0.4	20- 40	1,500- 3,000	4 - 8	0.25-0.5
Complete-mix	5 -15	0.2 -0.6	50- 120	3,000- 6,000	3 - 5	0.25-1.0
Step-aeration	5 -15	0.2 -0.4	40- 60	2,000- 3,500	3 - 5	0.25-0.75
Modified-aeration	0.2- 0.5	1.5 -5.0	75- 150	200- 500	1.5- 3	0.05-0.15
Contact-stabilization	5 -15	0.2 -0.6	60- 75	(1,000- 3,000)[1] (4,000-10,000)[2]	(0.5- 1.0)[1] (3- 6)[2]	0.25-1.0
Extended aeration	20 -30	0.05-0.15	10- 25	3,000- 6,000	18 -36	0.75-1.50
Kraus process	5 -15	0.3 -0.8	40- 100	2,000- 3,000	4 - 8	0.5 -1.0
High-rate aeration	5 -10	0.4 -1.5	100-1,000	4,000-10,000	0.5- 2	1.0 -5.0
Pure-oxygen systems	8 -20	0.25-1.0	100- 250	6,000- 8,000	1 - 3	0.25-0.5

[1]Contact unit.
[2]Solids stabilization unit.
SOURCE: Metcalf and Eddy, 1972.

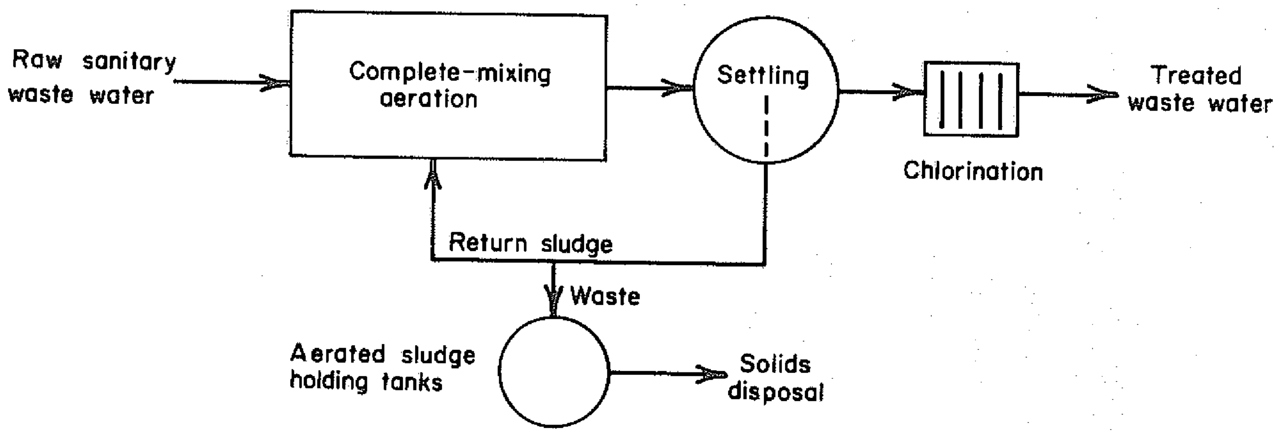


Fig. 13 ACTIVATED SLUDGE PROCESS

respiration phase. Even though some operators try to avoid separate sludge wasting, it should be utilized if the periodic discharge of objectionable solids in the effluent is not wanted. The basic reason for this is that no matter how long the mean resident time, not all the soluble organic matter is converted into innocuous end products. Some cell mass remains and accummulates until the cellular solids are delib-erately wasted.

Originally developed for small towns in the Netherlands, the oxidation ditch is an extended-aeration process that has found wide application in the United States. A schematic of one at College Utilities in Fairbanks is shown in Figure 14. Here, the air is supplied by rotors that agitate the air-liquid interface, and sludge is periodically recycled from the clarifiers (housed inside) back into the oxidation ditch (exposed to ambient). Velocities at which the mixed liquor circulates around the ditch are typically 1 to 2 ft/sec.

As can be seen from looking at Table 11, there are a few municipal activated sludge systems in Alaska for which data on effluent quality is available. According to Crevensten (1977), USEPA monitors effluent quality for the eighteen or so largest municipal plants. For none of these municipal activated-sludge systems is there a complete set of operating data including wastage rates, recycle ratios, and removal efficiencies. Hence, one can not see how well they are modeled by the basic kinetic equations for completely mixed flow with recycle (Metcalf and Eddy, 1972). But, one could calculate overall substrate removal rates from the available data. This will be done for available data collected for Fairbanks, Kenai, and Soldotna using equations of the form

$$S/S_o = (1 + K_1\theta)^{-n} \tag{1}$$

where S/S_o is the ratio of BOD_5 or SS in the effluent to that in the influent, θ is the hydraulic residence time, n is the number of aeration chambers in series, and K_1 is the substrate removal rate coefficient. Although this equation is typically applied to lagoons without recycle, we will apply it here to the available data. Hence, K_1 will provide an indication of how fast substrate is being removed for a given θ. But, the differing θ_c are not taken into account.

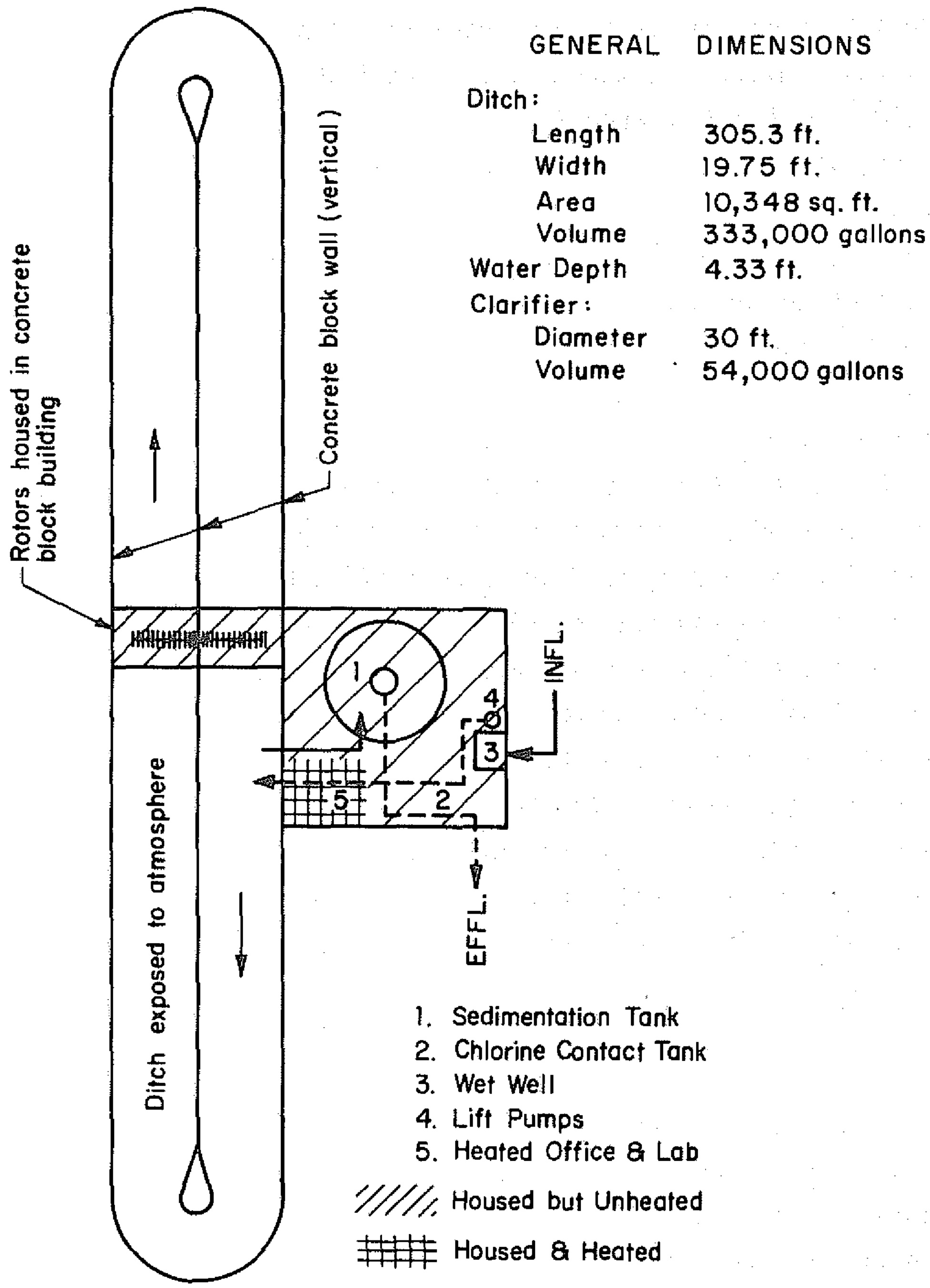


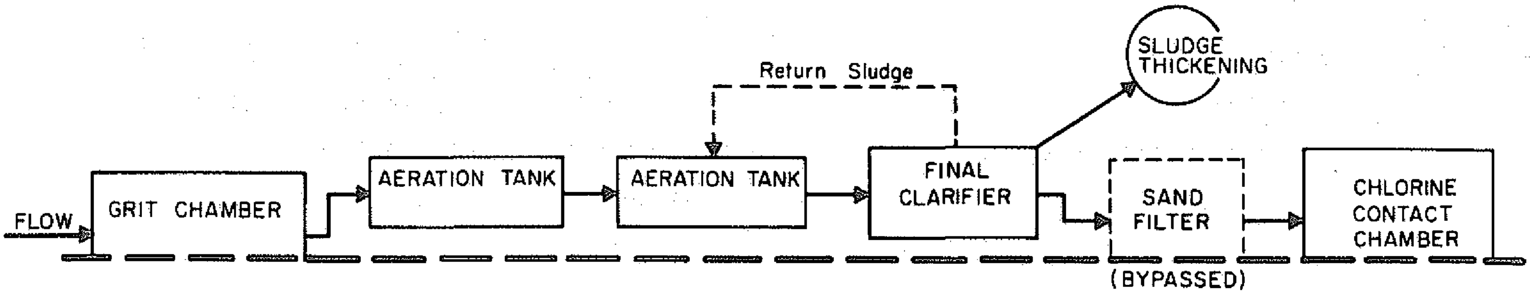
Fig. 14: COLLEGE UTILITIES CORP. OXIDATION DITCH DETAILS.
(Murphy and Ranganathan, 1974)

Kenai and Soldotna--

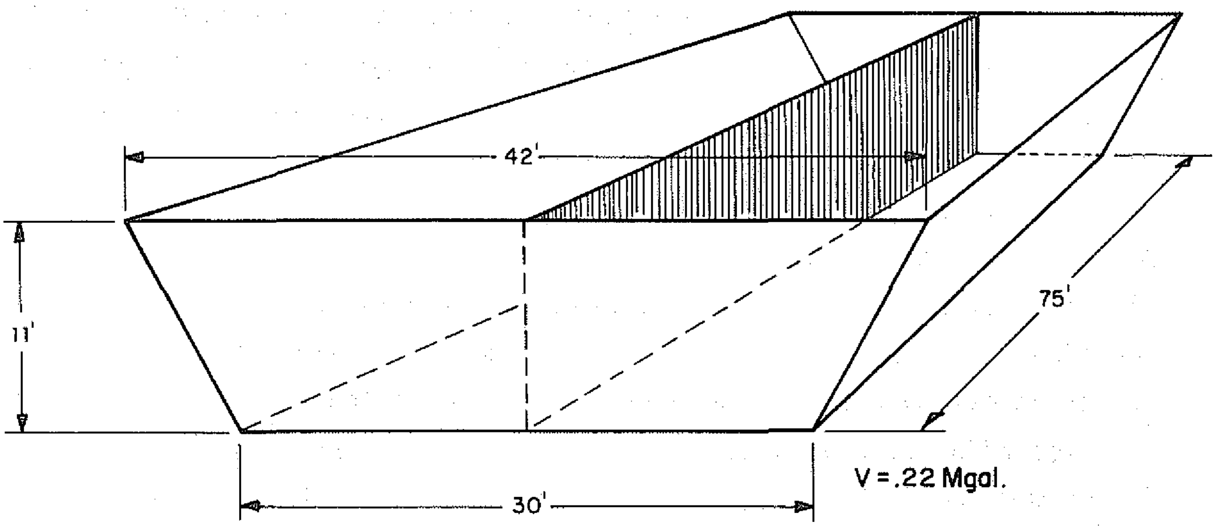
The Kenai plant, designed by Wince Corthell and Associates, is very similar to the Soldotna plant, the latter being about one half as large (Carnegie, 1976). Although originally designed for effluent polishing via a sand filter and sludge dewatering on drying beds, the filter is now bypassed and the excess sludge from Kenai is now hauled to a dump. With the volume of each aeration tank being about .22 Mgal (Figure 15) and a typical influent flow rate being .45 Mgd (Table 15), the hydraulic residence time θ is on the order of one day. The surface overflow rate (SOR) on the final clarifiers is about 400 gal/ft^2/day. This is well under a typical SOR of around 1K gal/ft^2/day for a secondary clarifier. Similarly, θ is high for a conventional AS process even though it is low for an extended-aeration process. Hence, from the viewpoint of loading rates, one is not surprised that the plant effluent often meets secondary standards. Since no data are available on sludge wastage, mean cell residence times cannot be calculated. Sludge wasting is accomplished on a batch basis by carting away every three months or so. The sludge is extracted from the final clarifiers using air lifts. Using Equation (1) with $S/S_0 = 35/104$, $n = 2$, and $\theta = .5$ days for once cell, we can calculate a substrate removal rate coefficient $K_1 = 3.3$ days^{-1}.

According to Crevensten (1977), effluent samples are taken at Kenai in the morning. Thus, they do reflect the solids carry-over that sometimes occurs during the higher flow period in the afternoon. Moreover, the gas-fired blowers have not proven reliable with shut-downs periodically occurring. USEPA has been urging Kenai to install electrically-driven blowers.

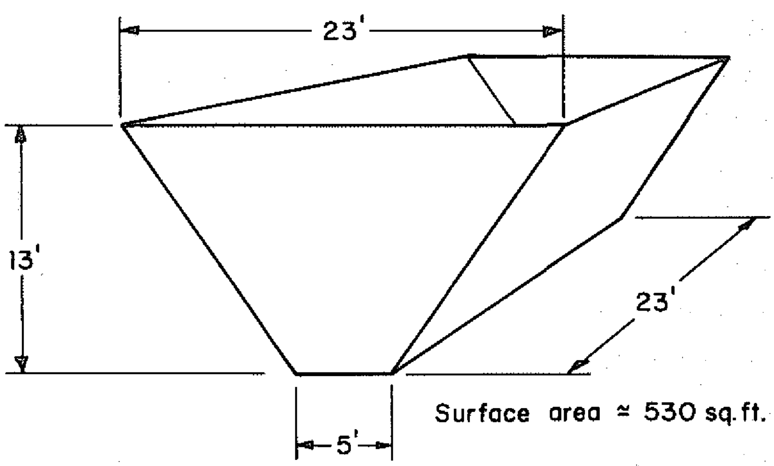
Using the data appearing in Table 16, with $S/S_0 = 22/164$, $\theta = .5$ days, and $n = 2$, we find $K_1 = 2.6$ days^{-1} for Soldotna. Here, the two aeration basins operate in series as in Kenai. Bulking was reported to have occurred in the Soldotna plant in February of 1976, but the problem was corrected shortly thereafter (Mills, 1976). Even though quantitative sludge wastage data were not available, Mills reported that most of the sludge is recycled to the aeration basin. The resultant low sludge wastage probably pushes the mean cell residence time θ_c into the extended-aeration range. The mixed liquor was brown in appearance on the day of our visit. This is indicative of good performance. Since both Kenai



(a.) OVERALL SCHEMATIC (one half of plant shown, mirror image of above for other half)



(b.) AERATION TANK (2 CELLS)



(c.) FINAL CLARIFIER

Fig. 15: KENAI SEWAGE TREATMENT PLANT

TABLE 15: KENAI PERFORMANCE DATA

Date 1976	Flow (Mgd)	BOD_5 (mg/l) inf[1]	BOD_5 (mg/l) eff[2]	SS (mg/l) inf	SS (mg/l) eff	Liquid Temp (°C)
March 4	.65	139	21	260	10	10
March 17[3]	.44	124	80	132	90	
May 13	.45	236	44	75	13	6
May 14	.45	111	24	59	1	6
September 15	.45	207	14	154	7	15
September 16	.45	63	4.5*	80	2	15
Average		164	35			

[1]influent BOD_5 values do not include septic tank contents which are occasionally dumped.

[2]all effluent BOD_5 values before chlorination except *.

[3]from EPA survey on March 17, 1976.

TABLE 16: SOLDOTNA PERFORMANCE DATA[1]

Date 1976	Flow (Mgd)	BOD_5 (mg/l) inf	BOD_5 (mg/l) eff	SS (mg/l) inf	SS (mg/l) eff	Liquid Temp (°C)
March 15	.31	108	15	76	14	7
May 20	.27	209	23	93	7	6
July 17	.15	185	49	114	1	11
September 16	.13	152	16	161	5	11
September 20	.15	168	7	261	4	11
Average	.20	164	22	141	6	9

[1]Recreational vehicles allowed to dump wastewater here.

and Soldotna have only one compressor, the oxygen input to the aeration chambers decreases each time the airlift is operated. The electrically-driven blowers at Soldotna have proven much more reliable than the gas-fired ones at Kenai (Crevensten, 1977).

Fairbanks--

The largest municipal AS plant in Alaska is located at Fairbanks. As shown in Figure 16, after entering the treatment plant through a 48-inch gravity sewer, the raw wastewater from Fairbanks flows through two mechanical bar screens each equipped with a shredder. Then, it passes through one of four mechanically cleaned, aerated-grit chambers. The grit is now transferred to either a sludge storage bin or trucks for disposal. At the present time, the wastewater is being next sent to the north aeration train only where four cells connected in series allow the wastewater to be aerated. The treated liquor then flows into one of the five secondary clarifiers currently being used (there are a total of eight). After passing through one of four chlorine contact chambers, the effluent flows by gravity into the Tanana River.

Some of the sludge from the secondary clarifiers is pumped into the digestor, but the majority is recycled into the aeration train. The digested sludge then is pumped into the sludge thickener and then to one of four dewatering units (all of which are used) ahead of which a polymer is added to aid in dewatering. Typically these units allow a solids content of 12% to be attained and are simply rotating screens that allow water to trickle through the mesh while retaining the sludge cake. This sludge and scum from the secondary clarifiers is then conveyed to trucks. The primary sludge from the grit chambers goes directly to the trucks. The thickener overflow goes back to the head end of the plant.

The Fairbanks Sewage Treatment Plant, which employs a complete-mix AS process, uses pure oxygen instead of air. The aeration tanks are covered with a pure-oxygen atmosphere maintained in the volume between the mixed liquor and cover. The CO_2 that is continually being produced by the microorganisms is displaced by fresh oxygen which is continually introduced into the aeration trains. Advantages claimed for this system are a reduced reactor volume (because a higher microorganism concentration--≈5K mg/l--can be maintained in the reactor) and increased sludge

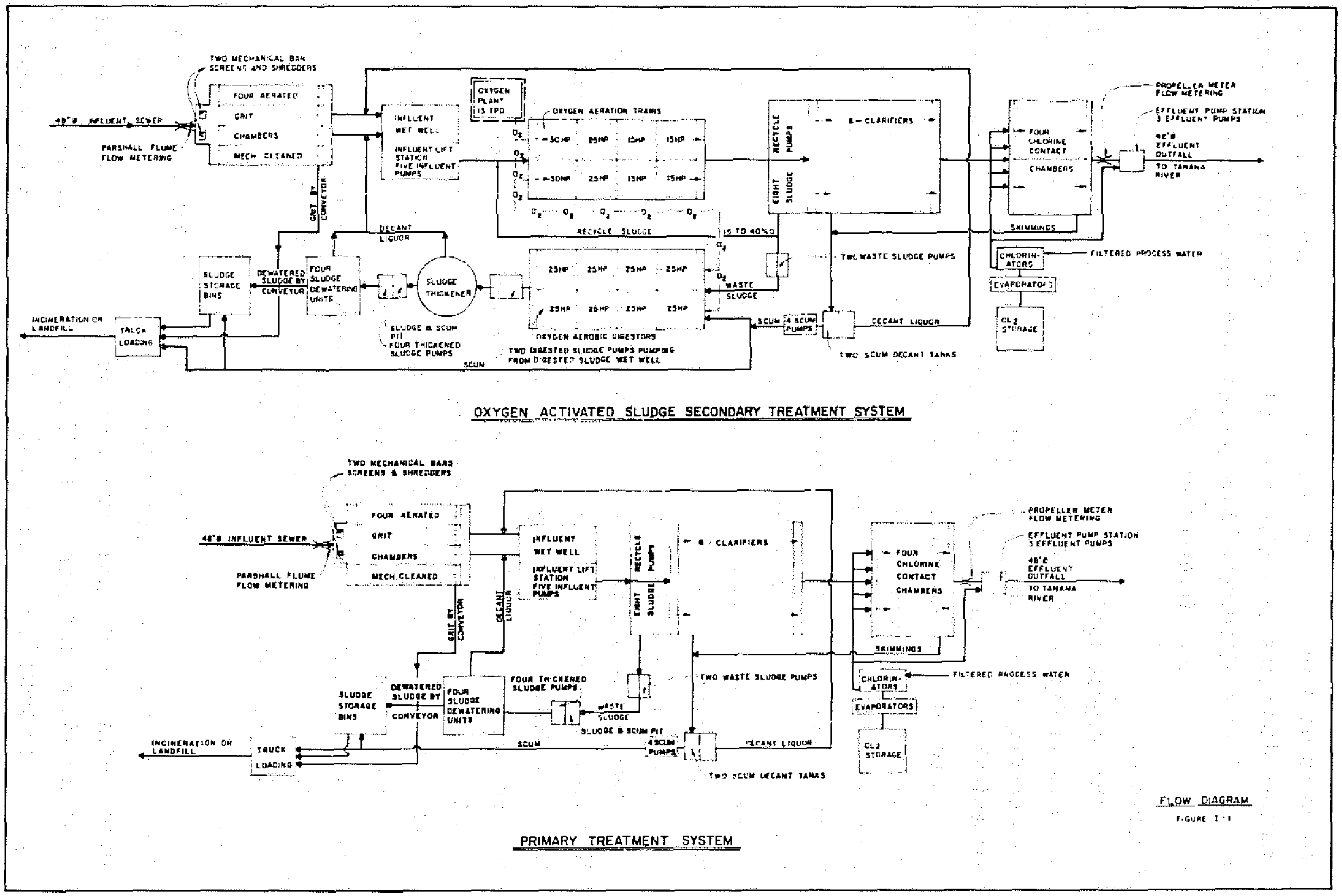


Fig. 16: FAIRBANKS SEWAGE TREATMENT PLANT

settleability (Kalinske, 1976). Chapman et al. (1976) claims the higher dissolved oxygen (DO) levels associated with the oxygen allow higher F/M ratios while still achieving the same effluent quality. McKinney (1975) claims that differences in results between pure-oxygen and conventional systems is basically the result of improperly designed and operated air-activated sludge systems. He implies these differences would result even if pure oxygen were not used. Kalinske (1976) concludes that air and oxygen activated-sludge systems are generally comparable with respect to solids settleability, effluent quality, and waste sludge production. He maintains that the process is not enhanced in any way by maintaining oxygen levels above 2 mg/l. Moreover, he states that both capital and operating costs are less for air activated-sludge plants. Parker and Merrill (1976) offer critiques of both Chapman's and Kalinske's reports. (It should be mentioned that Chapman et al. are employed by Union Carbide who is the largest supplier of pure-oxygen systems). They point out that oxygen systems can cope better with unexpected increases in organic loadings because of their abilities to operate at higher F/M ratios. But, they think this difference may be explained by better diffuser design with pure-oxygen systems. They do agree with Kalinske that there is no evidence to suggest that DO levels higher than 2 mg/l are necessary. Conversely, if DO levels fall much below 2 mg/l, performance can suffer. They argue, as does McKinney, that erroneous conclusions are too often drawn by comparing a well-operated, pure-oxygen system with an inadequate air system such that the DO level is less than 2 mg/l in the latter. They concur with Kalinske that air- and oxygen-activated sludge settleability will be comparable providing DO levels above 2 mg/l are maintained.

With saturation levels of DO on the order of 9 mg/l with an air atmosphere maintained above the mixed liquor, it is obvious that oxygen is not ordinarily a rate-limiting material. Some investigations support the argument that 2 mg/l is adequate to ensure floc penetration (Kalinske, 1976). Others (Chapman et al., 1976) state that critical DO levels may sometimes need to be as high as 5 mg/l depending on floc size. But, at the present time, further study is needed in this area. If higher microorganism concentrations necessitate higher oxygen levels, it is plausible that use of pure oxygen can help keep the levels high. This is because the rate at which oxygen can be transferred from a gas to a liquid is proportional to $(C_s - C)$ where C_s is the saturation concentration and C is the actual concentration of oxygen in the mixed liquor. Since C_s can be increased by a factor of 5 by using a pure oxygen atmosphere, it is clearly much easier to maintain a high C.

In the Fairbanks plant, the atmosphere is not pure oxygen but is highly enriched with oxygen (over 90% as it enters the first aeration cell) at a pressure slightly greater than one atmosphere. By the time these gases reach the fourth cell, the oxygen content has decreased to 40%. The system used is the Unox process developed by Union Carbide Corporation. The oxygen enriched atmosphere is created by injecting air at 40 PSI into granular beds of beads that absorb nitrogen. Hence, the air leaving the beds is enriched in oxygen. Regeneration is accomplished by lowering the pressure and passing air through the beds.

With only the north train, which has a total liquid volume of .36 Mgal, divided equally among four cells currently being used, the hydraulic residence time is typically about 2.2 hours. Using Equation 1 with $n = 4$ and $S/S_0 \approx 34/200$, the coefficient $K_1 \approx 5.6$ days^{-1}. This is much higher than the rate coefficients calculated for the Kenai and Soldotna AS treatment plants. Presumably, one important reason for this difference is a more efficient system at the Fairbanks plant with relatively young microorganisms (a mean cell residence time in July 1977 of 3.6 days was estimated by this author). By September, 1977, this had been increased to seven days by decreasing the sludge wastage rate.

Mass balances for solids performed on the clarifier, digester, the thickener, and dump truck system indicate that about 15K lbm/day of solids were autowasted into the digestor in the summer of 1977. If it is assumed that 40% of the solids are volatilized in the digester then 9K lbm/day enter the thickener. The dump trucks haul away about 5K lbm/day. The missing 4K lbm/day is accounted for by the thickener overflow which is recycled back to the head of the plant. This is consistent with the MLVSS concentration in the overflow of 17K mg/l with a flow rate of 30K gpd. It is believed that such high solids concentrations are caused by denitrification in the thickener carrying these solids to the surface.

Performance curves appear in Figure 17 and monthly averages for BOD_5, SS, flows, and temperature are listed in Table 17.

Even though the Fairbanks plant has been producing a reasonable effluent (BOD_5 slightly lower than the 25 mg/l permitted for April through July, 1977), there have been operational problems concerning sludge disposal. Initially, the seal between the doors on the bottom of the sludge storage hopper leaked when the sludge depth in the hopper was greater than 5 feet. Hence, the sludge had to be dumped directly into a truck without using a screw conveyer system which would increase the sludge solids fraction from 12% to 15%, say, in transporting the sludge to the top of the hopper 40 feet above. Moreover, the screw conveyer would only work when the door at the bottom of the feed sump was open. Hence, liquid from the sludge would spill out onto the floor (O'Neil, 1977).

Another operating problem was associated with the plant's design not being conducive to independent operation of the two aeration trains. The Unox system senses the average pressure in both aeration trains and supplies oxygen accordingly. This created problems with respect to disposal of the residual sludge from the drained aeration chambers and digester in the south train. In neither case was provision made for complete sludge removal other than by manual means once the liquid was drained. But, plant superintendent James O'Neil wouldn't let his people enter these tanks without safety harnesses and breathing apparatus which the plant didn't have in the winter of 1976-77 when this trouble arose. Because of the heat balance, it was undesirable to let the outside air help in the venting process during the winter. So, O'Neil continued to force oxygen into the south train to prevent development of anaerobic conditions. By continually purging the south train with water, O'Neil had managed to dilute the solids to <1K mg/l by May of 1977. After drainage, the remaining solids will be manually removed using a hose.

One feature of this plant that deserves mention is the use of heat exchangers to recover heat from the incoming sewage to heat the plant. With a feed temperature of around 15°C allowing a temperature drop of about -7°C before discharge into the Tanana, there is a potential daily heat supply of 650 MBtus for the 4 Mgpd flow. This is well in excess of

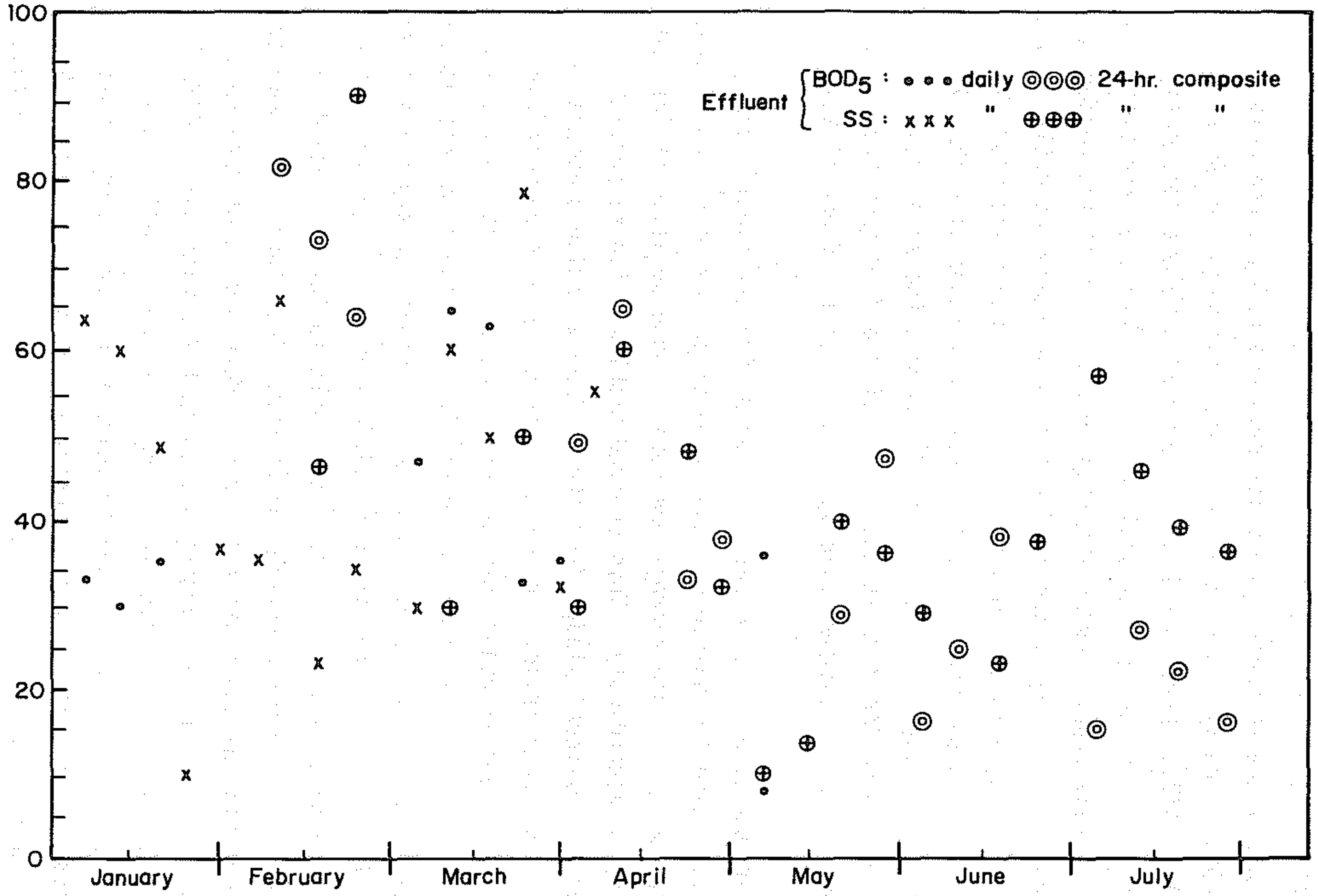


Fig. 17 PERFORMANCE DATA FOR FAIRBANKS SEWAGE TREATMENT PLANT

TABLE 17: FAIRBANKS PERFORMANCE DATA[1]

Month 1977	Flow (Mgd)	BOD$_5$ Inf	BOD$_5$ Eff	SS Inf	SS Eff	Liquid Temp (°C)
January	3.48		(32)		(37)	14.7
February	3.67		73		68	14.9
March	3.93	240	45	126	40	15.1
April	4.15	187	33	192	44	14.8
May	4.3	174	33	157	25	
June	5.3	205	26	165	30	
July	4.6	107	20	154	44	
April-July Average.		169	24	168	35	

[1]All numbers are monthly averages derived using 24-hour composite samples unless denoted by ( ), which denotes grab samples.

what is needed even on the coldest winter day and prevents the plant from adding further to the ice fog problem in Fairbanks. The heat from the wastewater is transferred to air circulating through the plant using glycol and Freon as heat exchanging media. This heat pump concept allows the 16°C sewage to heat the air to room temperature.

One operational cost that O'Neil is trying to reduce is the $800 per week currently being spent on a high molecular weight cationic polymer. Currently, a dosage of 200 mg/l is used on the sludge going to the dewatering screens from the thickener. Jar testing was used to arrive at this dosage. O'Neil is now (August, 1977) experimenting with other polymers to try to reduce these costs.

One type of extended aeration process used in Alaska is an oxidation ditch owned and operated by College Utilities in Fairbanks, Alaska. A study conducted on this plant in 1967-68 (Murphy and Ranganathan, 1974), concluded that the BOD_5 and solids removal efficiences were better than 90% at that time. At that time, the plant was loaded to only 50% capacity while treating wastewater from the University of Alaska, a primary school, and some residential apartment areas. At an average flow of 172K gpd, the hydraulic detention times were about 2.3 days in the ditch and eight hours in the clarifier (Grube and Murphy, 1969). Using this time and an average BOD_5 removal of 92%, one can use Equation 1 to calculate K_1 = 4.8 $days^{-1}$ with n = 1. The average influent strength was 289 mg/l with effluent BOD_5s less than 30 mg/l consistently achieved.

Two cage rotors 27.5 inches in diameter and 13 feet long, each driven by a 7.5 hp motor, provided the mixing and aeration energy. This amounts to only .05 hp/1K ft^3 of aeration volume. The raw waste is pumped into the ditch after passing through a bar screen. The return sludge drained into the wet well from the clarifier was pumped into the ditch along with the raw wastewater. The effluent from the 30' diameter settling tank is chlorinated before being discharged into the Chena River. At the average flow quoted above, the SOR is 243 gal/ft^2/day. Hence, the clarifier was nowhere near being hydraulically overloaded.

It should be mentioned that the quoted BOD_5 removals are less than actual since the septic tank wastes intermittently discharged into the ditch were not incorporated into the measured influent BOD_5s. Probability plots for the influent and effluent BOD_5 and SS levels are shown in Figures 18 and 19. Here the ordinate represents the percentage of the time during which the value is below that indicated. The fairly high temperatures of the incoming wastewater (typically at least 10°C) helps explain the good performance of this outside oxidation ditch in this northern climate. The high temperature, in turn, is caused largely by many sewers in this area being located in utilidors.

With no intentional sludge wasting being practiced, high SS in the effluent occasionally occurred. Sometimes the sedimentation tank was overloaded because the return sludge line became clogged with solids. This, of course, resulted in solids being carried over into the effluent. The situation has since been rectified by installing a sludge pump so that sludge drainage doesn't have to rely on gravity alone.

A sheet of frozen material about 10 feet long was noticed in the ditch in January of 1968. Its presence can be explained by a large surface area caused by foaming. This ice sheet grew to a length of 70 feet by April with a thickness of one foot. The low mixed-liquor solids level during the winter and subsequent increase in the summer were thought to be caused by their incorporation into the ice structure and its subsequent melting.

Detailed measurements of the spatial distribution of solids concentrations and DO throughout the ditch indicated that much internal sedimentation was occurring in the ditch. Hence, the mixed liquor was not completely mixed in the true sense of the word. This deposited sludge resulted in the release during the summer of floating solids. These solids did not settle readily in the clarifier and, hence, the quality of the effluent was lowered. This failure of the aeration to keep the sludge in suspension is consistent with the low calculated mixing power levels of only .05 hp/1000 ft^3. However, the College Utilities ditch had under 13K gal/ft of rotor length. This is conservative with respect to the normal standard design of 16K gal/ft. It appears that, with this ditch geometry, a standard design is not conservative enough.

The study by Murphy and Ranganathan (1974) concludes that an oxidation ditch can perform very well in the subarctic. They conclude that low temperatures do not inhibit the biological processes. However, the

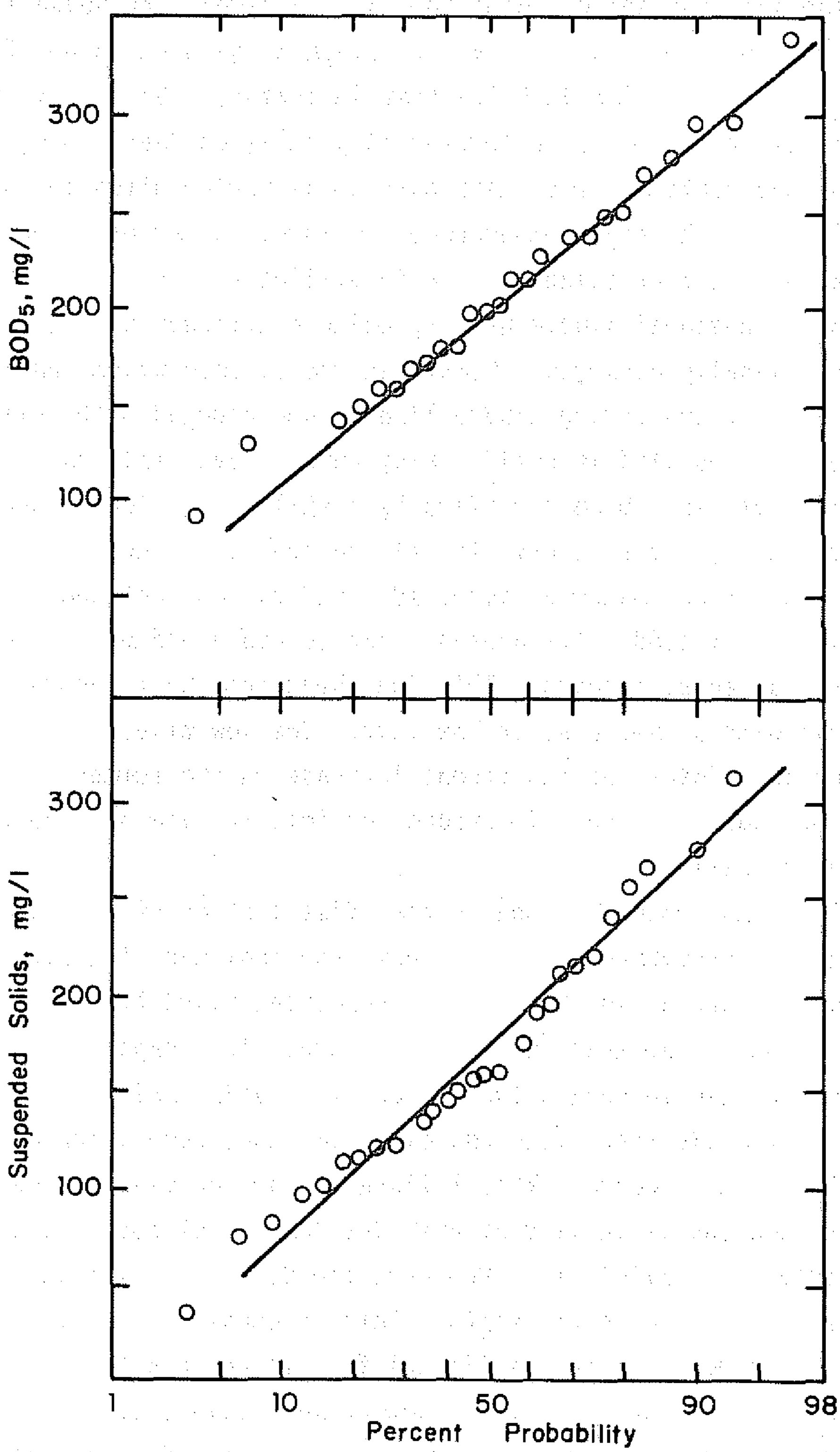


Fig. 18: OCCURRENCE PROBABILITY OF INFLUENT BOD_5 AND SUSPENDED SOLIDS. (Murphy and Ranganathan, 1974)

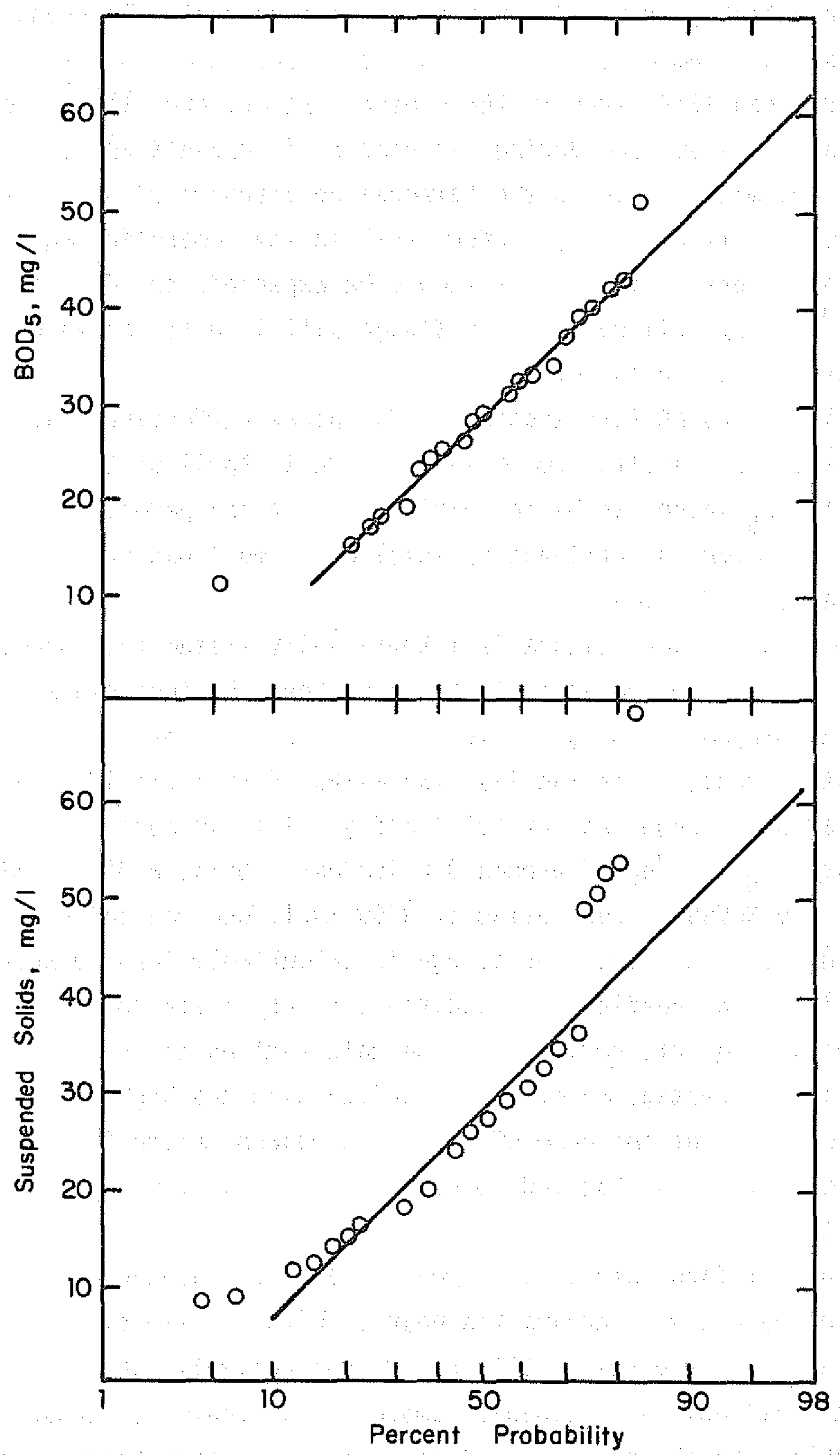


Fig. 19: OCCURRENCE PROBABILITY OF EFFLUENT BOD_5 AND SUSPENDED SOLIDS. (Murphy and Ranganathan, 1974)

limited data presented in the paper indicate the influent temperature was relatively constant year round. Subsequent measurements (Townsend, 1977) indicate the temperature of the mixed liquor entering the clarifier to be greater than 17°C, even in the winter. Hence, even though some ice formed near the surface during one winter, it doesn't appear that the bulk of the mixed liquor has a temperature anywhere close to freezing, indicating that this ditch may perform well in the subarctic because of warm influent waters. Finally, as should be expected, the operating data indicates that failure to waste sludge will lead to carryover of objectionable solids in the effluent.

According to USEPA (Crevensten, 1977), plant performance was poor over much of 1975. Results from a survey made in April of 1976 revealed the effluent BOD_5 values to be in compliance with the permit limitations (<30 mg/l). However the effluent SS level of 83 mg/l was well above the permitted value of 30 mg/l.

At the present time, sludge is intentionally wasted (Townsend, 1977). But not enough operational data have been obtained since the pumper trucks stopped dumping septic tank wastes into the ditch to quote average wastage rates. For the last two weeks of April in 1977, sludge was wasted at an average rate of 150 lbm/day. This is much less than the influent BOD_5 loading of around 800 lbm/day. Using a ditch volume of .33 Mgal and MLVSS concentration of 3700 mg/l, one can calculate a sludge age of 67 days. The average age is undoubtedly less because of solids leaving with overflow. In addition, steady-state conditions were not obtained during this period so these calculations are very crude. However, this low wastage rate, in comparison with the higher loading, is consistent with the increase of the MLVSS concentration in the mixed liquor. The latter was observed to occur during this time period (Townsend, 1977).

At a current flow rate on the order of 1 Mgpd, the hydraulic detention time in the ditch is around ten hours. With the two clarifiers now being used having diameters of 30' and 40' respectively, the SOR is about 300 gal/ft^2/day, a reasonable number. Nevertheless, Townsend is having trouble with the sludge settling in the clarifier because of filamentous growths which have led to bulking sludge. An attempt to kill these growths by adding hydrogen peroxide was unsuccessful.

Chemical analysis has shown adequate nutrients to be present and DO levels are greater than 2 mg/l in the ditch. Hence, it is difficult to see what is causing the filamentous growths. In spite of this problem, the effluent quality was good during runoff--a condition that may have been aided by the introduction of silt into the plant; the silt may be functioning as a coagulant aid.

Juneau--

Performance data for the Juneau-Douglas plant for a one-year period ending in June of 1977 appears in Table 18. In terms of monthly averages, minimal BOD_5 and SS removals of 93% and 87% respectively were attained. In spite of fluctuations of up to a factor of six in average daily flows, high BOD_5 and SS removals were achieved and good effluent quality attained. The treatment consists of grit removal followed by aeration, clarification, chlorination, and discharge into the bay via an outfall pipe. The sludge from the grit removal chambers and digestors is dumped into two nearby ponds. With the total aeration volume being 1.5 Mgal, the hydraulic residence time is quite long in the aeration basins. Since sufficient data on sludge wastage was not available, it was not possible to specify mean cell residence time. However, attempts are made to waste as little sludge as possible to prevent the MLVSS from falling to excessively low levels (Crevensten, 1977). Hence, the plant is operating in an extended-aeration mode (Kelton, 1977). Nitrification has been observed to occur in the final clarifiers (McCauslend, 1977). This is consistent with long detention times. Data on solids concentrations in the wasted sludge would help clarify this point.

With both 65-foot diameter clarifiers in operation, the SOR at a flow of 2 Mgpd is only around 300 gal/ft^2/day. Even with a storm-related flow of 6 Mgpd, the clarifiers are still not overloaded. But, one problem which still exists is the flushing of BOD_5 and SS through the system after heavy rainfalls (Shira, 1977). With the large annual rainfall experienced in Juneau (over 100 inches), these uncontrollable hydraulic fluctuations are a consequence of combined sewers coupled with the absence of an upstream holding basin. According to Shira (1977), the current plan is to phase out slowly the combined sewers over at least a 10-year period. Because of these large fluctuations, Shira wastes less sludge at high flows since the storm runoff then automatically flushes solids through the system. Another operational problem

TABLE 18: JUNEAU-DOUGLAS PERFORMANCE DATA[1]

	Flow Mgd		BOD_5		SS		% Rmvl		Electrical Usage
Month	Average	Range	Inf	Eff	Inf	Eff	BOD	SS	KW-HR
Jul 76	1.4	1.0-2.2	131	4	97	10	96	89	73K
Aug 76	1.5	1.1-2.6	135	5	134	10	96	93	74K
Sep 76	2.3	1.0-6.0	165	4	126	7	98	94	73K
Oct 76	2.3	1.4-5.1	156	3	99	7	98	93	76K
Nov 76	2.1	1.3-4.4	163	6	114	4	96	89	75K
Dec 76	2.0	1.2-3.7	97	6	88	5	94	94	77K
Jan 77	1.8	1.1-4.5	170	6	102	7	96	93	76K
Feb 77	2.2	1.5-3.8	113	5	72	8	96	89	76K
Mar 77	1.7	1.0-3.7	97	5	78	9	95	88	75K
Apr 77	1.6	1.1-2.5	55	4	69	9	93	87	73K
May 77	1.2	.9-2.9	148	9	132	15	94	89	72K
Jun 77	1.4	.9-2.9	100	6	130	12	97	92	68K
Average			127	5	103	9	96	94	

[1]Average liquid temperature = 15°C July, 3°C January.
All numbers monthly averages unless noted otherwise.
Influent BOD_5 ranged from 42 to 170 on daily basis.
Both aerators operated for this data.

is the backing up of salt water into the system at high tides. This has not resulted in any noticeable decrease in microorganism population, but large surges of dilute wastewaters into the system have caused the microorganism population in the mixed liquor to stay below 2K mg/l.

The idea of dumping secondary sludge into earthen ponds does not seem applicable to Juneau. The cool, moist climate does not give the sludge a chance to dry. On the day of my visit (July, 1977), front-end loaders were removing the accummulated sludge from the ponds. The purchase of additional sludge dewatering equipment is now being considered (Shira, 1977).

Another Juneau AS plant at Mendenhall is grossly overloaded with flows of the order of 1 Mgpd (Crevensten, 1977). A portion of the influent is almost continuously bypassed. Hence, even though the portion that is treated meets acceptable standards, the overall effluent quality is poor.

The final clarifiers in the five AS processes discussed here in some detail are all housed in buildings. But since the aeration cells were located outdoors, with the exception of Fairbanks and Juneau, the possibility of cold liquid temperatures in the clarifiers exists. For discrete particle settling at low Reynolds numbers, the influence of temperature on settling velocity can be calculated from Stokes Law (Metcalf and Eddy, 1972).

$$v_s = \frac{\text{"g"}}{18} \frac{d^2}{\nu} (\rho_s - 1) \qquad (2)$$

Here g is the acceleration due to gravity, d is particle diameter, ν is kinematic viscosity, and ρ_s is the specific gravity of the solid. The inverse relationship between settling velocity and viscosity exhibited in Equation 2 indicates the settling velocity to be almost a factor of two slower at 0°C than 20°C. Hence, for clarifiers operating at cold temperatures with dilute suspensions of solids, the surface overflow rate should be decreased by a factor of two compared to that required for a room-temperature application.

However, these same conclusions do not apply to low-temperature settling of concentrated suspensions of solids (Reed and Murphy, 1969). In this case the mutual interference of the solids cause hindered, rather than discrete, particle settling to occur (Maude and Whittemore, 1958). Data presented by Reed and Murphy indicate the reduction of "zone" settling velocity due to temperature to be minimal at initial particle concentration $\geq$6K mg/l. In fact, at 5K mg/l, there was only a 25% reduction in settling velocity between 18°C and 1°C. Hence, if one operates clarifers with MLSS levels $\geq$5K mg/l, one does not have to be overly concerned with providing larger areas for clarification at low temperatures. Moreover, Reed and Murphy (1969) tentatively concluded that sludge thickening is independent of fluid temperature.

Screening operations can be affected adversely by icing and/or the formation of frazil ice in the channel following the screen. Icing can be prevented by heating the screen. These problems have not occurred in the large municipalities just discussed because of the warm influent temperatures. Interestingly, the processes of skimming to remove oil and grease and coagulation can be enhanced by low temperatures (Alter, 1969).

At the Pitkas Point VSW facility, sewage is treated by four 1K gpd Multi-Flo units connected by a splitter box. Treated effluent is discharged to a leaching field. The Koyukuk VSW facility (under construction) will have a similar set up with the treated effluent being discharged to an intermittently flowing stream (Sargent and Scribner, 1976).

One military installation visited during the course of this study is the extended-aeration package plant at Murphy Dome (Figure 20). With a 34K gal aeration tank and a typical flow around 18K gpd, the hydraulic detention time in the clarifier is almost two days. The SOR on the final clarifier is only around 100 gal/ft^2/day. By either criteria, the system is not overloaded. Data was not available on MLVSS or sludge wastage so the mean cell residence time could not be deduced. The aerators are of the "sock" type with the DO measured at 1.5 mg/l on the day of my visit (10/28/76). The sludge is wasted only periodically. The inside of the building enclosing the plant was oppressively humid because of inadequate ventilation. The day I observed it, the overflow from the final clarifier appeared clear and the mixed liquor was brownish in color. Performance data since then reveals Murphy Dome to be operating satisfactorily with BOD_5 and SS levels in the effluent consistently <30 mg/l (Drum, 1977).

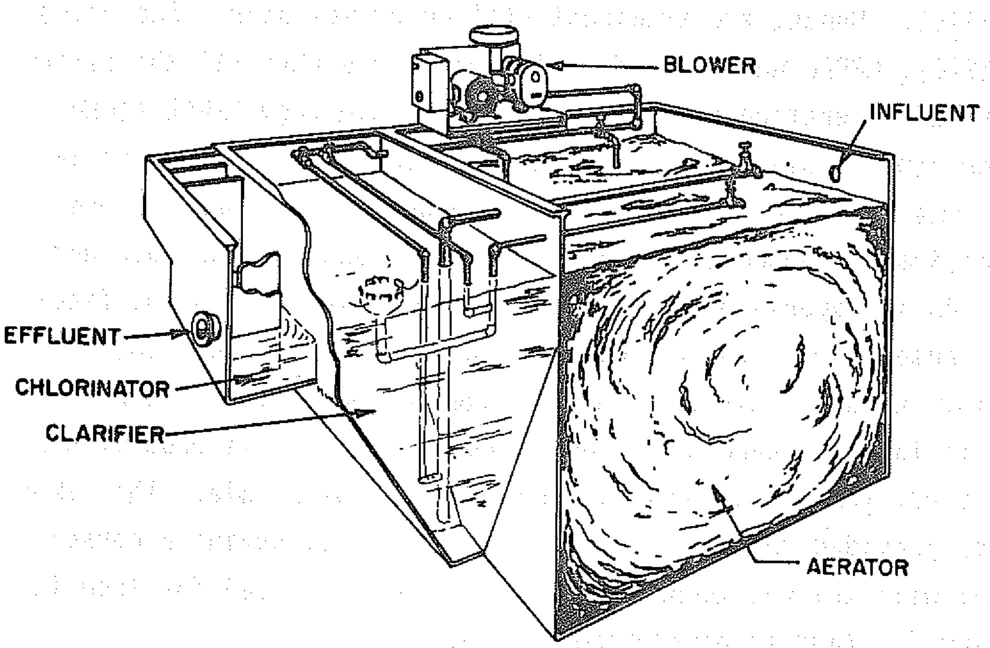


Fig. 20: EXTENDED-AERATION PACKAGED SEWAGE TREATMENT PLANT. (Steel Fabricators, Anchorage, Alaska)

For many months prior to the fall of 1976, the effluent contained high BOD_5 and SS levels. Data from July, 1976, indicated effluent BOD_5 levels in excess of 100 mg/l with influent levels averaging 135 mg/l (Thayer, 1976). Hence, the treatment efficiency was poor. According to Reed (1978), CRREL was assured by the Air Force that all the system components were operating properly. Hence, people from CRREL began to look for other explanations for the poor performance. One initial hypothesis was that the aeration tank was underloaded. But, before CRREL investigated further, Thayer located the cause of the poor performance. An improperly operating valve on the air lift in the final clarifier caused air or water to be forced down, rather than up, the lift. This, in turn, stirred up the sludge on the bottom of the clarifier producing solids carryover with the effluent. Since this problem has been rectified, the plant performance has been commendable. This short history was provided to illustrate the importance of having a conscientious and well-trained technician (Thayer in this case) for trouble shooting when a plant is performing poorly.

Other USAF extended-aeration package plants are located at Campion, Kotzebue, and Cold Bay. Their designs are similar to the unit at Murphy Dome and their performances are acceptable (Drum, 1977). At all of these, the sludge is withdrawn periodically. The most complete performance data is that available for Murphy Dome.

Lagoons

Aerated lagoons are frequently used in Alaska with at least twenty in use today (Reid, 1975). Here, mechanical or diffused-air aeration is used to supply oxygen with the process basically being an activated-sludge process without recycle. These facilities are profoundly affected by temperature because of their outside locations and high surface-to-volume ratios. In Alaska, special care is taken to build lagoons sufficiently deep that, even if the surface freezes in the winter, there will be sufficient volume under the ice to store the winter's sewage. For the facilities listed in Table 11, Eagle River, Homer, and Palmer have aerated lagoons. Operating results indicate that only the Palmer lagoon consistently meets the 1977 standards (Reid, 1975, and Christianson, 1976).

Christianson (1976) performed a study on cold-climate aerated lagoons in which operating data from lagoons in Alaska, Canada, and the northern tier of the lower 48 were incorporated. He pointed out that the lagoons built in Alaska are generally unable to meet 30/30 standards although they operate reasonably well. Alaskan lagoons studied in some detail are those located at Eielson Air Force Base (EAFB), Northway, Ft. Greely, Eagle River, and Palmer. The EAFB experimental lagoon had a volume of 14.8K ft^3, a total detention time of 30 days, and was operated using either four or six cells in series. After the fine-bubble diffusers were replaced with coarse-bubble diffusers, the lagoon and aeration system operated well over the subsequent two-year period. As shown in Figure 21, the winter and summer BOD_5 and SS removals were generally better for the six-cell than for the four-cell operation. This is consistent with the multiple reactor theory that assumes each reactor has CMF (Metcalf and Eddy, 1972). For a given rate coefficient K_1, this is exactly what Equation 1 predicts. Summer removals are not always thought better than winter removals because of the presence of algae in the summer effluent.

The two principal cells of the Fort Greely lagoon were operated in parallel for a one-year period. As shown in Figure 22, air was supplied to one cell via Chicago Pump Shearfusers and to the other via Aer-O-Flo diffusers. The effluent quality from either cell was similar with results for the Aer-O-Flo side shown on Figure 23. A significant feature here is the low BOD_5 levels for the filtered effluent. This illustrates the potential for good effluent quality with removal of algae. For the Northway lagoon, the overall BOD_5 removal over a five-year period was 86% with the detention time averaging 40 days. The influent was domestic sewage with temperatures varying from 2°C to 24°C.

The Eagle River and Palmer lagoons both treat domestic wastewaters with each having two separate cells. The former has performed poorly partially because the diffusers have been clogged continually. The Palmer setup consists of an aerated lagoon followed by a polishing pond with volumes of 868K ft^3 and 961K ft^3 and depths of 8.8 ft and 4.9 ft respectively. Operating results over a three-year period indicate a generally excellent performance (Figure 24). Data obtained over a three-month period in 1973 indicate that the aerated lagoon itself

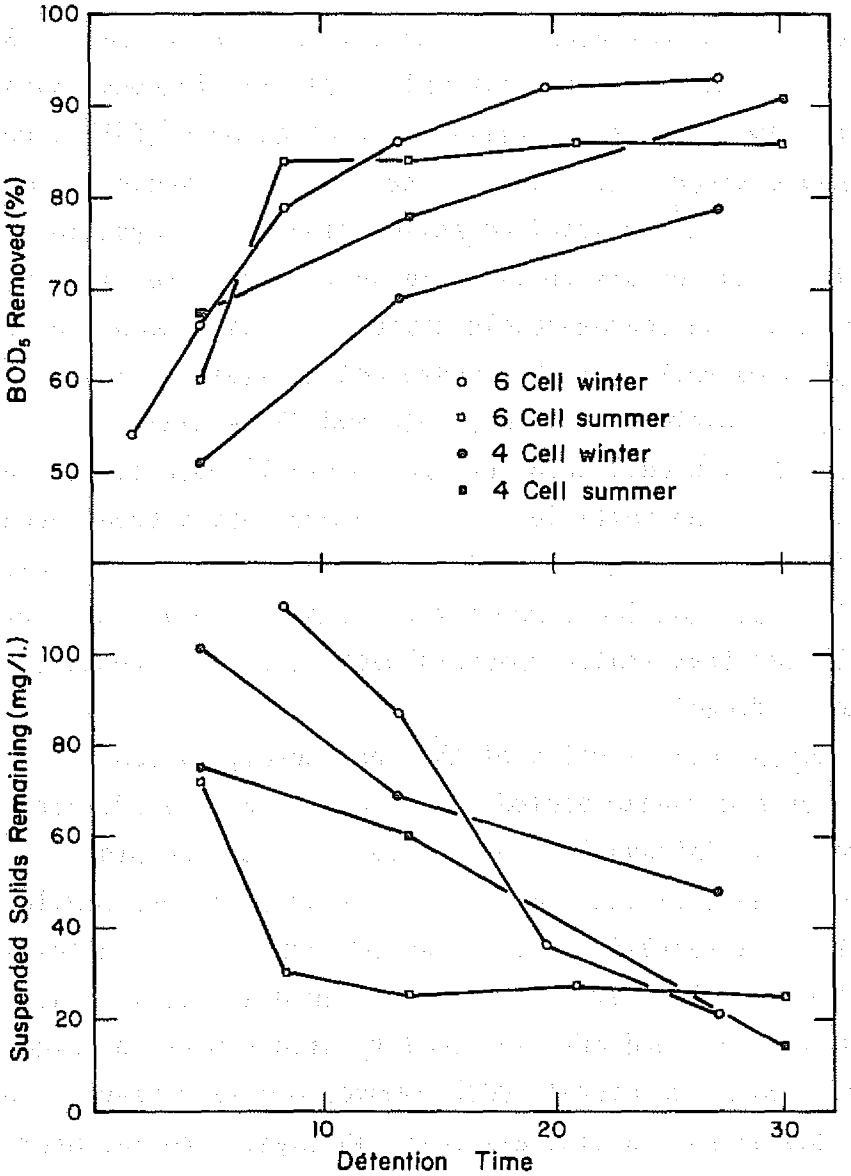
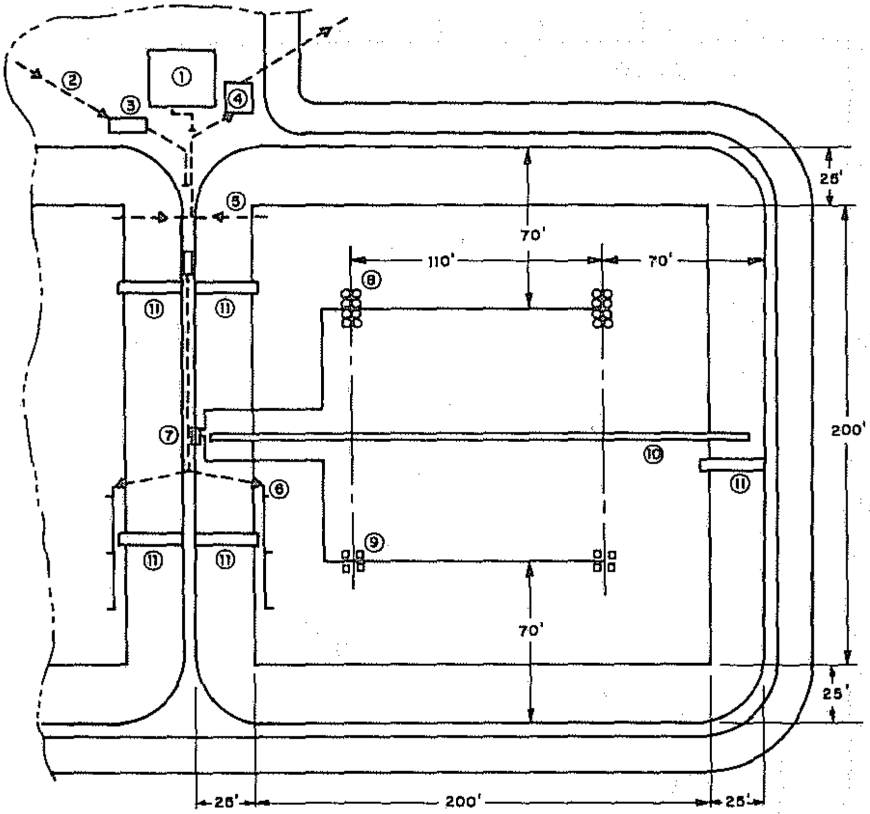


Fig. 21: EIELSON AIR FORCE BASE EXPERIMENTAL LAGOON WINTER AND SUMMER % BOD_5 REMOVAL AND SS REMAINING vs. DETENTION TIME. (Christianson, 1976)



① Laboratory Building
② Manhole-Parshall Flume
③ Wet Well and Lift Station
④ Chlorine Contact Chamber
⑤ Effluent Line
⑥ Influent Header
⑦ Air Header Box
⑧ Aeroflow Diffusers
⑨ Shearfusers
⑩ Baffle
⑪ Dock

Fig.22: PLAN VIEW OF FT. GREELY COARSE-BUBBLE AERATOR INSTALLATION. (Christianson, 1976)

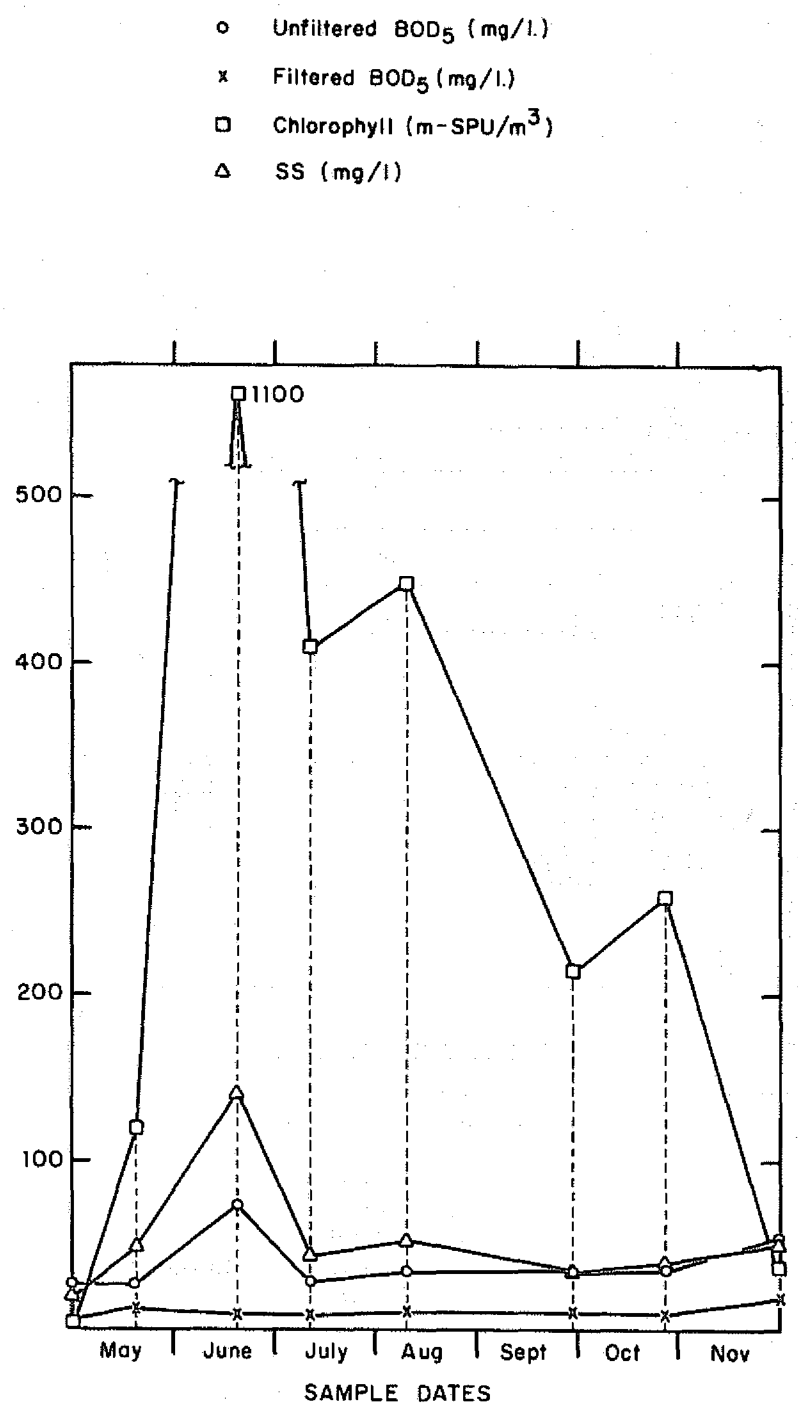


Fig. 23: FT. GREELY AERATED LAGOON. BOD_5, CHLOROPHYLL AND SUSPENDED SOLIDS vs. SUMMER SAMPLING DATES, 1972. (Christianson, 1976)

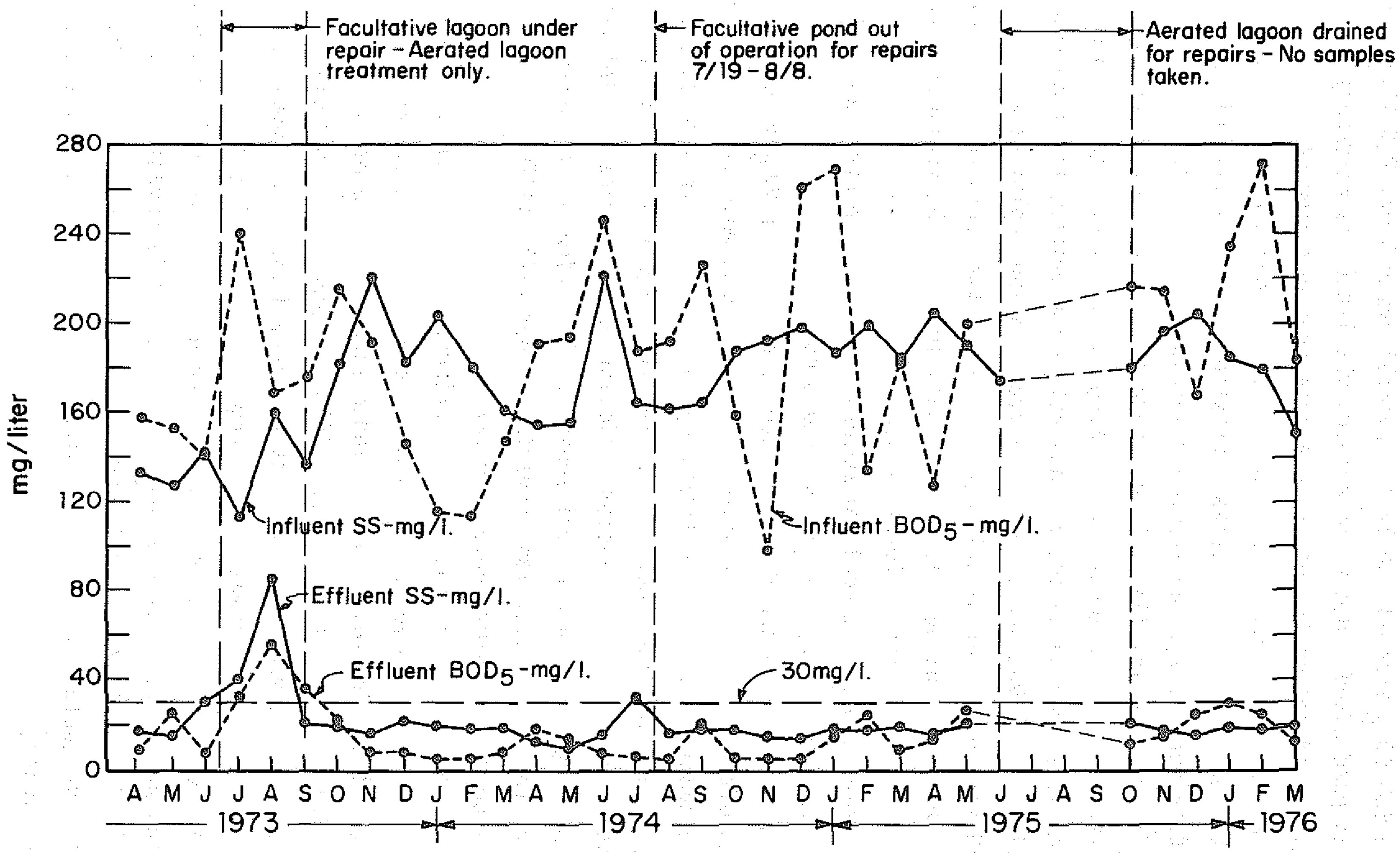


Fig. 24: PALMER LAGOON OPERATING RESULTS
(Christianson, 1976)

attained 55% BOD_5 removal, while the overall removal was 91% with a polishing pond added. Hence, it appears the latter is necessary to allow suspended solids to settle in a quiescent environment.

By adding data from lagoons in Manitoba, Minnesota, and North Dakota to that obtained in Alaska, Christianson (1976) was able to obtain data for average year-around removals for a variety of detention times (Figures 25a and 25b). Even though these data show a lot of scatter because of different lagoon designs, loadings, liquid temperatures, etc., removals seem to be better than 85% for detention times greater than fifty days. Not broken out separately on these plots are data for the EAFB six-cell experimental lagoon. These data indicated 85% removal after a 20-day detention time. Other data indicate the residual soluble BOD_5 to be 10 mg/l after a 20- to 40-day detention time.

By comparing operational parameters for the Eagle River and Palmer lagoons one can see why the latter should outperform the former. First, the loading on the Palmer lagoon is only 1.7 g BOD_5/m^3/day, compared with 10.2 for Eagle River. This is synonymous with the much longer detention time for the Palmer system (110 days vs ≈ 20 days). Second, as already mentioned, the Eagle River lagoon had frequent problems with its aeration system. Similar problems were avoided with the Palmer lagoon by careful maintenance. A portion of the diffuser system at the Eagle River was converted to Aer-O-Flo diffusers which have presented few maintenance problems.

In summarizing his findings, Christianson (1976) recommended that the aeration flow should be distributed to avoid sludge accumulation near the inlet. Second, adequate sludge storage should be provided in the initial sections of the lagoon (1500 l sludge/1K m^3 influent). Otherwise, the fermentation products released during the summer won't be treated before the effluent is discharged. Third, cells should be placed in series to avoid short circuiting with the last cell being a polishing pond (i.e. little or no aeration). While organic loadings should be low to prevent algal blooms in the polishing pond, it should provide a suitable environment for macroorganisms such as Daphnia which prey on algae.

Diffused-air systems have been found to be superior to mechanical aerators in cold climates because of the tendency for ice to form on the latter (Edde, 1972). Floating mechanical aerators, however, have been

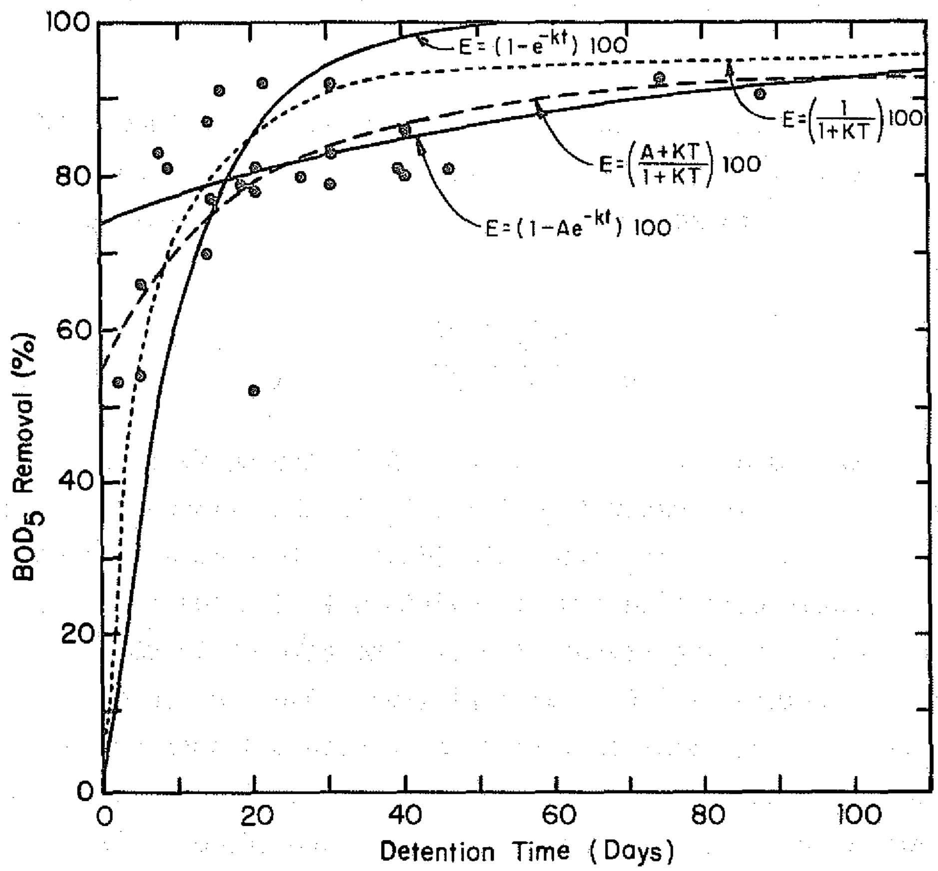


Fig. 25a: YEAR-ROUND PERCENT BOD_5 REMOVALS vs. DETENTION TIME (Av.)

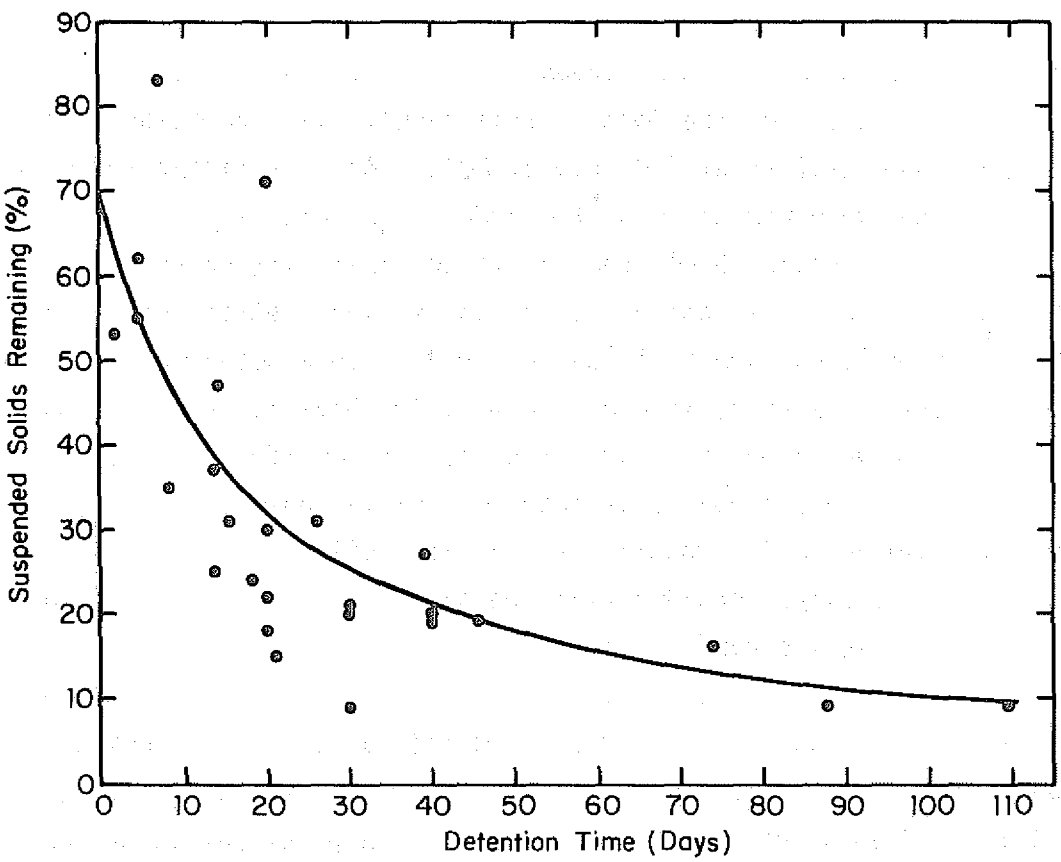


Fig. 25b: YEAR-ROUND PERCENT SUSPENDED SOLIDS REMAINING vs. DETENTION TIME (Av.) (Christianson, 1976)

used successfully at air temperatures as low as -35°C (Edde, 1972), but for a relatively warm industrial effluent. The average lagoon temperature T_w can be estimated from well-known relationships (Metcalf and Eddy, 1972).

$$T_w = \frac{AfT_a + QT_i}{Af + Q} \tag{3}$$

Here, a typical value for $f = 12 \times 10^{-6}$, A is the surface area in ft^2, T_a is the ambient air temperature (°F), T_i is the temperature of the influent (°F), and Q is the flow rate (Mgpd). In Figure 26 is shown the maximum allowable detention time to maintain $T_w > 2°C$ for a 20-foot deep aerated lagoon receiving sewage at 6°C. The attainable BOD_5 removal at a liquid temperature of 2°C is also plotted. One can see that one is constrained from both ends in trying to operate a lagoon in the winter.

Temperature dependencies of reaction rate coefficients for aerated lagoons are expressible by equations of the form (Edde, 1972)

$$K(T)/K(20°C) = \tilde{\theta}^{(T-20°C)} \tag{4}$$

Here $\tilde{\theta} = 1.035$ for an aerated lagoon and $\tilde{\theta} = 1.07$ for an aerated aerobic-faculative lagoon. For the former, this results in a reaction coefficent 1.86 times smaller at 2°C than at 20°C. At this latter temperature, the land requirement is on the order of 2 acres/Mgpd.

Special attention should be paid to the installation of piping either into or out of a lagoon and for connections between cells of a lagoon. These lines should all be submerged with a submerged discharge in the receiving water desirable. To minimize heat losses, lagoons should be shielded from the wind. Finally, in permafrost areas, construction of lagoons should take into account the possibility of surface subsidence caused by the thawing of ice-rich soils.

The long daylight hours during the summer in Alaska can present problems with algal blooms. This can lead to excessive SS levels in the effluent. Reed (1976) discusses three ways of reducing effluent SS levels (sedimentation, flotation, and filtration). For sedimentation to be effective, a quiescent settling zone must be established. Of the Alaskan lagoons cited in the literature, only the one in Palmer

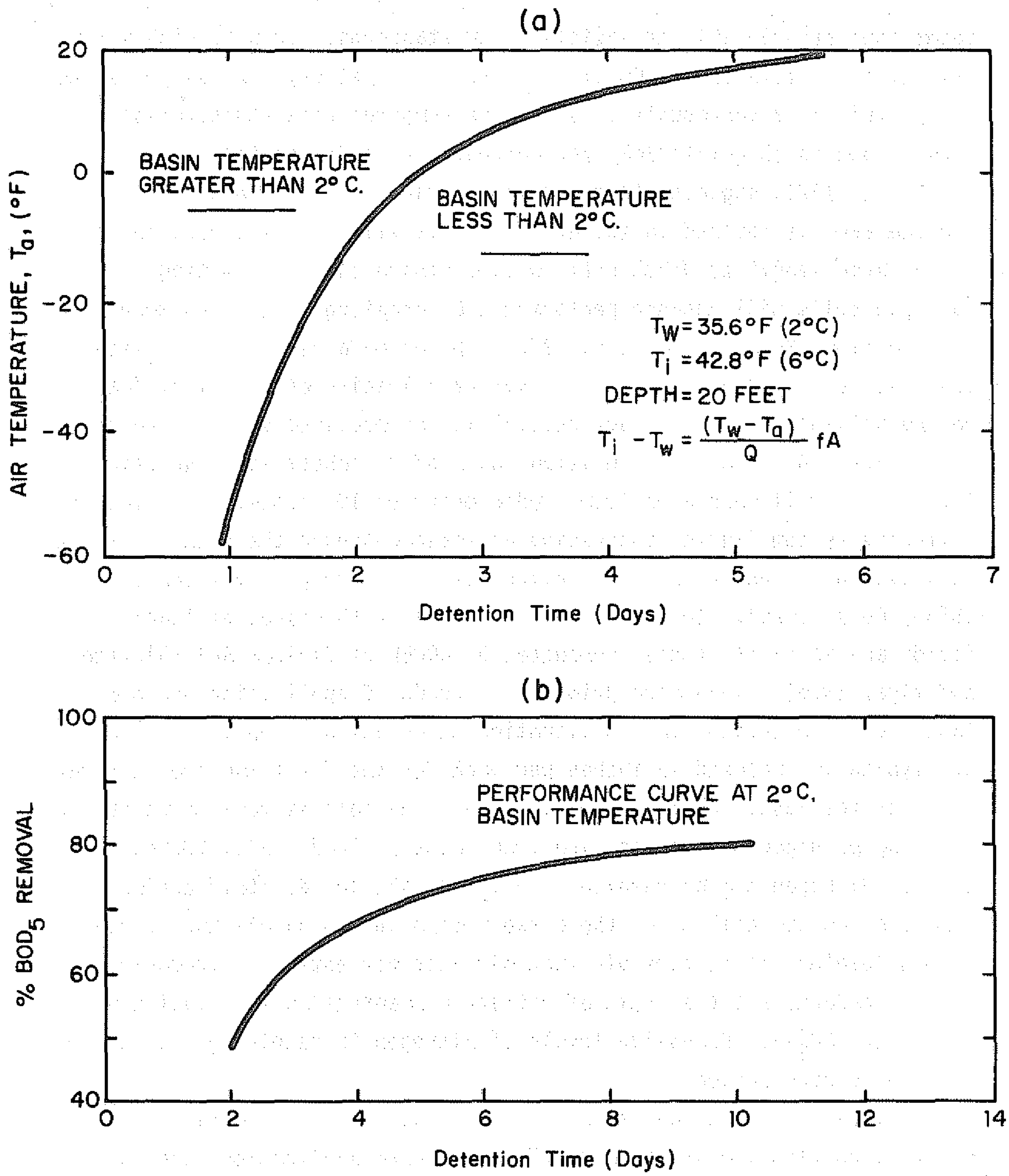


Fig. 26: OPERATION EXPECTATIONS OF AERATED LAGOON IN COLD-CLIMATE CONDITIONS. (Edde, 1972)

seems consistently able to satisfy 30/30 standards. This is with a 110-day detention time and a SOR of less than 2 gal/ft^{2}day for the polishing pond. This is tremendously conservative compared with characteristic SORs of 300 to 1K gal/ft^2/day for conventional sedimentation.

Reed (1976) suggests that a deep, compact system with minimum surface area is desired in the Arctic. This will minimize heat losses and a large length-to-width ratio will minimize short circuiting. Multiple cells will enhance performance by creating conditions more akin to plug flow (Metcalf and Eddy, 1972). Based on a review of the literature, Reid (1975) recommends a hydraulic loading of 20 gal/ft^2/day on the polishing pond. Sludge removal may be required every three to five years. At sludge accumulation rates of 2 inches/year (Christianson, 1977), this will mean a maximum sludge depth of 10 inches. One type of polishing system finding increasing acceptance around the country is the utilization of the natural filtration action of the ground (USEPA, 1973b; Reed, 1976). In Alaska, this is done at thousands of leach fields and in pilot studies conducted by CRREL at Eielson AFB (Sletten and viga, 1976). The three principal methods of application are overland flow, irrigation, and infiltration-percolation. Application rates are typically measured in inches per week for the first two and feet per week for the last. Available data suggest the soil is very effective in removing pathogens, organics, and trace metals (Reed, 1976; USEPA, 1973). Nitrogen can be removed by plant uptake and denitrification. Depending on the ability of these two mechanisms to handle the nitrogenous loading, it is possible that nitrates may enter the groundwater. USEPA standards set the limit of nitrate concentration in drinking water at 10 mg/l NO_3^--N. Excessive levels of nitrogen in receiving bodies can lead to eutrophication.

Relatively large land areas are required for the slow-rate systems of overland flow and irrigation. For a 4 in/wk application rate, at least fifteen acres would be required for a 100K gpd flow. If irrigation can be practiced for only five months per year, a storage volume of 21 Mgal must be provided for the winter flow. Many soils in Alaska are silty and not suitable for land treatment because of very low permeabilities, although there are regions where suitably granular deposits do exist. But, in general, slow-rate systems are probably only suitable south of the Alaska Range particularly where there is potential for agriculture.

Reed (1976) recommends that this natural filtering action be utilized to provide effluent polishing for the Air Force lagoons at Eielson, Galena, King Salmon, and Shemya. At Eielson, an existing borrow pit is available to serve as an infiltration cell.

According to Grainge (1977), sewage oxidation ponds have provided the most reliable method of sewage treatment in northern Canada, and that biological package plants have presented difficult operational problems. Of the seven such plants examined in the Northwest Territories, none operated properly.

Physical-Chemical Treatment

Physical and/or chemical treatment is practiced at the Pt. Woronzof Treatment Plant in Anchorage, at several VSW facilities, and at the Alaska Village Demonstration Projects at Wainwright and Emmonak. The Anchorage facility has been the subject of some controversy as there are no current plans to upgrade the plant from primary to secondary treatment as required by P.L. 92-500. Studies have been performed (Murphy and Carlson, 1972) to show that upgrading the facility would not improve the water quality in Cook Inlet, because of its tremendous dispersive powers. Hence, the Municipality of Anchorage argues that it would be a waste of the taxpayers' money to spend millions of dollars on removing additional organic matter from the wastewater if the effect of such an action on water quality in Cook Inlet couldn't be measured except immediately adjacent to the outfall. All this may be true, but the United States Environmental Protection Agency has to worry about the political consequences of allowing noncompliance with the law. Recent legislation may exempt Anchorage from secondary treatment.

After preliminary screening, the sewage at Point Woronzof flows to one of three clarifiers where gravity separation of solids occurs (Figure 27). The effluent is discharged into Cook Inlet after chlorination while the surface skim and settled solids are pumped to a single gravity thickener. From there, raw sludge, with a solids content of 3% to 5%, is pumped to a vacuum filter with which a cake of 25% solids is

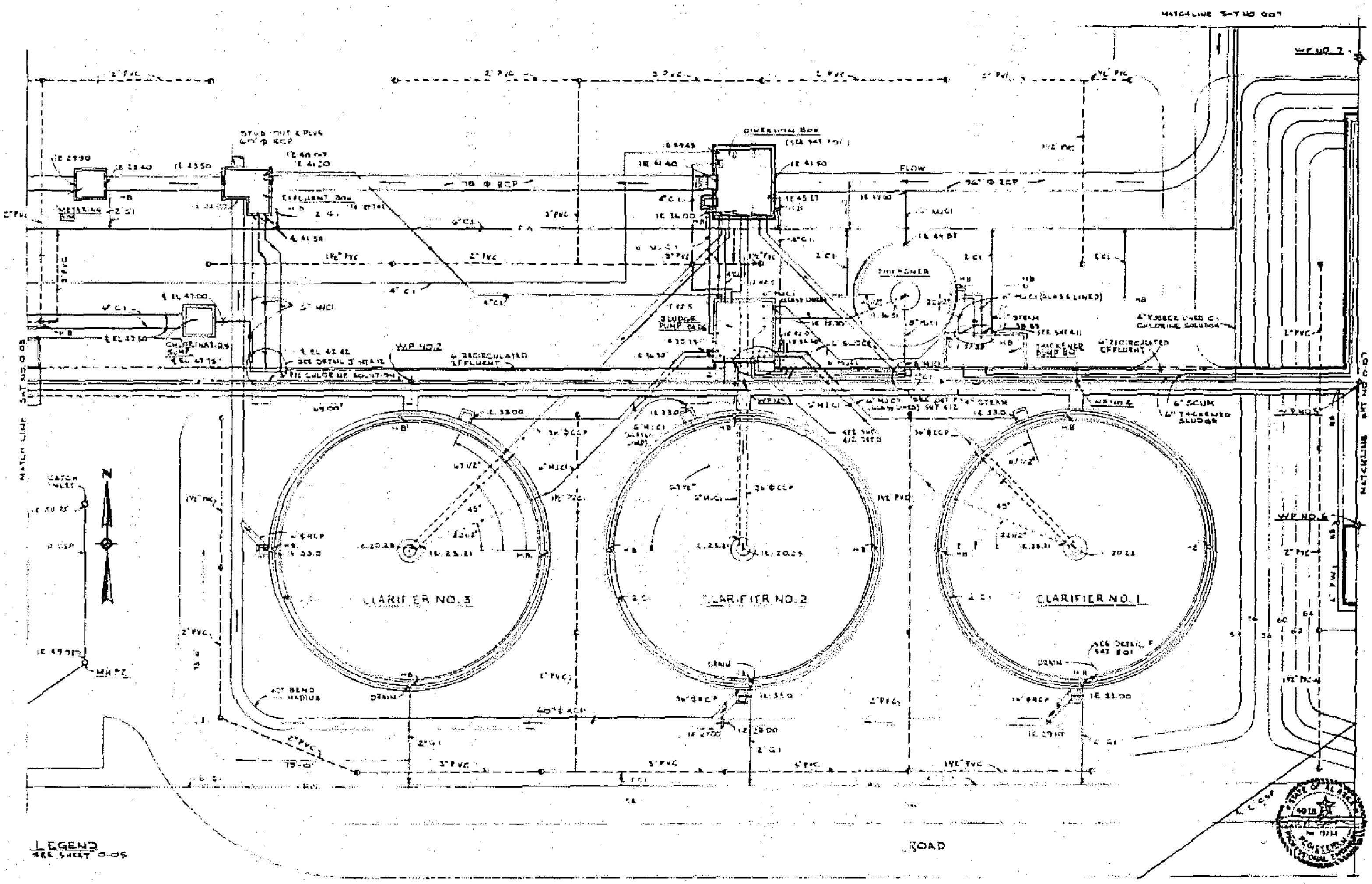


Fig. 27: GREATER ANCHORAGE BOROUGH SEWAGE PROJECT. (Borough Engineers, 1970)

created. This cake is fed to a multiple-hearth incinerator capable of reducing 4K lmb/hr of wet sludge to an inert material, an energy-intensive operation. The end product, representing only 10% to 20% of the original sludge volume, is trucked to dumpting sites on land.

Except during runoff, the flows average around 22 Mgpd (Hudson, 1976) with BOD_5 removals of around 25% for July, 1976. At this flow rate, the clarifier SOR is well under 1K gal/ft^2/day. At this loading, much of BOD_5 due to suspended solids should be removed. The SS removal efficiency for the same month was higher (around 50%). This is in the range of what is to be expected for primary treatment.

The VSW facilities at Nulato, Selawik, and Alakanuk utilize 14K gpd Met-Pro physical-chemical package plants for sewage treatment. The unit processes are alum coagulation, carbon adsorption, rapid sand filtration, and chlorination. The treated effluent is discharged to the land. After dewatering in a centrifuge, the sludge is incinerated. The plants of Selawik and Alakanuk appear to be operating well although effluent quality data are not available. Intermittent sludge buildup has been a problem at all three facilities (Sargent, 1977).

Two largely physical-chemical wastewater treatment facilities serving native villages today are located at Wainwright and Emmonak. Each facility was built as part of the Alaska Village Demonstration Project at a cost of over $1 million, including development costs. With each village having about 100 families, this represents an investment of over $10,000 per family. This is still considerably cheaper than a capital cost of $45,000 per home projected for an individual haul system of Nuiqsut (Johnson & Dreyer, 1977). Although water is hand carried from the facility or delivered by vehicles to the individual homes and disposal of wastes from homes is accomplished using honey buckets, the facilities serve as community centers for laundering and bathing with a few toilets provided. They also provide water for and treat wastewater from the schools.

The graywater and blackwater treatment systems at Emmonak are shown in Figure 28. The package unit supplied by Keystone Engineering, Inc., utilizes powdered, activated carbon adsorption followed by clarification, multi-media filtration, and chlorination. Blackwater centrate is treated along with the graywater. As of the fall of 1976, the powdered carbon

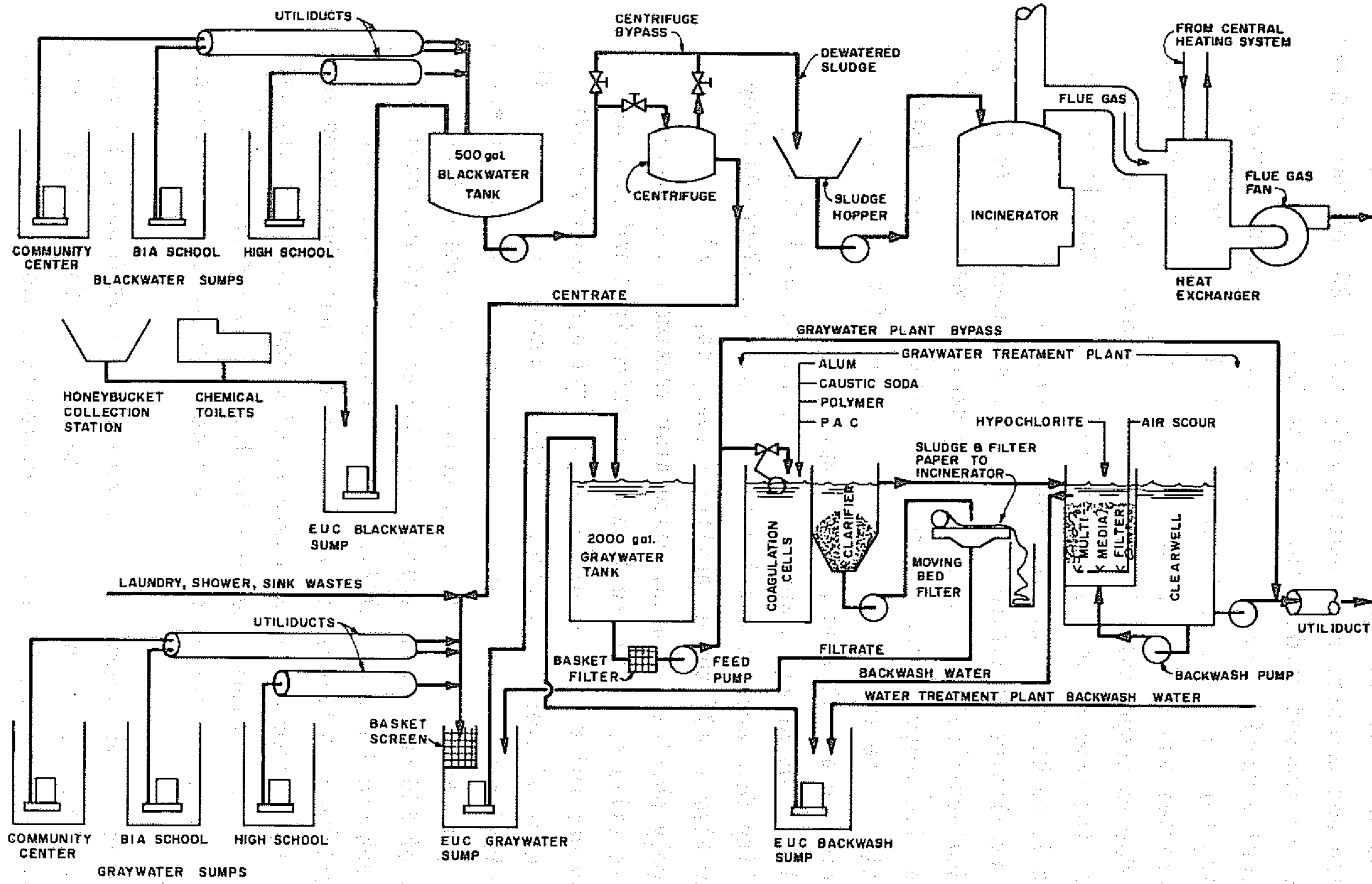


Fig. 28: GRAYWATER AND BLACKWATER SYSTEM AT EMMONAK. (Puchtler, Reid, and Christianson, 1976)

had not been used because of operational problems (Puchtler et al., 1976). Performance data (Table 19) obtained during a monitoring period show fair COD and SS removal. With flow rates averaging about 3,500 gpd, only 50% of the design capacity was used in an average sense.

One reason attributed for the inability of this plant to perform better was the presence of blackwater centrate and high solids levels in the graywater feed (Puchtler et al., 1976). Since these did not occur continuously, they helped to contribute to large variations both in the character and in the flow rates of the raw wastewater. It is believed that more careful flow control coupled with the addition of powdered activated carbon will help improve plant performance (Puchtler et al., 1976). Also, USEPA plans to add aeration to the holding tanks (Christianson, 1977). Power requirements are estimated to be 4.3 kwhrs/K gal. Other operational problems included clogging of cells in the coagulator, thermal currents in the clarifier, buildup of gum balls in the filter media, and lack of operator attention. Attempts are being made to correct these difficulties.

Blackwater from the community center, the BIA school, and toilets within the facility is very strong compared with typical domestic wastewater, with a COD around 8K mg/l. But, this is still more dilute than planned originally (Mitchell, 1977). Currently the blackwater is allowed to settle on a batch basis with the sludge dewatered after decanting. The power consumed in incinerating the concentrate is about 43 kwhr/K gal blackwater. Heat is recovered from the incinerator flue gas to help heat hot water. In this manner, about 25% of the net heat input to the incinerator is recovered. Currently, the incinerator is not being used because the dilute nature (2% solids) of the sludge results in escessive fuel requirements. Consideration is being given to disinfecting the sludge with lime (Mitchell, 1977). Information available in 1972 (Puchtler, 1973) projected an annual cost of $676 per household to operate the Emmonak facility. This does not include amortization of the capital costs which was taken care of by the federal government. Later information (Puchtler et al., 1976) revealed the actual operating expences over a 30-month period ending in October, 1975, were $189,000. With around 120 households, this is equivalent to an

TABLE 19: GRAYWATER AND TREATED GRAYWATER CHARACTERISTICS AT EMMONAK[1]

	Number of Samples[2]	Temp. °C	pH[3]	Conductivity UMHO	Turbidity JTU	Total Solids mg/l	Total Volatile Solids mg/l	SS mg/l	COD mg/l	TPO_4P mg/l	NH_3N mg/l	Alkalinity mg/l
Laundry	13	31 (6)	6.6-8.7 ---	1360 (690)	96 (32)	1770 (830)	550 (280)	240 (163)	960 (520)	30.5 (25.1)	6.2 (7.0)	248 (108)
Shower	9	25 (4)	5.6-7.3 ---	670 (300)	85 (27)	810 (367)	280 (170)	160 (74)	500 (369)	2.5 (3.3)	6.8 (8.6)	156 (46)
Graywater Composite	26	27 (3)	5.3-8.9 ---	1440 (1380)	109 (53)	2640 (2785)	990 (1190)	1680 (2930)	1510 (2480)	11.7 (11.9)	14.3 (36.8)	260 (355)
Treated Graywater Composite	24	23 (4)	3.0-7.2 ---	1330 (500)	47 (33)	1630 (1045)	520 (480)	330 (750)	300 (250)	4.4 (6.7)	7.6 (5.7)	125 (90)
Per cent Removal		---	---	8	57	38	47	80	80	62	47	52
Graywater Adjusted	19	28 (3)	5.3-8.4 ---	1035 (380)	104 (49)	1285 (450)	440 (210)	400 (210)	690 (345)	10.2 (8.0)	6.5 (4.3)	148 (82)
Treated Graywater Adjusted	18	24 (3)	3.0-7.4 ---	1280 (450)	51 (35)	1330 (610)	395 (260)	48 (35)	205 (100)	2.1 (1.2)	6.8 (4.0)	115 (52)
Per cent Removal		---	---	+24	51	+4	10	88	70	79	---	22

[1]Mean with standard deviation in parenthesis.
[2]In a limited number of cases, actual numbers of analysis for a given parameter may be 1 or 2 less than the number of samples shown.
[3]Range of values.
SOURCE: Puchtler et al., 1976.

annual expense of around $700 per household, close to Puchtler's 1972 estimate. During the first half of 1977, revenues derived from sales to the village were meeting over 80% of the operating expenses (Mitchell, 1977).

At the Wainwright facility, blackwater wastes from the six recirculating chemical toilets in the AVDP facility were originally incinerated. However, the incinerator was destroyed by fire in November, 1973. In the reconstructed facility, the blackwater wastes are treated biologically via extended aeration and lime disinfection. The modified graywater treatment system used today (Figure 29) consists of coagulation, sedimentation, sand filtration, carbon adsorption, and chlorination. Because of sludge carryover through the upflow clarifier, this component is completely bypassed for the modified system. Performance data in Table 20 indicates excellent COD and SS removal. But the plant also had periods of poor performance because of biological growth and decomposition in accumulated sludges. Neither the old nor the new plant has been operating long enough to prove long-term reliability. The current water charges paid by the villagers are 4¢ to 6¢ per gal. The actual costs are greater. Other reported charges are 25¢ per two-minute shower, 50¢ for a load of wash, and 10¢ for the dryer. To help reduce operating costs, heat is being recovered from the power plant at Wainwright.

According to Mitchell (1977), both Wainwright and Emmonak were experiencing operating problems at the time of his visit in July, 1977, and the Wainwright facility was shut down during his visit because of a power failure. As of August, 1977, the power plants were again operational. But the villagers still received chlorinated potable water during this time period. Although potable water was being produced at Emmonak, the graywater was not being treated. However, both facilities are believed to have operated for much of 1977. Mitchell believes that the shutdowns (either full or partial) at these facilities were partly caused by the lack of funds for assistance by USEPA personnel during 1977. To rectify this situation, it is hoped that a new USEPA grant will provide around $200,000 to each village for preventive maintenance and technical and management consulting for a one-year period. This outside help, according to Mitchell (1977), is essential if the facilities are to operate successfully.

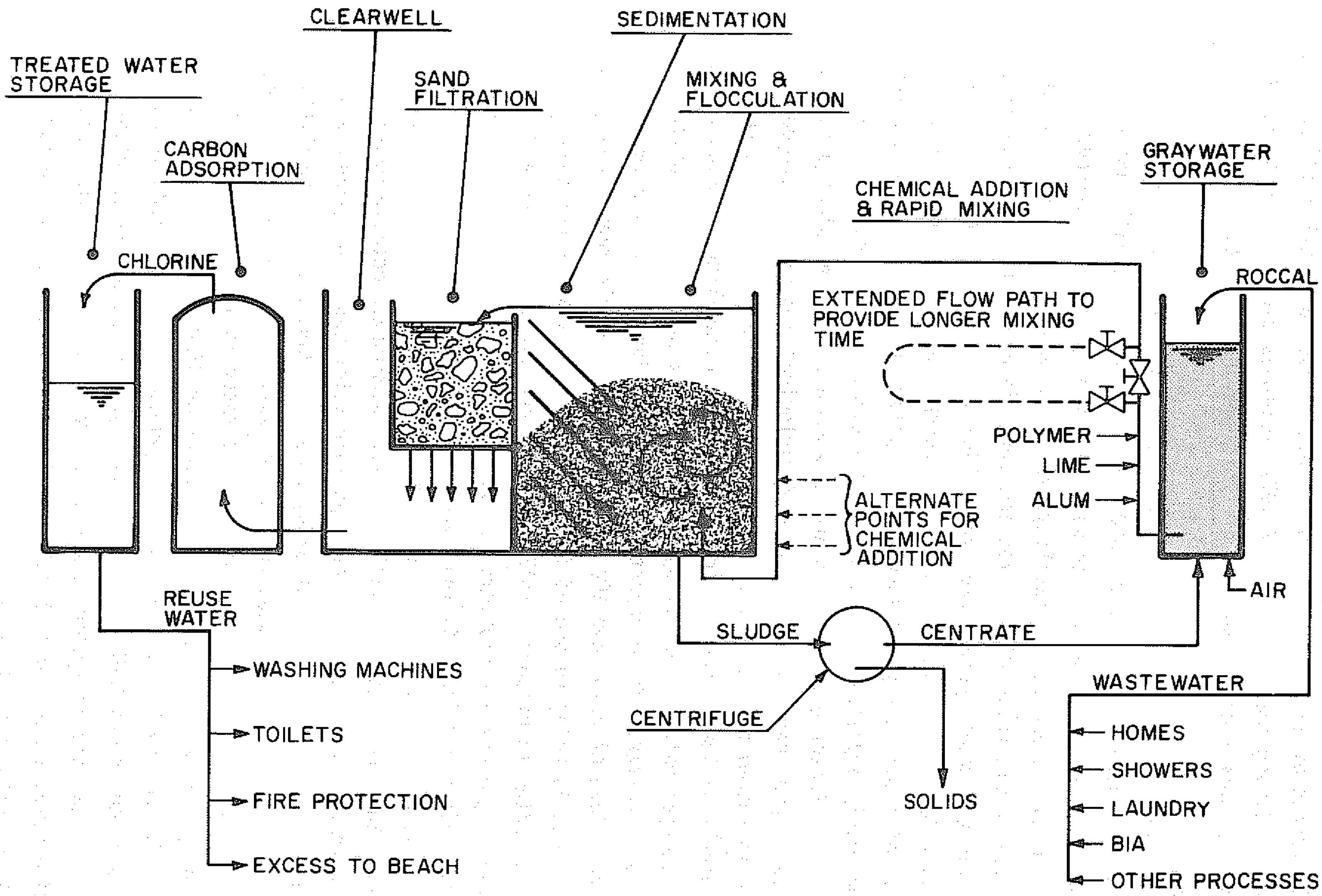


Fig. 29: MODIFIED WASTEWATER TREATMENT PLANT - WAINWRIGHT AVDP
(Puchtler, Reid, and Christianson, 1976)

TABLE 20: RESULTS OF PHYSICAL-CHEMICAL TREATMENT OF GRAYWATER AT THE ORIGINAL WAINWRIGHT UTILITIES CENTER

	Raw Gray-Water	P-C Plant Effluent	Carbon Column Effluent	Reduction Percentage
Color, PCU	35	25	5	86
Turbidity, JTU	200	16	10	95
COD, mg/l	840	284	25	97
Total Solids, mg/l	1910	1670	1240	35
Suspended Solids, mg/l	503	25	10	98
Total Volatile Solids, mg/l	624	304	162	74
Volatile Susp. Solids, mg/l	281	16	4	99

Source: Puchtler et al., 1976.

Special attention should be paid to the problems of disinfection at low temperatures. Not only does the efficiency of chemical disinfectants decrease with decreasing temperature but also the survival time of organisms increases (Chambers and Berg, 1970). Gordon and Davenport (1973) found the disinfecting ability of chlorine varied significantly from one wastewater source to another at temperatures less than 1°C. Hence an arbitrary chlorine residual after some specified contact time cannot be considered a reliable indication of satisfactory disinfection. Required instead is the establishment of a specific chlorine contact time for each effluent. But, they found low temperatures did not prevent disinfection to less than 200 fecal coliforms/100 ml from all sources tested. Gordon and Davenport cautioned against achieving chlorine residuals any higher than necessary because of possible toxic effects in receiving waters. Such effects had not been quantified in arctic and subarctic waters.

Freezing causes ice crystals to be formed in bacteria with the disruption of cellular membranes. The result is their gradual-to-rapid die-off depending on the species, temperature, and other environmental factors. Alternate freezing and thawing is a more effective means of disinfection than is freezing alone. In contrast, freezing does not destroy viruses because they are not cellular and, in fact, it preserves them (Chambers and Berg, 1970). Viruses may be stored at -70°C for a decade or more. Morrison et al. (1973) found that sewage can be disinfected to a safe level by the addition of lime to a pH of 12 even at cold temperatures. This can be done within 30 minutes at a temperature of 1°C.

Sludge disposal can represent up to 50% of the total operational and maintenance costs (Burd, 1968). The methods used most often at present include drying on sand beds, centrifugation, dewatering on screens, or vacuum filtration followed by disposal on land or incineration. The incinerating portion, especially, can be very expensive for small Alaskan communities. Operational difficulties forced Barrow to abandon its incinerator while the first Wainwright facility was destroyed by fire associated with the incinerator. One possible economic solution to the sludge disposal problem in Alaska is to utilize the climate to dewater sludge by freezing and then thawing. Remarkable improvements in dewatering rates following freezing have been found for sewage sludges. The mechanisms appear to be coagulation upon freezing followed by dehydration upon thawing. Tilsworth (1972) estimated total sludge dewatering costs using freezing at $50/ton of processed sludge compared with $70 for vacuum filtration or $100 for sand drying beds. This is assuming a sludge depth of 12 inches in Fairbanks. The reduced volume (higher solids content) of dewatered sludge could then be discharged to a sanitary land fill. For both biological and chemical sludges, slow freezing rates are best (Khan et al., 1976). According to a recent Canadian study (Environment Canada, 1977), land disposal using vehicles can be very difficult during the spring and fall when the ground is partially frozen. The method of spreading must be decided on a case-by-case basis.

Another technique that may prove useful for sludge disposal is to utilize waste heat for drying. Ryan (1973) noted that a 60-kw diesel generator operating at full capacity has recoverable waste heat of 270K Btu/hr in cooling water and exhaust gases. At 1K Btu/lbm water, this could ideally remove 270 lbm/hr of moisture from sludge. To maximize the heat recovered, it is necessary to recover the latent heat of condensation of the water vapor. One disadvantage of heat treatment compared with freezing is that a large fraction of the sludge is solubilized rather than destroyed resulting in a high-strength supernatent. Other uses of recovered heat include the preventing of pipes from freezing and the melting of ice to produce potable water. If this heat were used for this purpose with an efficiency of 50%, about 2 gpm of water would be produced.

Discussion

The treatment plants in Alaska's three largest cities (Anchorage, Fairbanks, and Juneau) are operating well at the present time. The secondary facilities at Fairbanks and Juneau are proof of the fact that the "bugs" can thrive in the north under proper conditions. The proper temperature can readily be achieved by utilizing the sensible heat available in the wastewater. The problems encountered with the Fairbanks plant are mainly those associated with the politics of sludge disposal. No one seems to mind a sanitary landfill as long as it is located near someone else. The complex controls on the pure-oxygen system have also created difficulties. Although the Anchorage plant has performed well, it will probably be the focus of controversy in the future as the municipality fights battles with USEPA over secondary treatment. At the present time, there are plans to extend the outfall pipe further into Cook Inlet to disperse the effluent more effectively (Merrell, 1977). It is the opinion of this writer that the tremendous dilution powers of Cook Inlet make additional treatment unnecessary at the present time. Recent federal legislation may exempt Anchorage from secondary treatment.

The situation in many of the villages with respect to wastewater treatment is not so good. According to a task force report for the Alaska Water Assessment (Sargent et al., 1976) in roughly half of all the villages, there are severe water contamination problems--meaning direct contact between people and untreated sewage is possible. This could include honey buckets dumped on a river bank or cesspool contents ponding on the ground surface. Agencies such as the USPHS-ANHS, the ADEC, and the USEPA are working hard to improve this situation. But even though water systems have been installed in many villages and wastewater treatment systems in a few, the track record is not fantastic. According to Ryan (1977b), about 10% of the sanitation systems installed by ANHS had failed completely. Sargent (1977) believes the failure rate is higher if one defines failure as inability to operate reliably. A previous VSW report (Carson, 1976) cited a 50% failure rate for systems installed by the USPHS. Ryan (1977b) said this was much too high and Sargent (1977) stated that it was impossible to be so quantitative at the present time. But, elsewhere Ryan (1977a) said proper operation and maintenance is unattainable 50% of the time in remote communities. The main reasons for failure are the lack of well-trained and conscientious operators, the lack of technical and management attention to the villages subsequent to plant installation, and the lack of financial reserves to maintain the facilities. Until a solution is found for these problems, no amount of technology will provide a cure. These difficulties are compounded by the harsh winters.

The question of how much a village should contribute toward operation and maintenance costs certainly influences deciding what system will be installed. If the user fees paid by the individual villagers are excessive, they may avoid using the facility and resort to unsanitary practices. This will defeat the purpose for which the facilities were installed and further drive up the user fees for the remaining residents.

The reason the largest muncipalities have better wastewater treatment systems is that this is where the money has been spent. In fiscal years 1974-1975, approximately \$100 M in state and federal funds were spent on construction of treatment facilities in the larger communities (Hammond and Mueller, 1977). According to the 1976 Needs Survey (Hargesheimer, 1976), about \$325 M is needed to attain the 1983 goal of best practical waste treatment technology. These funds would be used for providing secondary treatment, correct infiltration/inflow problems, replace or rehabilitate sewage collection systems, and construct new sewers. A recent USEPA survey (Ward, 1976) reveals that nationwide, 30% of the municipal facilities were not meeting BOD_5 design criteria and 45% were not meeting SS design criteria. A significant portion of the problem was attributed to poor operation and maintenance. So, the Alaskan municipalities are not atypical (Table 11).

The fact that these well-maintained facilities are able to operate without freezing up indicates the "distinctly northern" freezing problems can be overcome. The costs associated with the prevention of freezing are typically indirect and may include a homeowner's heating of relatively cold incoming water before it is discharged as sewage or the heating of water in a water treatment plant as it is in Fairbanks. In either case, tremendous amounts of energy in the form of heat can be transported to the sewage treatment facility in this fashion. In the case of Fairbanks, the heat from the incoming sewage proved sufficient to heat the treatment plant with a temperature drop of only 1°C. In Anchorage, the uncovered, primary sedimentation tanks do not freeze. In the bush, some villages have had problems with freezing (Rogness, 1977). This is typically caused by lack of operator attention.

The military installations typically employ aerated lagoons as the basic treatment technique with some extended aeration package plants being used. Of the twenty or so aerated lagoons in operation in Alaska in 1975 (Reid, 1975) only the one at Palmer consistently met 30/30 standards. By analyzing data from these facilities and from municipal lagoons, Christianson (1976) found that lagoons were able to achieve 85% removals for detention times greater than 50 days. For domestic wastewaters, this is sufficient to achieve 30/30 standards. But, for wastewaters from remote facilities, this may not be sufficient. Reed (1976) suggested that sufficient polishing could be obtained by utilizing the natural filtering action of the ground to allow a combination lagoon-natural filter to meet secondary standards. There is much evidence in the literature to indicate such land treatment schemes are very effective for pathogen removal (USEPA, 1973a). However, in northern regions, sufficient storage must be provided to allow the effluent to be retained during the winter months. In addition, more data are needed on pathogen die-off rates in the Arctic.

Christianson (1976) found better performance with several cells placed in series than with one large lagoon having the same total volume. This is consistent with the reactor dynamics theory (Metcalf and Eddy, 1972) which assumes completely mixed flow in each cell. Moreover, by having a final quiescent lagoon as a polishing pond, one is able to effect efficient settling of the suspended solids. Reed (1976) suggests building the cells with a minimum of surface area to avoid excessive heat losses. This phenomenon is self-regulating in some sense as heat losses will be reduced with the formation of ice over a portion of a lagoon.

Extended-aeration package plants were found to perform well on military sites when properly operated. The detailed discussion presented for the Air Force installation at Murphy Dome certainly confirms this.

Not enough operating experience is available to draw a definitive conclusion about physical-chemical treatment plants at remote villages. Preliminary data, however, indicates that finding and keeping qualified operators and maintaining the systems will be problem areas. When one realizes how much attention and manpower a giant corporation such as Alyeska has had to devote to its treatment plants to get them to operate properly, one realizes the magnitude of this problem for a native village.

Disinfection by either chlorine or lime has been shown to be effective at low temperatures. Because it is easier to handle, efforts should be devoted toward a greater use of lime as a disinfectant in the bush. It has been shown to be effective at low temperatures. The low temperatures can also be used to freeze sludge and thus aid in its dewatering.

Technological suitability is meaningless if the community cannot adequately finance and maintain the facility. Both Buzzell (1974) and Ryan (1973) suggest that the sophistication of a system should match the community's ability to operate and maintain it. For example, the advantage of more effective treatment in an activated-sludge system as compared to a septic system may be lost if it is not properly maintained and operated. The total cost of a facility includes initial capital, expenditures, maintenance costs, and operation costs. If a village only has to pay for operating costs, it might choose a different system than if it had to pay for total costs. Ryan (1973) pointed out that piped water distribution and sewage collection systems cost more to construct but are more convenient and less expensive to operate than haul-type operations. Sargent (1977) maintains that more data on total life-cycle costs is needed before definitive statements can be made on piped vs. hauled system costs. Alyeska Pipeline Service Company (1975) stated that the operating costs for its sewage treatment plants amount to 2.9¢/gal exclusive of fuel. If a village used a similar system and had a moderate sewage discharge of 20 gpcd, the operating costs would exceed $100/month/family. Statistics like this may encourage water conservation practices.

INDUSTRIAL TECHNOLOGIES

Introduction

Among the few industries located in Alaska are seafood processors, a few pulp mills, an ammonia fertilizer plant, a brewery, and energy-related activities. A significant part of the latter are the various work camps associated with the oil pipeline. As of the end of 1975, Alyeska was operating 31 sewage treatment camps with a total capacity of 1.56 Mgpd (Alyeska, 1975). By the summer of 1977, this number had been reduced to 14 (Pollen, 1977). Eventually there will just be vaporization at eight of the nine pump stations plus treatment plants at Valdez and Pump Station 5 (Eggener, 1977). There are about fifty other small sewage treatment plants on the north slope associated with oil and gas activities (Dietrick, 1977). These are centered mainly around Prudhoe Bay. Although the total discharge from these camps is not large, the ecological effects cannot be ignored because their discharges are often into a pristine environment. Certain of these plants are discusssed in more detail than others here because only a few of the plants were personally visited during the course of this study.

A significant percentage of all the seafood caught in the United States originate in Alaskan waters with much of it being processed there. For example, 80% of all the salmon harvested in the United States in 1972 were caught in Alaska and processed in 43 Alaskan plants (USEPA, 1975a). All of the king crab and much of the scallop harvest originates in Alaska. Much of the raw material is discarded as waste with the actual marketable yield being less than 30% of the total catch for king crab and shrimp (Jensen, 1965; USEPA, 1974). This has led to increasing efforts for by-product recovery (Paul, 1976). All of these factors lead to the conclusion that waste treatment technology and its accompanying by-product, recovery, in the seafood-processing industry will increase in importance in Alaska.

The two big pulp mills in Southeast Alaska, located at Ward Cove and Silver Bay, only employ primary treatment plus at least 90% chemical recovery (Dickason, 1977). As part of this recovery process, they are able to produce 75% of their electricity (Todd, 1977). Operators of the Ward Cove pulp mill in Ketchikan have maintained that the tidal flushing action is sufficiently great to satisfactorily disperse the effluent stream without need for further treatment. But, recent studies (Hammond and Mueller, 1977) have shown the DO levels at Ward Cove to be in non-compliance with the 6 mg/l criterion. With no pH control practiced at either mill, the water quality standards for pH are periodically violated near the two outfalls as are the standards for sulfite. To help meet these standards, both Alaska Lumber and Pulp and the Louisiana-Pacific Pump Company have recently agreed to install secondary treatment (Hammond and Mueller, 1977). This will result in the production of 20 tons of sludge per day at Sitka (Todd, 1977). It will be burned with the other waste products to generate electricity. The water withdrawals for this operation are around 50 Mgpd. The estimated capital investment at each mill is $35 M with annual operating and maintenance costs of $1.5 M.

The Collier Chemical Company nitrogen fertilizer complex utilizes natural gas from the Kenai gas field to produce almost half a million tons of ammonia annually (Rosenberg and Hood, 1975). In the process of synthesizing ammonia and urea, the plant uses over 700 tons/day of natural gas and produces almost 1 Mgpd of effluent containing about 10K lbm of nitrogen. Seventy percent of this is in the ammonia (NH_3) form (Crevensten, 1977). This nitrogen-fertilizer complex will soon be the largest on the west coast upon completion of the modifications now underway which will double its capacity. In 1978, this plant will have the capacity to provide 50% of the total west coast demand for ammonia used in fertilizers and for industrial uses (Alaska Construction and Oil, 1976).

As of the beginning of 1977, Collier had only holding basins for temperature and pH control before discharge. The treatment effected by these basins is minimal, and the salt sludges are then dumped on land. The wastewater consists of regenerating water from the demineralizers, process water from the ammonia and urea plants, steam boiler and cooling tower blowdown, and pump seal water. Most arises from the water used to regenerate the three cation and three anion beds which purify boiler feed water. The process water flow of 200K gpd is formed in the urea plant and from steam provided by the boilers for the ammonia plant. The sanitary wastes go to a separate septic tank system (Wright, 1976).

By practicing water conservation, Collier claims to use less water per unit mass of ammonia produced than any other plant in the world (Alaska Construction and Oil, 1976). Moreover, in a recent study, Rosenberg and Hood (1975), detected no change in the ammonia balance in Cook Inlet over the last five years. The ammonia input into Cook Inlet should be decreased by the installation of a stream-stripping tower upstream of the holding basin to eliminate most of the ammonia. This method has proved effective elsewhere (Schroeder, 1977). But, performance may suffer during the winter because of both the increased solubility of ammonia in water and operational difficulties at colder temperatures (McGowan, 1975). Some of the ammonia transferred to the air will find its way back into Cook Inlet anyway. The ureas will be hydrolyzed and stripped also (Wright, 1976). Moreover, an outfall and diffuser system will allow effluent to be discharged even at low tides (Crevensten, 1977). This system should result in a daily ammonia-nitrogen discharge of less than the maximum allowable of 700 lbm, which is less than what is discharged by the Borough of Anchorage.

Seafood Processing

Information presented by Collins (1977) and Buck et al. (1975) reveals that water supply and wastewater disposal problems can arise when there is a heavy concentration of seafood processing in one area. As of 1970, the 15 fish and shellfish processing plants on the Kodiak waterfront discharged over 70 M pounds of wastes annually into the harbor (USEPA, 1971a) (Figure 30). Seventy-five percent of this was shrimp waste material. Now, thanks largely to the installation of screens in 1973 on the individual effluent streams and the incentive for by-product recovery, the annual waste discharge is closer to 20 M pounds (USEPA, 1974). In the more remote locations where concentrated groups of processors do not exist, screening is not practiced. But where treatment is practiced in Alaska, screening is used.

As with many industries, seafood processing uses a great deal of water per unit of material processed. The general total usage range is from 500 to 3K gal/ton of raw product processed (USEPA, 1975a). Hence, the minimum water usage is at least twice the weight of material processed

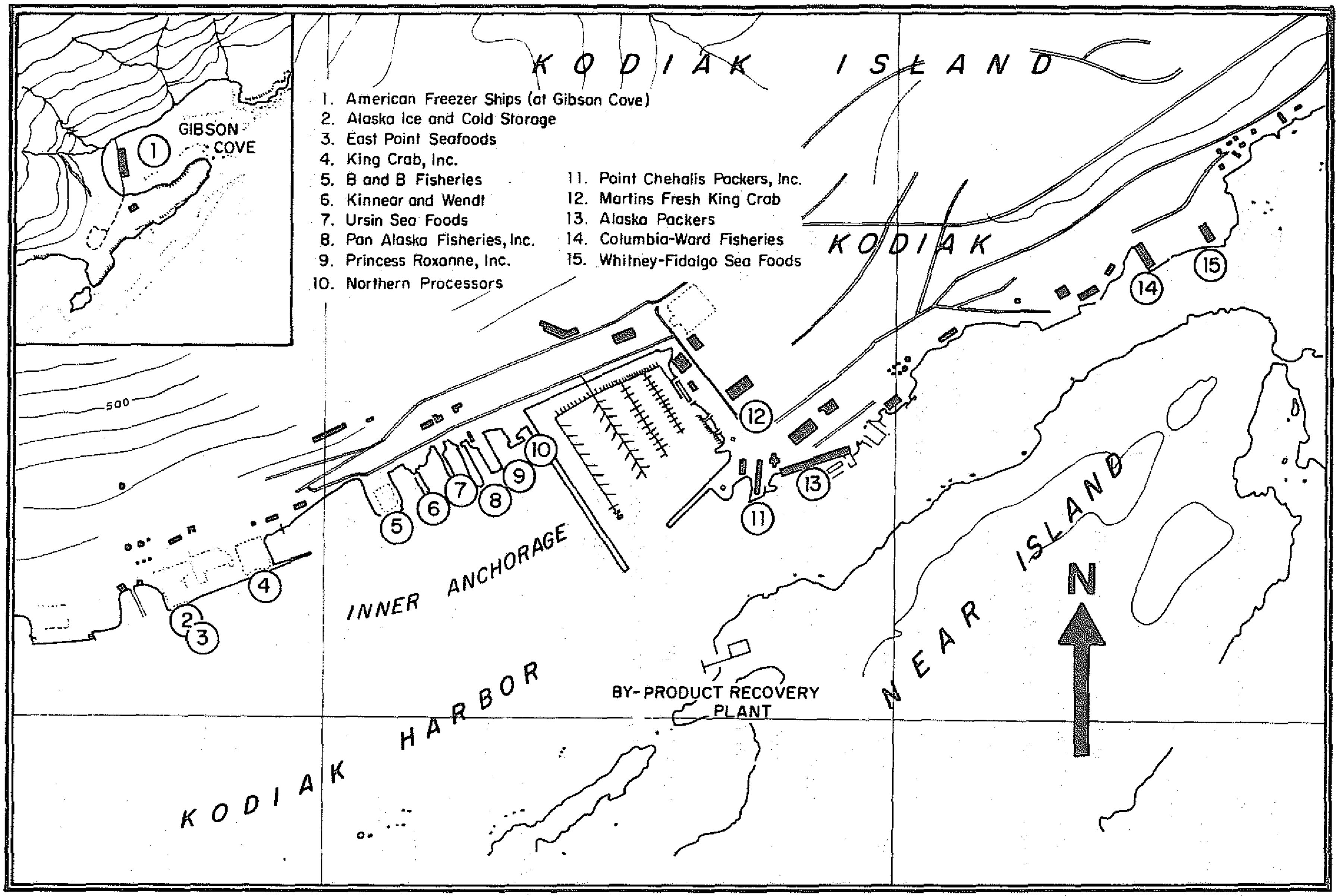


Fig. 30: PROCESSING PLANT LOCATIONS AT KODIAK HARBOR (from E.P.A., 1971)

while an average usage is closer to 40 times. These wastewaters originate from unloading and fluming the fish or shellfish, butchering, filleting or peeling, cooking, can washing, and cooling.

The unit operations involved in a typical shrimp-processing plant are shown in Figure 31 (USEPA, 1974). One can see that there are many places where solid or liquid wastes are generated. The peeling operation generates the largest volume of wastewater and mass of pollutants. At canneries in the Gulf of Alaska, 58% of the wastewater originates in the peeling operation (Mauldin and Szabo, 1974). In a study of shrimp processing along the Gulf of Mexico coast, Mauldin and Szabo (1974) found the process wastewaters were around ten times as strong (as measured by BOD) as domestic sewage.

Even though a sequence of shrimp-processing operations only is shown in Figure 31, similar techniques are employed in processing other kinds of seafood. In a study of salmon processing in Alaska, Jensen (1965) found that the BOD of salmon wastewaters is also very high (≈30K mg/l).

The USEPA (1974) presents results for the catfish, crab, shrimp, and tuna segments of the seafood-processing industry. The authors point out differences between crab processors in Alaska and in the lower 48 states with respect to water usage. Water usage is greater in the former because of greater mechanization. Attention is paid to the portion of crab and shrimp wastes which result from processing (around 80%). Results are presented for the water usages, wastewater characterizations, in-plant controls, and external-treatment technologies. It is pointed out that the highly proteinaceous nature of the wastewaters results in significant nitrogen demands. This has implications for the type of secondary treatment required if the oxygen demand of the wastewaters is to be minimized.

Crucial to the choice of wastewater treatment for the future is reuse and by-product recovery. This factor will obviously play an important role in deciding whether or not a given process is worthy of adoption. Also important are the health implications involved whenever water is reused in a food-processing application. Besides the possibility of water reuse in either the same or another part of the process,

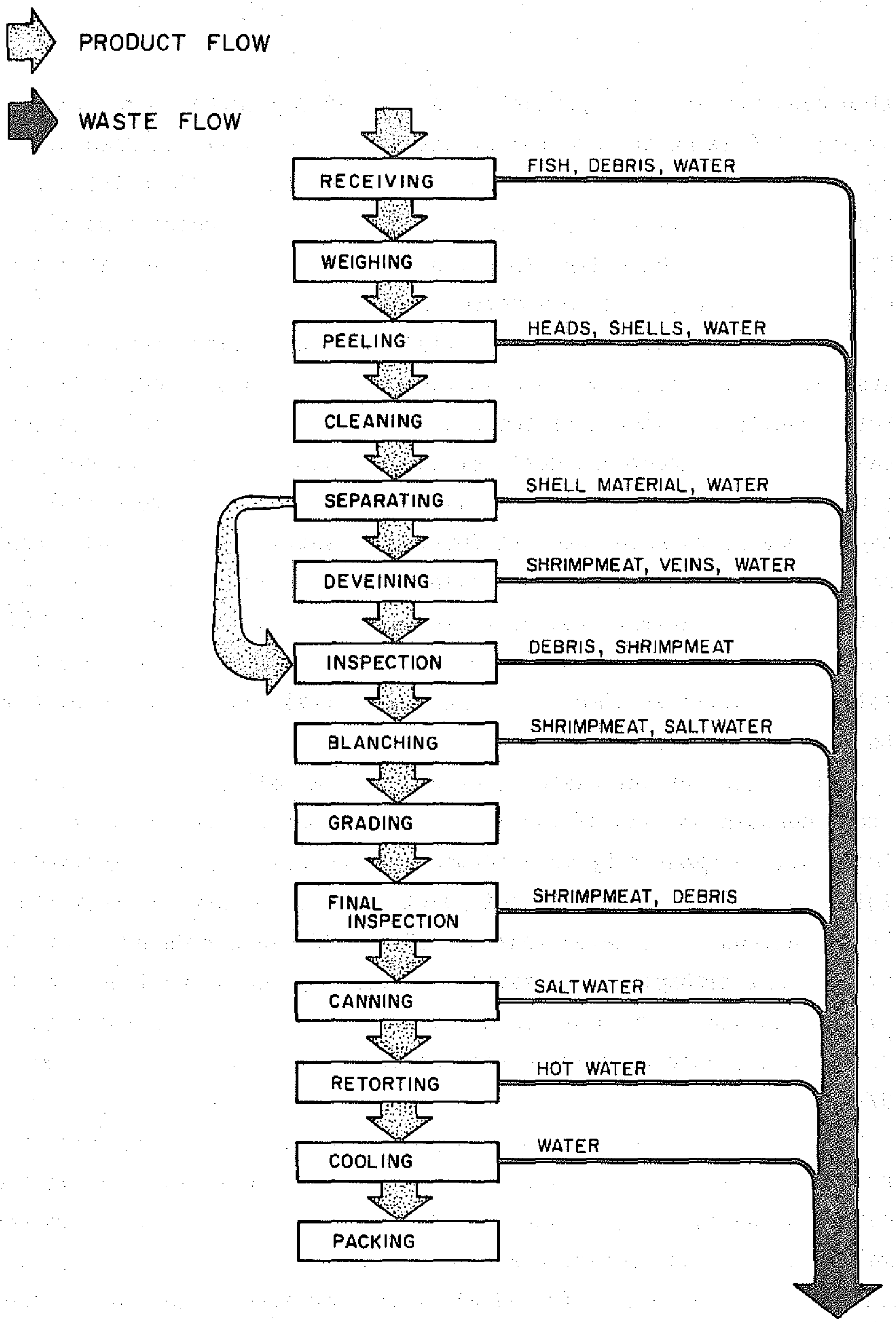


Fig. 31: GENERAL PROCESS SCHEMATIC SHRIMP CANNING (from US E.P.A., 1974)

water conservation is important. The use of dry conveying equipment instead of fluming and control of washdown flows will cut down on water use and pollutant discharge. One Alaskan plant uses 260 gal/ton of salmon merely to flume fish from holding bins to butchering machines (USEPA, 1975a). The extent to which the costs of treating water for reuse can be offset with by-product recovery is important.

Paul (1976) and the USEPA (1971a) present results relating to the feasibility of enhancing yield in conjunction with by-product recovery. These results are important because they indicate a partial recovery, at least, of waste treatment costs by selling some of the recovered solids is possible. Paul pointed out that recovery of high-protein fish meal may be more profitable when the 200-mile fishery zone is established. Then, the higher-protein bottom fish such as halibut may become a larger portion of the harvest brought to the area than at present. In Kodiak, Bio-Dry is the largest producer of shrimp and crab meal in the United States. It utilizes almost all the waste offal (40 M tons/year) from the Kodiak processors.

Since much of the waste material is of a colloidal or smaller size, a considerable portion of the SS and COD is not removed by screening. These must be removed by more advanced processes such as dissolved air flotation (DAF). A 50 gpm pilot plant study using DAF to treat Alaskan shrimp wastewater revealed that the SS and COD were reduced about 75%. With present technology, operation of such a process requires a highly skilled operator. Optimum coagulant dosages must be determined and system upsets must be guarded against carefully (Mauldin and Szabo, 1974).

Mauldin and Szabo (1974) surveyed shrimp-canning plants processing about 20 tons/day. Screening was found to be an efficient way to remove heads and shells. They found a large settleable solids reduction and a small COD reduction achieved via screening. The Bauer (Hydrasieve) tangential screen out performed all the other screens tested. Screening plus DAF removed 75% of the BOD_5, 99% of the SS, and 90% of the oil and grease. Centrifugation was found to decrease the volume of the DAF sludge by a factor of four. In looking at DAF, the study pointed out that the air-to-solids ratio was about three times that used for other industrial wastes. Since these processes don't remove dissolved solids, the amount of wastes discharged would be decreased if flumes were replaced by conveyers. System costs were estimated for screening in combination with DAF, with the solids subsequently hauled to a landfill area after dewatering.

The Best Applicable Technology Economically Achievable (BATEA) guidelines to be met in 1983 require DAF and aeration to treat much of the seafood processing wastes. Even though biological treatment is used at only three locations for the United States seafood industry, most seafood wastes should be amenable to biological treatment. There will certainly be an adequate supply of nutrients. A combined activated sludge-lagoon system in Florida removes 97% of the BOD_5 and 94% of the suspended solids from shrimp-processing wastewater. Land disposal is incompatible with the highly saline wastes at many sites.

In Alaska there exists the possibility of utilizing the cold provided by nature in the winter to freeze wastewater or sludge. This allows the possibility of decreasing the dissolved solids content of the wastewater because the salt will be excluded to the outside of each individual ice crystal (Stepakoff et al., 1974). In warmer climates, the same principles can be utilized, but the process energy is no longer "free." Similarly, sludge, once frozen, possesses enchanced settleability properties. Hence, it can be dewatered more readily than sludge that has never been frozen (Ali Khan et al., 1976). If this method is to be used for all the sludge, storage must be provided during the summer.

The National Marine Fisheries Service (Nelson, 1977) has done some work on water reuse in seafood processing. They envision the possibility of using peeling water further downstream but are worried about cost and sanitation problems. There also exists the possibility of technology transfer from other industries. For potato processing (Environmental Science and Technology, 1973), considerable progress has been made in reducing water usage in the peeling operation. The olive industry found that activated carbon beds would greatly lower the organic levels in storage brines and processing waters from the production of canned ripe and glass-packed green olives. These reconditioned brines could then be used to store freshly harvested olives. The cost was estimated to be \$36/1K gals of reconditioned brine produced (USEPA, 1971b).

Package Plants on Pipeline

Background--

Of the 31 sewage plants along the pipeline in 1975, seven were extended aeration, 23 were physical-chemical and the Valdez plant is biological followed by physical-chemical. According to Alyeska (1975), all these plants were designed to achieve 85% removal of BOD_5 and SS. But with the high strength incoming wastewater, an 85% removal is not equivalent to a 30/30 effluent. For typical domestic wastewaters having influent BOD_5 and SS levels around 200 mg/l, an 85% removal level would just meet secondary standards. For the Valdez terminal for the 11 months ending December, 1974, the average BOD_5 in the influent was 432 mg/l (Alyeska, 1975). Now, the influent BOD_5 levels at Valdez are closer to 800 mg/l (Eggener, 1977). Hence, an 85% BOD removal would produce an effluent containing at least 65 mg/l BOD_5. High BOD_5 and SS levels are typical of construction camp wastewaters. Reed noticed a similar problem with aerated lagoons treating wastewater from air force installations (Reed, 1976).

In the late 1960s, Alyeska's road construction camps north of the Yukon River used extended aeration activated sludge sewage treatment. Prospect and Coldfoot Camps initially used oxidation ditches fabricated out of half sections of culvert and framing lumber. Aeration was provided by standard rotors or brushes, and was followed by clarification and chlorine disinfection. Mechanical equipment was housed in heated structures to provide freezing protection. However the ditches themselves were not enclosed. Some degree of mixed liquor temperature control was provided by covering the ditch with plywood or other material which would allow a build up of an insulating snow cover. The success of these early systems is not well documented as few operational data exist. As the camp populations increased and greater treatment capacities were required, Alyeska abandoned the ditches and replaced them with package extended aeration treatment sytems.

Prospect, Dietrich, and Coldfoot were not the only camps to use extended aeration package treatment. Franklin Bluffs, Atigun, and Chandalar camps employed the use of Smith and Loveless' package extended aeration treatment plants, Galbraith and Happy Valley Camps used Clow package extended aeration treatment, and Five Mile camp used a Chicago Pump extended aeration package plant. The ditches at Prospect, Dietrich, and Coldfoot, as well as the newer camp at Old Man were outfitted with Steel Fabricator's package extended aeration treatment plants. Toolik was the only Alyeska camp north of the Yukon in the early to mid-1970s which did not use package extended aeration treatment. Instead, a Met-Pro physical-chemical treatment plant was installed at this site.

By 1974, Alyeska's camp population projections were once again dictating that additional treatment capacity would be required for the pipeline construction phase of the project. Consequently, Alyeska purchased and installed Neptune Microfloc physical-chemical wastewater treatment plants in all of their pipeline construction camps with the exception of Sourdough camp which was constructed at a later date. All the project's pump station construction camps followed by purchasing and installing Environmental Conditioner's (ECI) package physical-chemical treatment plants. These statements apply to those plants south of the Yukon also where there were no plants prior to the actual pipeline construction since there was no need to build a road there. Hence, the first plants to be installed in this region were physical-chemical plants in 1975.

In addition to the domestic wastes treated at Valdez, ballast water from the tankers is also treated to remove oil. The chemical sludge resulting from this treatment is returned to the crude oil system.

At the permanent pipeline camps, all liquid in the sewage will be evaporated at 427°C by recovering heat from the stack gases. This extra liquid is only a small fraction of the waters of combustion so this should not create additional ice fog problems. Pathogen die-off studies have showed this is a sanitary procedure (Eggener, 1977). This will be the scheme used at all facilities except Valdez and Pump Station 5.

In the following discussion, attention is focused on the particular plants visited by this author. This included Pump Station 1 and 5 and the Livengood, Five Mile, and Galbraith camps. But, the processes discussed are common to other pump stations or camps.

The ECI units (Figure 32), with a design capacity of 25K gpd, use a flow sequence as follows. Initially the wastewater is passed through a set of roto screens to avoid having the pumps destroyed by solids. These

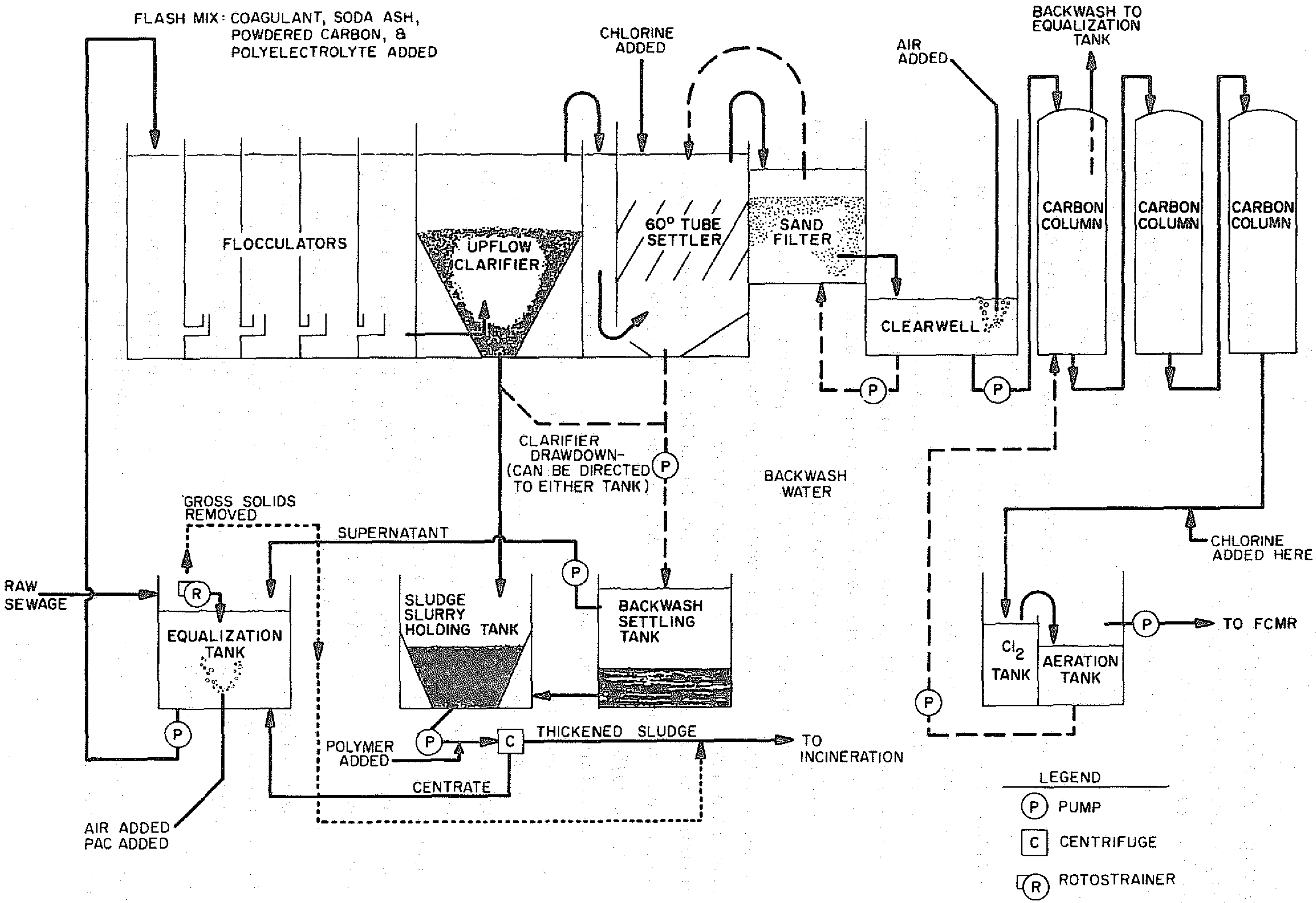


Fig. 32: ECI AST-25 PHYSICAL/CHEMICAL SEWAGE TREATMENT PLANT (Dames and Moore, 1975)

screens remove solids larger than .01" (250μ). Then, the flow is pumped into a 10K gal equalization tank. Although initially intended to serve purely as a surge tank to provide a uniform flow at a constant rate into the rest of the plant, the equalization tank eventually served as a treatment system itself. By both addition powdered activated carbon (PAC) and providing aeration, soluble COD removal efficiencies of 40% to 80% have been measured in the surge tank (Eggener, 1977).

At Pump Station 5, air is blown into the surge tank through 1/8" openings and the resulting DO levels are typically around 8 mg/l according to the operator (Stewart, 1976). No PAC is added in the surge tank. By running his surge tank only half full with a flow of 30K gpd, Stewart achieves a residence time of only four hours. The surge tank is normally only filled halfway to provide reserve capacity in the event of sudden increases in hydraulic loading. The operators are encouraged to run the surge tanks at least half full (Eggener, 1977). With no sludge recycling, this would also be the mean cell resident time θ_c. One would not expect a high biological treatment efficiency at such short θ_cs. However, there are some cells recycled back to the surge tank in the centrate (Eggener, 1977). At Pump Station 1 having a typical flow of 20K gpd, both air and PAC are added in the surge tank. Air is sparged through 1/8" diameter holes at a rate sufficient to maintain DO levels of 8-10 mg/l in the surge tank (Nelson, 1977). Approximately 125 lbm of PAC is added each 24 hours. This maintains a carbon concentration in the surge tank of around 500 mg/l according to Nelson. (These numbers are only estimates as 125 lbm per 20K gals is equivalent to something like 700 mg/l.) Another feature of the Pump Station 1 setup includes 18 inches of plastic trickling filter media at the bottom of the surge tank. This characteristically has a large surface area-to-volume ratio and consequently provides sites for the development of biological films and slimes similar to the growths occurring in fixed growth biological treatment systems. The residence time in the surge tank, assuming it is filled, is about 16 hours at a flow of 20K gpd. At a camp population of 330, this corresponds to a per capita flow of 65 gpd.

After the surge tank, the wastewater (now partially treated) flows to a contact or mixing chamber where various chemicals are added. At Pump Station 5, alum, PAC, soda ash, and a polymer are added with the dosages being 977, 440, 345, and 8.5 mg/l respectively on November 15, 1976 (Stewart, 1976). By July, 1977, these dosages had been reduced to 470, 317, 0, and 3.9 mg/l respectively (Eggener, 1977). At Pump Station 1, lime is added instead of soda ash and only a small part of the PAC dosage (1/6) is added in the mixing tank. The alum dosage is generally under 500 mg/l (Eggener, 1977). Since fixed baffles are used in the flocculator, there is no independent control over (only flow rate) the energy input for mixing. Sludge must be periodically pumped from the bottom of the flocculator to avoid clogging for these ECI units (Eggener, 1977).

After flocculation, solids are removed by a hopper-bottomed upflow clarifier followed by 60° settling tubes followed by a multi-media filter. A top view of this part of the process appears on Figure 33. The SOR on the clarifier at a flow of 20K gpd is about 1K gal/ft^2/day, a reasonable number. SOR for primary clarifiers generally fall in the range between 400 and 800 gal/ft^2/day (Hammer, 1975). The granular carbon columns located after the filters had to be abandoned because of problems with anaerobic growths even though the addition of sodium nitrate helped (Jones, 1977). It was then decided to introduce PAC to the equalization tank. The filter bed is about 4 feet deep with one layer of carbon supported by three layers of sand on top of three layers of gravel (Table 21). The fact that the specific densities of the media materials decreased with elevation allows bed stratification to be maintained after backwashing. At Pump Station 1, the effluent looked very clear in November of 1977. Even the overflow out of the 60° tubes appeared clear. These tubes are set at such a high angle to be self cleaning (Hansen et al., 1969). Nevertheless, they had to be periodically cleaned, mainly for oil and grease removal. The filter is backwashed with 1200 gal of effluent every 6 hours at Pump Station 1 (Nelson, 1977). According to Eggener (1977), the filters are usually backwashed every 12-24 hours.

After chlorination, the effluent passes through a clear well before being discharged into a flow control management reservoir. At Pump Station 5, the chlorine dosage was around 8 mg/l (Stewart, 1976). The latter typically allows a detention time of over 200 days to be attained. A lined holding pond is also available to provide around five days storage capacity in the event of plant shutdown. This effluent is

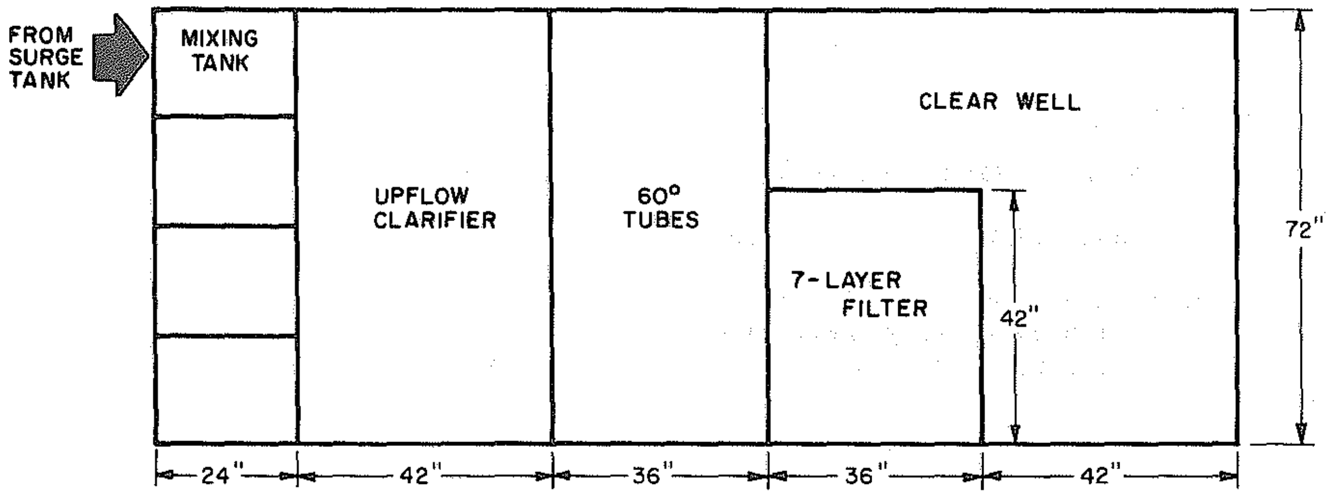


Fig. 33: ECI MIXING, SETTLING, AND FILTRATION MODULE

TABLE 21: FILTER BED MATERIALS

	Depth	Quantity (ft^3)
Support Bed Base		
#1 Gravel (1 1/2" x 3/4")	6"	6
#2 Gravel (3/4" x 1/2")	4"	4
Sand Bed		
#F5 Sand 2.0 mm Effective Size	4"	3
#F9 Sand 1.0 mm Effective Size	4"	3
#F16 Sand 0.45-0.55 Effective Size with uniformity coefficient of 1.45 to 1.60	16"	13
Carbon		
F300 (300 mesh) Granular Carbon	12"	--

SOURCE: Jones, 1977.

supposed to (and typically does) meet secondary standards before being discharged into the reservoir (Dietrick, 1977). The reservoir walls have varying degrees of permeability. If the waters in the reservoir meet 30/30 standards, they are sometimes pumped onto the tundra in the spring (Eggener, 1977). According to Zemansky (1975), sometimes excessive percolation through gravel berms surrounding the reservoirs created stagnant ponds of treated wastewater adjacent to these lagoons. He further claimed that percolation of treated wastewater into streams at Happy Vally and Prospect occurred although data was not presented to verify this. At the two pump stations that do not have reservoirs because of poor ground conditions, long infiltration pads are used (Alyeska, 1975). At Valdez, the effluent from the physical-chemical plant is discharged through a submerged diffuser into the Valdez Arm.

Part of the operating problems at these plants is related to sludge disposal, the details of which will be discussed later. This condition is aggravated by excessive sludge production caused by high chemical doses. Typical doses required when Pump Station 1 was operating using a flocculite "600" polyelectrolyte (prior to July of 1976) are shown in Table 22. Under these conditions, the plant contents were very turbid with filter runs less than five hours. Jar test results indicated these chemical dosages were not really adequate to remove the suspended and colloidal matter present in the effluent from the surge tank. This led to the jar testing of a Magnifloc "837A" polymer at the conditions shown in Table 22. The results revealed dramatic reductions in turbidity over a pH range from five to eight. This extension of an allowable pH range reduced the lime required dramatically. Note that less than 100 mg/l of lime were needed to raise the pH above five. The tube settlers became clearly visible and the floc blanket in the primary clarifier began to fall. This allowed the time allotted for sludge withdrawal from the clarifier to be cut in half (Jones and Pollen, 1976).

After the combination of utilizing PAC plus aeration in the surge tank, the effluent quality from Pump Station 1 improved dramatically. According to Pollen (1977) the effluent BOD_5 dropped from over 100 to at or below 10 mg/l and generally remained at these low levels through May of 1977. Part of the explanation for these low values is associated with the high PAC dosages. Similar comments can be made about the SS.

TABLE 22: CHEMICAL DOSES AT PUMP STATION 1 IN 1976 (Flow = 24 gpm)

	Operating with	
	Flocculite 600	Magnifloc 837A
Alum	1100	800
Lime	400	0-100
Poly (in mixing tank)	15	3.5
Poly (in centrifuge)	70	70 (F600)
Carbon	470	470

SOURCE: Jones and Pollen, 1976.

It should be mentioned that effluent samples at the pump stations were taken right before weekly plant maintenance. Since the latter results in the discharge of some accumulated solids, effluent quality deteriorates immediately after maintenance. But, this phenomenon is short lived (Jones, 1977). This improvement in effluent quality is obviously related to the changes in the operation of the equalization tank and perhaps to the change in polymer. Moreover, Pollen 11977) believes one important reason for the high quality was the establishment of a skilled and highly motivated operator, George Nelson, of Pump Station 1. The single most important technological improvement was the addition of air in the surge tank. With just air at Pump Station 1 without carbon, Bud Mitchell thought he was able to attain a 30/30 effluent over the one week test period (Pollen, 1977). According to Eggener (1977), hard data was not taken to substantiate this. This result is consistent with a correlation between decrease in effluent quality and blower failure at the pump stations (Pollen, 1977). Adding the PAC to the surge tank increased its residence time in contact with the sewage before settling from less than .1 day (when added only in mixing tank), to greater than .5 days. This not only provided more time for the soluble organics to be adsorbed onto the carbon, but also allowed more time for the PAC to serve as seeding material for bioflocculation. The MLVSS levels in the surge tank are sufficiently low (1K mg/l) as to prevent the biological conversion efficiency there from being comparable to that achieved in conventional extended-aeration plants (Eggener, 1977).

Data shown on Table 23 reveals that the alum doses at Pump Station 1 were reduced by almost a factor of two over those shown in Table 22 by July, 1977. According to Eggener (1977), chemical doses were cut significantly when more power was added to the flocculators. Furthermore, effluent BOD_5 levels of less than 30 mg/l were attained at carbon dosages of 100 to 300 mg/l. According to USEPA (1973a), dosages around 200 mg/l are sufficient to treat raw municipal wastewater. Of course, this is less concentrated than that from the pipeline. The higher PAC dosages used at Pump Station 1 (Table 23) resulted in effluent BOD_5 often less than 10 mg/l.

TABLE 23: ALYESKA MONTHLY P/C WASTEWATER TREATMENT PLANT
REPORT FOR PUMP STATION 1, JUNE 1977

Date 1977	Camp Population	Average Chemical Dosage Rate mg/l Alum	Carbon	Poly	Chlorine	Lime	Effluent Discharge gal	Average Effluent pH	Avg. Chl. Residual mg/l	Sludge Burned gal
6/01	250	485	631	3.2	6.8	56	19,188	6.5	1.1	192
6/02	262	475	497	3.0	6.3	52	24,100	6.5	1.2	288
6/03	265	492	620	2.7	6.6	59	19,339	6.2	1.5	258
6/04	277	475	528	2.8	5.9	55	22,698	6.6	1.4	246
6/05	275	288	569	2.3	6.4	off	21,060	6.6	1.0	264
6/06	284	305	472	1.9	11.7		25,389	6.4	1.8	276
6/07	287	305	539	1.8	11.7		22,230	6.3	1.4	288
6/08	305	254	578	2.0	9.5		15,561	6.3	1.5	240
6/09	286	271	592	2.2	9.6		20,252	6.4	1.2	246
6/10	276	254	567	2.1	11.0		21,243	6.4	1.3	288
6/11	271	163	472	1.7	11.6		25,389	6.3	1.4	288
6/12	258	221	449	1.5	10.9		26,676	6.2	1.2	276
6/13	262	237	264	1.8	10.9		22,698	6.2	1.3	288
6/14	274	229	481	1.9	11.0		24,900	6.3	1.0	288
6/15	287	254	541	2.0	9.3		16,614	6.5	1.1	288
6/16	305	245	481	1.8	10.5		24,921	6.3	1.1	288
6/17	315	298	479	2.3	11.6		25,000	6.2	1.0	288
6/18	319	487	452	2.1	10.5	56	26,500	6.3	2.0	288
6/19	317	625	481	2.4	10.5	110	24,921	6.3	1.8	288
6/20	316	625	468	2.5	9.4	112	25,629	6.4	1.9	288
6/21	307	516	482	2.5	8.2	108	24,895	6.2	1.9	288
6/22	289	489	500	2.4	8.2	50	23,984	6.4	1.6	288
6/23	257	475	492	2.4	7.6	52	24,391	6.4	1.8	264
6/24	263	489	507	2.3	6.2	51	23,649	6.4	1.5	288
6/25	241	503	490	2.4	5.9	51	24,452	6.4	1.5	276
6/26	241	475	528	2.1	6.4	50	22,695	6.3	1.5	252
6/27	228	462	565	2.0	6.2	49	21,210	6.5	1.5	288
6/28	235	475	639	2.2	6.2	50	18,749	6.5	1.4	288
6/29	250	489	501	2.1	6.6	52	17,950	6.5	1.6	264
6/30	243	502	466	2.3	6.6	53	20,560	6.7	1.4	288
Total							676,843			
Average	275						22,561		1.4	275

Notes: Reservoir 5% full by end of month. Amount of potable water pumped unknown as system has no meters.
Estimated daily use of potable water: 30K gal.

SOURCE: Eggener, 1977.

According to USEPA (1975b), the adsorption of organics onto PAC is very fast, with equilibrium commonly being reached in less than ten minutes. Hence, it appears that the increased contact time is more important for bioflocculation than increased adsorption. Moreover, Fluor personnel at Valdez (Jones, 1977) found the addition of PAC prior to coagulation reportedly results in greater organic adsorption than its addition simultaneously with coagulants in a flush-mixing tank. This observation conflicts with those of Stukenberg (1975) who reported no significant differences in PAC adsorption when added before or during chemical coagulation. This latter observation was made on normal strength wastewaters, whereas the observations at Valdez were made on camp wastewaters requiring higher chemical dosage rates. Eggener (1977) believes the data obtained at Valdez is inconclusive.

Ultimate sludge disposal is via incineration. At Pump Stations 1 and 5, the sludge is centrifuged after being collected as underflow from the settling tubes and clarifier. The concentrate from the centrifuge is fed into an incinerator via nozzles. The centrifuge typically captures only about 50% of the solids from the combined carbon-alum sludge. The solids left in the centrate are believed to serve as "seeds" for the microorganisms in the equalization tank after it is recycled to the head end of the plant (Eggener, 1977). At Pump Station 5, the dewatered sludge has to be diluted somewhat to avoid clogging the nozzles, which is a waste of energy (Stewart, 1976). One hundred gpd of waste oil is burned with the sludge at Pump Station 5. At Pump Station 1, waste oil was not normally burned, and the total volume of sludge burned per day is about 330 gals. With a camp population of around 330, this is equivalent to 1 gpcd. Ninety percent of this is from the centrifuge and 10% from chemical toilets. The sludge is exposed to three different burning zones: 1) a low-temperature muffle furnace, 2) a higher-temperature main chamber, and 3) an afterburner. The garbage only gets burned twice by not passing through the muffle burner. The incinerators use about .3 gal fuel per lbm sludge which amounts to .3 gpcd. At 75¢/gal, this amounts to 20¢ pcd (Jones, 1977). About two barrels per day of ash get produced by the incinerator with one half of this originating from the sludge. There have been problems with permafrost melting under the incinerator and, to a lesser extent, under the rest of the plant. This has occurred even with a gravel pad 5 ft thick (Zemansky, 1975; Eggener, 1977). The chemical sludges containing alum tend to form a glossy coating on the inside incinerator surface. This must be removed occasionally (Eggener, 1977).

Neptune-Microfloc Package Plants--

The second kind of physical-chemical plant used is a Neptune-Microfloc unit having a nominal capacity of 75K gpd. Unlike the ECI units, this system was meant to be used with PAC. After comminution, the flow passes through a 40K gal aerated equalization tank (Figure 34). At Five Mile, periodic steam cleaning is required to remove the heavy grease buildup on the sides of the surge tank. At Galbraith, the lack of a screen at the inlet led to solids clogging up the system components such as the incinerator nozzles. It should be mentioned that there was no screen initially at Five Mile but that one was installed by the operators (Alleva, 1976). The liquid drawn off from the bottom of the surge tank passes into a chemical-mixing tank where alum, PAC, and a polymer are added. On the days we visited the camps (November, 1976), the dosages were 400, 900, and 1 mg/l for Livengood, and 378, 480, and 1.2 mg/l for Five Mile. At Galbraith, the PAC dosage varied from 600-900 mg/l with the alum dosage a function of pH. According to Eggener (1977), 100-300 mg/l of PAC is sufficient. Alum dosage rates required to achieve clarity in the physical-chemical units were a function of the efficiency of the sludge-dewatering process. For example, if the centrifuges in the ECI system were overloaded or operated without the benefit of proper polyelectrolytes, the centrate would return a high concentration of solids to the equalization tank. This would then increase the solids loading to the chemical coagulation process and cause the operators to use increasing amounts of chemicals in order to achieve the same level of clarity in the process flow. Increased chemical usage would then create more sludge to be processed and thus place a greater burden on an already overloaded centrifuge. Because of this, some unusually high sustained alum dosage rates were observed, sometimes in excess of 1K mg/l. In contrast, with good solids separation in the sludge dewatering step, alum dosage rates were observed to be significantly reduced, often around 750 mg/l or less. Eggener (1977) says the required dosages are

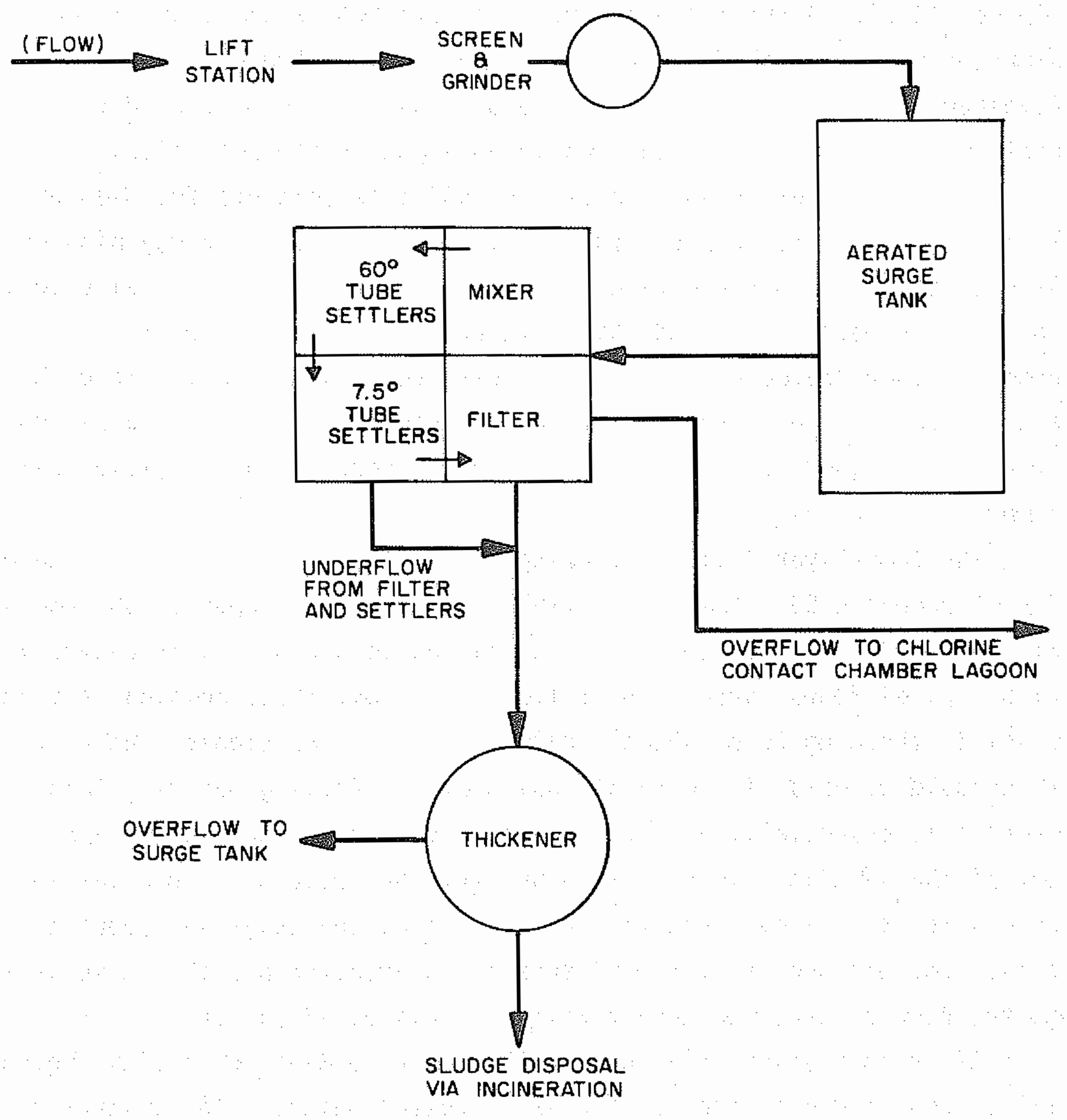


Fig. 34: SCHEMATIC OF NEPTUNE-MICROFLOC PACKAGE PLANT PLUS SURGE TANK AND SLUDGE DISPOSAL ADDITIONS.

now <500 mg/l. The pipeline camps using gravity thickening to dewater sludge have developed higher solids capture efficiencies than the pump station camps and consequently have slightly lower alum dosage rates (Jones, 1977). After flash mixing, flocculation is aided by mixing blades on vertical shafts whose rotational speeds could be adjusted (Eggener, 1977). These adjustable energy-input flocculation devices performed better than the baffled arrangement in the ECI units.

The sewage next goes to a bank of 60° tube settlers followed by 7.5° tubes. The residence time in the former is about thirty minutes. At Galbraith, an operator reported that grease balls periodically collect on a screen mounted over the 60° settlers. They have to be manually removed after drawdown. The tubes themselves have to be washed by hand about once per week. Eggener (1977) reported that the 7.5° tubes sometimes got clogged with anaerobic growths. These could be removed using compressed air when backwashing.

The four-layer filter following the tube settlers does an excellent job of removing SS. Tom Cannon (1976) at Livengood reported SS removals of 90% and 99% in the tube settlers and settlers plus filter respectively. With typical flows through the filter of 60 gpm, these removals are at specific throughputs of about 5 gal/ft^2/min, a much higher loading rate than could be used with conventional gravity settling basins. This means that essentially all the PAC is removed from the sludge dropping out of the settling tubes. At Galbraith, the twice-daily backwashing used a total of about 3K gals. At Five Mile, the usage was about twice this. The effluent from the filters is chlorinated and then discharged to the flow control management reservoir discussed earlier.

After thickening in a conical thickener to maybe 6% solids (Eggener, 1977), the sludge is incinerated as discussed before. The thickener overflow goes back to the head end of the plant. This thickener replaced a sidehill screen and two settling tanks which didn't perform well. According to Cannon (1976), about 250 gpd of fuel is used in two incinerators to help combust 2800 lbm of garbage in one burner at 650°C and 700 gal of sludge in the other at 1100°C. About 25 ft^3 of ash is produced from this operation. At Five Mile, about 165 gpd of fuel were used for 1K gal of sludge, thus producing about ½ barrel of dry ash (Alleva, 1976). Besides this ash, about another 1½ barrel of solid waste is discarded daily. Sludge from the bottom of the five-day holding pond is shoveled directly into the incinerator. Alleva reported he was hard pressed to incinerate the sludge fast enough. The operators at Galbraith said the incinerator caused them the most problems. It had to be shut down every three months for repairs. Eggener (1977) reports a sludge solids production of around .7 lbm pcd which is double the influent solids. This is consistent with an influent SS level around 400 mg/l and the addition of over 1K mg/l of chemicals.

According to Tom Cannon (1976), the Livengood plant had been operating well in 1976. Of the weekly samples, 18% revealed BOD_5 effluent values over 30 mg/l. The data we saw revealed generally the BOD_5 removals to be better than 90% and the SS removals to exceed 97%. It appeared that the most significant variable upon which BOD_5 removal depended was PAC dosage. As long as it was kept $\geq$300 mg/l, good removal efficiency was achieved. The critically important biological conversion in the aerated surge tank is assumed to be constant in this discussion. Limited data (Eggener, 1977) indicate 85% COD removals with no PAC. Chlorine dosages required to achieve a residual of 1 mg/l varied from 3-8 mg/l. At Five Mile, the BOD_5 removal efficiency had only been less than 85% three times in 1976. Alleva (1976) thinks this can be traced to sludge carry-over from the thickener. Eggener (1977) reports that this seldom happened.

Comments on Operation of ECI and Microfloc Plants--

From this discussion, one can notice that the main differences between the ECI and Microfloc units are: 1) the former was initially designed for carbon columns and the latter for PAC and 2) the former utilizes centrifuges for dewatering while the latter uses thickeners. Here, by ECI unit, we mean the complete waste treatment system at the pump stations even though Environmental Conditioners only supplied the basic physical-chemical treatment unit. The surge tanks and sludge dewatering systems were additional equipment. Similar comments apply for the Microfloc units at the pipeline camps.

From discussions with operators and others familiar with physical-chemical plants along the pipeline, I've heard many comments about how they should be operated, what improvements could be made, etc. Eggener

(1977) mentioned the abrasiveness of the PAC wearing out the centrifuge desludge tips and pumps. Diaphragms on the chemical feed pumps kept wearing out until diaphragm pumps were replaced by peristaltic pumps. Other problems mentioned were tank corrosion and control panel malfunction. The former was aggravated by low pH conditions caused by alum and chlorine. Protective coatings would help here.

Operators at Livengood and Five Mile (Cannon, 1976; Alleva, 1976) mentioned disposal of sludge from the holding pond or lagoon as being a headache. Eggener (1977) reported these ponds were hardly ever used. At Livengood, the five-day holding pond was 70% filled in November with 40% of this being sludge. It was unclear at the end of 1976 on which land site this was to be disposed of as the camp was due to be closed. Alleva (1976) suggested that more flush tubes on the tanks would make cleaning easier and that a more complete operating manual would be helpful. Eggener (1977) mentioned that it is difficult to have a complete operating manual when the plants are being modified. Operators at both Five Mile and Galbraith pointed out that the basic conditions created by large amounts of laundry wastes fouled the system by killing microorganisms. This may happen when a large number of personnel is about to vacate the camp. Alleva (1976) said this problem was not so severe in the past when there were two surge tanks. For reasons not known, the DO levels in the surge tanks at Five Mile were frequently zero in the fall of 1976 (Alleva, 1976). This might have been caused by blower failure.

The lagoon at Five Mile is lined with rip-rap on the order of 8 in. in diameter. According to Dietrick (1976), the walls are highly permeable with effluent quickly draining out the low end. Dietrick has seen data indicating that part of the lagoon was aerobic and the lagoon effluent anaerobic. We noticed the effluent from the French drain at Pump Station 5 was septic. Such conditions, maybe arising in part from sludge at the bottoms of the lagoons, could lead to unpleasant conditions in the receiving environment. Dames and Moore (Pollen, 1977) positively identified sulfur bacteria in the effluent from the French drain at Pump Station 6. This is a good indication of anaerobic conditions and the sulfur can perhaps be attributed to the presence of contributing sulfate in the effluent.

At Pump Station 5, Ray Stewart reported his main problem was related to the plant operating continually at or above its rated capacity of 20 gpm. But, plant records indicate the maximum flow never exceeded 20 gpm in 1976. At Galbraith, the system will be overloaded if the camp population climbs much above 1K (design flow 60 gpm). This population was never approached in 1976 or 1977. Similar comments can be made about other plants in 1976 and 1977. Zemansky (1975) indicates the controlling factor on plant performance, during the period when his study occurred, was loading rate. Performance suffered when plants were overloaded both hydraulically and organically. Eggener (1977) said he believed there was a correlation between discharge violations and camp population. But, he added that the physical-chemical treatment plants designed for 60 gpm could be run at 100 gpm with good performance. Recent data indicate the plants are not hydraulically overloaded. Moreover, the blowers have adequate capacity to maintain reasonable DO levels in the surge tanks for the organic loading experienced (Eggener, 1977).

Biological Package Plants--

In addition to the physical-chemical plants discussed so far, there are also biological plants both along the pipeline and at living quarters for private contractors at Prudhoe Bay. Valdez has a combination biological plant followed by a physical-chemical plant. Of course, all the physical-chemical plants incorporate some biological treatment, as discussed. Limited data indicate the reaction rate constants for these degradation processes are not different from those for domestic wastes (Eggener, 1977). The Happy Valley biological treatment plant has consistently produced effluents meeting 30/30 standards. It is important to mention that its loading is steady without large fluctuations. Because of fluctuating camp populations, modular construction is advisable for biological treatment plants (Eggener, 1977). The sludge produced biologically doesn't thicken under gravity as well as that produced in a physical-chemical plant (approximately 1.5% vs 6% solids) (Eggener, 1977).

Biological treatment with extended-aeration package treatment systems has been proven effective in producing high-quality, treated effluents in Alaskan construction camps. The design of these systems must incorporate features which will handle the wastewater adequately. One important item is the inclusion of an aerated surge tank upstream of the system's main aeration tank. The surge tank should be sized to handle between 20% and 25% of the total, daily, influent flow. The aeration tank of the treatment plant should be fed from the surge tank at a constant feed rate and its size should be such as to provide a full 24-hour detention period. The clarifier should have provisions for producing a continuous stream of return sludge to the aeration tank as well as provisions for wasting concentrated settled sludge. No obstructions should impede the operator's efforts to scrape down the sides of the clarifier--a daily task. Sludge return rates vary depending upon operator preferences. However, return sludge ratios of .75 to 1.50 are common. Clarifier overflow rates vary among manufacturers. Typical SORs for larger municipal extended aeration systems are around 300 to 600 gal/ft^2/day. Smaller package, extended-aeration plants tend to use lower overflow rates, as the cost for doing so is not prohibitive. Typical SORs in camp, package, extended-aeration systems are 100 to 250 gal/ft^2/day. However, it should be noted that oversized clarifiers may require higher return sludge rates to avoid excessive denitrification in the settled sludge (Jones, 1977).

Aeration capacity is also a prime factor in the successful performance of construction-camp, wastewater-treatment systems. Aeration recommendations outlined in the Ten State Standards (American Society of Civil Engineers, 1975) suggest that, for a diffused aeration system, an air supply capacity of 1500 ft^3 per lbm of influent BOD_5 be provided. This is for conventional activated sludge processes with MLVSS concentrations under 5000 mg/l. One would assume that higher MLVSS concentrations in the ranges encountered in construction camps (i.e. 5000 to 7500 mg/l) would have somewhat higher oxygen uptake rates and correspondingly higher aeration requirements. In addition to supplying air for aeration-tank oxygen and mixing, air is used for surge-tank aeration, sludge-air lift pumps and, in some installations, to operate aerobic digestors. The total air requirement is usually exceeded with a comfortable factor of safety, and often a secondary blower is provided for backup in the event one blower should fail. For example, at Happyhorse, the blowers supply air for the aerobic digestor, aeration tank, surge tank and lift pumps. The mininal capacity of the treatment unit is 16.5K gpd. The two blowers supplied with the unit are each capable of supplying 240 cfm of air at standard conditions. Hence, one of these blowers supplies all the air required for normal operation (Jones, 1977). With S_0=600 mg/l, the influent BOD_5 loading is about 80 lbm/day while each blower can supply 350K ft^3 day of air. This corresponds to 4200 ft^3 per pound of influent BOD_5. Not all of this, of course, goes into the aeration tank.

The two biological treatment plants which were personally examined on our visit north of the Yukon were used by private contractors. They both were of the extended-aeration type. The one at Mukluk utilized a rotating bio-filter. With no attempt at sludge wastage, it is not surprising that the effluent appeared very turbid. It is essential to waste some cells even in an extended-aeration process because the conversion efficiency into innocuous end products is not 100%, even when the microorganisms are in the endogenous respiration phase (Schroeder, 1977). Data was not available at Mukluk with respect to plant performance.

Alaska Constructors, Inc., has a sewage treatment plant at Prudhoe Bay consisting of one 10K gpd Steel Fabricator plant and two Biopure units. This discussion will focus on the former. A 16K gal surge tank precedes these three extended-aeration units to serve a camp population of about 390. For the month of October, 1976, the flows into the 10K gal Steel Fabricator aeration unit varied between 10K and 20K gpd. Air is provided via diffusers with the sludge being continuously recycled from the clarifier (Figure 20). About once every two weeks, the recycle pump is shut down and the sludge allowed to settle to the bottom of the clarifier. It is then pumped to an aerobic-anaerobic digester where it is stored an average of 10 days prior to incineration. After chlorination, the effluent goes into a lagoon (Miller, 1976). The clarifier supernatant appeared clear on the day of our visit except for occasional bits of floc.

With a typical flow into the steel fabricator aeration tank of 15K gpd, the hydraulic residence time is around 16 hours. Using data supplied by Henry (1976) with MLVSS levels around 5K mg/l, one can calculate that about 4K lbm of cells are in the aerator at any one time. With the average daily wasting of 40 gals of sludge from the clarifier, we can calculate a wastage rate of 32 lbm/day if the sludge wasted is assumed to be 1% solids. These two numbers, when combined, yield a mean cell residence on of the order of 12 days, a reasonable number for an extended-aeration plant. This combined with an SOR in the clarifier of around 150 gal/ft^2/day suggest the plant is being run in a reasonable way. Dissolved oxygen levels in the mixed liquor for the months of October and November, 1976, were on the low side, averaging around .6 mg/l. This may be marginally adequate.

Performance data for October 28, 1976, revealed the BOD_5 and SS removals to be 97 and 93% respectively. Data for the six-week period ending November 16, 1976, reveals the average BOD_5 and SS removals were 93 and 82% respectively. The mixed liquor temperatures averaged over 22°C so the microorganisms were certainly not feeling the effects of the Arctic. Performance plots appear on Figure 35.

A recent comparison of data for northern and nonnorthern extended-aeration plants concluded the northern plants performed significantly worse (Smith and Given, 1976). Here, northern plants were defined as those north of the mean annual 0°C isotherm. As shown in Figure 36, ten of the thirty-five northern plants examined were along the pipeline. The reasons given for this poor performance were poor design and operation. Two of the problems were hydraulic and organic overloading. Another operational problem was too low a level of MLVSS in the aeration tank because of solids accumulation in the clarifier or clogging of the sludge return pump. Lack of trained operators was cited as a main reason for this poor performance.

Results for BOD_5 and SS removals are shown in Figures 37 and 38. It is seen that the plants tested by the NSF performed significantly better than either of the two sets of plants in the field. This illustrates the importance of operating under controlled conditions with qualified operators. Another contributing factor is the much greater variation in wastewater strengths and flows for the field data compared to the NSF data. When poor plant performances were catalogued as functions of specific operating and design problems, no one problem emerged as dominant. Any explanation could best be summarized by invoking Murphy's Law:

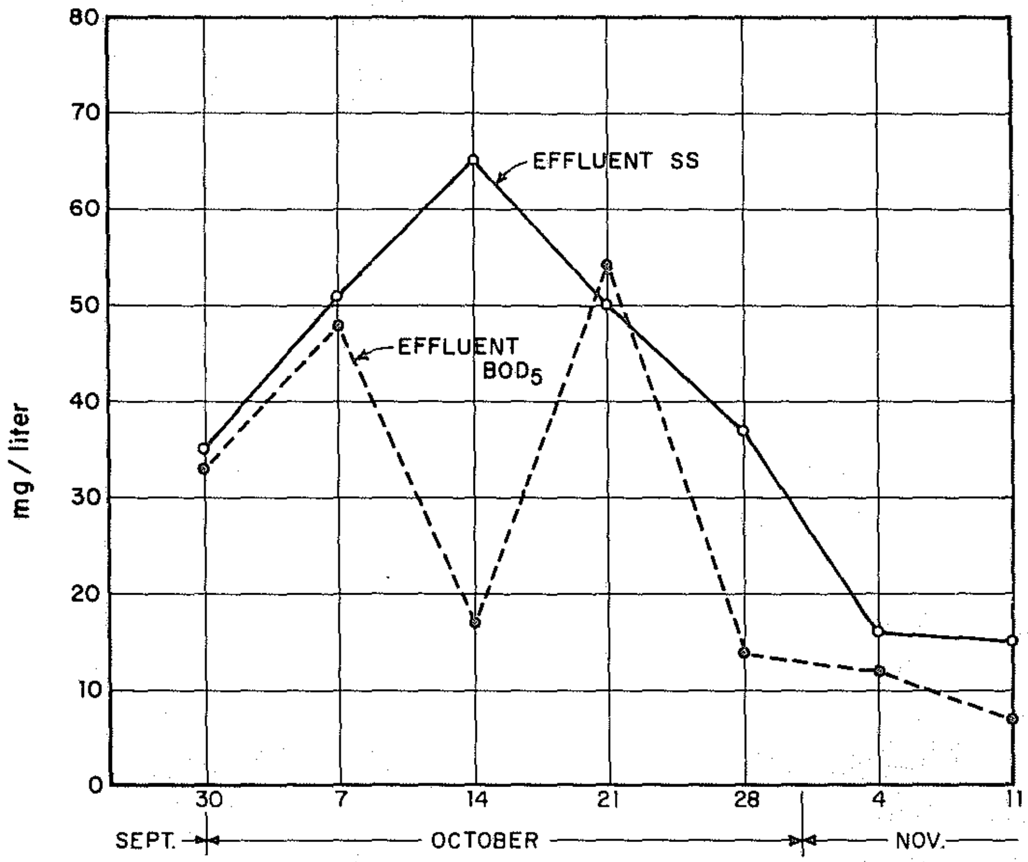


Fig. 35: PERFORMANCE OF 10 K GPD STEEL FAB PLANT AT CRAZYHORSE. (Henry, 1976)

Fig. 36: LOCATIONS OF NORTHERN AND NON NORTHERN EXTENDED-AERATION PLANTS (Smith and Given, 1976)

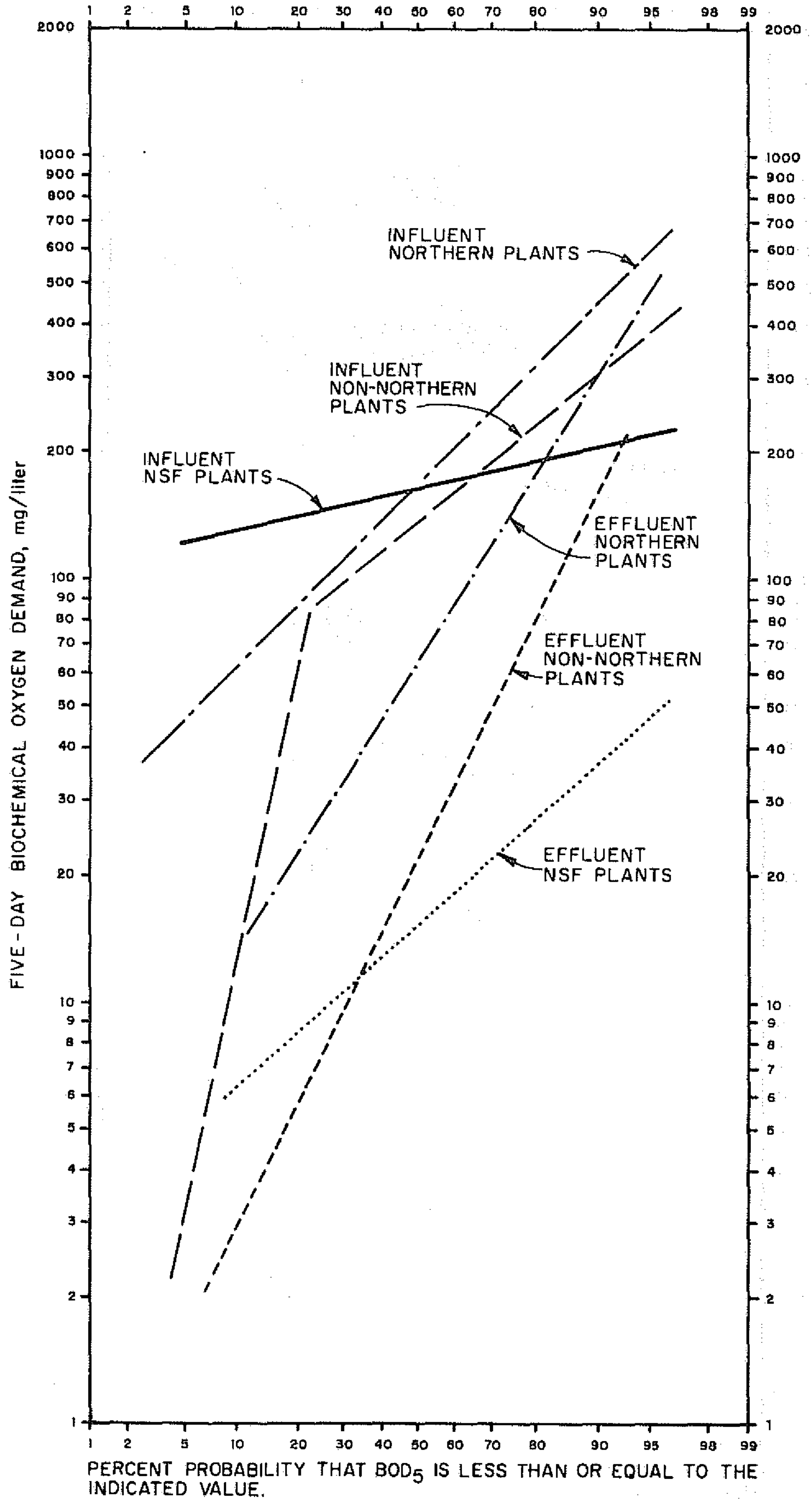


Fig. 37: COMPARISON OF INFLUENT AND EFFLUENT BOD_5 VALUES FOR NORTHERN, NON NORTHERN, AND NSF EXTENDED-AERATION PLANTS (Smith and Given, 1976)

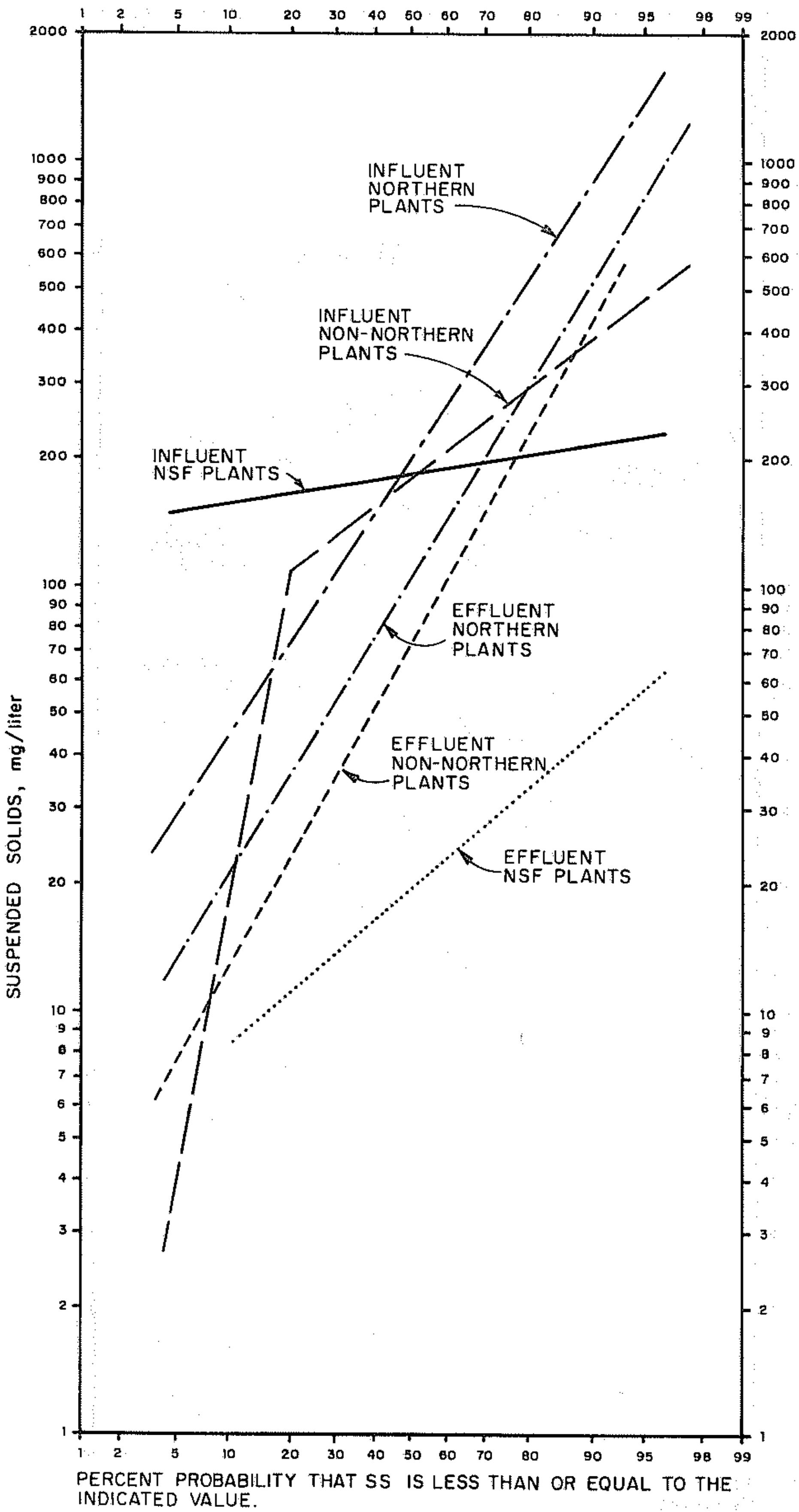


Fig. 38: COMPARISON OF INFLUENT AND EFFLUENT SUSPENDED SOLIDS FOR NORTHERN, NON NORTHERN, AND NSF EXTENDED-AERATION PLANTS. (Smith and Given, 1976)

"Anything that can go wrong will go wrong." In addition to the several problems already mentioned, poor selection of equipment and pipe sizing were included among several dozen deficiencies.

To correct the solids buildup and/or return sludge pump clogging problems, operators should scrape the clarifier and/or check the sludge-return pumps regularly. Smith and Given (1976) suggested that perhaps the designer could "design around" this problem. For example, one may be able to employ floating tube settlers in the aeration tank. This writer believes that this, in turn, would not work unless the operators were enough to clean the tubes periodically. Once again, we are back to the need for good operators. This idea is consistent with only very few northern problems being identified in Smith and Given's study. The three mentioned included freezing problems with exposed oxidation ditches and even in encapsulated plants at lower elevations and the possibility of plant flooding if the effluent line froze.

Pipeline Performance Summary--

In general, the performance of the plants along the pipeline has improved (Tables 24 and 25) since Smith and Given's study. It has continued to improve through 1976 and 1977. The data in Table 25 indicates superb BOD_5 removals at the pump stations (Typically >98%) with the effluent quality at Five Mile being very good. Galbraith appeared to violate a discharge standard on the order of once a month. SS removals at the pump stations were also excellent. Other data (not presented here) reveal the high BOD_5 removals found for the first half of 1976 (Table 24) to be equaled or surpassed since then. Undoubtedly, much of this improvement can be attributed to better-trained operators. It must be remembered that many of these operators had little or no experience with sewage treatment plants when they first were assigned to these positions (Eggener, 1977). Moreover, the frequency of plant overloading has diminished since the break-in period. The use of surge tanks has also helped to alleviate the overloading problems.

TABLE 24: MONTHLY AVERAGE % BOD_5 REMOVED FOR PIPELINE & PUMP STATION CAMPS

	1975													1976
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec		Jan-Jun
Camps														
Sheep Creek	87	87	86	96	88	96	83	88	85	88	93	93		
Tonsina	97	87	83	93	81	77	84	82	82	83	94	74		
Glennallen	93	93	86	90	80	81	74	88	64	86	90	88		
Isabel Pass Bio										88	94	95		
Isabel Pass P/C		92	97	70	92	93	92	84	67	84	81	97		
Delta P/C	94	94	96	92	93	83	90	92	91	86	84	89		
Delta Bio								96	88	82	89	81		
Livengood	93	95	78	78	57	78	84	83	87	90	78	84		
Five Mile			96	74	84	56	71	76	83	84	91	87		
Old Man			94	87	89	74	85	86	88	90	90	96		
Prospect			96	91	93	86	79	88	89	91	94	97		
Coldfoot			95	91	93	84	92	90	87	93	96	95		
Dietrich			94	92	62	82	74	87	89	89	92	86		
Chandalar	90	96	98	88	91	95	98	83	66	93	96			
Atigun I	93	90	94	90	81	84	79	96	88	83	94	93		
Atigun II							83	92	86	81	94	80		
Galbraith					86	85	89	86	87	93	88	90		
Toolik		75	96	84	90	94	92	92	79	90	94	95		
Happy Valley P/C					89	93	90	95	86	89	91	99		
Happy Valley Bio	95	94	96	96	96	96	97	96	90	92	95	96		
Franklin Bluffs			75	74	70	81	68	79	63	82	79	70		
Average														94
Pump Stations														
# 1	63	78	84	80	79	81	91	85	83	86	84	90		
# 3		77	82	95	87	94	92	93	94	91	90	92		
# 4		83	88	97	89	96	96	94	91	96	90	91		
# 5					68	82	90	90	81	85	78	92		
# 6						82	92	95	95	90	84	79		
# 8			65	76	89	93	96	98	94	92	89	90		
# 9						80	88	96	95	88	88	91		
#10			79	72	83	87	72	92	75	83	83	87		
#12						79	74	89	84	90	82	83		
Valdez														98
Average														90
Averages														
85% or above	89	64	67	62	58	44	57	79	62	73	70	76	67*	
80% or above	89	87	86	72	79	81	71	93	79	100	90	90	85*	
75% or above	89	100	90	81	88	93	82	97	90	100	100	93	92*	

* Overall average during 1975.
SOURCE: from Alyeska, 1975, and Eggener, 1977.

TABLE 25: RECENT EFFLUENT DATA NORTH OF YUKON

Date (1977)	BOD$_5$					
	Pump 1	Pump 3	Pump 4	Pump 5	Galbraith	5-Mile
2/03	<6	17	< 6	< 6	15	
2/08		29			22	11
2/14		15			55	8
2/21		12			33	19
2/28		< 6			21	20
3/07					17	25
3/14					28	31
3/21			9		< 6	61
3/28		9	< 6		51	11
4/04		7			85	11
4/11		< 6			< 6	20
4/16		7			53	15
4/24		7			17	17
5/01		11			31	16
5/09		13			19	15
5/16		7			21	8
5/23		8		12	17	< 6
5/30		< 6	27	< 6	15	7
6/06		< 6	30	13	33	12
6/13			< 6	< 6	7	6
6/20			9		>170	< 6

Note: some of the dates may be off by one day.
SOURCE: Dietrick, 1977.

Early Arctic Experience--

Consideration of earlier Alaska industry experience in Arctic Sewage Treatment (Clark et al., 1971; Coutts, 1972) lets us see how much progress has been made in the last six years. At the time the physical-chemical treatment plants for the pipeline were purchased in 1975, very little operating experience had been acquired with them. As of 1971 (Clark et al., 1971), there were only two physical-chemical plants in operation on the North Slope and over a dozen biological plants. The physical-chemical plants all incorporated alum flocculation, upflow clarification, downflow carbon adsorption-filtration, upflow carbon adsorption, and chlorination. The biological plants were mostly of the extended-aeration type as can be seen from Table 26. As can be seen from the data on Table 27, the effluent quality was not very good in general.

In general, the physical-chemical plants outperformed the biological plants. The good performance of Plant E was the result of a conscientious operator who supplemented the feed with kitchen wastes because the plant was so lightly loaded. Since these biological plants did not practice sludge wasting, it is not surprising that the effluent BOD_5 or SS levels were higher than 30 mg/l. Some reasons why the physical-chemical plants did not meet 30/30 standards may include greatly varying feed flow rates, infrequent carbon backwashing, and untrained operators.

Discussion

Alaskan industries are in a unique position with respect to those in the rest of the country both because of the low population densities and the unspoiled nature of much of the environment. The fact that such a situation exists could serve as a basis for a philisophical discussion of whether or not they should be required to attain the same effluent standards as industries in the lower 48. However, such is not the scope of this report. Instead, we will focus purely on the technological aspects of industrial wastewater treatment.

Of the four kinds of industries mentioned in this study, the wastewater treatment schemes adopted to date by the seafood industry (screening) has resulted in minimal additional costs (Hammond and Mueller, 1977).

TABLE 26: NORTH SLOPE AND ARCTIC SECTOR PIPELINE WASTEWATER TREATMENT FACILITIES, MARCH 1971

Activity	Location	Type	Unit Capacity GPD	Discharge	Design Population	Effluent Chlorinated
Pipeline	Toolik	Phys-Chem	24,000	Lined lagoon	340	yes
Oil field	Deadhorse	Phys-Chem	24,000	Lined lagoon	75	yes
Oil field	Mikkelson	Incinceration	5,000	Tundra pond	50	yes
Oil field	Deadhorse	Ext. Aer.	2,500	Tundra pond	50	yes
Pipeline	Galbraith	Ext. Aer.	19,000	Lagoon	320	yes
Pipeline	Happy Valley	Ext. Aer.	24,000	Sagavanirktok	220	yes
Oil field	Kuparuk	Ext. Aer.	6,000	Kuparuk	60	yes
Oil field	Deadhorse	Ext. Aer.	5,000	Tundra	50	yes
Oil field	Deadhorse	Ext. Aer.	7,000	Tundra pond	50	yes
Pipeline	Chandalar	Ext. Aer.	7,000	Chandalar	92	yes
Oil field	Deadhorse	Ext. Aer.	8,333	Tundra pond	75	yes
Oil field	Prudhoe	Ext. Aer.	15,000	Tundra pond	200	yes
Pipeline	Prospect	Ext. Aer.	18,000	Jim River	250	yes
Pipeline	Coldfoot	Ext. Aer.	15,000	Koyukuk	190	yes
Pipeline	Dietrich	Ext. Aer.	15,000	Dietrich	190	yes
Pipeline	5-Mile Camp	Rated Aer.	15,000	Surface	208	yes
Pipeline	Crazyhorse	Aer. Lagoon	15,000	Sagavanirktok	160	yes
Oilfield	Deadhorse	Pit		Tundra		no
Oilfield	Deadhorse	Pit		Tundra		no

SOURCE: Clark et al., 1971.

TABLE 27: NORTH SLOPE SEWAGE TREATMENT PLANT EFFLUENT CHARACTERISTICS

System	Date	BOD_5	COD	SS
Toolik Phys-Chem	7/16/70	78	160	69
	8/20/70	46	221	111
	9/16/70	--	318	16
	9/30/70	--	309	11
	3/17/71	--	373	25
Deadhorse Phys-Chem	8/20/70	320	435	61
Deadhorse Ext. Aer.	9/16/70	--	696	291
Galbraith Ext. Aer.	7/16/70	33	--	68
	9/01/70	28	--	15
	9/30/70	--	113	28
Happy Valley Ext. Aer.	9/30/70	0	324	147
Prudhoe Ext. Aer.	7/30/70	180	364	34
	7/30/70[1]	10	49	10
	8/20/70	153	371	106
	8/20/70[1]	15	106	22
	9/16/70	--	1167	1049
	9/16/70[1]	--	357	388
	3/12/71	40	170	68
Crazyhorse Ext. Aer.	9/16/70	--	268	148

[1]Sample at end of long detention lagoon.
Influent Values: BOD_5 - 380-1100 mg/l; COD - 232-7610 mg/l;
SS - 582-3316 mg/l.
SOURCE: Clark et al., 1971

At the same time it has had a dramatic effect on the amount of wastes discharged into Kodiak Harbor. Screening, of course, only removes part of the suspended matter. To remove the considerable portion of the processing wastes in the colloidal or dissolved forms requires more costly processes. If the powers-that-be decide that such a removal is desirable, technology is available to do the job. These include dissolved air flotation and biological treatment. Each of these or a combination would remove a considerable fraction of the organic matter remaining after screening. But, their use would impose economic burdens on the Alaskan processers and require highly trained operators--in short supply in Alaska. Complete treatment such as reverse osmosis to remove essentially all the dissolved matter is economically prohibitive. The immediate steps that should be taken to reduce wastewater discharges include water conservation, reuse, and by-product recovery. In fact, by selling some of the recovered by-product, the operator may be able to help pay for part of the wastewater treatment.

Available information indicates that steam stripping should be very effective in removing ammonia from the effluent at Collier Chemical. This appears to be a reasonable process from an economic point of view. Secondary treatment to be installed at the paper mills may prove to be economically prohibitive (Hammond and Mueller, 1977).

A great deal of experience has been gained in operating package plants at remote sites as a result of the pipeline. There were only two physical-chemical plants in operation on the North Slope as of 1971 (Clark et al., 1971) and a dozen biological plants. With biological plants mainly being used for the first years of pipeline development (Jones, 1977), very little operating experience had been acquired with the physical-chemical plants purchased in 1975. This is reflected in the variation in performance in Table 24. This, plus the lack of trained operators is reflected in the problems encountered initially, many of which are summarized in the paper by Smith and Given (1976). But as can be seen from Table 24, the performance by the middle of 1976 was commendable, with the average BOD_5 removal at the camps being 94%. This is to be compared with less than half of the federally-funded municipal waste treatment facilities as of 1975 achieving secondary treatment of 85% removal (Gilbert, 1975). Two of the camps were using biological treatment successfully in 1977.

Even though this pipeline-related experience has shown package plants can be successful in a technological sense, there still remain serious environmental and economic questions. Is 94% removal good enough when the influent BOD_5 is 600 mg/l and the effluent is being discharged into a pristine environment? Can the solid waste from these plants be disposed of in an ecologically acceptable manner? The answers to these and similar questions, if they exist, will not be found here. But they are obviously of paramount importance with regard to future development in the bush. What should be clear from this study, however, is that BOD_5 and SS removals greater than 90% can be achieve by <u>well-operated</u> biological and physical-chemical package plants in the bush. The underlined words are key. To do this, including the training of good operators, is expensive. At operating costs of 4¢/gal of wastewater treated (Eggener, 1977), and a usage of 100 gpcd, the yearly costs for a family of five would amount to $7000. This illustrates the extravagant costs of modern living in the Arctic while attempting to preserve the environment. Someone has to decide if such expenses are justified.

As long as the wastewater treatment is accomplished in an indoor heated facility, the physical, chemical, or biological processes are the same as those which occur in a more moderate climate. The usual considerations of providing proper detention times for the microorganisms and letting the cells settle in a quiescent basin are crucial for proper performance of a biological treatment plant. Similar comments can be made for physical-chemical plants regarding coagulation dosages, proper surface overflow rates, filter backwashing, etc. For either type of plant, disposal of waste sludge can create problems. Incineration works, but it is energy intensive and requires much maintenance. Land disposal, while acceptable perhaps in a moderate climate, presents many possible problems in permafrost areas. These include very slow die-off rates for pathogens and surface runoff of pathogens. Further studies are desirable concerning the fate of pathogens in the Arctic. Other distinctly northern problems are all associated with the possibilities of either effluent lines or even internal piping freezing up. Such conditions can be avoided by careful monitoring. One last point that should be made is that there were no purely physical-chemical plants along the pipeline by the end of 1976. All employed at least some aeration in equalization basins to allow biological treatment to occur.

For the future, all pump stations will utilize waste heat to vaporize water from the waste solids while simultaneously killing the pathogens. The combination biological followed by physical-chemical plant at Valdez should provide satisfactory treatment. By the middle of 1976, its BOD_5 removal efficiency was 98%.

CONCLUSIONS

Alaska is in a unique position in the United States with respect to the tremendous dilution capacity of its coastal and inland waterways compared with its small population and lack of a significant industrial base. However, the tendency toward solution by dilution, even if the law permitted this, must be weighed against the desirability of preserving the environment. These considerations, coupled with the isolated nature of many bush communities and the inherently primitive living conditions have led to tremendous diversity in degree of wastewater treatment practiced.

Some of the larger municipalities such as Juneau and Fairbanks have biological treatment plants that are meeting secondary standards. By the middle of 1976, the pipeline camps and pump stations were averaging around 92% BOD_5 removal using mainly a combination of biological and physical-chemical treatment. Even though the high strength of the incoming wastewater requires such a high removal efficiency to meet secondary standards, the performance data indicates that current technology will work in Alaska, providing one is willing to pay for it. At 4¢ per gal of wastewater treated, this translates into an annual operation and maintenance cost of $7000 for a family of five using 100 gpcd. Even though these costs may be reduced somewhat by careful economizing, they support the basic conclusion that it is impossible to live cheaply in the Arctic and still have modern sanitation facilities. A discussion of the necessity or desirability of having such facilities is out of the scope of this paper. For the future, the wastewater from the pump stations will be evaporated using the excess heat in the stack gases. The pathogens will be simultaneously disinfected.

Most of the industries, including the pulp mills and seafood processers, and the largest municipality, Anchorage, utilize only primary treatment. The pulp mills, however, have recently agreed to install additional treatment capacity to be in compliance with PL 92-500. The primary treatment plant at Anchorage employs sedimentation followed by chlorination with the sludge being incinerated. It is achieving around 50% BOD_5 and SS removal. With a population of close to 200K, this means tens of thousands of pounds of organic matter are being discharged daily into Cook Inlet. But, the tremendous dispersive powers of the inlet result in negligible effects on water quality except in the immediate vicinity of the outfall pipe. The seafood processes in Kodiak utilize screening and by-product recovery. Screening or barging to sea is practiced at the remote processing locations. The adoption of these first-order, waste-treatment technologies at Kodiak has greatly reduced the pollution occurring in the inner harbor while simultaneously helping to recover a valuable commodity.

Wastewater treatment is virtually nonexistent at many remote villages. Often the human excretia is disposed of in honey buckets which are either hauled to a dump or, sometimes in the case of coastal villages, placed on sea ice in the spring. The hope here is that the waste matter will be carried out to sea during breakup. Unfortunately, this is not always the case. If the honey buckets are left in the village proper, fly and odor problems can result. Moreover, the proximity of the wastes to the village inhabitants can lead to the spread of diseases because pathogens are present in the honey bucket wastes. Even in the case of human wastes being dumped onto the tundra a mile or so from the village, there is still the danger of water supplies being contaminated. This danger is probably greatest during the breakup period when there can be a lot of runoff flow. Contrary to popular opinion, some decay of waste matter that is exposed to the elements does occur in the Arctic. Such degradation is slow and occurs mostly in the summer. More research is needed to ascertain the magnitudes of these decay rates and pathogen die-off rates under field conditions. Such information, when coupled with transport calculations, will help villagers to make rational decisions concerning the location of disposal areas with respect to water supplies and residential areas.

Some progress is being made in upgrading sanitation facilities in some of the villages. The USPHS has installed around sixty sanitation systems with running water. The ADEC is installing some physical-chemical and extended-aeration package plants as part of a Village Safe Water Program. A project recently completed by the USEPA is the AVDP. This involved the installation of physical-chemical water and wastewater treatment systems in Wainwright and Emmonak. In each case, the watering point, showers, laundry, sauna, and toilets are all located in one building to minimize the distribution and collection costs. Water is sold for a few cents a gallon and there are user charges for the other facilities. Although both are operational, each has had its difficulties. The common thread here with the experience at the pipeline camps is the need for well-trained and conscientious operators and adequate financing. Moreover, if a major piece of equipment fails in the bush, it may take weeks to find a replacement. Only time will tell if such relatively complex systems can perform reliably in the villages over long periods of time.

Detailed operating data is not available for all but a few of the the largest treatment plants. Such data must be collected before definitive comparisons can be made. But, there are sufficient data to verify that encapsulated plants can work well in cold climates. But, one must ensure there is an adequate supply of ventilating air to prevent excessive condensation. This will increase the required heat input. For individual homes or other small-scale users, the options are septic tanks, aerobic package plants, or composting toilets. Field data verifies that small plants many times do not perform well in practice because of poor maintenance and operation procedures. Anyone installing such a system should be willing to devote some of his own time to maintain it or be willing to pay a qualified person to do so. If this is done, septic and aerobic systems should work in suitable soils. Composting systems offer a plausible alternative for those without running water or suitable soils for leach fields.

Only a small percentage of the operational problems associated with wastewater treatment are distinctly northern (i.e. associated with freezing). Much of what goes wrong could easily occur (and often does) in the lower 48. Difficulties in the bush are typically those associated with remote, as opposed to northern, facilities. The fact that most of the larger muncipalities do not routinely produce secondary effluent is not inconsistent with the situation in the lower 48.

As in the lower 48, a large portion of operating costs of both small or large treatment plants are related to sludge disposal. In many villages, the "sludge" is fecal matter in honey buckets. Care must be taken to ensure that its disposal does not contaminate groundwater. One promising method of disinfection that deserves wider consideration is lime addition, which has been shown to be effective at low temperatures and is simpler to use than chlorine addition. Some of the larger municipalities including Fairbanks and Juneau are experiencing problems in finding sites for sludge disposal. Drying ponds have not worked because of Juneau's cool damp climate. For those many communities where long, cold winters are the rule, freezing should be considered as a means of dewatering sludge. According to a 1972 study in Fairbanks, dewatering by freezing was found to be cheaper than drying on sand beds or incineration. Use of this method would allow Alaska's most abundant resource, cold, to be utilized. The spreading of sludge on land should be studied as another alternative for ultimate disposal.

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APPENDIX A

APPENDIX A

ALASKA DEPARTMENT OF ENVIRONMENTAL CONSERVATION
PRIVATE WASTE DISPOSAL SYSTEMS

1. The following summary of the Department of Environmental Conservation's Wastewater Regulation is provided to assist you in installing an approved private waste disposal system.

   a. The minimum size septic tank for a 1- to 3-bedroom home is 1000 gallons. For each additional bedroom 250 gallons should be added.

   b. A private waste disposal system is not permitted if a community sewer system is available within 200 feet.

   c. 100 feet must be provided between a lake, creek, stream, river or coastal waters, and a septic tank or soil absorption system.

   d. 100 feet must be provided between well and septic tank or soil absorption system.

   e. Cesspools will not be approved.

   f. The bottom of the seepage pit must be a minimum of four feet from the groundwater.

2. In areas of questionable soil conditions percolation tests will be required. These tests must be conducted by a professional engineer licensed to practice in the State of Alaska.

3. In areas where the percolation rate is less than one inch in 60 minutes, soil absorbent systems will not be permitted.

4. In areas where percolation tests reveal that a conventional system, i.e., septic tank and subsurface absorbent system, can not function, variations of this type system or different type systems may be permitted provided that the system is designed and submitted to the Department of Environmental Conservation by a professional engineer licensed to practice in the State of Alaska for review. This is no guarantee of approval by the Department of Environmental Conservation.

5. A system will not be approved that is malfunctioning.

6. Recommended reference material:

   a. Manual of Septic Tank Practice, U. S. Department of Health, Education and Welfare. Available from the Superintendent of Documents, U. S. Printing Office, Washington, D. C. 20201.

b. Department of Environmental Conservation's Wastewater Regulation 18 AAC 72.010. Available from the Department of Environmental Conservation. (No Cost).

SEPTIC TANK-SEEPAGE PIT SYSTEM

The septic tank conditions sewage so that it may readily percolate into the porous soil. Waste flows into the septic tank. Most of it is liquid; only a very small part is solid matter. The liquid flows out of the tank into the disposal field, consisting of a seepage pit, drain field, or absorption bed, and then into the porous gravel. The solid matter, remaining in the tank, is broken up by bacteria--the heavy parts settling to the bottom (sludge) and the lighter parts rising to the top, creating scum.

If adequately designed, constructed, maintained, and operated, septic tanks are effective in accomplishing their purpose. That is to prevent clogging of the soil. If the soil is allowed to become clogged, the sewer system discontinues to operate effectively. Only with periodic checking and preventative maintenance, can the homeowner expect to receive satisfactory service from the on-site sewer system. Consider the following factors:

A. Toilet paper substitutes, paper towels, newspaper, wrapping paper, rags, sticks, etc., are not likely to decompose and should not be flushed into the septic tank.

B. Drainage from garage floors or other sources of oily waste should be excluded from the septic tank.

C. Roof drains, foundation drains and drainage from other sources, producing large intermittent or constant volumes of clear water, should not be piped into the septic tank or absorption area.

D. Waste drains, from household water softener units, have no adverse effect on the action of the septic tank but may cause a shortening of the life of a disposal field installed in clay-type soil.

E. The functional operation of septic tanks is not improved by the addition of disinfectants or other chemicals. The addition of chemicals is not recommended. Soaps, detergents, bleaches, drain cleaners, or other material, as normally used in the household, will have no appreciable adverse effect on the system. Moderation should be the rule.

F. The septic tank capacities recommended are sufficient to receive grease normally discharged from a house.

G. A septic tank of adequate size can handle all the wastes from kitchen, laundry, or bathroom.

H. Abandoned septic tanks, leaching pits, and cesspools should be filled with earth or rock.

I. Do not enter a septic tank until it has been thoroughly ventilated and gases have been removed to prevent explosion hazards or asphyxiation of the worker. Anyone entering the tank should have one end of a stout rope tied around his waist, with the other end above ground level by another person strong enough to pull him out if he should be overcome by any gas remaining in the tank.

J. Septic tanks should be inspected at least once a year and cleaned when necessary. They should not be washed or disinfected after pumping. A small residual of sludge should be left in the tank for seeding purposes. To check your septic tank, a long stick, approximately six feet, is wrapped in white toweling and inserted through the septic tank siphon pipe. Push the stick to the bottom of the tank and let stand for twenty to thirty minutes. Remove the stick and observe two dark bands. One band on the bottom and one halfway up the stick. This represents the sludge and scum and if equal to one-third or more of the total liquid depth of the tank, it is time to have the tank cleaned.

K. Keep airtight caps on the siphon pipes of the seepage pit and septic tank. The sewer system will vent properly. The removal of one cap results in frozen sewer lines. (During winter, a cold wind across the house vent on the roof draws freezing air through the sewer system and moisture is picked up as it passes through the septic tank. The moist air condenses and freezes in the line from the house to the tank.) Having caps prevents children from dropping strange objects down the siphon pipes causing plugged lines. Having caps prevents unpleasant odors in the area.

L. A chart showing the location of the septic tank and disposal system should be placed at a suitable location in dwellings along with such information as to size of septic tank, material it is made of, size of drain field, etc.

SOILS CLASSIFICATION

Unified Class	Description	Seepage Area Required
GW	Well-graded gravel	85 Ft^2/bedroom
GP	Poorly graded gravel	85 Ft^2/bedroom
GM	Silty gravel	225 Ft^2/bedroom
SW	Well-graded sand	125 Ft^2/bedroom
SP	Poorly graded sand	125 Ft^2/bedroom
SM	Silty sand	250 Ft^2/bedroom
ML*	Silt	275 Ft^2/bedroom

*Percolation test required.

Pit

Multiply the soil rating times the number of bedrooms. Divide the result by the depth of the seepage pit below the discharge pipe times 4. The result is the length of the side of the seepage pit required.

Trench

Multiply the soil rating times the number of bedrooms. Divide the result by the width of the trench in feet. The result is the length of trench required.

APPENDIX B

APPENDIX B

DEPARTMENT OF HEALTH, EDUCATION, AND WELFARE
PUBLIC HEALTH SERVICE
INDIAN HEALTH SERVICE
ALASKA AREA NATIVE HEALTH SERVICE

STATUS OF SANITATION FACILITIES CONSTRUCTION BRANCH, PUBLIC LAW 86-121 (as of 9/1/76)

Village Name and Project Number	Description	New Homes	Total Homes Served	Est. Pop.	Construction Status % Completed 9/1/76	Date Construction Completed
Akhiok (73-802)	Emergency Water System Repairs		25	130	Completed	8-74
*Akhiok (64-442)	Community Water & Individual Waste (septic tanks)		24	130	Completed	12-66
*Akhiok (77-171)	Community Sewer & Water Extension & Treatment	15	35	140	Under Design	
Akiachak (61-420)	Watering Point & Individual Waste		37	226	Completed	8-64
*Akutan (73-621B)	Watering Point at Each Home		21	101	Completed	10-74
Aleknagik (73-621N)	Water Source		15		Completed	10-74
Allakaket (62-428)	Watering Point & Individual Waste		26	129	Completed	12-64
Allakaket (73-621D)	Water Source at School		26	129	Completed	8-75
*Ambler (72-915)	Community Water & Sewer & Treatment	16	45	115	80%	
*Anaktuvik Pass (7 -105)	Community Water & Sewer & Treatment	12	22	120	15%	
*Andreafsky (See St. Marys)						
*Angoon (60-1E) (63-339 APW)	Community Water & Sewer		60	429	Completed	10-6[illegible]
Angoon EM (68-680)	Replace Submarine Hose		60	429	Completed	12-70
Angoon EM (70-690)	Corrosion Protection		60	429	Completed	C-70

SOURCE: Rogness, 1976

*Angoon (73-928)	Community Water & Sewer Ext. & Treatment	30	85	450	85%	
*Angoon (75-134)	Community Water & Sewer Ext.	25	25	100	Completed	10-75
*Arctic Village (73-103)	Hauled Water & Sewage & Treatment (new lagoon)	19	28	140	Completed	9-76
*Atka	Community Water & Sewer & Treatment	11	20	90	Prelim. Planning	
*Atkasook	Community Water & Sewer & Treatment	30	30	140	Prelim. Planning	
Barrow (68-606)	Waste Collection & Disposal (activated sludge plus lagoon)		310	2,000	Completed	5-72
Barrow (74-624)	Water Source & Treatment		310	1,400	80%	
Barrow (74-943)	Haul Vehicles, Incinerator, Etc.	40	310	1,400	Completed	6-75
*Bethel (69-991)	Community Water & Sewer & Treatment	200	200	1,200	Completed	12-71
	Hauled Water		150	1,200	Completed	
*Bethel (77-153)	Watering Point Expansion & Sewer Ext. (Apts., Etc.)	70	80	300	90%	
Birch Creek (7 -131)	Watering Point & Individual Waste (privies)	08	08	36	Under Design	
Brevig Mission (76-130)	Watering Point & Individual Waste	10	20	123	85%	
Buckland (76-114)	Watering Point & Individual Waste	22	22	95	85%	

* Running Water In Home

Chalkyitsik (76-129)	Watering Point & Individual Waste	12	30	130	Under Design	
Chefornak (64-441A)	Watering Point & Individual Waste		30	135	Completed	8-66
Chevak (63-435)	Watering Point & Individual Waste		57	350	Completed	5-65
*Chistochina (70-905)	Community Water & Individual Waste	06	08	70	Completed	8-71
Chuathbaluk (70-685D)	Water Source at School		11	50	Completed	11-70
*Chuathbaluk (71-926)	Community Water & Sewer & Treatment	10	20	100	Completed	10-75
*Copper Center (69-615)	Individual Water & Waste		30	174	Completed	7-70
Copper Center (72-685B)	Iron Removal Equipment (Individual)		30	174	Completed	10-72
*Craig (75-135)	Water & Sewer System & Treatment Extensions	15	15	100	85%	
Crooked Creek (70-685A)	Water Source at School		25	140	Completed	11-70
Crooked Creek (76-101)	Watering Point & Individual Waste	10	13	80	Prelim. Planning	
Deering (65-450B)	Watering Point & Individual Waste		17	65	Completed	11-75
Deering (77-166)	Watering Point Repairs & Expansion	8	25	70	60%	
*Dot Lake (70-908)	Community Water & Individual Waste	7	08	70	Completed	11-70
Ekwak (73-621G)	Watering Point		21	120	Completed	10-74
*Elim (73-115)	Community Water & Sewer System & Treatment (extended aeration)	20	47	175	Completed	10-74

English Bay (64-447)	Community Water & Individual Waste		16	80	Completed	10-66
*English Bay	Community Water Extensions & Sewer System & Treatment	8	24	100	Prelim. Planning	
*Eskimo Village (73-128)	Water & Sewer System Extensions	6	06	30	Completed	8-74
*Fort Yukon (77-946)	Community Water & Sewer & Treatment	40	90	450	Under Design	
Galena (75-916)	Hauled Water & Sewer & Treatment	10	12	200	90%	
*Galena (77-150)	Hauled Water & Sewer Expansion (lagoon)	30	30	120	Under Design	
*Gambell (77-947)	Community Water & Sewer & Treatment	30	70	400	Under Design	
Golovin (76-119)	Watering Point & Individual Waste	10	34	180	70%	
*Goodnews Bay (68-601)	Community Water & Sewer & Treatment (septic tank and ocean outfall)		39	213	Completed	11-71
*Goodnews Bay (72-917)	Community Water & Sewer System Extensions	20	20	100	Completed	11-72
Goodnews Bay (72-685B)	Repair Water & Sewer Lines		40	230	Completed	11-71
*Grayling (66-452)	Community Water & Individual Waste		27	142	Completed	2-68
Grayling EM (70-698)	Repair Water System		28	142	Completed	11-71
*Grayling (77-177)	Community Water Expansion, Sewer System & Treatment	15	35	155	Under Design	

Gulkana (68-670)	Water Source		13	54	Completed	5-69
*Gulkana (69-614)	Community Water & Sewer Treatment (septic tank and drain field)		13	54	Completed	12-69
*Gulkana (69-902)	Water & Sewer System Extensions	9	09	40	Completed	
Gulkana EM (72-685B)	Replace Transmission Line		26	100	Completed	9-71
* Gulkana EM (76-807)	Water System Repairs		26	100	50%	
*Haines (75-136)	Water & Sewer System Extensions	25	25	130	Completed	10-75
*Holy Cross (68-603B) (70-616)	Community Water & Sewer & Treatment (lagoon)		38	200	Completed	9-71
*Holy Cross (72-918)	Water & Sewer System Extensions	15	53	90	Completed	11-72
*Holy Cross (76-147)	Water & Sewer System Extensions & Repairs	5	05	30	Completed	9-76
*Hoonah (60-3E/65-448/69-612)	Community Water & Sewer (Refuse Truck)		175	1,000	Completed	7-70
Hoonah EM (68-681)	Relocate Transmission Main		175	1,000	Completed	10-68
*Hoonah (68-900)	Community Water & Sewer Extensions	15	15	90	Completed	8-70
*Hoonah (72-929)	Water & Sewer System Extensions & Treatment	35	215	1,100	Completed	8-74
*Hoonah (75-137)	Water & Sewer System Extensions	30	30	100	Completed	7-76
Hooper Bay (63-436)	Watering Point & Individual Waste		70	430	Completed	5-65
Hooper Bay EM (71-685A)	Replace Watering Point		70	430	Completed	11-71
Hooper Bay (76-934)	Watering Point & Individual Waste	30	30	100	30%	
*Hughes (68-607)	Community Water & Individual Waste		10	75	Completed	7-70

Hughes (71-685C)	Sanitary Landfill		10	75	Completed	10-71
*Hughes (73-133)	Water System Extension & Individual Waste	6	16	30	Completed	8-76
Huslia (62-429)	Individual Water & Waste		28	163	Completed	5-64
*Huslia (73-100)	Community Water & Sewer & Treatment (septic tanks)	15	40	160	Completed	8-75
*Hydaburg (63-338 APW)	Community Water & Sewer		70	300	Completed	1-69
Hydaburg EM (69-683B)	Repair Transmission Line		70	300	Completed	10-70
*Hydaburg (72-930)	Water & Sewer System Extensions & Treatment	25	85	300	95%	
*Hydaburg (75-138)	Water & Sewer System Extensions	15	15	60	Completed	10-75
Igiugig (73-621Q)	Watering Point (No Water Obtained)		8	30	Completed	8-76
*Kake (67-454) (69-611)	Community Water & Sewer & Treatment		105	500	Completed	9-70
*Kake (73-941)	Water & Sewer System Extensions & Treatment Expansion	30	120	160	90%	
*Kake (75-139)	Water & Sewer System Extensions	20	20	70	Completed	10-75
Kalskag (68-603A)	Watering Point & Individual Waste		28	180	Completed	2-71
*Kalskag (71-927)	Community Water & Sewer & Treatment	25	33	200	Completed	10-73

Kaltag (65-449B)	Watering Point & Individual Waste		29	179	Completed	6-66
*Kaltag (72-920)	Community Water & Sewer & Treatment (extended aeration)	19	45	225	Completed	5-72
Kaltag (75-806)	Filter Repairs		45	225	50%	
Karluk (71-685B)	Septic Tanks (2)		02	8	Completed	10-72
*Karluk (77-175)	Water & Sewer Systems & Treatment	15	35	90	Under Design	
*Kiana (71-913)	Community Water & Sewer & Treatment	18	51	275	Completed	12-72
*Kiana	Community Water & Sewer Extensions	15	15	50	Under Design	
*King Cove (68-605) (72-685)	Community Sewer & Treatment & Refuse		70	330	Completed	9-70
Kipnuk 73-631B						
Kipnuk (64-441B)	Watering Point & Individual Waste		43	240	Completed	12-75
Kivalina (76-113)	Watering Point & Individual Waste	12	33	195	80%	
*Klawock (72-942)	Community Water System Extension & Sewer System Treatment (septic tanks)	20	65	225	90%	
*Klawock (75-140)	Water & Sewer System Extensions	12	12	60	Completed	10-75
Klukwan EM (69-683A)	Chlorination Equipment		25	160	Completed	6-70
*Klukwan (72-939)	Community Water & Sewer & Treatment	25	38	115	Completed	11-72
*Klukwan (75-141)	Water & Sewer System Extensions	23	23	100	Completed	4-75
Kokhanok Bay (70-685B)	Water Source at School		16	90	Completed	11-70
Kotlik (65-449C)	Water Point & Individual Waste		29	160	Completed	2-68
Kotlik EM (69-684)	Repair Water Tank		28	160	Completed	9-69
*Kotzebue (66-451/68-600/69-609)	Community Water & Sewer		103	450	Completed	7-72
Kotzebue EM (70-687)	Repair Transmission Line		300	450	Completed	5-70
*Kotzebue (71-619/72-620)	Water & Sewer System Ext. & Sewage Treatment		370	500	Completed	10-74

*Kotzebue (71-911) (72-931)	Water & Sewer System Extensions	25	145	470	Completed	8-76
Kotzebue EM (72-687)	Replace Frozen Sewer Lines		53	470	Completed	10-74
*Kotzebue (76-952)	Water & Sewer System Extensions & Treatment Expansion	48	348	500	95%	
Koyuk (65-449A)	Watering Point & Individual Waste		32	132	Completed	11-68
Koyuk EM (72-685C)	Recharge Well		32	132	Completed	12-72
Koyuk (73-621L)	New Source		29	130	Completed	12-74
Kwethluk (67-455)	Watering Point & Individual Waste		75	370	Completed	12-68
*Kwethluk (72-919)	Hauled Water & Sewer & Treatment	30	80	300	Completed	8-74
Little Diomede (73-132)	Watering Point & Individual Waste	19	20	100	Completed	9-74
Lime Village (7 -120)	Watering Point & Individual Waste	5	09	40	Prelim. Planning	
*Lower Kalskag (70-904)	Community Water & Sewer & Treatment	25	40	180	Completed	2-72
Lower Kalskag EM (74-803)	Repair of Water Line		40	180	Completed	5-74
Manokotak (70-686)	Refuse Equipment		32	210	Completed	11-72
*Manokotak (71-921)	Community Water & Sewer & Treatment	19	44	210	Completed	10-73
Marshall (68-682)	Water Source at School		33	150	Completed	2-68

Marshall (75-110) (77-143)	Community Water & Sewer & Treatment	31	55	200	30%	
McGrath (77-169)	Community Water & Sewer & Treatment	30	90	279	Under Design	
Mekoryuk (64-441C)	Watering Point & Individual Waste		40	250	Completed	4-66
*Mekoryuk (72-937)	Community Water & Sewer & Treatment	20	55	270	Completed	10-75
Mentasta Lake (77-162)	Watering Point & Individual Waste	6	9	45	10%	
Metlakatla EM (61-481)	Water Line Repairs		160	990	Completed	10-61
*Metlakatla (62-430) (66-4X1)	Community Water & Sewer & Refuse (aerated lagoon with post chlorination plus ocean outfall)		160	990	Completed	7-65
*Metlakatla (65-924)	Community Water & Sewer Extension	15	15	90	Completed	10-67
*Metlakatla (67-977)	Water & Sewer System Extension	15	16	90	Completed	10-68
*Metlakatla (71-912)	Community Water & Sewer System Ext. & Treatment	25	225	1,000	Completed	10-74
*Metlakatla (72-953)	Water & Sewer System Extensions	40	40	180	Completed	10-72
*Metlakatla (76-163)	Water & Sewer System Extensions	25	25	120	50%	
Minto (62-427)	Watering Point & Individual Waste (Abandoned when village moved)		37	187	Completed	10-64
*Minto (70-903)	Community Water & Sewer & Treatment (lagoon)	38	38	200	Completed	7-72
Minto EM (74-804) (76-156)	Repair of Facilities		42		85%	
*Mountain Village (70-909)	Community Water & Sewer & Treatment (septic tanks)	30	90	420	Completed	12-74
*Mountain Village (76-155)	New Well & Sewage Treatment Plant	14	90	420	75%	
Napakiak (61-422)	Watering Point & Individual Waste		39	197	Completed	4-66

Napaskiak (61-421)	Watering Point & Individual Waste		35	185	Completed	5-65
Napaskiak (76-808)	Relocate Watering Point		38	190	30%	
Nelson Lagoon (73-621A)	Watering Source at School		14	78	Completed	7-76
Nenana (77-174)*	Community Water & Sewer & Treatment	20	120	550	Under Design	
Newhalen (73-621O)	Water Source		12	70	Completed	7-74
*New Koliganek (68-602/70-618)	Community Water & Sewer & Treatment		26	114	Completed	11-72
New Koliganek EM (72-685B/73-800)	Repair Frozen Lines		26	114	Completed	10-71
*New Stuyahok (71-922)	Community Water & Sewer & Treatment	17	44	200	Completed	11-75
Nikolai (73-621F/74-622/70-685E)	Watering Point & Individual Waste		25	140	Completed	10-76
Nikolski (73-621I)	Community Water & Individual Waste		12	57	Completed	9-74
*Noatak (74-107/76-151)	Community Water & Sewer & Treatment	38	55	200	70%	
*Nome (69-613)	Individual Connections to Existing Water & Sewer System		43	234	Completed	9-69
*Nome (70-906) (75-805)	Water & Sewer System Extensions & Repairs	50	65	300	Completed	9-74
Nome (76-158)	Water & Sewer System Extensions	24	40		Under Design	
Nondalton (70-685C)	Water Source at School		36	190	Completed	11-70
*Nondalton (71-914)	Community Water & Sewer & Treatment	15	38	230	Completed	11-75
Nondalton (76-167)	Community Water & Sewer System Repairs	9	42	240	70%	
*Noorvik (73-951)	Community Water & Sewer & Treatment	23	27	250	Completed	10-74
*Noorvik (76-145)	Water & Sewer System Extensions	20	20	90	95%	
*Noorvik (77-168)	Water & Sewer System Extensions	10	10	40	Under Design	

Northway (70-689)	Water Source		27	180	Completed	9-71
*Nuiqsut (77-165)	Water & Sewer System	35	35	180	Under Design	
Nulato (70-685F)	Water Source at School		56	300	Completed	11-70
Ohgsenakale (70-685G) (Portgage Creek)	Water Source at School		15	90	Completed	11-70
*Old Harbor (64-444) (68-4X3)	Community Water & Sewer & Treatment & Refuse		41	200	Completed	10-65
*Old Harbor (77-176)	Community Water & Sewer Extensions	45	5	150	Under Design	
Olsenville (73-621K)	Water Source		11	75	Completed	5-74
Oscarville (62-425)	Watering Point & Individual Waste		14	38	Completed	5-67
*Ouzinkie (64-445)	Community Water & Sewer & Treatment		43	250	Completed	11-67
Ouzinkie EM (71-689)	Sewer System Repairs		43	250	Completed	7-72
Ouzinkie (77-148)	Water & Sewer System Extensions	12	12	50	40%	
Pedro Bay (70-685H)	Water Source at School		12	70	Completed	11-70
Pilot Station - 77-309						
*Pilot Station (70-910)	Community Water & Sewer & Treatment	22	51	290	Completed	7-74
Pitka's Point (70-685I)	Water Source at School		14	50	Completed	11-70
Point Hope (77-157)	Watering Point & Individual Waste	19	65	380	Under Design	

*Port Graham (70-907)	Community Water & Sewer & Treatment	17	39	150	Completed	11-72
*Port Lions (64-446) (68-4X2)	Community Water & Sewer & Treatment & Refuse		42	250	Completed	11-65
Quinhagak (77-172)	Watering Point & Individual Waste	55	90	330	Under Design	
*Russian Mission (68-603C) (70-617)	Community Water & Sewer & Treatment (lagoon)		28	130	Completed	11-71
*Sand Point (74-954)	Community Water & Sewer & Treatment	10	100	350	90%	
Savoonga (76-948)	Watering Point & Individual Waste	25	70	400	30%	
*Saxman (68-604) (69-610)	Community Water & Sewer & Treatment		40	200	Completed	
*Saxman (FHA) (72-933)	Community Water & Sewer & Treatment	20	20	100	Completed	4-74
*Saxman (72-935)	Water & Sewer Extensions	20	20	100	Completed	10-75
St. Marys (66-453)	Watering Point & Individual Waste		30	210	Completed	4-69
*St. Marys (73-932)	Community Water & Sewer & Treatment	7	50	250	Completed	9-76
*St. Marys-Andreafsky (76-143)	Water & Sewer System Extensions	20	34	180	Completed	9-76

St. Michael (65-449D) (73-801)	Watering Point & Individual Waste		32	176	Completed	6-74
St. Michael (77-109)	Watering Point & Individual Waste	25	50	195	Under Design	
Scammon Bay (63-437)	Watering Point & Individual Waste		24	115	Completed	3-67
*Scammon Bay (73-124)	Community Water & Sewer & Treatment	15	29	180	80%	
*Shageluk (75-149)	Community Water & Sewer & Treatment	24	26	160	30%	
Shaktoolik (63-432)	Watering Point & Individual Waste		23	184	Completed	5-70
Shaktoolik (77-104)	Watering Point & Individual Waste	20	20	190	10%	
Shishmaref (65-450A)	Watering Point & Individual Waste		44	220	Completed	9-71
Shishmaref (72-936)	Watering Point & Individual Waste	24	68	250	90%	
*Shungnak (77-957)	Community Water & Sewer & Treatment	18	38	165	Under Design	
*Sitka (77-161)	Community Water & Sewer Repairs & Extensions	45	55	220	Under Design	
Sleetmute (7 -102)	Watering Point & Individual Waste	10	17	130	Prelim. Planning	
South Naknek (73-621P)	Water Source		34	150	Completed	7-74
Stebbins (63-433)	Watering Point & Individual Waste		28	160	Completed	11-65
Stebbins (77-958)	Watering Point & Individual Waste	20	45	220	Under Design	
*Takotna (77-170)	Individual Water & Waste	5	10	25	Under Design	
Tanacross (67-457)	Individual Water & Waste		21	76	Completed	7-70
*Tanacross (72-940)	Community Water & Individual Waste (septic tanks)	12	12	120	Completed	10-74

*Tanacross (75-144)	Water & Sewer System Extensions	15	15	60	Completed	9-76
Tanana (67-456)	Individual Water & Waste		65	270	Completed	
*Tatitlek (64-443)	Community Water & Sewer & Treatment		35	195	Completed	10-67
Tatitlek EM (72-685E)	Replace Roof on Water Tank		35	195	Completed	5-73
*Teller (76-950)	Community Water & Sewer & Treatment Watering Point for Existing Homes	30	30 10	150 70	10% Under Design	
Tetlin (67-458)	Watering Point & Individual Waste		27	92	Completed	6-73
Tetlin (7 -116)	Expand & Repair Watering Point	13	38	100	Under Design	
*Togiak (74-623/75-623/75-955)	Community Water & Sewer & Treatment	30	95	400	60%	
*Toksook Bay (74-121)	Community Water & Sewer & Treatment	26	53	240	98%	
Tuluksak (62-426)	Watering Point & Individual Waste		30	146	Completed	2-67
Tuntatuliak (61-423)	Watering Point & Individual Waste		26	132	Completed	4-66
*Tyonek (61-424)	Community Water & Individual Waste (Abandoned)		28	180	Completed	10-64
Tyonek (73-621C)	Repairs to Water Treatment Plant		28	180	Completed	9-73
Twin Hills (72-685D)	Watering Point		10	70	Completed	11-72
* Twin Hills (77-164)	Community Water & Sewer System & Treatment	14	17	67	Under Design	
*Unalakleet (64-440)	Community Water & Individual Waste		88	550	Completed	2-67
Unalakleet EM (68-486)	Rehabilitate Pumphouse		88	550	Completed	3-68
Unalakleet (69-608)	New Water Source		88	550	Completed	7-71

*Unalakleet (72-923) (72-949)	Community Water Extensions & Sewer & Treatment	27	119	550	Completed	10-74
*Unalakleet (77-154)	Water & Sewer System Extensions	20	20	100	95%	
Unalaska (72-685F)	Refuse Equipment		50	180	Completed	5-73
Wainwright (72-925)	Hauled Water & Sewer & Treatment	25	55	300	Completed	2-74
Wainwright EM (74-146)	Repair of Facilities		55	300	Completed	5-75
Wales (65-450C)	Watering Point & Individual Waste (summer)		27	136	Completed	5-68
Wales (77-956)	Watering Point & Laundromat	18	41	150	Under Design	
White Mountain (63-434)	Watering Point & Individual Waste		16	129	Completed	12-65
*Yakutat (63-431)	Community Water & Individual Waste		55	300	Completed	10-64
*Yakutat (72-938) (72-945)	Community Water Extensions & Sewer & Treatment	20	80	300	Completed	7-74
*Yakutat (75-142)	Water & Sewer System Extensions	35	35	120	Completed	8-76

STUDIES AND WATER EXPLORATION PROJECTS:

Kuskokwim River Area (60-2S)	Water Reconnaissance Study	Completed	6-61
Kuskokwim Area (61-470)	Water Exploration & Development	Completed	6-63
Native Villages (62-471) Emergency Supply (62-482)	Water Exploration & Development	Completed	6-64
Kotzebue Area (63-472) Emergency Supply (64-483)	Water Exploration & Development	Completed	6-65
Bethel - Seward Peninsula (64-473)	Water Exploration & Development	Completed	12-69
Kodiak Area (64-474)	Engineering Studies for Projects	Completed	12-64
Statewide (AHRL) (71-671)	Desalinization Using Freezing (Other Studies)	Completed	8-73
Copper River Valley Area (73-621H)	Individual Wells for Tazlina, Chistochina, Gakona	40%	
Well Drilling (73-621J)	Equipment and Driller's Salaries	50%	

NOTE: None of the PHS-constructed lagoons have operated long enough to require sludge removal. They are all built large enough to store the winter flow. Performance data invariably not taken for village sewage treatment systems.