THE CONSTRUCTION AND EVALUATION OF A WATER RECLAMATION KIT PROTOTYPE FOR ISOLATED APPLICATION

By

MALCOLM JOHN ROBERTS, B. ENG.

A Project Report

Submitted to the School of Graduate Studies

in Partial Fulfilment of the Requirements

for the Degree

Master of Engineering

McMaster University

(October) 1978

WATREK: A WATER RECLAMATION KIT FOR ISOLATED APPLICATION

MASTER OF ENGINEERING (1978)
(Engineering)

McMASTER UNIVERSITY
Hamilton, Ontario

TITLE: The Construction and Evaluation of a Water Reclamation Kit Prototype for Isolated Application.

AUTHOR: Malcolm John Roberts, B. Eng. (Memorial)

SUPERVISOR: Dr. A. Benedek

NUMBER OF PAGES: xv, 226

ABSTRACT

Unique sanitation problems exist in isolated communities, particularly those in Canada's North. These problems are compounded in the North by an under-supply of safe potable water and unsuitable ground conditions for most common Southern sanitation systems.

In an effort to meet some of these problems, a packaged water reclamation kit (WatRek) was designed. The kit was designed to be easily transported and assembled, and to produce an effluent suitable for recycle for non-potable use with a minimum of operator attention. Unit processes utilized in the prototype were: biological treatment, clarification, flow equalization, aerobic sludge digestion and activated carbon adsorption. An effluent quality of 30 mg/l COD and SS was considered adequate for recycle.

During evaluation the prototype met the effluent criteria at all times. The solids removal efficiency of the floating tube clarifier was found to be sensitive to energy dissipations in the aeration tank, of greater than 0.6 HP/1000 Igal. Overall net yield of micro-organisms during the experimental period was estimated to be 0.08 g MLVSS/g COD removed. Seventeen days were

required to develop a biological floc in the aeration tank without an activated sludge seed.

The floating tube clarifier and hence the overall prototype operation were sensitive to hydraulic conditions. Prior to installation the prototype would require modification of the clarifier and the operational mode of the carbon column which was subject to inadequate backwashing. Improvements in the aeration tank system to maximize oxygen transfer and minimize agitation of the floating clarifier would also be required.

ACKNOWLEDGEMENTS

The author would like to thank his supervisor, Dr. A. Benedek, for his support, constructive criticism and guidance, without which this study would not have been possible. The author also wishes to gratefully acknowledge the comments and understanding of Dr. W.J. Snodgrass.

The assistance and suggestions of Messrs. J. Bansci, R. Dunn, J. Newton and H. Behmann in building and testing the prototype and performing analyses were invaluable.

The provision of facilities by the Canada Centre for Inland Waters, Wastewater Technology Centre is gratefully acknowledged.

Last, but not least, special thanks must go to Miss Nancy Lyons for her encouragement and typing of the manuscript, and to the Hall Society, for tea and sympathy.

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LIST OF ABBREVIATIONS

BOD	Biochemical oxygen demand
COD	Chemical oxygen demand
D.O.	Dissolved oxygen
F/M	Food to microorganism ratio
MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids
MLX	Mixed liquor extraction
PAC	Powdered activated carbon
PCT	Physical-chemical treatment
PVC	Polyvinylchloride
R.O.	Reverse osmosis
SRT	Solids retention time
SS	Suspended solids
SVI	Sludge volume index
TDS	Total dissolved solids
TOC	Total organic carbon
VS	Volatile solids

CHAPTER 1

INTRODUCTION

Under certain conditions of topography, climate and location, the provision of sanitary water supply and wastewater disposal becomes a difficult and costly exercise. One area in which these conditions exist is in the far northern regions of Canada. The difficulty in the Canadian North is accentuated by the widely scattered, small communities and a dry, cold climate.

Today's North is on the verge of great changes due to the impending development of energy resources (Berger, 1977) and increasing recreational interest. This development will bring large numbers of people to the North. With this influx of population will come the necessity for improved sanitation services and sewage disposal.

1.1 Wastewater Disposal

Many methods of wastewater treatment have been utilized and developed for application in watershort situations such as exist in the North. The concept of producing a high quality effluent water which would be suitable for recycle as a non-potable source holds great merit.

Bromley (1977) reported on a waste treatment system, intended for northern areas, which produced a high quality effluent. It was the objective of this present work to use the process information reported by Bromley to design, construct and test a workable prototype treatment plant. The system was to be compact, reliable and efficient while being simple to operate and maintain.

The package treatment system developed was called "WatRek" for "Water Reclamation Kit".

CHAPTER 2

LITERATURE REVIEW

2.1 Water Supply in the North

Several important climatic factors combine to make the North a water short area: (i) there is little precipitation throughout the year. Most areas receive 25 cm (10 inches) or less (Climatological Atlas, 1953); (ii) the permanently frozen ground, permafrost, does not permit infiltration and most precipitation is carried away as surface runoff; (iii) the moisture which does fall and does not runoff immediately is retained in the form of ice and snow for much of the year.

Lakes and rivers make good water sources during the summer months but care must be taken to choose sources which do not freeze to the bottom in winter. Even in those which do not freeze solid the quality of the source may become so impaired, by the concentration of salts in winter, that they have to be abandoned (Boyd & Boyd, 1965).

The biological quality of surface waters in the North is generally acceptable for potable use. However, delivery practices and sanitation procedures, or the lack thereof, are such that the water is often contaminated before use (Suk, 1975). ... gives incidence of enteric disease in the population (Dingman, 1971; Fournelle *et al.*, 1958). Improvements in existing water supply and waste disposal techniques will greatly aid in decreasing this health hazard.

2.1.1 Water Delivery

In some places in the North water delivery is still based on hand-toting buckets. Ice has been collected from lakes in the fall and stored in bunkers in the permafrost, to be thawed when needed (Boyd & Boyd, 1965). These methods are becoming increasingly rare as more modern techniques are employed.

Standard practise of water supply in the south, *i.e.* buried pressure mains, is not usually adequate for northern applications. Buried mains tend to freeze and rupture unless extensively protected from freezing temperatures. Because of the scarcity of water in many places the practice of bleeding water to prevent freezing is not acceptable. Some methods commonly used in the North are trucking, recirculating systems and intermittent or pulsed systems.

Suk (1975) has concluded that the intermittent or pulsed system is the optimum, even though it has been used only for delivery to central storage facilities to date. The system has the advantage of requiring little heat input to keep the lines from freezing. Heat is required only when the system is full of water, during delivery periods every few days.

The recirculation system is continually full of water and, therefore, requires that heat be added to the pipe, the water or both. The capital cost of the system is high because the piping system must form a closed loop, thereby effectively doubling the length of pipe required.

Some settlements use tank vehicles to haul water from the source to the houses. This system is very vulnerable to equipment failure in an area where parts for repair may be scarce. It has been shown that the water is all too often contaminated during delivery. For a more detailed description of northern water supply systems the reader should refer to Suk (1975).

2.2 Wastewater Systems

The sparse population, irregular development patterns and harsh climate make disposal of human wastes a difficult problem in the North. Wastewater disposal practices must be convenient and sanitary as well as aesthetically and environmentally acceptable.

According to Deans and Heinke (1972) services in 54 settlements in the Northwest Territories range from no service at all to some form of piped service with treatment. Half of these communities used tank truck pick-up and disposal of either "honey-bags" and/or holding tank pump-out.[7] Most of the remaining settlements discharged their waste to a waterbody. The balance use an open dump.

2.2.1 Box and Can

This is the most primitive and least sanitary of the methods presently used. Plastic bags called honey-bags are used to line a bucket which serves as a toilet. After use the bag is closed and removed from the bucket. These bags are then disposed of individually or collected periodically by a truck. Frequent breakage of the bags near dwellings cause an unsightly and unsanitary situation.

2.2.2 Water Closets

The water closet is the standard flush toilet familiar in southern areas. For northern use modifications are often added to the standard unit. The most common modifications are aimed at lowering the water use of the standard southern model. Units with a flushing requirement of as little as 1.1 liters (1 quart) have been successfully demonstrated (Deans & Heinke, 1972; Clark *et al.*, 1962). In present installations the waste from water closets is discharged either to the wastewater system or to a holding tank within the home. This tank is then periodically emptied by a tank truck.

Other modifications to the water closet involve the use of liquids other than water for carriage purposes. Boyd & Boyd (1965) reported on experiments with oil as a carriage medium. The oil/sewage mixture was then burned, with some difficulty, in an incinerator. Today, the high cost of oil, particularly in the North, makes this alternative unattractive.

2.2.3 Chemical and Incinerating Toilets

Chemical toilets utilize a holding tank and a strong chemical to render the wastes less objectionable. Commercial units are quite economical and may be a sound alternative to the box and can. The problem of disposing of a high strength waste is not alleviated by chemical toilets.

Incinerating toilets reduce human wastes to an inert ash, with the aid of an electrical element or natural gas. The electrical variety uses significant amounts of high voltage electricity and, therefore, may be expensive as well as unsafe. Both varieties allow odors to escape from their combustion compartments (Deans & Heinke, 1972; Boyd & Boyd, 1965).

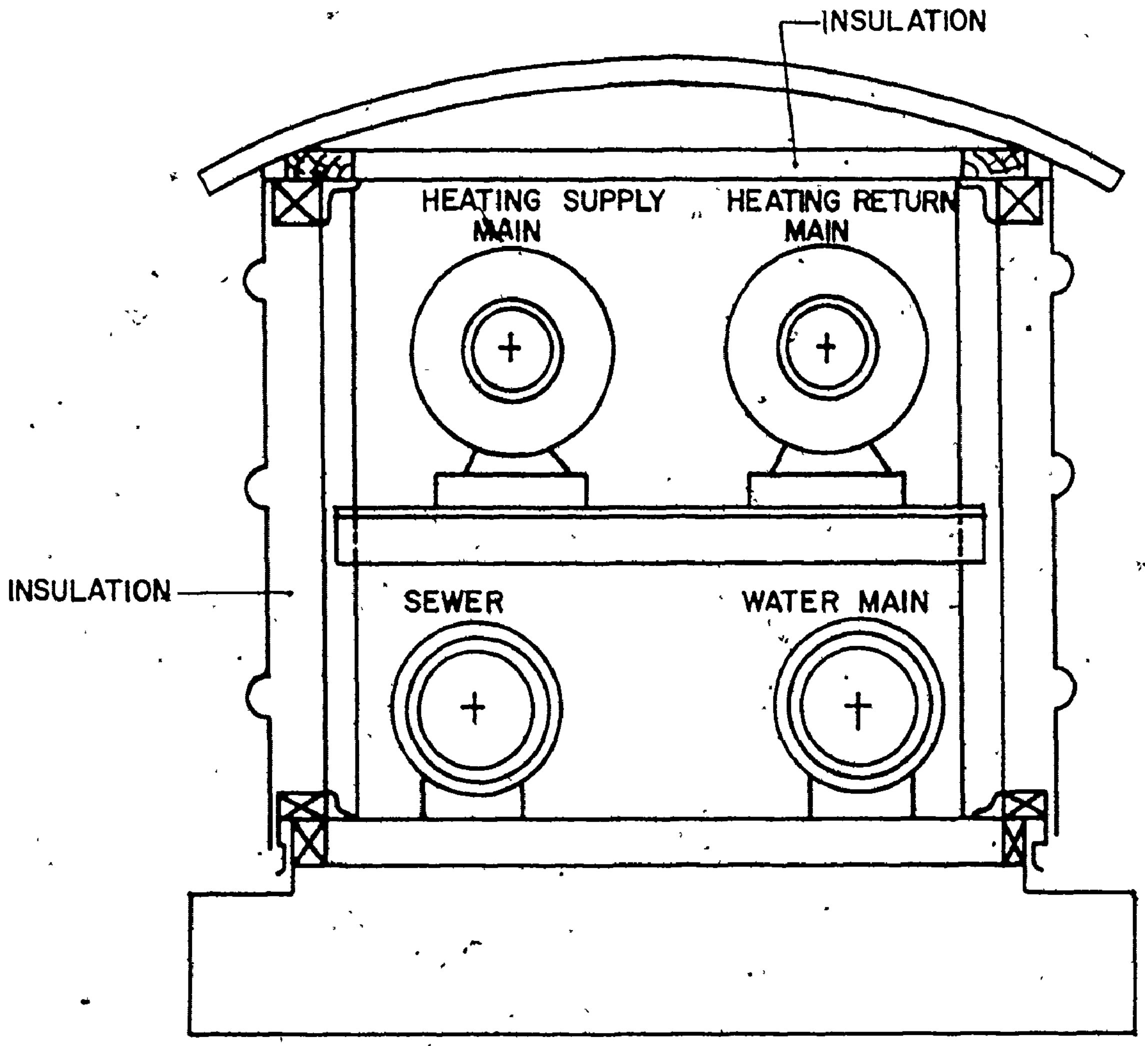
Most investigators agree that the low volume flush toilet is the best alternative for northern application. It should be coupled with the appropriate disposal system to be truly effective. Ideally, the entire sewage system should be chosen, designed and implemented as one project in order to have an efficiently integrated system.

Transmission and ultimate disposal of the waste from any sewage system must be sanitary and environmentally acceptable. Some of the systems that have been used and/or experimented with are mentioned in the following section.

2.3 Wastewater Transmission and Disposal

Most communities in the North were established before the efficient handling of human waste was considered very important. Therefore, primitive methods of transmission and disposal have been prevalent. The practice of filling empty oil drums with waste and leaving them on the sea ice has largely been discontinued (Boyd & Boyd, 1965). Still prevalent is the disposal of honey-bags on land sites after collection by the community truck. Likewise, holding tank pumpout is often dumped on land (Deans & Heinke, 1972).

Piped conveyance of sewage poses unique problems in the North. Buried gravity sewers are difficult to install and maintain because of rough terrain and freezing conditions. Pressure piping systems which overcome the rough terrain, must be protected from freezing either by heat tracing or installing them in a utilidor (Deans & Heinke, 1972). (See Figure 2.1.)



INSULATION
HEATING SUPPLY MAIN
HEATING RETURN MAIN
INSULATION
SEWER
WATER MAIN

The use of vacuum sewage systems has been studied in Scandinavia and Bermuda (Deans & Heinke, 1972; Heinke, 1974). The systems use an air pressure differential of about one half atmosphere to provide the energy to move the sewage through small pipes at high speed. The combination of small diameter pipes, and the fact that each flush travels as a plug through the pipe, leaving it mostly dry, means that the heating costs are reduced. Vacuum systems must be operated in conjunction with vacuum toilets.

The disposal of sewage collected by a piped system can be accomplished through direct untreated discharge to a water body, conventional treatment before discharge, or a high degree of treatment and recycling.

A major drawback of the discharge of untreated sewage to a waterbody is the prospect of pathogenic organisms contaminating surface water. Clark *et al.* (1962) felt that pathogenic contamination was probably the only problem with small discharges. They, therefore, recommended that disinfection be the only required treatment for small discharges.

In situations where raw sewage discharge is not permitted or not desirable, some form of treatment must be instituted. Such treatment, *e.g.*, biological or physical-chemical, may be carried out in lagoons or package treatment units. The treatment alternatives available in the North and their performance is discussed in the succeeding section. An alternative which has also been studied is the renovation and reuse of water. The water has usually been reused as a flushing liquid in water closets. This will also be discussed in the following section.

2.4 Biological Waste Treatment

The treatment of wastewater through biological means has been practiced in many situations and locations for many years. Biological treatment processes have been the subject of much research over the years and have been applied in the North.

2.4.1 Temperature Effects on Biological Processes

The Arrhenius relationship

$$k_T = Ae^{\Delta E/RT}$$

where
k_T = reaction rate constant
A = Arrhenius constant
ΔE = activation energy
R = gas constant
T = absolute temperature

indicates that as temperature decreases, the rate of reaction decreases, and this relationship has been found to model the behaviour of biological systems. Busch (1971) reported that little or no effect of temperature on biochemical oxygen demand (BOD) removals is noticed down to $10^{o}C$; particularly in food limited systems. He also stated that reduced sedimentation efficiency and solids carryover, accounted for most temperature effects observed between $12^{o}C$ and $24^{o}C$, in the activated sludge system.

For design purposes the following relationship between the reaction rate coefficients at any two temperatures has been developed:

$$k_T = k\theta^{(T^1 - T)}$$

where k_T = reaction rate at temperature T^1

k = reaction rate at temperature T and

θ = the temperature coefficient.

The value of θ has been seen to vary widely with process type and loading. Eckenfelder (1970), citing various literature data, reported values of 1.0 to 1.135 for θ (see Table 2-1).

The constant Q_{10} has been defined as, "the ratio of rate of substrate utilization at temperature T, to the rate at temperature $T-10^{o}C$ (Busch, 1971):

$$Q_{10} = \frac{k_T}{k_{T-10}}$$

Values of Q_{10}, like those of θ will vary from one process to another. It is also dependent upon which 10^{o} decrease in temperature is chosen. The state of nutrient limitation

Table 2-1

Temperature Coefficients

(after Eckenfelder, 1970)

Process	θ
Activated sludge	
F/M <0.5	1.0
>0.5	1.0 - 1.04
Trickling filters	1.035
Aerobic lagoons	1.035
Aerobic-facultative lagoons	1.07 - 1.08
BOD bottle	
(20 - 30°C)	1.056
(4 - 20°C)	1.135

in which the process operates is also a cause of variability in Q_{10}. Because of these effects the value of Q_{10} has been seen to vary from 1.4 to 2.1 (Busch, 1971).

Howland (1953) found, for trickling filters, that as organic loading increases, so does the effect of temperature. He also noted that increases in fractional removal decrease the value of θ. The statement that θ varies directly with loading has been supported by Busch (1971) and Keefer (1962). Busch has said that the reduced reaction rate caused by reduced temperature can be offset by increasing the solids concentration in the reactor. In effect this is identical to decreasing the loading, or food to microorganism ratio (F/M) of the system. Keefer attributed similar removals of BOD at temperatures of 12°C and 24°C to longer aeration times at the low temperature. This is also the same as decreasing the system loading rate. Henry (1974), in a bacteriological study, found that the proportion of psychrophilic organisms in an activated sludge increased at low temperatures and that the proportion decreased with loading. He also stated that an increase in psychrophiles helps moderate the effect of cold temperatures on sewage treatment. Sutton (1976) found that temperature sensitivity decreased with increasing sludge age.

It appears that there is a definite advantage to carrying a high solids concentration and solids retention time (SRT) in the aeration tank if the treatment plant is operating at low temperature.

2.4.2 Solids Separation

The sedimentation and recycling of the microbiologic... sludge is critical to the function of the activated sludge process. The settling of flocculant solids must overcome the resistance of two forces, interparticle forces and fluid drag (Dick, 1970). Fluid drag is a direct function of viscosity, which is in turn an inverse function of temperatu... From Stoke's law it can be shown that:

$$\frac{V_1}{V_2} = \frac{\mu_2}{\mu_1}$$

where V = the particle settling velocity and

μ = the absolute viscosity (Reed & Murphy, 1969).

Stoke's law applies to a single ~~particle~~ settling in a fluid and as such could be used only when the mixed liquor suspended solids (MLSS) entering the final clarifier is very low. As concentration increases, resistance to settling, due to interparticle forces, increases to the point where it is the larger portion of the resisting forces (Dick,1970). These principles lend credence to the second conclusion of Reed and Murphy (1969), that the influence of temperature on the settling velocity of activated sludge decreases as concentration increases. This demonstrates that the thickening function of secondary clarifiers can be carried out successfully at low temperature, which was the third conclusion of Reed & Murphy. However, the efficiency of the clarification function of the secondary clarifier may be very important in the overall picture. Small flocs which may be left behind by the settling sludge mass will settle as individual particles in accordance with Stoke's law. They, therefore, settle more slowly at low temperatures and may be carried over the clarifier weir. It is, therefore, important to provide a solids separation system which operates well in both the hindered and discrete settling regimes.

2.4.3 Lagoons for Northern Waste Treatment

Lagoons as a method of waste treatment have seen extensive use in northern Canada and in Alaska. In Alaska, lagoons are particularly popular at military outposts and bases. Clark *et al.* (1970a) noted that seventeen lagoons, both facultative and aerated were in operation. Each type of lagoon has its advantages, disadvantages and treatment capabilities. A brief summary of lagooning in the North follows.

2.4.3.1 Facultative-anaerobic lagoons

These are lagoons which have no forced aeration system and as such are anaerobic in winter and facultative in summer. Typical summer BOD removals are around 70%

while those observed in winter may be less than 50% (Clark *et al.*, 1970a). Deans & Heinke (1972) reported that such lagoons are essentially dormant during the winter. The proposal for winter storage-summer treatment advanced by Clark *et al.* (1970b) appears to hold some merit in light of poor winter performance. However, Deans & Heinke (1972) felt that algal blooms in receiving waters would cause downstream dissolved oxygen depression.

The waste loading on a facultative-anaerobic lagoon varies from one installation to another. Loading rates of from 11 to 5268 kg BOD/ha/day (10 to 4700 lb BOD/acre/day) have been reported by Clark *et al.* (1970b). Odor production is thought to be a function of loading rate. Dawson & Grainge (1969) recommend loading rates of 22.4 kg BOD/ha/day (20 lb BOD/acre/day) in order to control odors. Clark *et al.* (1970b), however, report that lagoons with loadings as high as 5268 kg BOD/ha/day (4700 lb BOD/acre/day) experienced no odor problems while odors did exist at another installation with loadings of 278 kg BOD/ha/day (248 lb BOD/acre/day). Clark *et al.* (1970a) recommend loadings less than 56 kg BOD/ha/day (50 lb BOD/acre/day). High rate primary basins followed by long detention time basins have been used successfully and are recommended by Dawson & Grainge (1970).

Pathogenic contamination of surface and groundwater is a concern in the use of facultative anaerobic ponds in the North. The ponds must eventually be abandoned due to sludge accumulation. Sludge entering the lagoon is not rapidly decomposed and accumulation rates of 249-396 l/1000 people/day (8.8-14 ft^3/1000 people/day) have been reported (Clark *et al.*, 1970a). Even though high rates of coliform removals have been observed there remains the possibility of pathogen survival. Since viable pathogens have been isolated from the wastes of early Arctic explorers, it is apparent that the hazard of pathogenic infection will remain for some time after the facultative-anaerobic lagoon has been abandoned.

Dawson & Grainge (1969) have recommended that facultative-anaerobic ponds have a summer liquid depth of 1.2-1.5 meters (4-5 ft) and an unfrozen depth of 0.9 meters (3 ft) in winter. A retention time of 8 to 12 months and a loading rate of 22.4 kg BOD/ha/day (20 lb BOD/acre/day) was suggested in order to achieve 80% summer removals. In multiple cell systems, they recommend a high rate primary cell depth 3 to 7.6 meters (10-25 ft). This recommendation for great depth may cause problems in permafrost areas with frost heave and permafrost thawing. Thornton (1974) has noted that it is essential to maintain an impermeable frozen core in impounding embankments in the North.

2.4.3.2 Aerobic lagoons

Aerobic lagoons have a system of forced aeration which serves two functions: (1) oxygen transfer for the biological population, and (2) mixing. Of these, the mixing requirement is most often the limiting factor. Aeration devices should be selected on the basis of operational considerations. Due to winter icing problems, surface aerators are generally considered unsuitable (Deans & Heinke, 1972; Dawson & Grainge, 1969; Clark *et al.*, 1970b).

Christianson & Smith (1974) have reported that while fine bubble diffusers are more efficient in terms of oxygen transfer they may not be more economical. The main fault with fine bubble diffusers has been the problem of clogging with oil, sand, biological solids and other debris. Cleaning procedures recommended by the manufacturer are often only marginally successful. For these reasons Clark *et al.* (1970b) have concluded that coarse bubble diffusers are better from both the maintenance and efficiency standpoints. They reported that with a fine bubble system 70-85% removals could be achieved, in comparison to the 80-90% removals observed in the coarse bubble system.

Most aerobic lagoons in the North typically have detention times of 15-30 days (Dawson & Grainge, 1969; Clark *et al.*, 1970a; Clark *et al.*, 1970b; Reid, 1966). These same authors reported removals of 80% BOD even at reduced temperatures. Loadings were between 0.006 kg/m^3/day and 0.21 kg/m^3/day (4 and 1.3 lb BOD/1000 ft^3/day). Algae is present in the effluent from aerated lagoons in the summer and may reduce efficiencies due to their oxygen demand (Reid, 1966; Clark *et al.*, 1970a).

Since the aerobic system is more efficient than the facultative variety, sludge accumulation is greatly reduced. Accumulations of 42.5-113 l/capita/year (1.5-4 ft^3/capita/year) were reported by Clark *et al.* (1970b).

Lagoons often have to be lined in order to control seepage and precautions against ice damage of air headers and berms must be taken.

Even though, in winter, the mixing imparted by the aeration system helps reduce ice cover, freezing has been reported by most authors. Solids entrainment in the frozen mass, along with low reaction rates in the 1°C liquid phase combine to reduce the efficiency in winter. When spring arrives, the untreated mass which has been deposited in the lagoon becomes available for treatment and oxygen utilization increases. Christianson & Smith (1974) reported that this increased activity lowered the dissolved oxygen in the lagoon to near zero. No odors were produced, however, even at these low dissolved oxygen (D.O.) levels. The absence of odors from aerated lagoons is one of the attractive features of the system.

2.4.4 Activated Sludge

The extended aeration variation of the activated sludge process has become the most popular system for use in Arctic communities (Alferova *et al.*, 1974; Clark *et al.*, 1970b; Deans & Heinke, 1972). Extended aeration relies on low loading rates to maintain the sludge in the endogenous phase where excess sludge production is kept to a minimum. Low loading rates can be achieved by extending the detention times to as much as 24 hours or by increasing the mass of biological solids (Busch, 1971). Typical loading rates for the extended aeration process are 0.05-0.15 g BOD/g MLVSS/day as opposed to 0.2-0.6 for conventional activated sludge (Metcalf & Eddy, 1972). A flowsheet for a typical extended aeration system is included as Figure 2.2.

Primary settling of the influent sewage is not usually practised in extended aeration. This fact, in combination with a reduced sludge wasting schedule, help make its operation relatively simple (Deans & Heinke, 1972). In Section 2.4.1, it was noted that temperature effects on activated sludge decrease as loading rates decrease. Since extended aeration is a lightly loaded or low rate system, it is a viable alternative in cold climate applications. Clark *et al.* (1970b) have reported that extended aeration treatment has been successful at temperatures as low as 2°C. Most applications of extended

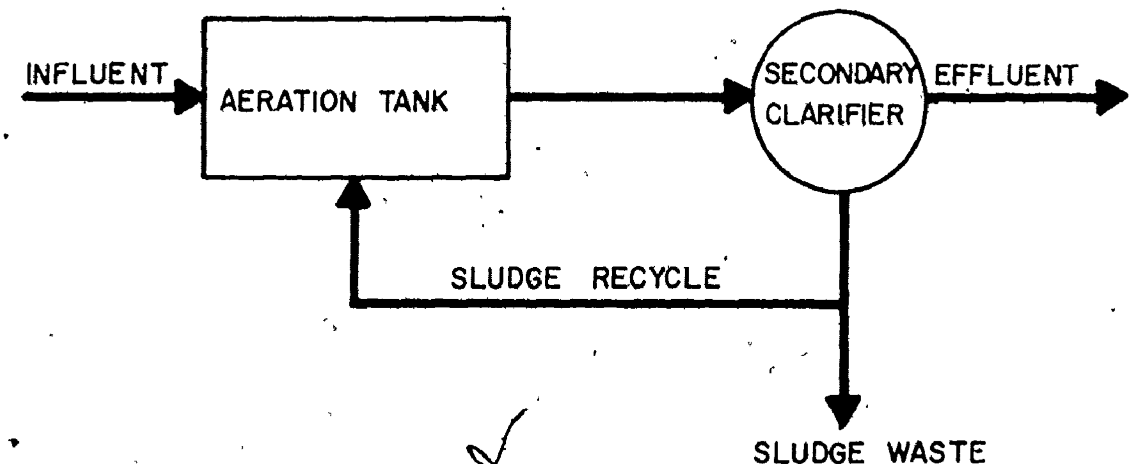


FIG. 2.2 TYP ION FI T

aeration in the North are at small installations where they are subject to highly varying flow rates and pollutant loads. By virtue of its low loading rate, extended aeration has some excess capacity which enables it to absorb such variations without serious upsets (Deans & Heinke, 1972).

Despite the apparent advantages to be gained by the use of extended aeration, significant drawbacks do exist. Extended aeration effluents are characteristically somewhat turbid. Clark *et al.* (1970b) quoting Pipes (1969) suggest that this may be the result of some aerobic digestion of the sludge in the aeration tank. Solids separation is, therefore, impaired as seen by the SVI values of 150-300.

Soluble BOD removals are high, even though total removals vary from 75-95% (Metcalf & Eddy, 1972). The bulking of extended aeration is accentuated at low temperatures. On the other hand, efficient suspended solids removal will offset the above noted problem. Upflow and tube type clarifiers have shown promise in this area (Clark *et al.*, 1970b; Buzzell *et al.*, 1974).

As designed at present most extended aeration package plants require an operator at least part time. Heuchert (1974) reported the total failure of package plants in the absence of operator attention. The operator is required for maintenance of mechanical equipment and for periodic sludge wasting.

The extended aeration process has been incorporated into package treatment systems for small applications. These are usually compact in order to be thermally and spatially efficient (Deans & Heinke, 1972). Experience with package extended aeration plants will be related in Section 2.6.

2.5 Physical-Chemical Treatment

A variety of physical and chemical processes are used at present for the treatment of wastewater. Broad categories for these processes are: membrane processes, ion exchange, adsorption, precipitation, coagulation-flocculation, sedimentation, and disinfection. Within each of these categories is a wide range of variations which can be tailored to the particular problem. The best known and most widely used physical-chemical treatment (PCT) system is the clarification-adsorption system (Cohen, 1974). A flowsheet for this type of system is shown in Figure 2.3.

2.5.1 Coagulation-Flocculation

Since 80% of raw sewage COD is in colloidal or larger solid form (Weber, 1972) a significant reduction can be achieved if these particles are removed. A large proportion of these particles must be destabilized and

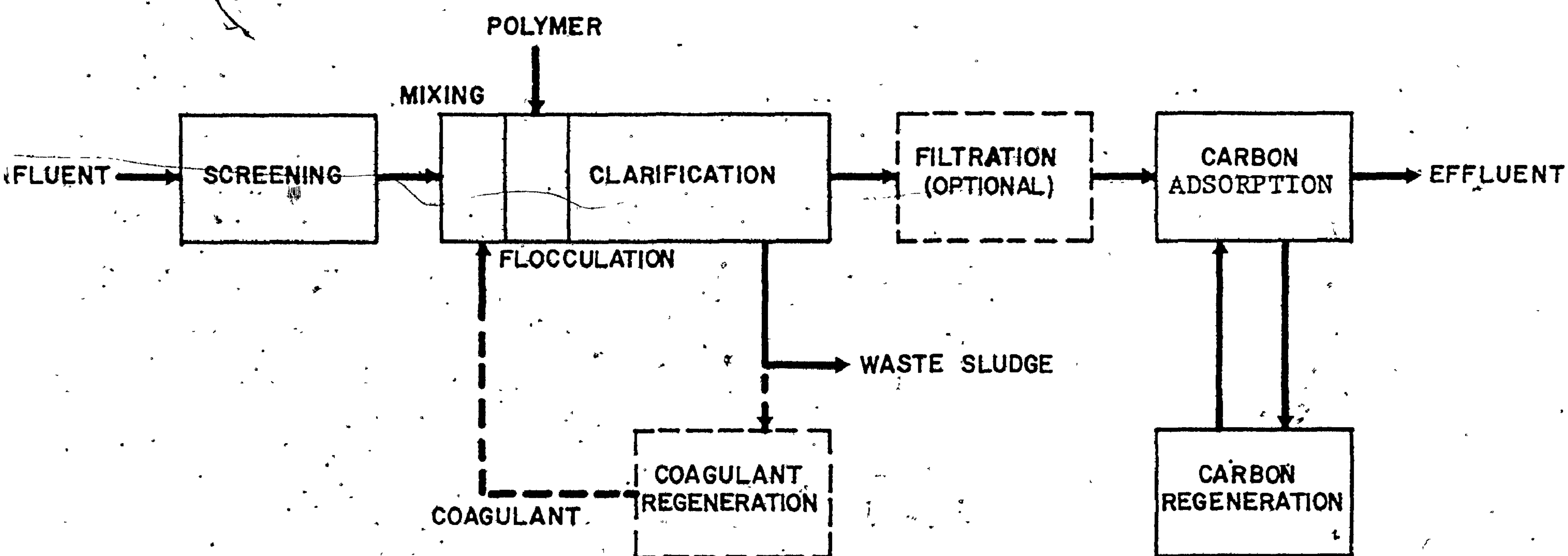


FIG. 2.3 TYPICAL PCT FLOWSHEET (AFTER MAQSOOD, 1975)

aggregated before they will settle out. The addition of coagulant chemicals, such as salts of iron and aluminum, serve to lower the surface charges of the particles. Particles are thereby encouraged to flocculate and settle out. Flocculation is sometimes carried out with the aid of polymers which hasten floc formation and sedimentation.

2.5.2 Sedimentation

The flocculated suspension is then passed to the clarifier where the flocs are permitted to settle out. Clarifiers of any type may be used for this process.

2.5.3 Filtration

Filtration is an optional process. It serves to remove any solids which might not settle out in the clarifier. Deep granular filters of sand, dual-media or multimedia configuration have been used. Depending on the application of the system, filter precoats of diatomaceous earth or powdered activated carbon have been used (Weber, 1972). Another function of the filtration step is to minimize the backwash requirement of the adsorption column.

2.5.4 Adsorption

The adsorption process usually utilizes activated carbon as the solid phase. Activated carbon has a very high surface area (Rankin, 1975) and is highly nonspecific in its adsorptive capabilities (Benedek, 1973). This makes it an ideal material for use in removing the wide variety of dissolved organic compounds found in sewage. Typical wastewater is percolated through activated carbon columns. The columns are designed to provide contact times of between 20-60 minutes and loading levels of approximately 0.5 g COD/g carbon (Cohen, 1974). With these loadings levels, and in conjunction with the previously mentioned processes, an effluent of secondary quality can be achieved.

The activity of biological films on carbon serves to increase its removal capacity. This may be due to *in situ* biological regeneration of the carbon (Weber *et al.*, 1972) as well as soluble substrate uptake from the liquid phase by the biological mass.

2.5.5 Regeneration

Coagulant chemicals are not usually regenerated, although it is possible. Carbon, however, can be regenerated by heating in an oxygen limited steam atmosphere. Some carbon is oxidized in the process and a quantity of make-up carbon is required. This process is usually only feasible in large installations and would, therefore, not be considered in a system such as WatRek.

2.5.6 Other Processes

Disinfection is an important process in any wastewater treatment scheme. It is usually accomplished by using some form of chlorine or other halogen. Ozone, heat and ultraviolet radiation have also been used. It is important to maximize the percentage of bacterial kill for a safe effluent discharge.

Membrane processes, ion exchange, and precipitation are generally used in very specific situations and have seen limited use in sewage treatment. They are useful when a high quality renovated water is required.

2.5.7 Temperature Effect on PCT

The properties of viscosity, density, diffusivity and solubility are all affected by temperature and in turn affect the processes utilized in PCT.

Liquid viscosity is an inverse function of temperature. Over the range of interest in PCT, viscosity varies by a factor of 2. There is, therefore, an increase in resistance to movement between solid and liquid. Although density varies inversely with temperature, the range is negligible over the region of interest.

Diffusivity varies directly as the square of temperature when the effect of viscosity is included. The variation over a 30°C range, is about 23% (Maqsood, 1975). The solubility of metallic salt coagulants is reduced in cold water.

Maqsood (1975) has reviewed the theoretical effects of temperature on the various PCT processes. His major conclusions are summarized below on a process by process basis.

2.5.7.1 Coagulation-flocculation

Reduced temperature affects coagulation through the lowered solubility of the destabilizing chemical. The limiting factor in wastewater flocculation is usually bulk fluid motion or orthokinetic flocculation which is proportional to the inverse square of temperature. Flocculation is, therefore, only a weak function of temperature.

2.5.7.2 Sedimentation

The effect of temperature on activated sludge settling was discussed previously in section 2.4.2. Changes in fluid viscosity with temperature will have an effect on the settling of individual particles. However, Maqsood has concluded that settling was not crucially affected by temperature.

2.5.7.3 Porous media filtration

The headloss through any granular bed should increase as temperature decreases because of the greater viscosity of the liquid.

2.5.7.4 Adsorption

Since the solubility of most substances decreases with temperature adsorption should increase with decreasing temperature. Maqsood found that low temperature had a midly negative effect on adsorptive kinetics but a positive effect on adsorption capacity.

Removals by the biological film in a carbon column should be subject to the same temperature effects as discussed previously in Section 2.4.1.

2.5.8. Advantages of PCT (Cohen, 1974)

1) Stability. The fact that, in PCT, there are essentially three possible ways of removing solids, adds an inherent stability to the system. Cohen (1974) states that solids passing the sedimentation tank are trapped by the filtration step. Should solids breakthrough occur in the filter, the carbon column would serve as a backup filter. Also, unlike biological systems, PCT systems are not affected by the presence of toxins in the feed.
2) Space. A minimum of land area is required for PCT in large applications.
3) Rapid startup. No extended period of sludge accumulation and acclimation is required for PCT is

4) Treatment. Acceptable removals of organic compounds are consistently met and frequently surpass secondary treatment levels.

5) Metals removal. The capacity for heavy metals removal is easily added to the system by the inclusion of a precipitable step.

2.5.9 Disadvantages of PCT in the North

1) Cost: The requirement for a continuing addition of chemicals is a cost disadvantage. The costs of chemical addition include the chemical itself and the transportation, holding and metering of the chemical.

2) Operators. In northern communities where operators are scarce, a treatment system must be simple and automated. Since chemical addition and the other unit processes require mechanical facilities, maintenance adds an additional need for a skilled operator.

3) Sludge disposal. Significant volumes of sludge are produced in the PCT process and must be disposed of. An additional difficulty is the poor dewatering characteristics of most PCT sludges (Cohen, 1974).

4) Flow rate. Since automatic chemical addition would likely be tied to the flow rate, a constant [illegible] the

requirement. It is characteristic of small systems, in any area, that peak flows may be 2-3 times the daily average. Thus, flow equalization may be necessary for proper PCT design.

2.6 Package Sewage Treatment Plant Experience

A package sewage treatment plant has been defined by Kolbe (1975), as a prefabricated and pre-engineered sewage treatment unit. He went on to describe five situations in which packaged treatment was applicable:

1). When sewage treatment is required quickly, *e.g.*, construction camp sites;
2) To relieve pressure on present sewage works, during periods of high occupancy;
3) When sewage treatment is required on a temporary basis prior to connection to a larger system;
4) Where modular construction is an advantage, and
5) When it is economically advantageous.

The fact that these plants are usually designed for small populations less than 2000, means that they are subjected to large peak flows. Furthermore, routine operation and maintenance can be prohibitively expensive due to the lack of onsite personnel. Hence, in small systems, careful design is required to overcome the high flow variability and to minimize the common lack of operator attention.

Lack of attention has been cited by Heuchert (1974) and Turvey (1975) as reasons for poor performance in package systems. Turvey was assessing a number of South African plants and went on to say that under-design, unsatisfactory design and misuse through poisoning were also problems. Heuchert was evaluating two extended aeration plants on an artificial island in the Beaufort Sea. He concluded that a complete lack of attention was the reason for their failure.

Operational difficulties with biological package treatment plants center around sludge conditioning. Vosloo (1975) reported that light, bulky sludges, with filamentous growth and SVI's greater than 400 ml/g, were probably due to overaeration and/or poor sludge return. He stated aeration should be set up such that at high flows dissolved oxygen levels should be around 1 mg/l. He considered that the return of sludge, through a small slot, from the clarifier to the aeration tank, by gravity, was unsatisfactory due to the complex hydraulics at the slot, and the possibility of clogging.

Deans & Heinke (1972) also noted the poor control of air flow in most biological package plants. They also note other disadvantages: (i) turbid effluents; (ii) the need for power and maintenance facilities. They listed advantages including the thermal efficiency of a compact package plant, low sludge production, no primary sedimentation and a tolerance to shock loadings due to the low F/M ratio.

These authors also suggested some considerations for the design of packaged treatment facilities for use in Arctic work camps. The items noted may be extended to other small applications in the North. Designers should: (i) consider strong wastes; (ii) provide for sludge handling; (iii) install bypasses; (iv) add a comminutor for kitchen waste; and (v) heat trace exposed lines.

2.6.1 PCT Package Plants

Package treatment plants utilizing PCT processes are available from several manufacturers. They include a variety of process combinations but the coagulation-adsorption sequence is the most popular. In this section, a few package plants will be mentioned with respect to their concept, purpose and performance. Although several of these plants were neither designed for, nor tested in northern application, all were designed to handle a strong waste. The level of automation and consideration of a recyclable effluent also make them of interest to this study.

Kreissel & Cohen (1973) reported on the evaluation of a commercially available package plant of 91000 l/day (24000 US ... itv ... of is shown in Figure 2.4. The comminution and degritting facilities are not shown. Aluminum and ferric sulfates were used to coagulate the sewage. The carbon loading rate was 0.05-0.1 m/hr (1.3-2.5 US qpm/ft^2).

The system performance was satisfactory despite poor solids capture in the clarifier. A clear effluent of consistently high quality was obtained, probably due mainly to the filtering and adsorbing capabilities of the carbon columns. Phosphorous in the effluent was less than 0.4 mg/l, 80% of the time, and the COD was less than 35 mg/l, 90% of the time. Color and turbidity levels were below those prescribed in drinking water standards. The removal of coliforms was comparable to that of an activated sludge plant. Pilot studies showed that disinfection could be accomplished with halogens or ultraviolet radiation. The authors reported that the system reached turbidity equilibrium, after startup, in 2-4 hours.

Compact PCT packages have been developed for marine applications. Qasim *et al.* (1973) described a shipboard unit which consisted of a recirculating chemical toilet and an evaporation system for liquid/solid separation (see Figure 2.5). The system had the two-fold purpose of treating wastes and reducing water use. The chemical toilet successfully reduced water use for toilet flushing from 100 l/day (26.2 US qpd) to 3.8 l/day (1 US qpd).

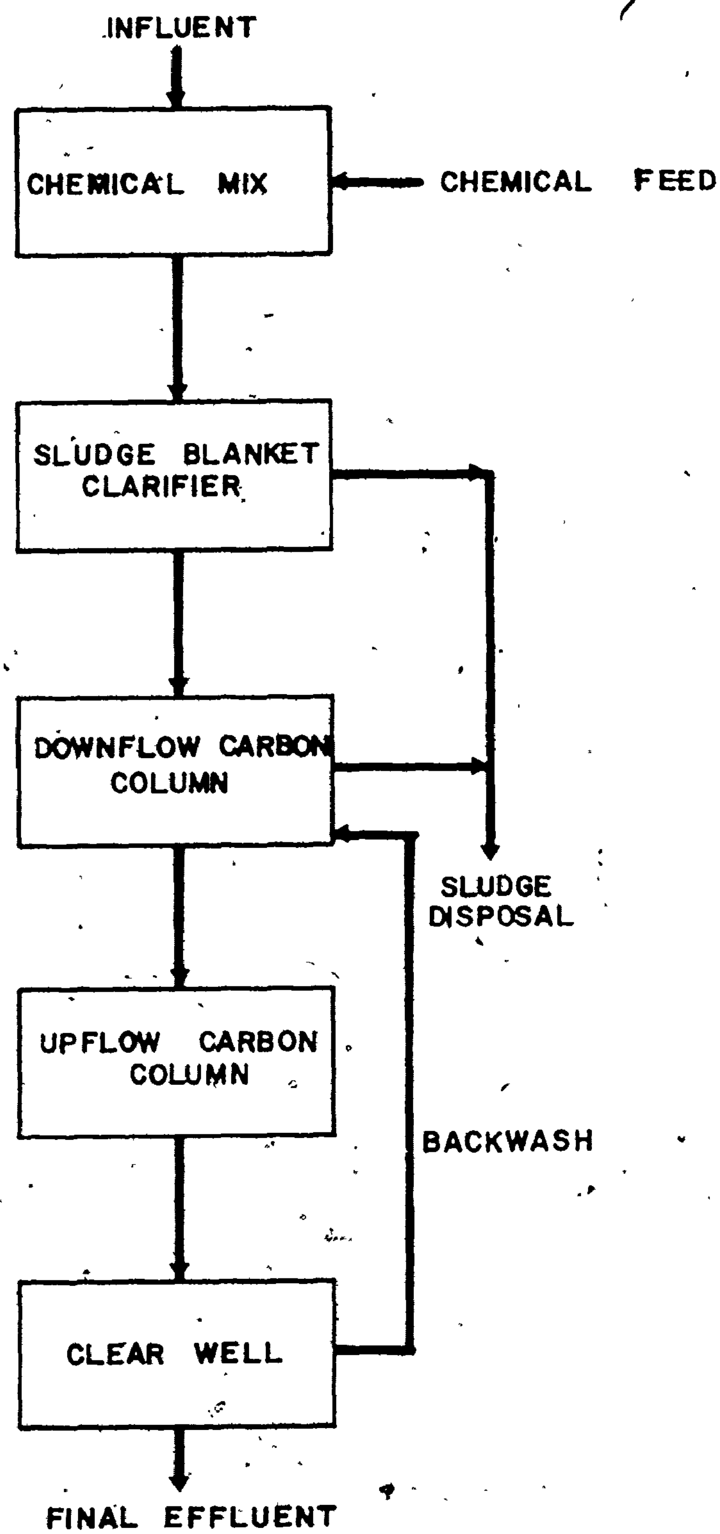


FIG. 2.4 PCT FLOWSHEET (AFTER KREISSEL AND COHEN, 1973)

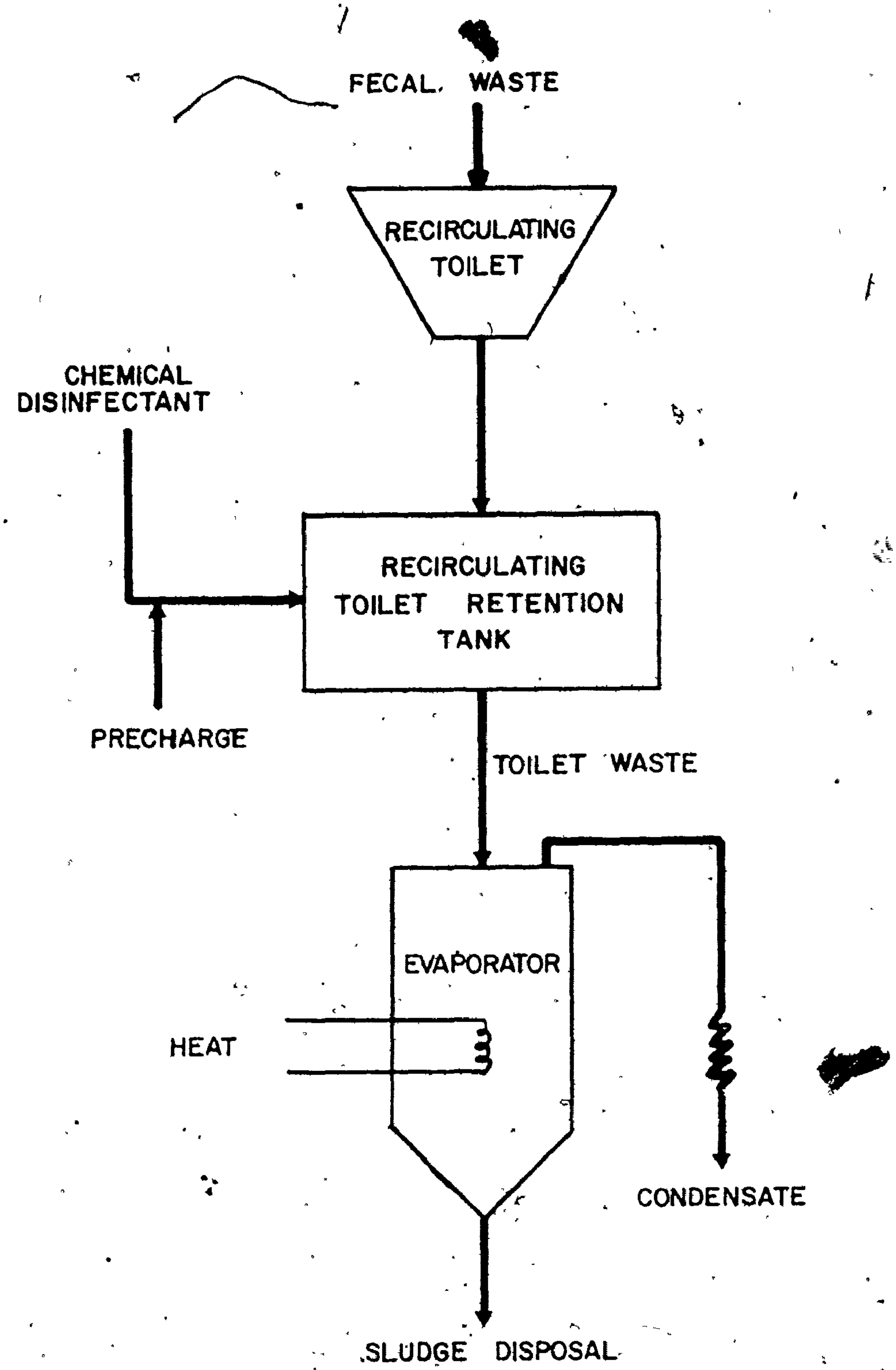


FIG. 2.5 EXPERIMENTAL MARINE TREATMENT SYSTEM (AFTER
ET AL

The evaporator, which was operated daily, produced a sterile sludge of 65% solids. The condensate, however, was not always within the design objective of <50 mg/l BOD. This was particularly the case when the evaporator elements were not cleaned regularly. Chlorination of the condensate was shown to reduce the condensate BOD to design levels. Chlorination may render the organics less biodegradable or even toxic, thereby inhibiting the measurement of BOD_5, however, it is unlikely to lead to significant reductions in COD or ultimate BOD.

Kaminsky *et al.* (1973) demonstrated a marine sanitation system in both the flowthrough and recycle modes. A system schematic has been included as Figure 2.6.

Salt water was used as a flushing medium. Black water was carried to the vibratory screen for removal of large solids. The filtrate passed to a flow equalization tank and then to a centrifuge which removed fine solids. Prior to the carbon adsorption columns the wastewater was chlorinated in order to reduce bacterial growth and the resultant clogging in the columns. Following adsorption, the wastewater was chlorinated again and either discharged overboard or returned as a flushing medium. The carbon column backwash water was either returned to the equalization tank or wasted overboard. The capacity of the system was 19000 l/day (5000 US gpd).

Pilot plant performance in the flowthrough mode

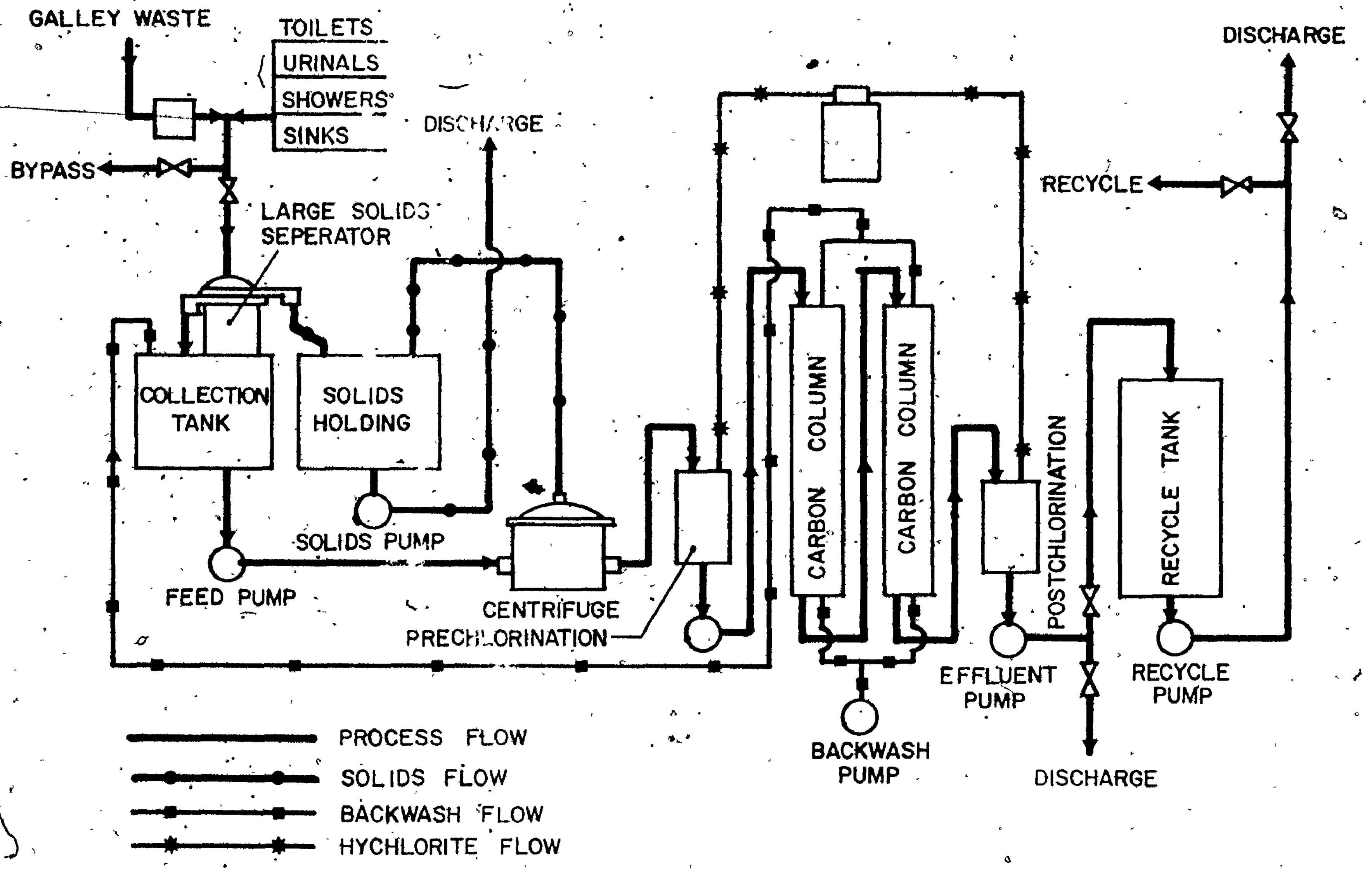


FIG. 2.6 MARINE SANITATION SYSTEM (KAMINSKY AND ROBERTS, 1973)

was adequate, achieving BOD and SS removals of 81.7% and 94.9%, respectively. Chlorination, both before and after the carbon step, was required to produce a bacteriologically acceptable effluent. This indicates that despite prechlorination, some growth may have occurred in the carbon columns.

When the system was in the recycle mode, an odor of ammonia was noted in the recycle tank after two days of operation. During the five day recycle runs, an increase in the effluent BOD and a decrease in removal efficiency across the system was observed. The addition of permanganate to the postchlorination step failed to reduce the BOD buildup. Effluent BOD values increased to 400 mg/l at the end of some of the recycle runs. Flushing liquid went from clear, to milky white, to grey during these runs.

The mechanical operation of the system was trouble-free. The effluent did show a variability and did not always reach the treatment goal of 50 mg/l for both BOD and SS.

Robins and Green (1974) developed a highly automated system for treating the wastes from pleasure boats at dockside pump-out stations. The wastes from these vessels, which used chemical toilets and various chemical additives was found to be quite high. BOD concentrations varied from 1700 to 3500 mg/l and showed questionable biological treatability due to the presence of the chemicals.

Zinc and formaldehyde were found to be particularly inhibiting.

The system as reported is shown in the schematic drawing, Figure 2.7. It included disinfection and comminution, powdered activated carbon adsorption, alum flocculation and vacuum filtration with the aid of diatomaceous earth. Following a period of laboratory evaluation, alum dosage was increased and filter aid (diatomaceous earth) dosage was decreased. Pre-aeration was added to enhance BOD and ammonia removals. Zinc removal by precipitation was suggested and post-chlorination was required for disinfection.

Removals of SS, BOD and COD averaged greater than 97%, without post-chlorination and greater than 95% with post-chlorination. Zinc removals were greater than 90% when precipitation was included.

The system operated satisfactorily but produced very high dissolved solids in the effluent. Without the zinc removal, 4945 mg/l TDS was observed. This increased to 11000 mg/l when zinc removal was included. These values can be largely attributed to residual chemicals. Chemical addition amounted to 16 kg/1000 l treated.

Deans & Heinke (1972) reported on two systems which, although they had not been used in the North, the authors felt had promise. They were the Liljendahl Chemical Treatment System (Figure 2.8) and the Elsan-Yarrow

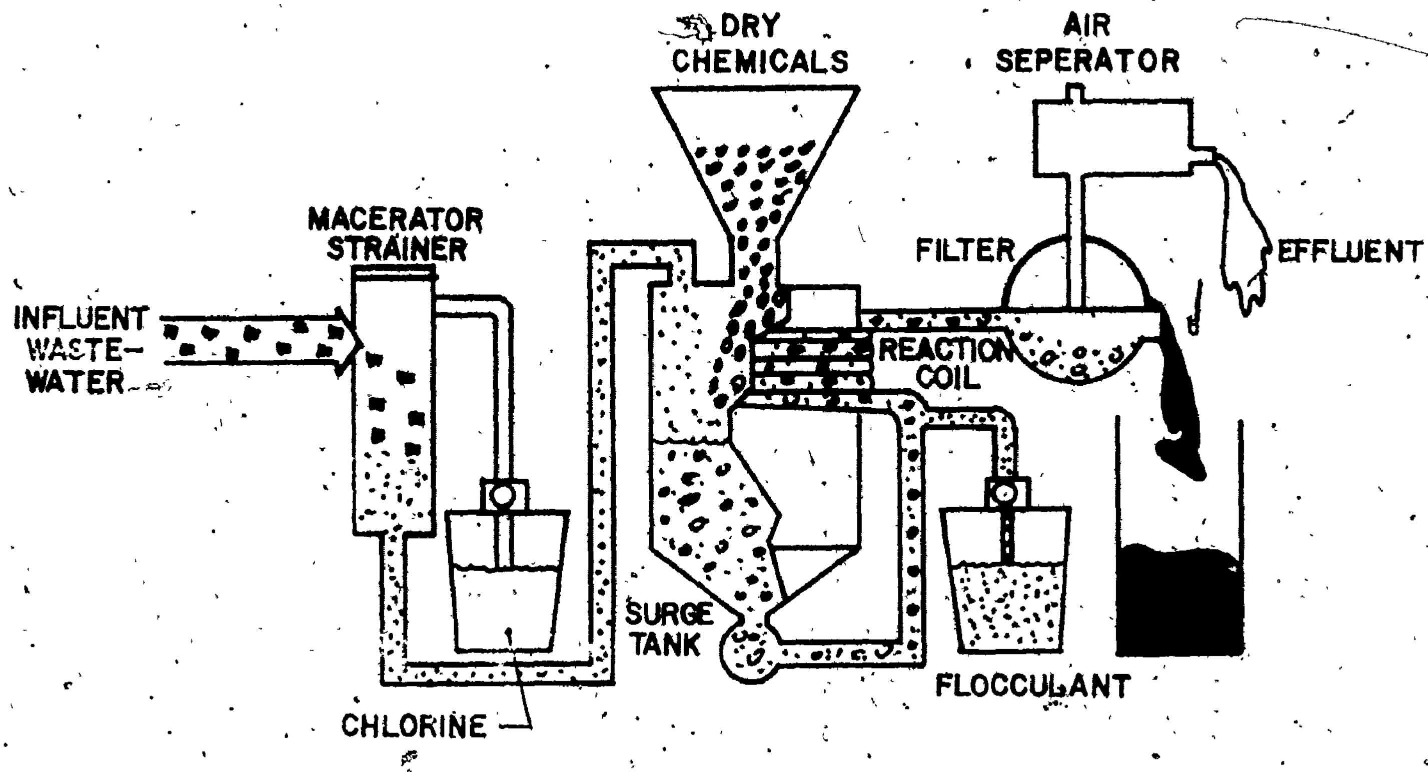


FIG. 2.7 FMC WASTE TREATMENT SYSTEM
(AFTER ROBINS AND GREEN, 1974)

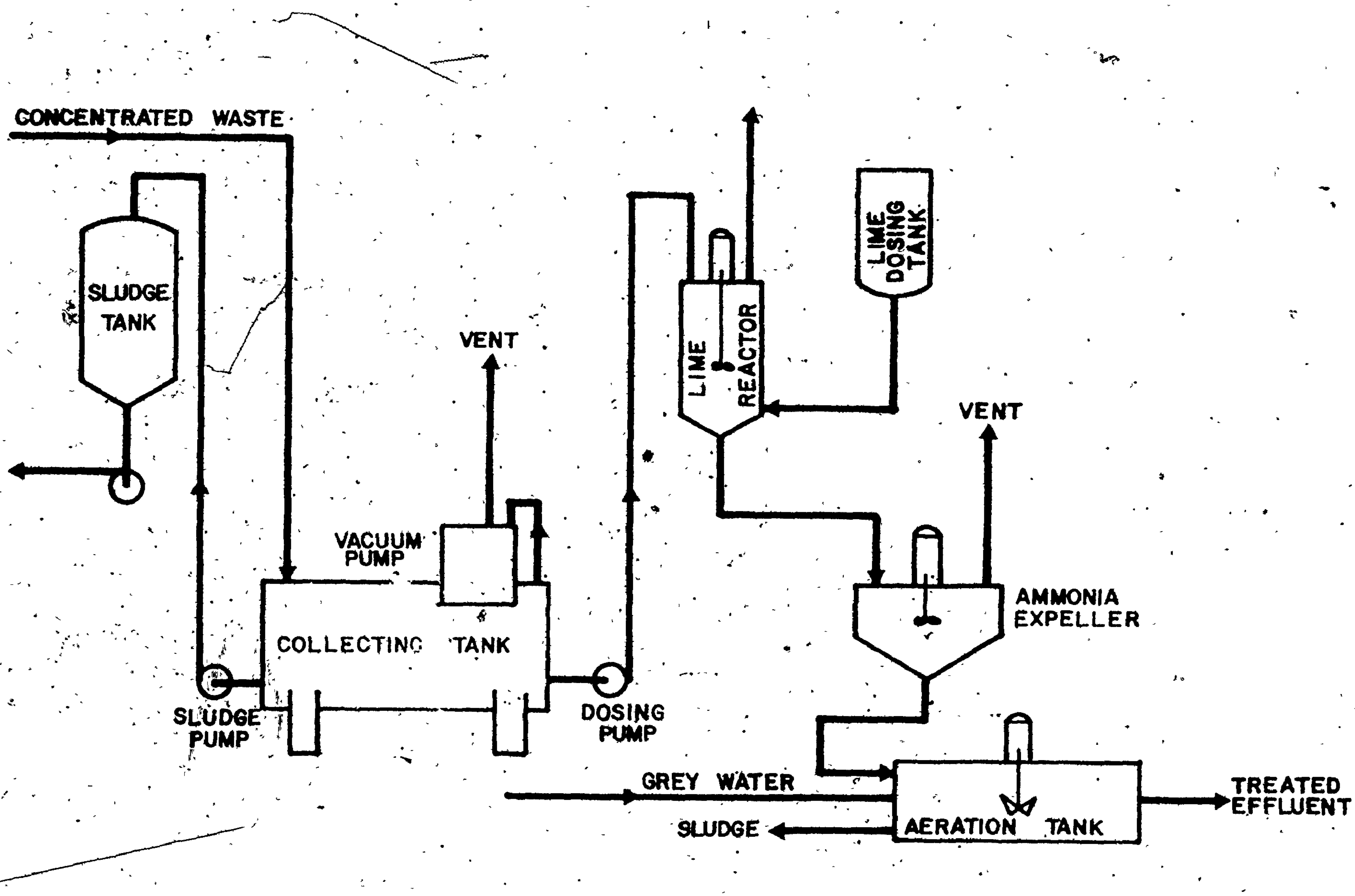


FIG. 2.8 LILJENDAHL CHEMICAL TREATMENT SYSTEM (DEANS & HEINKE, 1972)

System (Figure 2.9). Both of these were designed for use on small flows of highly concentrated waste and were compact.

The Liljendahl system is amenable to use with a vacuum collection system. Solids are settled out in the collecting tank and the sludge anaerobically digested. Lime is then added to the supernatant for disinfection and the conversion of urea to NH_3. The NH_3 is air-stripped and the effluent discharged.

While outwardly simple, a number of mixers and pumps are required, thereby adding a possible maintenance problem. Energy, in addition to that necessary to drive the various electric motors, would be required to heat the digester to a temperature at which decomposition rates would be acceptable.

The Elsan-Yarrow system used chlorine in tablet form to improve the odor and color of the waste. Following comminution, sodium hydroxide was added to further breakdown the waste. The manufacturer claimed that after settling the effluent could be recycled as flushwater. Drawbacks of this system include the need for technical supervision and the strong chemical nature of the sludge. This sludge might pose disposal problems.

The systems described above, while, for the most part, operating satisfactorily, have the disadvantage common to most physical chemical processes. They are highly mechanized and, therefore, are maintenance and

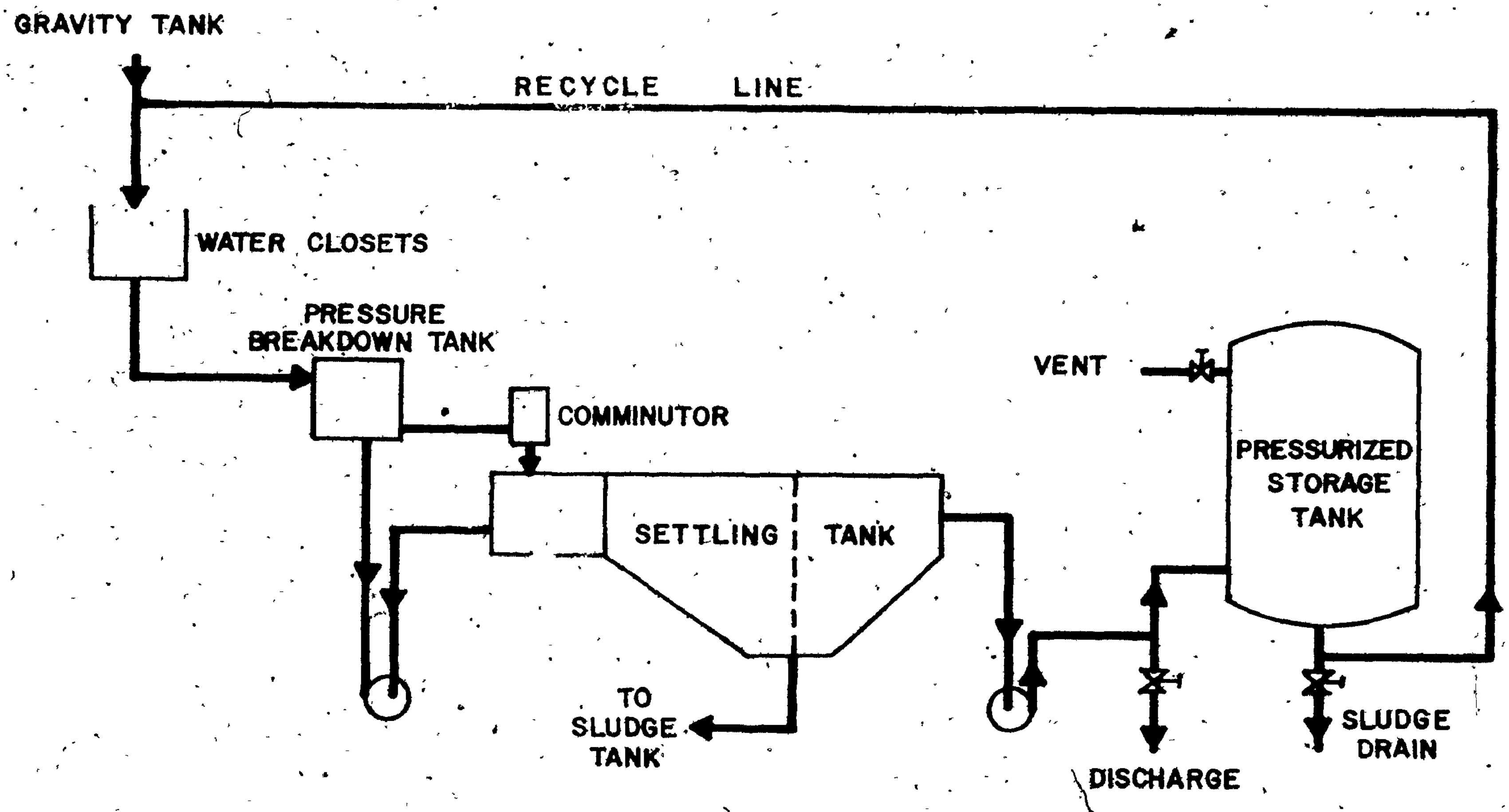


FIG. 2.9 ELSAN-YARROW CHEMICAL TREATMENT SYSTEM (DEANS & HEINKE, 1973)

energy intensive. All but one require significant chemical addition in addition to chlorination. The use of some of these chemicals, the corrosive ones, require special corrosion resistant tankage. Such materials add significantly to the cost of these systems. Without exception, they involve the disposal of large quantities of chemical sludges which may not be suitable for biological digestion and in any event, digestion would require further energy inputs. Open dumping of strong biological/chemical sludges is a practice that should be avoided.

2.6.2 Biological Package Treatment Units

Packaged biological treatment plants in the North are usually poorly operated and/or overloaded (Clark *et al.*, 1970b; Heuchert, 1974). These units are generally of the type often used in southern applications. Contact stabilization and extended aeration are the most common activated sludge variations used. Sludge recycle is often by gravity and wasting rarely carried out. Some difficulties with this mode of operation have been outlined previously (see Section 2.1).

The need for new and effective package treatment for the North has been accentuated by the failure of transplanted southern systems.

Buzzell *et al.* (1974) and Reid & Crowther (preprint) have reported on a system which accomplished biological treatment, flow equalization and secondary clarification in a single wood stave tank. Central to the system was a tube settler which utilized the classical sedimentation prinicples advanced by Hazen (1904) and applied by Culp *et al.* (1968). The settler was constructed of tubes angled at 60° to the horizontal and floated in the mixed liquor by means of carbuoys partially filled with water.

Floating the clarifier in the aeration tank has a three-fold benefit. First, it allows for efficient upflow clarification within the tubes. Secondly, it is thermally efficient. Heat from the clarifier is not lost to the environment, and the problem of clarifier freezing is alleviated. And finally, the ability of the clarifier to be floated at a given level, so that overflow rates are constant, provides flow equalization within the aeration tank free-board. Since flow over the clarifier weir is constant a hydraulic pulse simply increases the level in the aeration tank. The surcharge is then carried over the weir at a constant rate until the level returns to normal.

The aeration system was a pulsed, coarse air system which helped minimize the energy requirements. The tankage was redwood stave construction and was easily fabricated, transported and thermally efficient.

The performance of the system was within the range of extended aeration. Suspended solids and BOD removals were 77% and 87%, respectively. The authors thought that the gentle agitation of the clarifier, due to the aeration, aided in removing the sludge from the tubes.

Although, theoretically, the control of the depth over the weirs by flotation is sufficient to ensure constant flow rate, the authors used a ball valve in the exit line to control the flow rate. Any valves on a clarifier effluent line are subject to clogging with biological growth and escaping solids.

Assembly of the tank was completed onsite in eight man-hours. Figure 2.10 shows a sketch of the system. Operational requirements were small for this system. Daily checks were sufficient to look for breakdowns, stoppages, periodic sludge removal from the clarifier tubes and occasional excess sludge wasting. The system was a good example of thoughtful waste treatment design for the North, where simplicity is important.

Lomas and Townshend (1976) developed a carbon-adsorption, bio-oxidation prototype (CABOS) for use on ships. Following initial laboratory work a pilot plant was built and tested. The chosen processes were biological oxidation with powdered activated carbon (PAC) in the aeration tank, secondary clarification, multimedia filtration and ozonation. By virtue of the F/M of 0.14 g BOD/g MLSS in the aeration tank, the system may be considered to be extended aeration.

Startup of the CABOS was fast in terms of removals, likely because of the presence of the PAC, which would

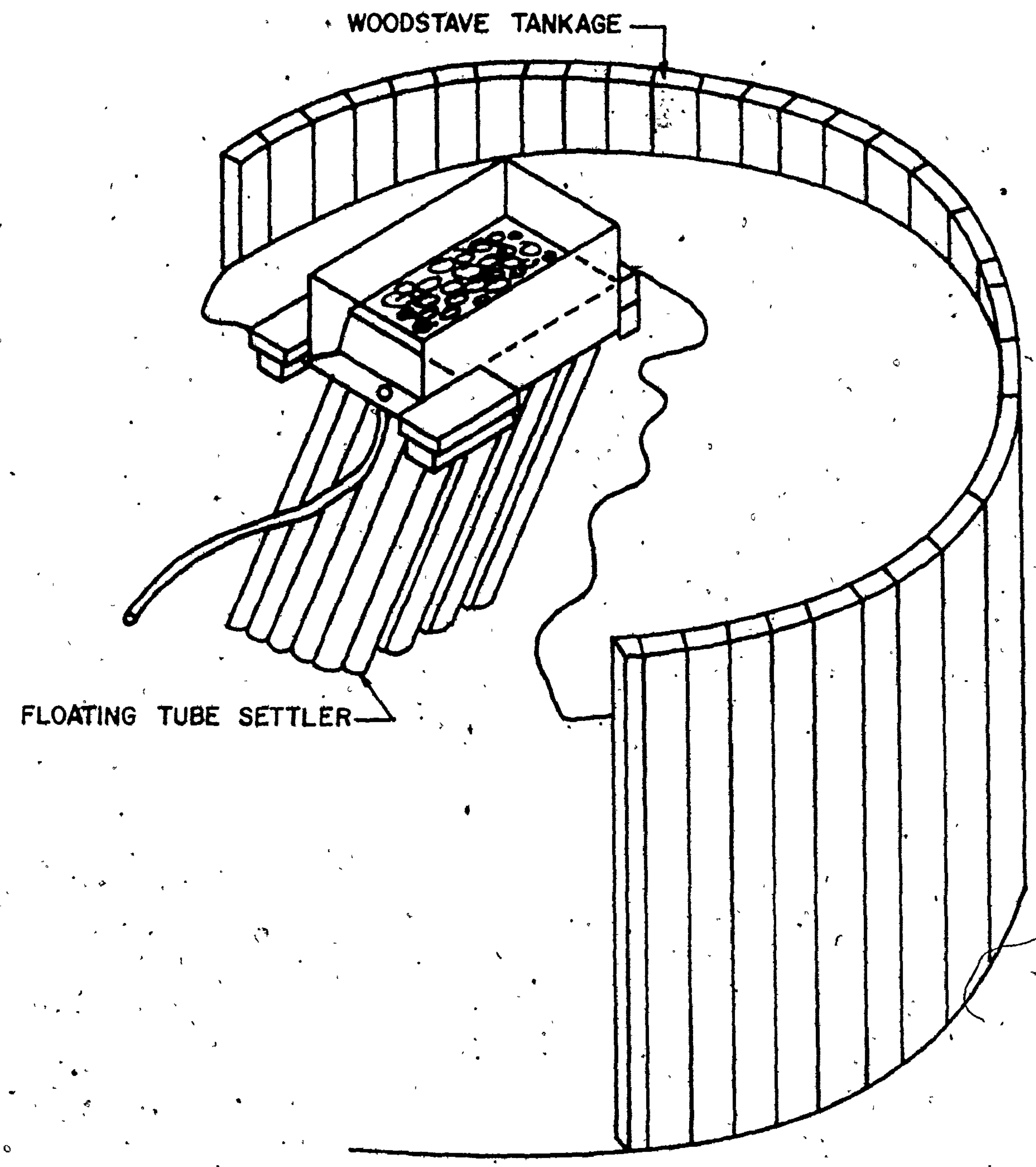


FIG. 2.10 SINGLE TANK EXTENDED AERATION SYSTEM
(AFTER BUZZELL AND REID, 1974)

adsorb organics before a sludge was developed. BOD removal through the aeration tank/clarifier system was 98%. Further removal was noticed across the mixed media filter. Ozonation caused no further removals, but proved to be an effective disinfectant. Excess biological sludge was periodically wasted. When the feed was interrupted for several days there was a negligible effect on the performance of the system.

The effect of toxic substances was investigated. Bleach was found to have no significant effect but efficiency was impaired by the addition of Pine-Sol.

CABOS appears to be an efficient system for shipboard application where there is mechanical expertise. Such a system, however, is too mechanized for northern application. Also, it appears that energy intensive processes such as ozonation may not be economical in the North.

Brown *et al.* (1975) describe the conceptual design of an environmental service module. Such systems, which amount to community water service centers, hold promise for the North. Brown's design includes renovation of water to a quality sufficient for non-potable re-use. PCT and conventional biological waste treatment systems were rejected and wet oxidation was chosen.

In wet oxidation the organics in the waste are oxidized at high temperature and pressure. The reaction is carried out at temperatures of 175-315°C (350-600°F) and 10.5-210 kg/sq. cm (150-3000 psig). Average removals are in the range of 80-90% (Metcalf & Eddy, 1972).

In the process described by Brown *et al.*, organic refuse and sewage are fed to a hammer mill for grinding and then macerated. The slurry is then pumped to the wet oxidation chamber and combusted. Liquid from the combustion chamber is then combined with greywater, coagulants are added and the liquid filtered through 50 micron screens. Further filtering is carried out in cartridge filters before a three stage reverse osmosis (R.O.) step. The concentrated R.O. waste is evaporated and the steam wasted or recovered. The final permeate of the R.O. unit is then ozonated for disinfection. The resulting effluent should be recyclable as flushing and washing water.

While there is no doubt that such a system would produce a recyclable effluent, there is also little doubt that there would be significant operation and maintenance problems. The high pressures required for wet oxidation and reverse osmosis would require careful monitoring and the equipment would need regular maintenance.

Such a complex system is almost the direct antithesis of the simple systems recommended by most of the authors summarized in this review. The power costs alone for the above system are almost 80% higher than those of the biological system which the authors rejected.

2.6.3 Community and Household Treatment Systems

In the previous section, the wastewater treatment system of a community service module was discussed. The concept of centralized water service facilities holds merit on economic grounds in isolated applications.

In facilities such as those described by Reid (1974) and Edwards & Fahlman (1974) incineration was used as final destruction of the waste. Water was reclaimed for flushing purposes, thereby reducing the total treatment requirement.

The other end of the service spectrum is the installation of individual household treatment units for treatment of domestic wastes. Typically, in the south, septic tanks have been used in this application. These depend on well drained tile fields for satisfactory performance. Conditions for a tile field may not be ideal in most northern regions, particularly in permafrost areas.

Duncan (1964) demonstrated a recirculating system which was simple and required no energy input, other than that used to operate a hand pump. The system used a marine toilet, the wastes from which were simply chlorinated and settled. The supernatant from the settling tank was recycled, via a hand pump, for flushing. For winter operation, the system was precharged with antifreeze. No freezing problems were encountered. Users of the unit were generally satisfied with it except for occasional odor complaints. Sludge from the settling tank had to be wasted periodically.

While the effluent produced by this unit may not have been acceptable for extensive recycle, the concept of a self contained, manually operated, recycling system is a good one.

CHAPTER 3

WATREK SYSTEM DESIGN

3.1 Design Criteria

The system has been designed to provide an alternative for the treatment of domestic wastes from construction camps, institutions such as orphanages and multiple family dwellings. It might also be incorporated as the waste treatment system, or a component thereof, in a central, integrated utilities facility such as those described by Reid (1974) and Edwards & Fahlman (1974).

Alter (1974) has outlined the factors to be considered in the provision of water supply and sewage disposal services to northern communities. These factors include:

1) High quality effluent;
2) Simple operation and minimal maintenance;
3) Reliability;
4) Low energy use, and
5) Minimum heat loss.

The treatment plant must be portable so that in a camp installation it can be easily moved and reused when the camp is relocated or shut down. In conjunction with this, the process must be capable of producing a high quality effluent without a long startup period.

3.1.1 Process Selection

In order to accomplish the objectives outlined above, a system as illustrated in Figure 3.1 was selected. This system is similar to that studied by Bromley (1977).

Raw domestic wastewater entered the aeration tank where COD was consumed by the biomass under aeration. The extended aeration modification of the activated sludge process was employed here. Waste activated sludge was conveyed to the aerobic digester where its mass was reduced. Digested sludge was returned to the aeration tank.

A floating tube clarifier was placed in the aeration tank to provide flow equalization, secondary clarification and gravity sludge recycle. An activated carbon column was added to the system as a tertiary treatment step.

The present system differed from Bromley's in the addition of the simultaneous digestion feature and the floating tube clarifier and the omission of coagulant addition. Disinfectant was not included in the WatRek prototype but could easily be added.

The rationale for selecting these unit processes has been discussed by Bromley and in Sections 2.4 and 2.5 of this work.

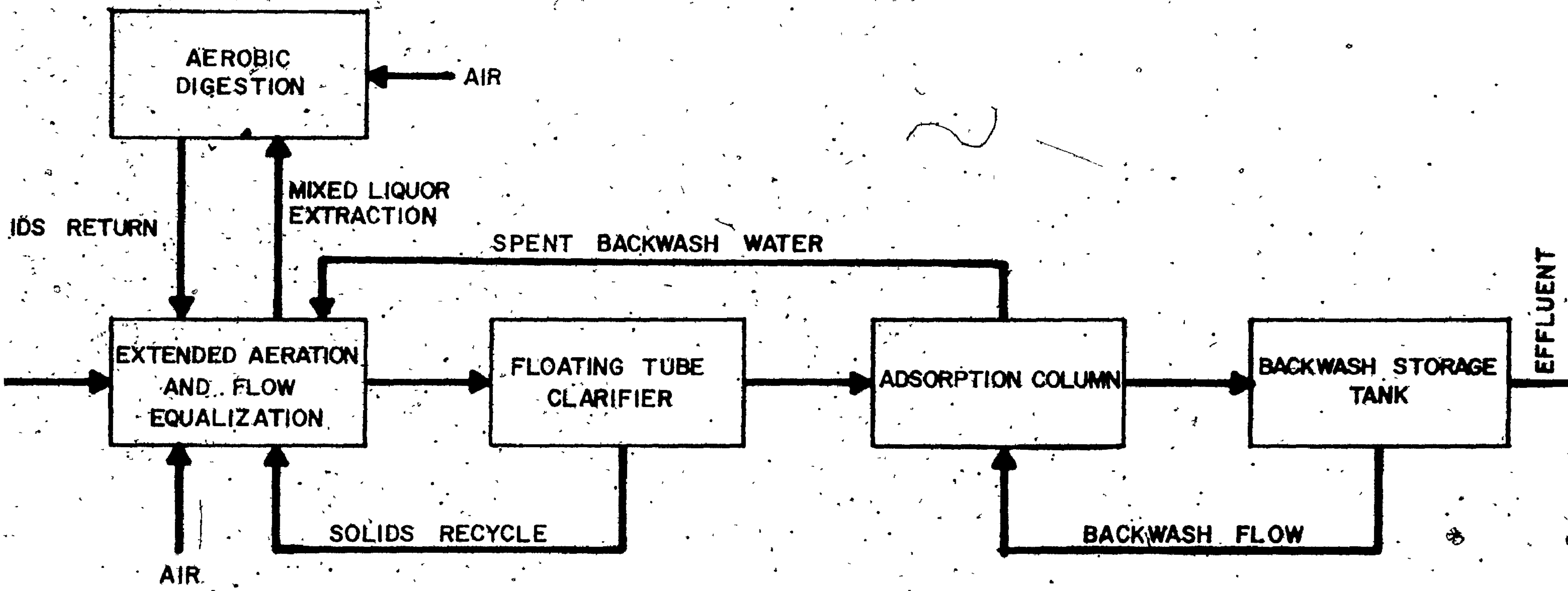


FIG. 3.1 WATREK FLOWSHEET

3.1.2 Kit Concept

In order to fulfill the requirement for portability within the constraint of costly air transportation, the concept of a prefabricated "kit" package treatment plant was evolved. The plant was designed to be dismantled into components which could be handled by two men. Assembly and disassembly times were not to exceed four man-days and were to be possible without special tools or skills.

A rectangular plan was chosen because the assembled unit had to be of minimum dimensions and have a high volume to wall area ratio to reduce heat loss.

3.1.3 Construction Materials

On the basis of weight, ease of assembly and cost, wood was chosen to be the major construction material. Table 3-1 summarizes the characteristics of the three materials investigated and Appendix C goes into more detail.

The final system was a compromise of the three materials; wood, steel and plastic. Framing was of steel angle section faced with a "skin" of 1.91 cm (3/4 inch) plywood. Polyvinylchloride (PVC) liners were installed within the tanks to make them water-tight and resistant to decay.

Table 3-1

Construction Materials

	Metal	Wood	Plastic
Advantages	High strength Water-tight Not attacked by microorganisms Acceptable cost	Light weight No special tools High strength in laminates Acceptable cost	Moderate weight Resists micro-organisms Resists water attack
Disadvantages	Requires rust-proofing Requires special tools High weight	Bulky construction Attacked by micro-organisms Subject to water attack	Low strength to weight relationship Requires molding High cost

3.2 WatRek Prototype Description

The WatRek prototype, as illustrated in Figure 3.2, consists of an aeration tank, activated carbon adsorption column, effluent holding tank and aerobic digestion tank. The air compressor and adsorption column backwash pump are situated at the front operational area of the plant.

3.2.1 Flow Scheme

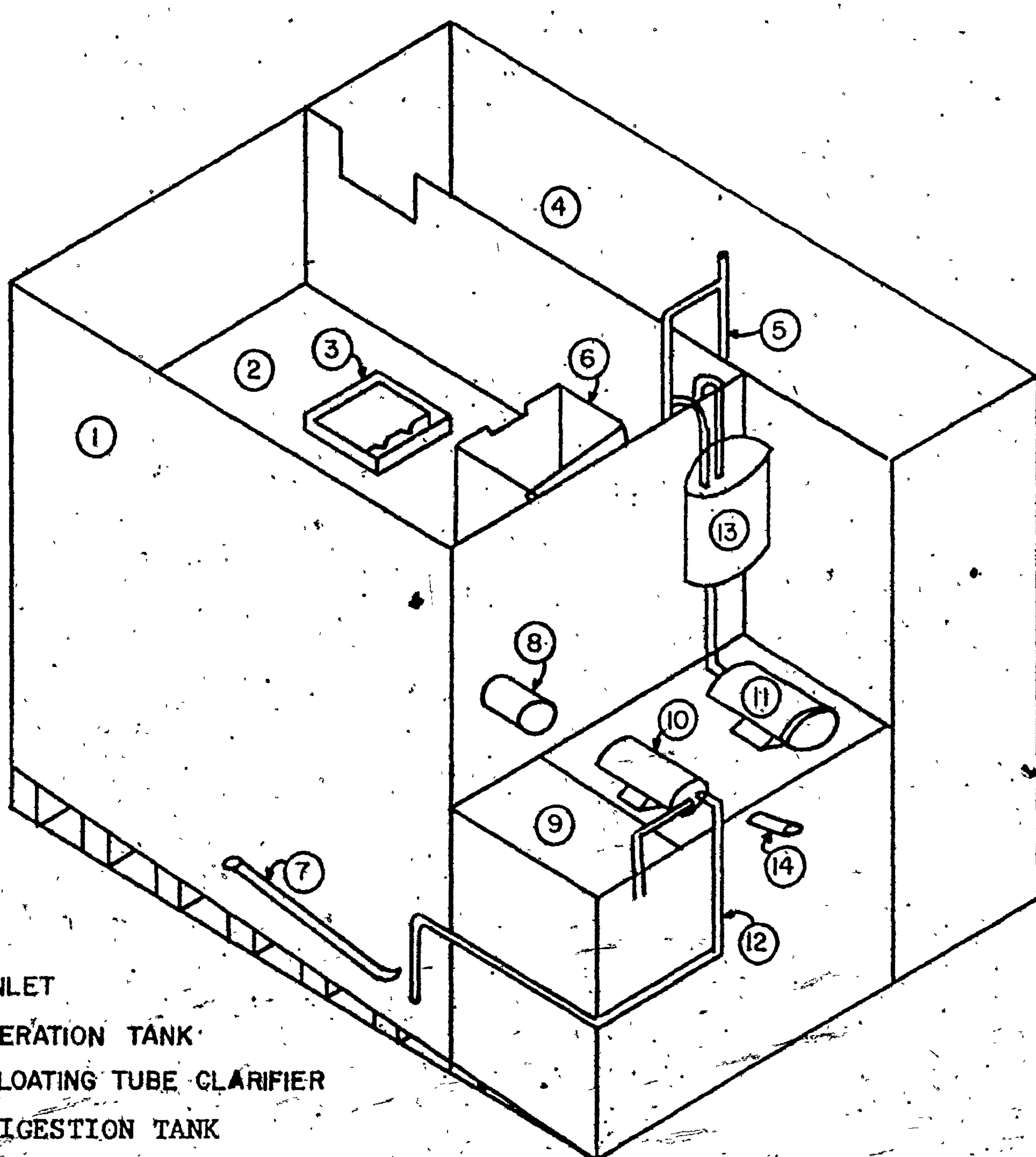
3.2.1.1 Aeration tank and aerobic digester

Waste entered the WatRek unit via the inlet port in the side wall of the aeration tank. This tank had an operating volume of 2143 l (75.7 ft^3) and freeboard sufficient to contain an additional 733 l (25.9 ft^3). The freeboard volume was intended to be used as flow equalization during times of high flow.

A design loading or food to microorganism ratio (F/M ratio) of 0.15 mg COD/mg MLVSS-day, was used to design the extended aeration process. When applied to the assumed feed concentration (Table 3-2) a value of 4000 mg/l mixed liquor volatile suspended solids (MLVSS) was obtained.

Assuming an apparent yield value of 0.3 g MLVSS/g COD removed, approximately 100 l (22 I gal) of mixed liquor at a concentration of 4000 mg/l MLVSS, must be wasted daily. Since the digester must provide >15 days detention to ensure adequate digestion in cold temperatures, its volume was set at 2492 l (88 ft^3).

FIG. 3.2 WATREK PROTOTYPE



(1) INLET
(2) AERATION TANK
(3) FLOATING TUBE CLARIFIER
(4) DIGESTION TANK
(5) MIXED LIQUOR EXTRACTION
(6) ADSORPTION COLUMN
(7) CLARIFIER EFFLUENT LINE
(8) CLARIFIER OVERFLOW
(9) BACKWASH STORAGE TANK
(10) BACKWASH PUMP
(11) AIR COMPRESSOR
(12)

Table 3-2

Summary of Aeration Tank and Digester Design Parameters

Design population	10
Per capita flow (l/day)	227
Per capita loading (g COD/capita/day)	151
System loading (g COD/day)	1517
Influent concentration (mg COD/l)	667
Aeration Tank	
Operating volume (l)	2143
Freeboard volume (l)	733
F/M (mg COD/mg MLVSS-day)	0.15
Apparent yield (g MLVSS/g COD)	0.3
Detention time (hr)	23
MLVSS (mg/l)	3500 - 4500
Aeration rate (l/min)	22
Digestion Tank	
Operating volume (l)	2496
Feed rate (l/day)	100
Detention time (days)	25

Both the aeration tank and the digester were aerated. In the activated sludge process aeration serves two purposes: 1) provides dissolved oxygen, and 2) provides mixing. Usually the mixing function governs the volume of air required by the system. This was not found to be the case in the calculations shown in Appendix B, and resulted in a calculated air flow requirement of 22 l/min (0.77 scfm).

The air for the system was supplied by a GAST compressor (Model No. 0322-P102G18D). This was an oil-less, graphite vane model and had a rated capacity of 70.8 l/min (2.5 scfm), free air.

The compressed air stream was fed through a pressure tank to a distribution manifold where it was split into digester, aeration tank and air lift pump streams.

The diffusers in the aeration and digestion tanks were of perforated 13 mm (1/2 inch) nylon tubing with 1.6 mm (1/16 inch) diameter holes spaced at 78/m (2/inch). The aeration tank was equipped with two sections of this tubing, one of 1.5 m (5 ft) and the other 0.2 m (8 inches). A 46 cm (18 inch) section was employed in the digestion tank.

A 1.25 cm (1/2 inch) air lift pump was used for mixed liquor extraction. The air supply from the compressor was controlled by a normally closed solenoid valve operated by a 60-second timer. Air was injected into the pump standpipe at a "T" below the surface of the aeration tank.

3.2.1.2 Floating tube settler

A floating tube settler, as shown in Figure 3.3 was used in the WatRek system to serve three functions:

1) Flow equalization;
2) Secondary clarification, and
3) Sludge recycle.

In a small installation, the rate of wastewater flow varies greatly over the course of a day. Total flows, however, fluctuate little from day to day. By using the aeration tank freeboard as flow equalization volume, the use of another, equalization basin was not required. Flow equalization also protected the secondary clarifier from increases in flow which would have impaired its efficiency.

A constant overflow rate was provided by designing the submergence of the unit to allow a specific depth over the four V-notch weirs. This depth could be varied by adding weights to the flotation tray.

Culp *et al.* (1968, 1969) and Hansen *et al.* (1969) reported that a significant increase in clarifier overflow rates could be achieved through the use of inclined tubes. Rates as high as 4.05 m/hr (2000 Igpd/ ft^2) were reported. Mendis (1976) reported that, when applied to mixed liquor separation in clarification

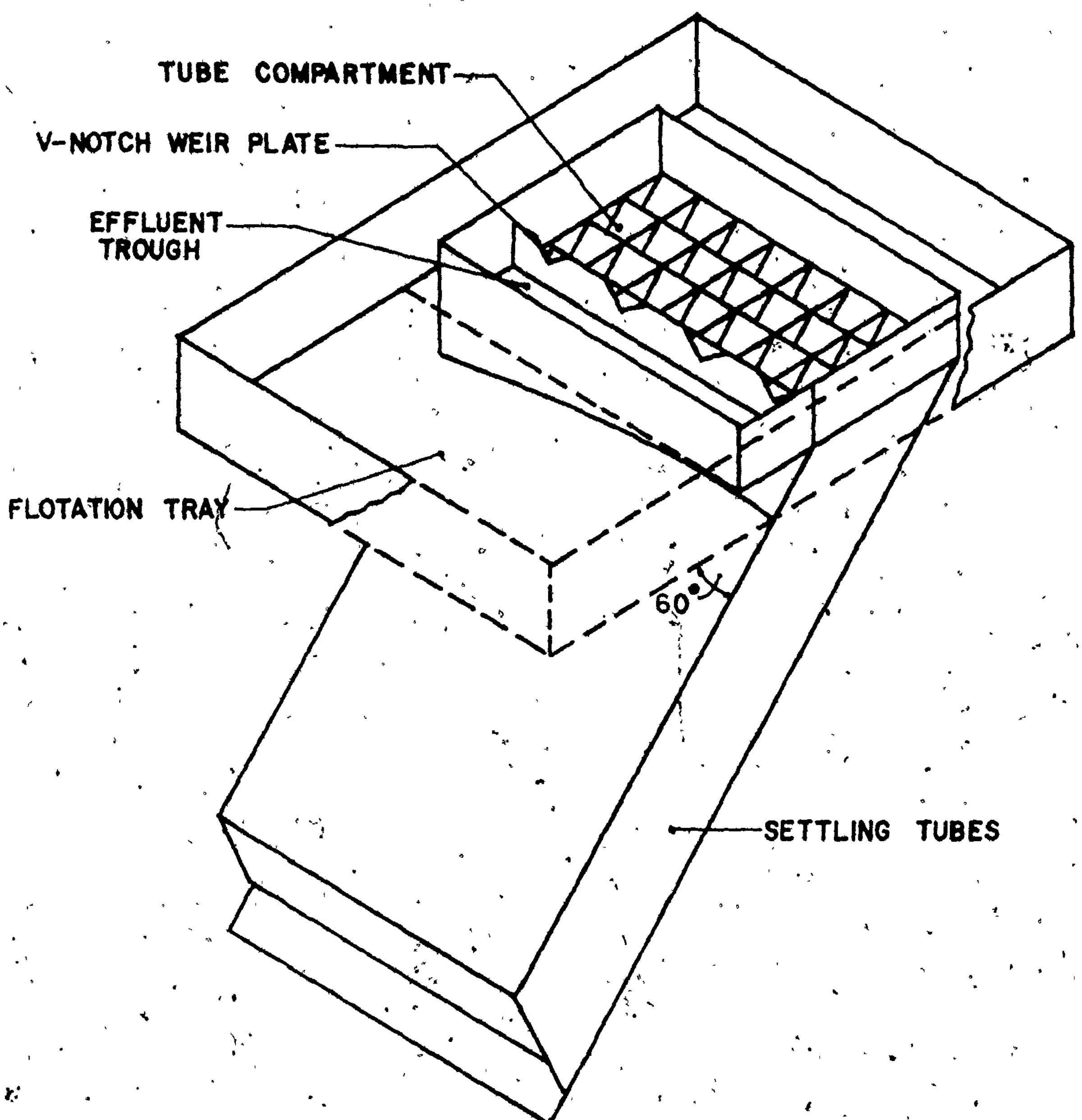


FIG. 3.3 FLOATING TUBE CLARIFIER

limited situations, an overflow rate of 0.33 m/hr (163 Igpd/ft^2) based on tube surface area, was possible.

The WatRek floating clarifier was designed to operate in the upflow mode in order to take advantage of gravity flow to the carbon column. The tubes were square and fabricated of PVC sheeting. Clarified effluent overflowed the weirs into a trough. From this effluent trough, two 2.5 cm (1 inch) downcomers carried the effluent to the base of the carbon column. Details of the clarifier design are shown in Appendix B and summarized in Table 3-3.

Sludge recycle was provided as the sludge settled through the open bottom ends of the tubes into the aeration tank. This flow was aided by gravity and fluid motion around the tube ends. Placement of the clarifier in the tank was such that air bubbles would not enter the tubes.

3.2.1.3 <u>Tertiary treatment</u>

An upflow activated carbon column was employed as the tertiary treatment step in the WatRek system. This unit had the function of removing, through adsorption, organics remaining in the waste following biological treatment. The column also acted as a granular filter for removal of solids which had escaped the clarifier.

Table 3-3

Floating Tube Settler

Liquid surface area	929 cm^2
Overflow rate	21.02 m/hr
No. of tubes	36
Size of tubes	5.1 cm x 5.1 cm
Length of tubes	61 cm
Angle of tubes to horizontal	60^o
Tube settling area (see Appendix C)	1.13 m^2
Tube overflow rate	0.082 m/hr

During the startup phase of the WatRek unit, the biological treatment process would not be operating efficiently and effluent quality would be substandard. The carbon column would remove some of the waste constituents during this startup period while the sludge was being conditioned. Figure 3.4 serves to illustrate the expected performance of the overall system during startup.

In designing the carbon column, adsorption was considered to be the critical function and filtration a beneficial side effect not requiring detailed consideration. A carbon life of 6 months was chosen. Equations given by Benedek (1973) were used to design the unit and are shown in Appendix B. "Filtrasorb 400", a 10 x 40 mesh, activated carbon manufactured by Calgon Corp., was used as the medium in the carbon column. Table 3-4 gives the properties of this carbon.

Periodic backwashing of a carbon column is required to remove entrapped solids. In the WatRek system, backwash could be initiated either manually or automatically, by a timer. Effluent from the carbon column was used for backwash. The column backwash flow was upward and supplied by a pump situated in the backwash storage tank. Spent backwash water and accompanying solids were returned to the aeration tank.

3-5 a of

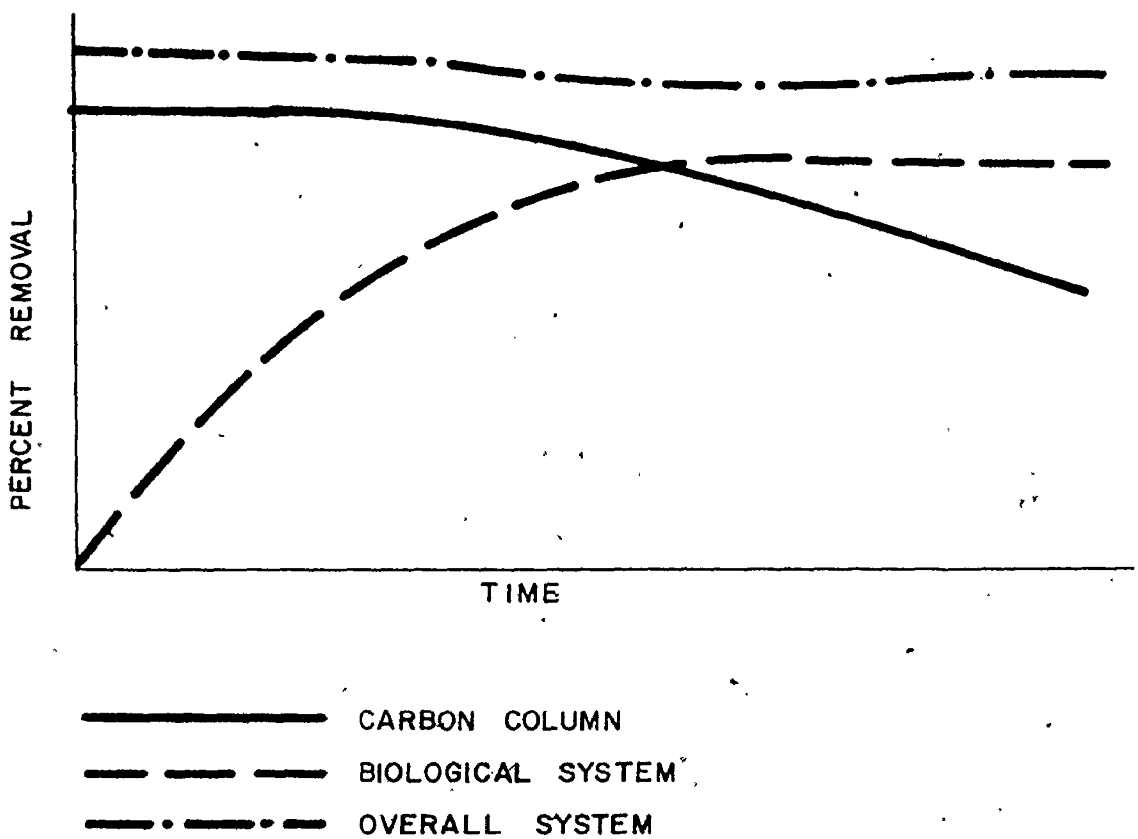


FIG. 3.4 THEORETICAL STARTUP CHARACTERISTICS

Table 3-4

Filtrasorb 400

Raw material	Bituminous coal
Surface area (m^2/g)	900 - 1050
Backwashed and drained density (lb/ft^3)	26
Average particle size (mm)	0.8 - 0.9
Uniformity coefficient	1.0
D_{10} (mm)	0.7
Approximate loading for municipal waste $\frac{g\ COD}{g\ carbon}$	0.8

Table 3-5

Activated Carbon Column

Carbon	Filtrasorb 400
Mesh size	10 x 40
Loading $\frac{g\ COD}{g\ carbon}$	0.6
Bed volume, liters	132
Surface area (sq cm)	1265
Mass of carbon bed (kg)	55.4
Hydraulic loading rate (m/hr)	0.73
Contact time (min)	85
Carbon life (months)	6
Backwash rate (l/min/m^2)	390 - 490
Backwash duration (min)	10 - 15
Backwash volume (l)	625
Backwash pump rate (l/min)	189

3.3 Structural Design

The structural details of the tankage in the WatRek system are shown in Appendix N. From the drawings included there, the sizes and exact locations of the various members may be determined.

Basically, the structure was one of an exterior frame of angle steel, along the inside of which a skin of 1.91 cm (3/4 inch) sheeting grade plywood was bolted. Five such pieces (shown in the Appendix) were prefabricated in such a way as to be easily joined by 1.3 cm (1/2 inch) structural bolts at each corner. Each prefabricated unit consisted of the bracing system and the plywood skin, since these were not separated for transportation purposes.

The floor consisted of two modules of 1.91 cm (3/4 inch) plywood, underlain by 5 cm (2 inch) thick timber stringers. The floor was sloped at 6.3% downwards toward the control panel end. The tank connections were made water-tight by using bulkhead fittings with gaskets or "O" rings.

The activated carbon column was constructed of 1.91 cm (3/4 inch) plywood sheeting. Since it was to be immersed in the mixed liquor of the aeration tank, it was given a double coat of epoxy paint to protect it from biological attack.

3.3.1

Engineering machine shop at McMaster University. Standard techniques of construction were used.

The steel reinforcement frame was of welded construction, with the exception of the corner points which were bolted. This frame was then lined with the plywood skin which was bolted into place. Floor beams and panels were then cut and assembled to give the desired slope. Interior wall sections were then installed to form the backwash storage tank, and the aeration and digestion tanks. The required fitting holes were then cut to size.

Polyvinylchloride swimming pool liners were manufactured to order by Acorn Pools Ltd. These were suspended by sandwich boards along the top rims of the tanks. Fittings were then installed.

The carbon column was constructed of plywood with a double layer of epoxy paint to protect the wood. Joints were sealed with silicone sealants. A 50 mesh screen, supported on a 4 mesh screen was used to hold the carbon bed.

The floating tube settler was built of rigid PVC sheeting. The tubes were formed of 3.2 mm (1/8 inch) material while the flotation tray was of 6.4 mm (1/4 inch). All this was assembled using PVC pipe cement.

3.4

prototype are given in Appendix as approximately $1800. Labor costs were not available, but it is estimated that several hundred man-hours were required to build the prototype.

CHAPTER 4

SYSTEM EVALUATION METHODS

4.1 Objectives

Bromley (1977) has demonstrated that the processes included in the WatRek system can produce an effluent which satisfies the design criteria in this study. Therefore, the experimental programme was designed to isolate operational problem areas which existed in the WatRek prototype; as opposed to determining the quantitative and predictive design relationships of the system. Because of this, no attempt was made to duplicate the high concentrations of pollutants prevalent in the North. Rather, easily available municipal wastewater was utilized.

Experiments were carried out primarily on the aeration tank-clarifier-adsorption column system. The effect of variation in digestion tank operation was not investigated in detail.

4.2 Apparatus

The equipment used in the evaluation programme may be divided into two parts: 1) WatRek, and 2) auxiliary equipment. A detailed inventory of the WatRek is presented in Appendix F. Auxiliary equipment, as designated below, were needed to connect the WatRek unit to the wastewater source and the sampling equipment.

Raw sewage was fed to the system by a Robbins and Myers "Moyno" pump (Springfield, Ohio). The pump discharged through a 25 mm (1 inch) flexible hose, to the aeration tank. The pump was driven by a Sterling variable speed drive (3/4 hp, Type WPFF).

Daily composite samples were collected by Sigmamotor, automatic samplers. These were equipped with 60-second timers (Singer Industrial Time Corp). The samples were drawn through 3.2 mm (1/8 inch) diameter tubing.

4.3 Procedures

4.3.1 Startup

In an isolated installation a suitable seed of activated sludge may not be available. For this reason no seed was used in the startup of the WatRek unit. During the startup period, samples of the developing mixed liquor were collected daily and analyzed for solids content.

4.3.2 Operation

During the experimental period the unit was usually attended twice each day, in the morning and in the evening.

The morning procedures comprised the actual plant operation, while the evening visit was normally an inspection.

On both visits aeration and digestion tank liquid levels were recorded by reading scales located on the tank walls. The aeration tank depth data served to illustrate the hydraulic characteristics of the clarifier-adsorption column system.

Liquid flow rates were measured by the bucket and stopwatch method on each visit. Two measurements of each flow were made in order to verify the value. Raw sewage feed flow was measured at the influent point by breaking the line at a quick-fit coupling. The effluent flow rate was measured at the carbon column overflow to the backwash storage tank. Mixed liquor extraction and digestion tank return flow rates were also measured twice daily. The flow rates of air to the aeration tank and digestion tank were measured by means of pre-calibrated rotameters.

If the measured flow rates deviated slightly from the nominal values, they were readjusted. However, if the observed deviations were large and the cause found to have been the physical failure of some component, the run was stopped.

The dissolved oxygen concentration in the aeration tank, digestion tank and the carbon column was measured twice daily. The sludge volume index (SVI) of the

Along with these quantitative observations, a qualitative assessment of the system was carried out. This usually took the form of recording pertinent visual and olfactory observations.

4.3.2.1 Backwash

The carbon column was backwashed whenever the level in the aeration tank approached 1.16 m (3.8 ft). The normal operating level of the aeration tank was between 1 m and 1.1 m (3.25 and 3.5 ft). Since the unit would overflow at a level of 1.35 m (4.45 ft) the level of 1.16 m (3.8 ft) was chosen.

Because the hydraulic characteristics of the system were unknown at the outset, the backwash sequence was initiated manually. Before backwashing the column, valve no. 1 in Figure 4.1 was closed so that backwash flow would not be carried upward through the clarifier downcomer. The backwash pump was then turned on. Simultaneously, the normally open solenoid valve on the carbon column overflow closed, preventing solids-laden backwash water from entering the backwash storage tank. The backwash pump was turned off by a low level control in the aeration tank. After a short period, during which the fluidized solids in the column were allowed to settle, the solenoid was reset to the open position manually and the excess

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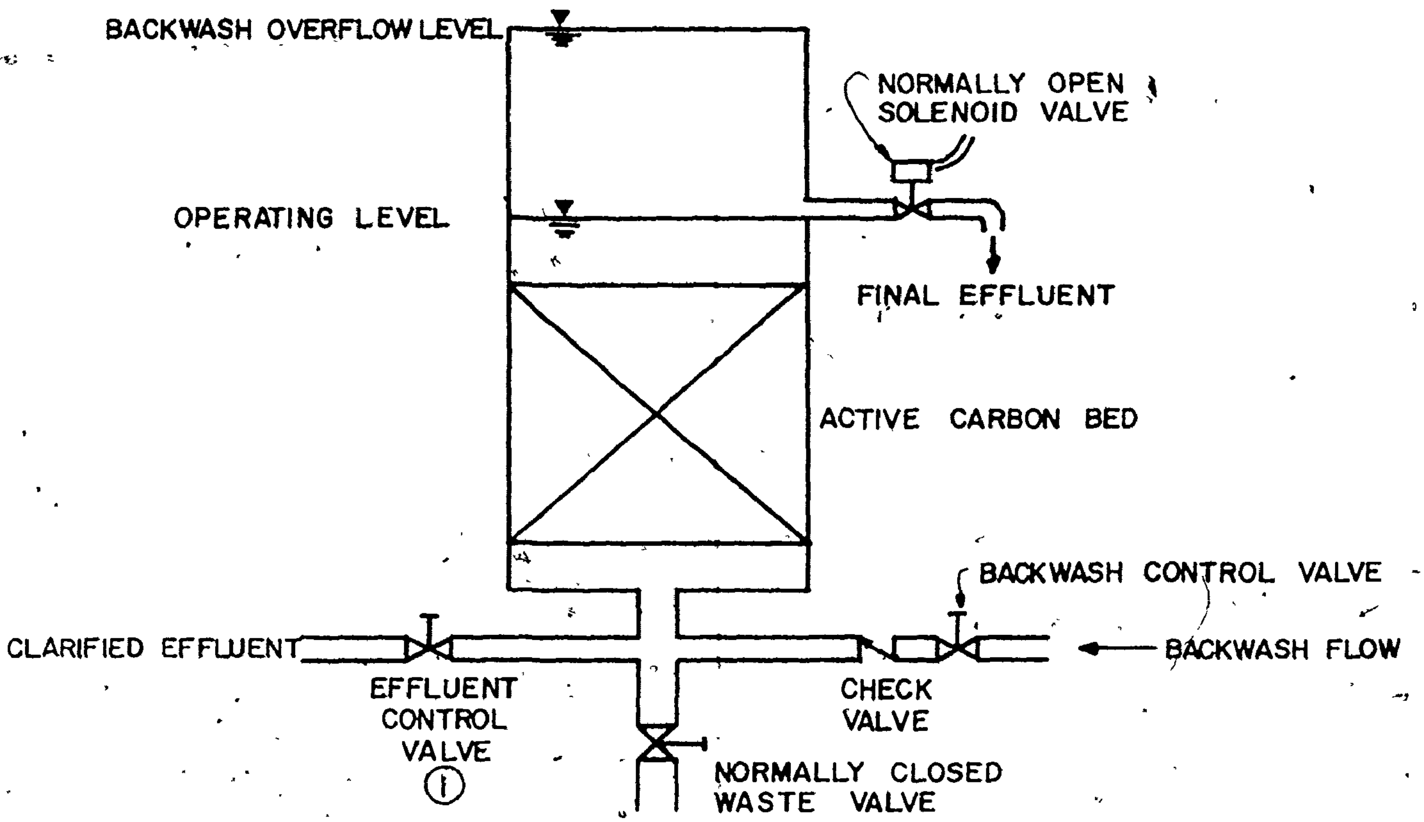


FIG. 4.1 ADSORPTION COLUMN PIPING DIAGRAM

Each time the column was backwashed the clarifier tubes were cleaned. This was done by pushing a 5 cm (2 inch) cleaning brush into the tubes from the top.

4.3.2.2 Digestion tank

Sludge wastage from the digestion tank was not necessary as the evaluation time lasted less than the design holding period of 6 months.

4.3.3 Sample Collection and Analysis

Twenty-four hour composite samples of the influent sewage, clarifier overflow and final effluent were collected daily by automatic samplers. A portion of the sample was collected for two minutes every half hour. The liquid was transferred, by the sampling pumps, to 4.5 l (1 I gal) plastic containers stored inside refrigerators which were kept at 4°C. In this way the degradation of the sample between collection and analysis was minimized.

These samples were analyzed for chemical oxygen demand and suspended solids. Samples were usually analyzed on the same day as they were collected. If this was not possible, the samples were chilled or frozen until analysis could be carried out. This was required on very few occasions.

routinely for suspended solids and periodically for total and volatile solids.

All analyses were carried out according to the methods outlined in *Standard Methods for the Analysis of Water and Wastewater* (1971). COD was measured by Method *220*. Suspended solids analyses were according to Method *537*, using a 0.45 μ membrane filter. Volatile and total solids were done by ashing and evaporative techniques as outlined in Method *538*.

4.4 Pre-Startup

Prior to startup, a programme of "debugging" was carried out. Investigations of the floating tube settler, aeration system and backwash pump were carried out.

The effect of the depth of weir submergence on the clarifier operation was investigated.

The oxygen transfer coefficient of the aeration system was measured using the method reported by Kayser (1969). Sodium sulfite in the presence of cobalt chloride catalyst was used as the oxygen scavenger. Dissolved oxygen concentrations were read using a Delta Scientific, automatic dissolved oxygen analyzer, Model 2010 and probe.

By backwashing the carbon column and measuring the extent of bed fluidization, the backwash pump was determined to be adequate.

4.5 Calibrations

The rotameters used to measure the air flows were calibrated using wet test meters and standard orifices.

The air lift pump used for mixed liquor extraction was calibrated using the bucket and stopwatch method. Flow rate was checked daily due to level fluctuation in the aeration tank.

CHAPTER 5

RESULTS AND DISCUSSION

5.1 Pre-Startup Activities

5.1.1 Structural

Structural testing consisted of filling the tanks with water and examining the system for structural problems as well as leaks. Two structural inadequacies were noted.

First, the wall separating the aeration tank and the digestion tank (the inter-tank wall) was observed to deflect by about 1.3 cm (0.5 inches) when the water level difference between the tanks was 15 cm (6 inches). Since, during operation, such level differentials would be prevalent, reinforcement was required. The reinforcement was provided by fitting a pair of adjustable jacks into the digestion tank, to bear between the exterior wall and that facing the inter-tank wall.

In conjunction with this modification, a change in the proposed operation scheme was made. It had originally been intended to air-lift mixed liquor into the digester and allow it to overflow a weir, back to the aeration tank. With the jacks in place, however, the level in the digestion tank had to be below the level in the aeration tank. Thus a peristaltic pump was used to return the digester liquid at the same rate as the air-lift and the digestion tank level would be held at a constant depth of 1.07 cm (3.5 ft). This reduced the residence time in the digestion tank from 25 to 17 days, at a mixed liquor extraction rate (MLX) of 100 l/day (22 Igpd).

The second structural defect noted was insufficient strength in the exterior wall opposite the control center, as viewed in Figure 3.2. When the system was water tested, a deflection of approximately 2 cm (0.8 inches) was observed at the base of the wall. The deflection was alleviated by the addition of two additional structural members as noted in Appendix G.

5.1.2 Hydraulic Considerations

5.1.2.1 Sealing

Persistent leaks were found at the fittings around the tubes entering the carbon column. This was attributed to the difficulty in securing the carbon column into the corner of the aeration tank. These leaks were alleviated by removing the carbon column to a more central position in the aeration tank, away from the walls. It was then possible to further tighten the fittings and thereby stem the leak.

Another, more persistent leak was found at the fitting in the wall common to the aeration and digestion tanks. The liner was eventually found to have been folded underneath the shoulder of the fitting. Inserts were added to the wall in order that the liner material could be stretched to a snug fit, thereby eliminating the leak.

5.1.2.2 Floating clarifier

During hydraulic testing, the clarifier effluent trough was observed to flood. The additional weight of the supernatant in the trough caused the clarifier to sink. Initially this resulted in an increase in head over the weirs and a concomitant increase in flow. As more water flowed over to the effluent tray, this process continued until equilibrium was reached and the water level in the effluent trough reached that in the tube compartment (Figure 3.3).

The downstream headloss analysis performed in Appendix E indicates that 0.15 m (5.7 inches) of head would be required for the tray to be self-draining. Since this head was not available in the system, the flow equalization capacity of the unit was negated. This condition forced the elimination of the flow equalization facet of the study. The revised study schedule is shown in Table 5-2.

A possible method of alleviating the flooding problem would be to install a low limit on the clarifitravel. This stop would of necessity be above the level of the carbon column overflow. In this way, at zero flow conditions the effluent trough would be dry and the static water level situated somewhere in the effluent downcomer.

5.1.3 Aeration

As noted in Appendix B, an aeration mass transfer coefficient of 1.825 hr^{-1} was required in the aeration tank. Prior to startup, measurements of the value of k_la were carried out using tap water.

As shown in Table 5-1, an unacceptably low value was found for the very course bubble diffusers. Following a modification in the size and spacing of the diffuser orifices, the test was repeated. Results improved, but remained below the desired value.

It was observed during both experiments that the bubble pattern in the aeration tank was uneven. A majority of the bubbles were being released along the central portion of the aerator tube, and very few bubbles evolved at either end of the tube. This was attributed to insufficient pressure drop through the diffuser orifices to provide even flow throughout the length of the aerator tube. In order to correct this problem an auxiliary source of air was acquired from the W.T.C. facilities. This source had a pressure of 6.3 kg/sq cm

Table 5-1

Determination of Mass Transfer Coefficient and Efficiency in the Aeration Tank

Diffusers	Air source	Air flow rate (scfm)	$k_1 a$ (hr^{-1})	$\frac{lb\ O_2}{HP\text{-}hr}$
5 ft 1/2" tube with 1/4" holes (1 inch)	WatRek	1.7	1.02	0.11
5' 8" of 1/2" tube with 2 x 1/16 holes/ inch	WatRek	1.25	1.25	0.21
5' 8" of 1/2" tube with 2 x 1/16 holes/ inch	W C Auxiliary air	5	5.47	0.23

(90 psi). The system compressor continued to supply air to the digestion tank.

As shown in Table 5-1, a satisfactory value of k_1a was obtained with the new air source. An even pattern of bubble release was also noted along the length of the aerator tube. These two factors led to the acceptance of the test as satisfactory.

Calculation of the efficiency of air transfer is shown in Appendix H and also shown in Table 5-1. The calculated values of pounds of oxygen transferred per horsepower hour show that the system falls somewhat below normally accepted levels of 0.5 to 2 lb O_2/HP hr.

5.2 Startup

5.2.1 Procedure

The system was started up by first filling it with tap water and then starting the wastewater feed. Seed organisms (sludge) were not used, as suitable sources of seed would not be available during eventual system startups in isolated locations. Observations were made on the length of time required to reach a satisfactory condition for the biological components of the systems.

The startup feed rate was 2275 l/day (500 Igpd). The aeration rate varied over the three startup attempts. These are discussed in subsequent sections. During startup mixed liquor extraction was not practiced.

Analyses performed during the startup period were influent and clarifier effluent COD and SS. Samples were collected as daily composites. Also, daily grab samples of the mixed liquor were collected for suspended solids determination.

5.2.2 Initial Startup Attempts

The first two startup attempts failed due to plugging in the clarifier-carbon column system. Flow was so restricted by the accumulation of solids in the system that an unacceptable rise in the depth of the aeration tank occurred. Both these attempts were carried out under aeration rates of 2.4 l/sec (5 scfm).

Following these two failures, the 1.25 cm (1/2 inch) clarifier overflow downcomer was replaced by 2 x 2.5 cm (1 inch) fixtures meeting in a "Y" connection to form a single 2.5 cm (1 inch) line. This improved flow through the downcomer.

Following each unsuccessful startup attempt, both the digestion and aeration tanks were drained.

5.2.3 Startup No. 3

The third attempt at startup proved to be successful. Certain changes were made in the startup procedure in order to facilitate successful operation.

It had been noted during the previously unsuccessful attempts that the clarifier was subjected to turbulence due to the mixing in the aeration tank. For this third startup the aeration rate was reduced from 2.4 l/sec (5 scfm) to 0.9 l/sec (2 scfm). This produced a noticeable reduction in the agitation of the clarifier. Also, during startup no. 3, the carbon column was not initially included in the flowsheet. The clarifier effluent was run to waste through a gate valve. This prevented the clogging of the carbon column which in the initial two startups showed signs of plugging.

The results of the startup are illustrated in Figures 5.1 through 5.4. Figures 5.1 and 5.2 show the solids history for the startup. The mixed liquor suspended solids concentration increased steadily over a period of 27 days to a level of approximately 2800 mg/l. This represented a total solids mass of approximately 5000 gm (11 lb) in the aeration tank.

Figure 5.2 shows that total COD removal climbed rapidly to 85%, after 13 days operation. This may be attributed, in part, to the fact that the SS removal efficiency reached 90% after only 12 days of operation.

It appears possible, due to the high SS removals, that the carbon column could have been put on line after 17 or 18 days of operation. As it was, however, the carbon column was introduced on day 27. The aeration tank depth record (Figure 5.3) indicates that no serious plugging of the carbon column occurred. Increases in

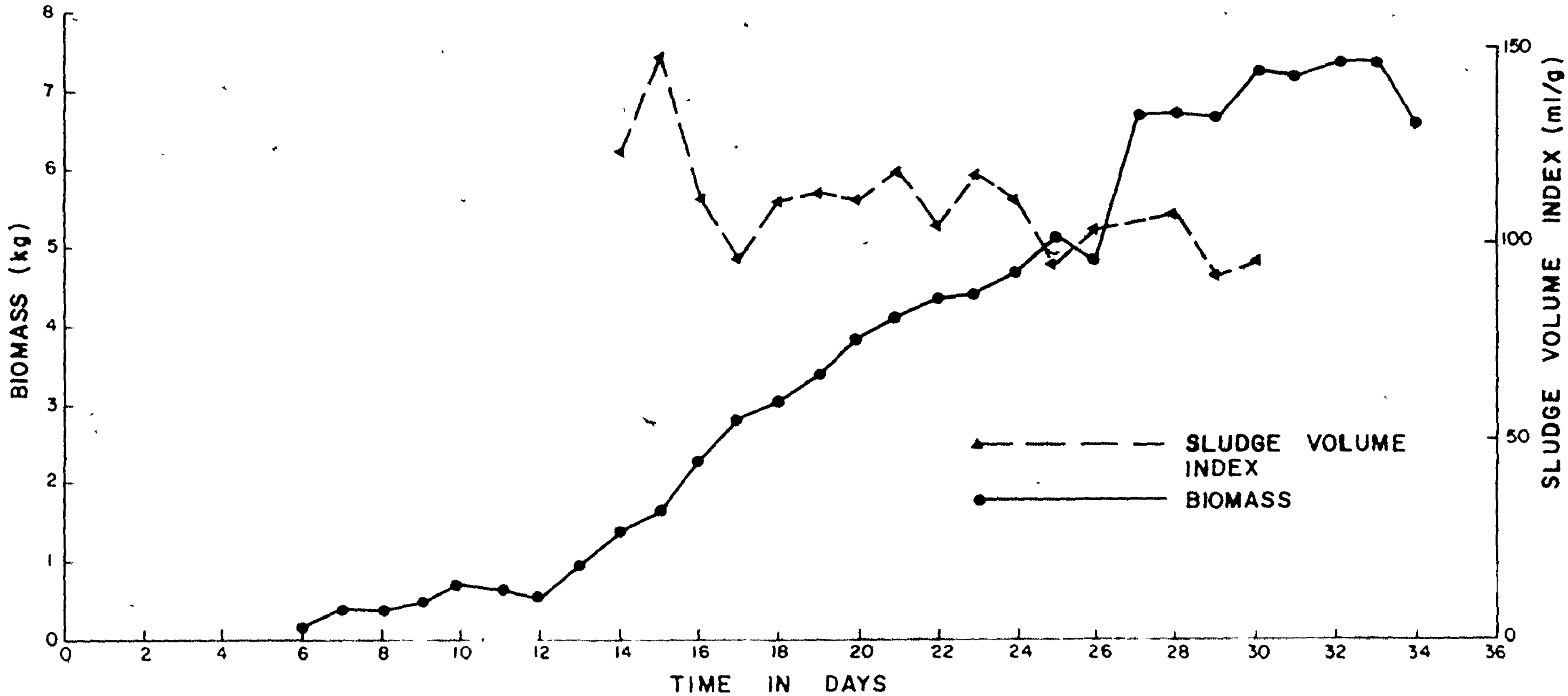


FIG. 5.1 TOTAL BIOMASS AND SVI DURING STARTUP

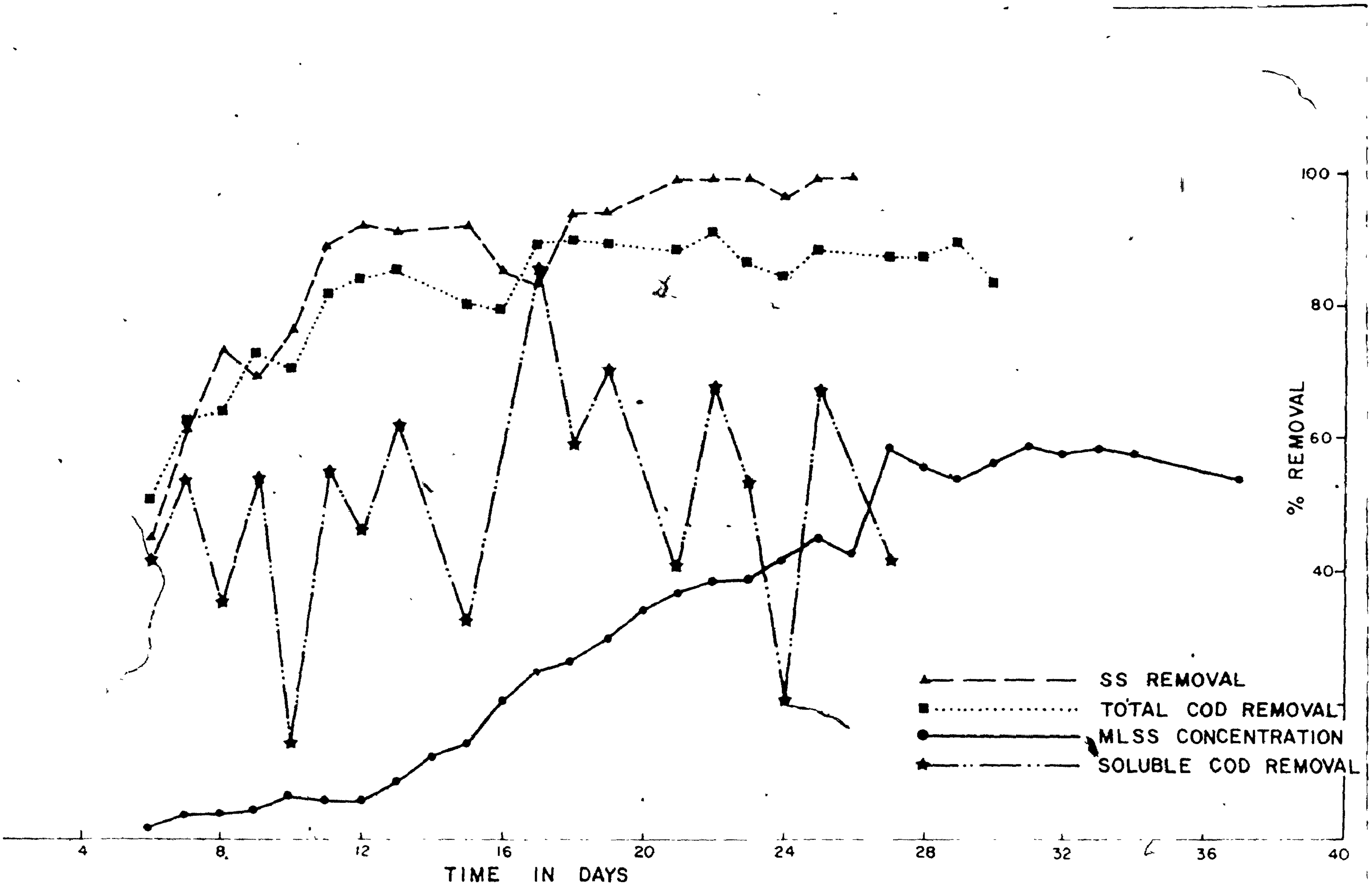


FIG. 5.2 MLSS AND REMOVAL EFFICIENCIES DURING STARTUP

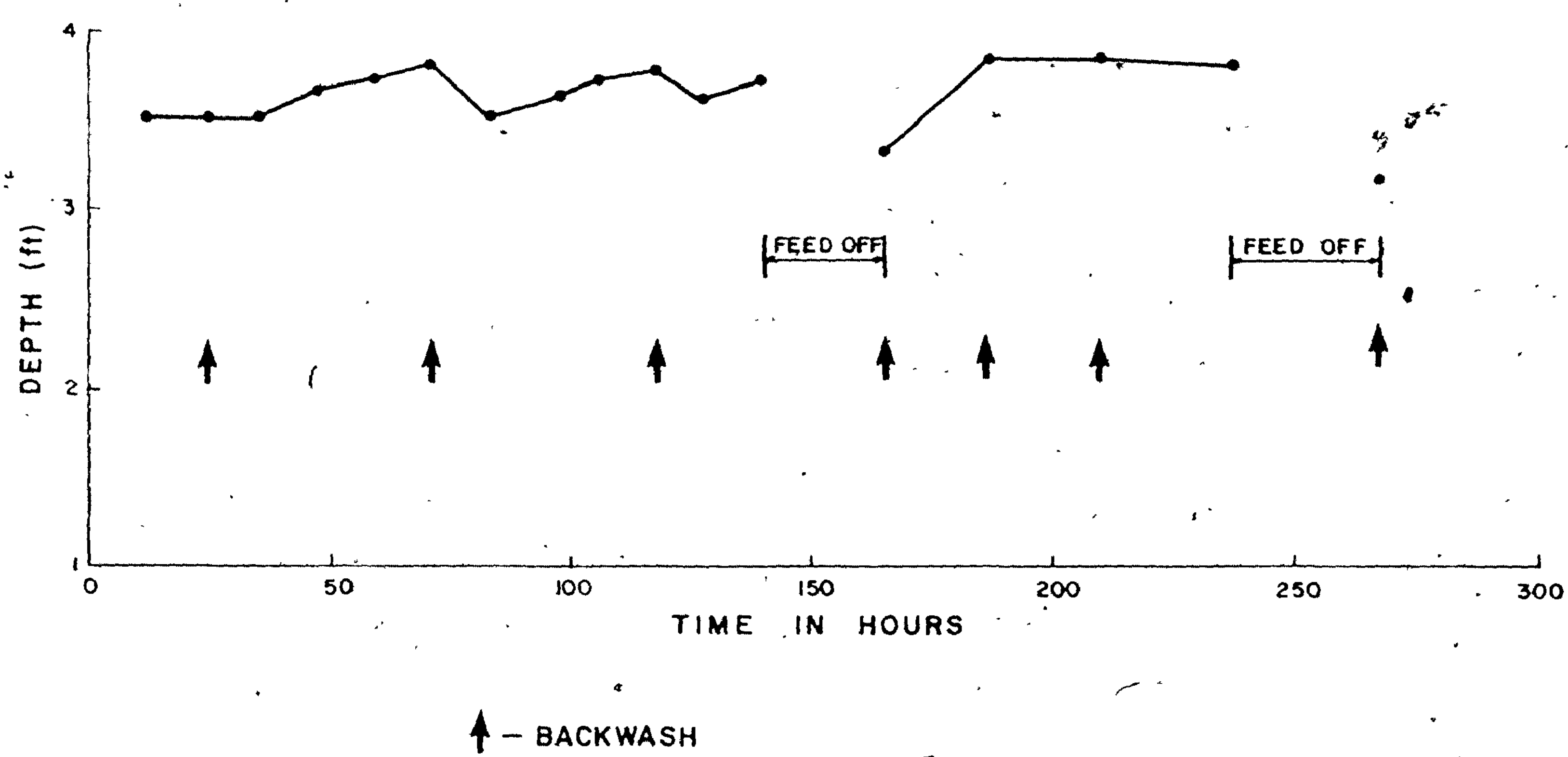


FIG. 5.3 AERATION TANK DEPTH RECORD

depth in the aeration tank were remedied by simple backwashing of the carbon column.

The dissolved oxygen (D.O.) history (Figure [illegible].4) can be taken as a further indication that experimental runs could have been initiated at approximately day 1[illegible]. The decreasing D.O. indicates an increase in biological activity within the aeration tank.

5.3 Evaluation Programme

The WatRek prototype was evaluated at the Wastewater Technology Centre of the Canada Centre for Inland Waters in Burlington, Ontario. Raw municipal wastewater was supplied to this facility from the Burlington Skyway Water Pollution Control Plant. Prior to entering the apparatus, this wastewater was degritted and passed through a primary settler.

A programme of seven runs was drawn up, in which volumetric feed rate (Q), air flow rate to the aeration tank and mixed liquor extraction (MLX) rate were varied. Table 5-2 is a summary of the experimental runs.

The length of time over which each run would extend was not firmly set. Such open ended runs were considered necessary due to the uncertainty in the performance and operating characteristics of the prototype. The criteria chosen for terminating the runs were based on qualitative analysis of system operation. If the

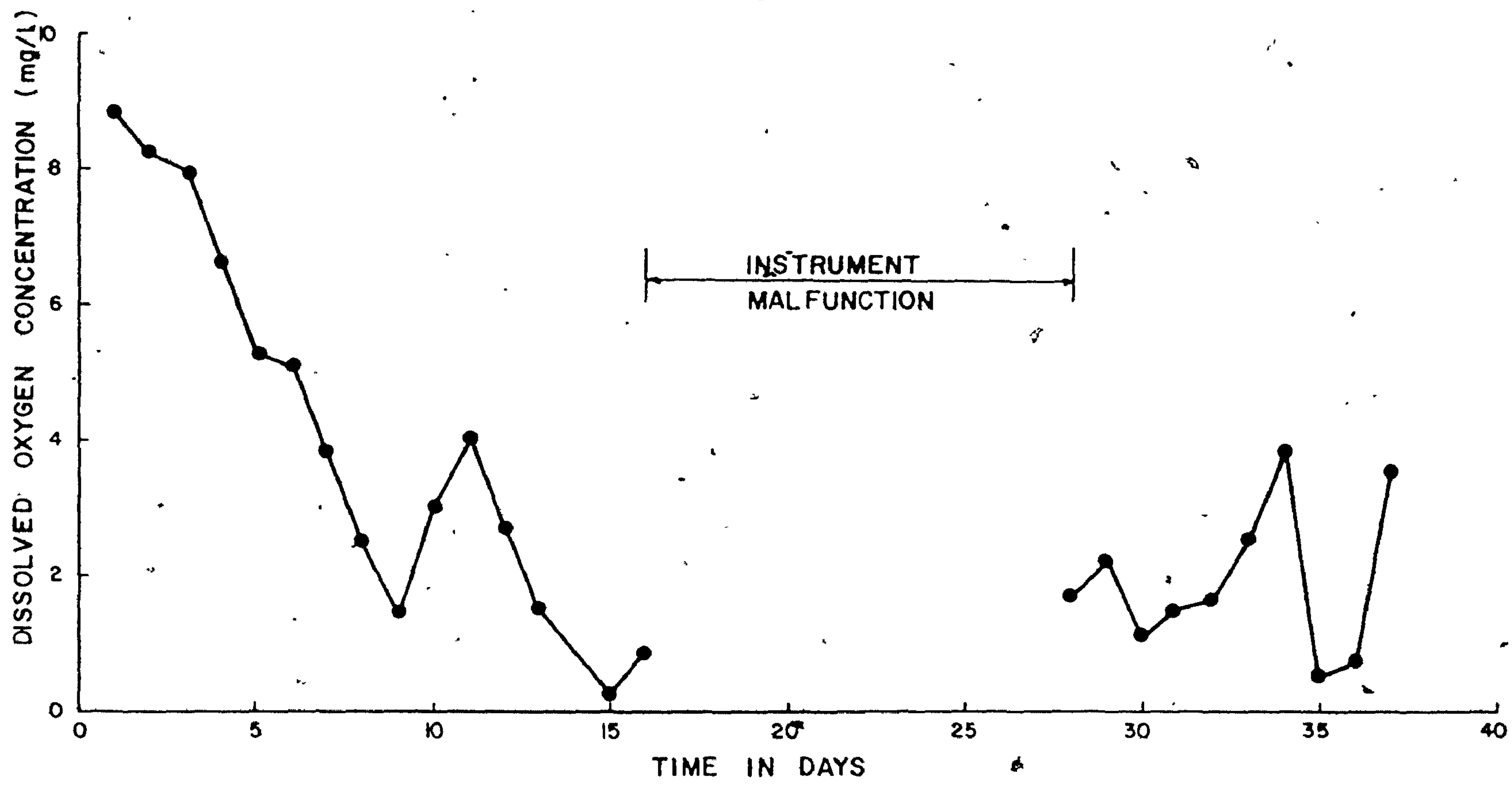


FIG. 5.4 AERATION TANK D.O. HISTORY DURING STARTUP 3

Table 5-2

Summary of Experimental Programme

Run #	Wastewater feed rate, Q l/day (scfm)	Aeration rate l/s as S.C. (scfm)	Mixed liquor extraction rate (MLX) l/day (Igpd)
1	2273 (500)	1.9 (4)	150 (33)
2	2273 (500)	0.9 (2)	50 (11)
3	2273 (500)	1.4 (3)	150 (33)
4	4545 (1000)	1.4 (3)	150 (33)
5	4545 (1000)	0.9 (2)	150 (33)
6	3410 (750)	0.9 (2)	50 (11)
7	4545 (1000)	0.9 (2)	50 (11)

system operated satisfactorily the run would be continued for at least six days.

If the run was not considered successful, it was continued until failure of some form was observed. Failure was subdivided into three categories: process, hydraulic, and physical. Process failure was defined as the system not meeting the design criteria set out in Section 3.1.1. If these criteria were not met with minor adjustments in equipment or operational procedures, the run was considered a failure but continued.

Hydraulic failure was defined as conditions of a hydraulic nature, which so impaired the system that safe and successful operation was not possible. Included in this category were continual clogging, excessive aeration tank levels and inadequate level control.

Physical failure referred to any equipment failure which forced termination of the run. In the event of physical failure the run was repeated. In the event of process or hydraulic failure, however, the run was considered useful for isolating sources of potential failures.

5.4 Process Performance

5.4.1 COD Removal

The removal of total COD across the treatment system averaged 93% over runs 2 through 7. Run 1 has been excluded from this discussion because, as noted in Sections 5.4.2 and 5.6.1, the air flow rate had an extraordinary effect on the solids removal efficiency of the clarifier.

Figure 5.5 shows the cumulative totals of COD fed *vs* COD removed as well as the removal efficiencies through the unit. From this figure it may be seen that all runs exceeded 90% total COD removal with Run 3 the highest at 95%.

Figure 5.6 shows that the majority of the total COD, 85% on average, was removed by the aeration tank. Best removals were achieved in Run 2, at 88%. These values are within the range of treatment which may be expected of an extended aeration system.

As shown by Figure 5.7, the average removal of total COD in the carbon column was 53%. However, since the removal in the carbon column is dependent upon the driving force and therefore the influent COD concentration, the mass of total COD removed is a more important parameter. A total of 3120 g (6.9 lb) of COD was removed in the carbon column. This value is small compared to the 16,300 g (35.9 lb) removed by the aeration tank, however, and it indicates that the carbon column acts as an effluent polishing unit rather than a major COD removal unit.

This is further indicated by Figure 5.8. It is interesting to note in Figure 5.8 that despite the high effluent total COD escaping the aeration tank in Run 1,

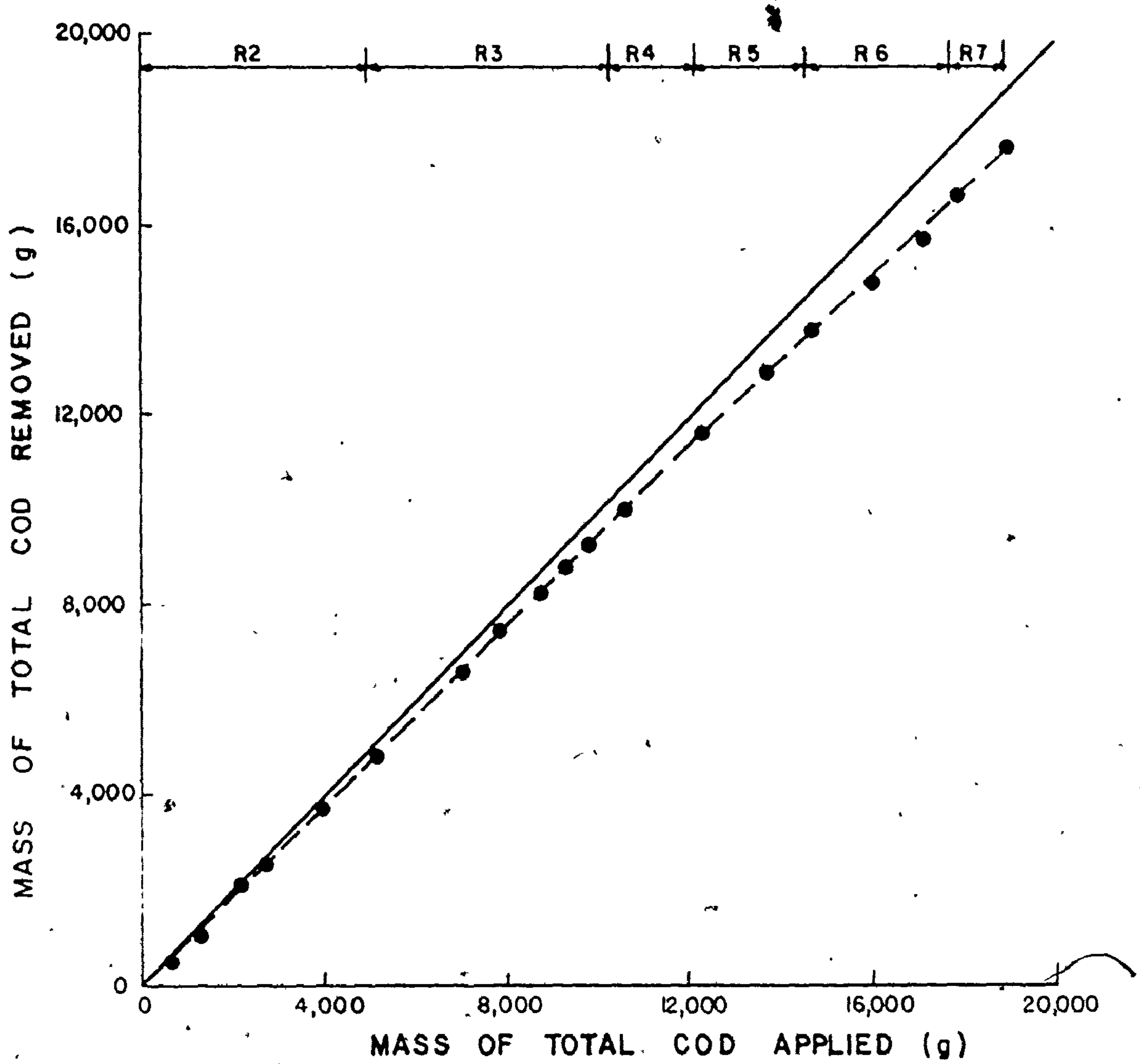


FIG. 5.5 COD - MASS REMOVED vs. MASS APPLIED (OVERALL)

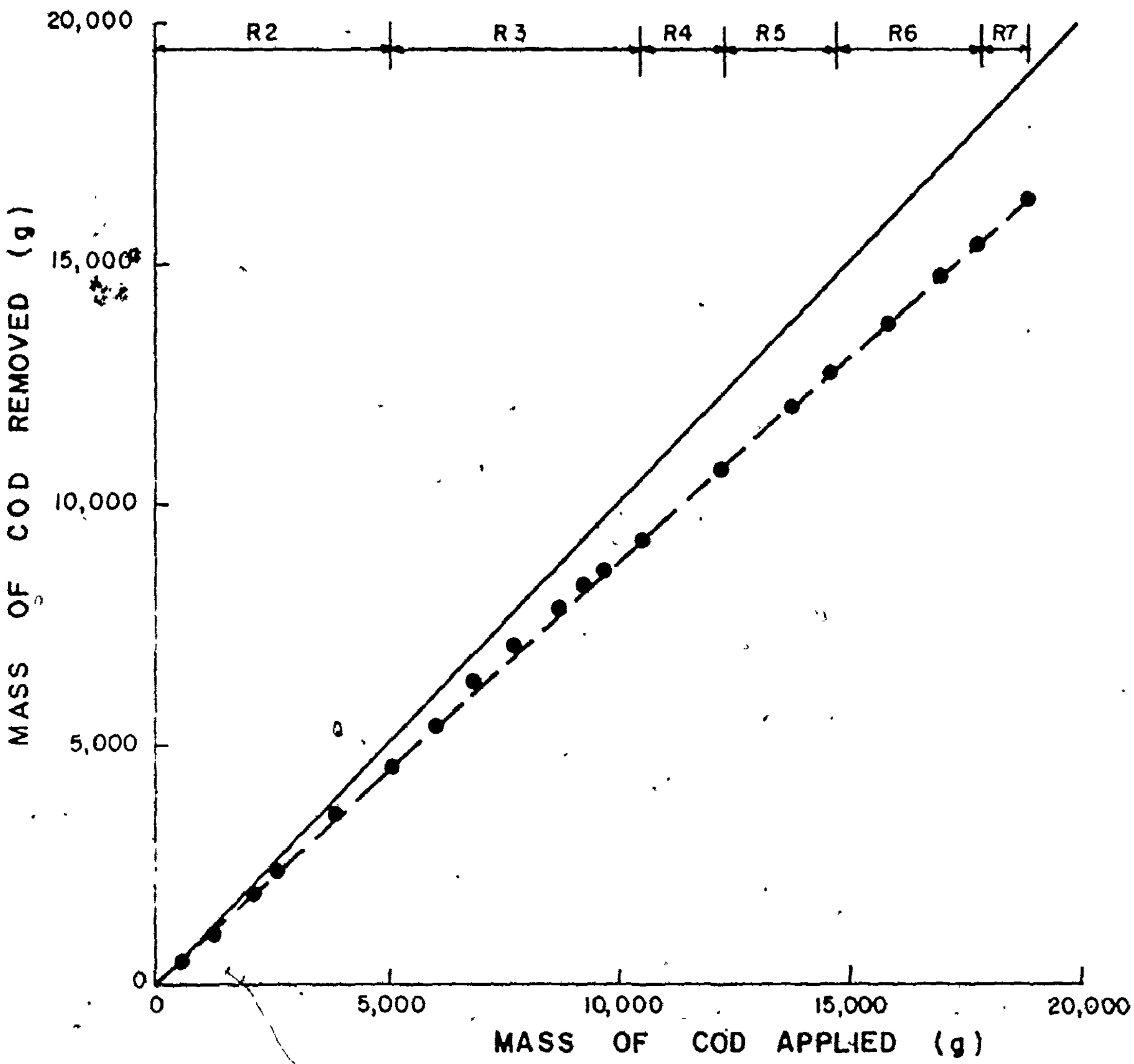


FIG. 5.6 COD- MA ... VS MASS

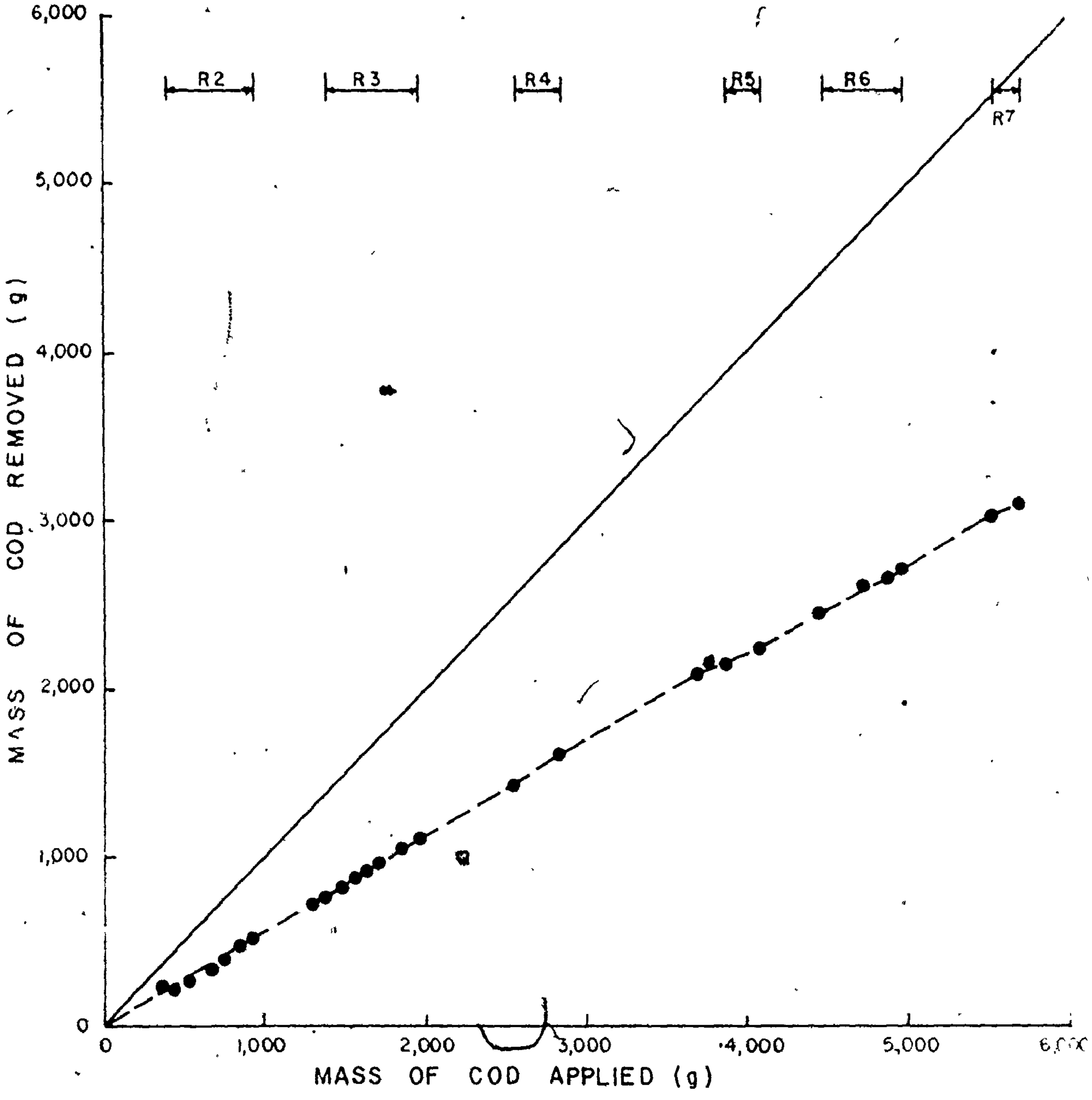
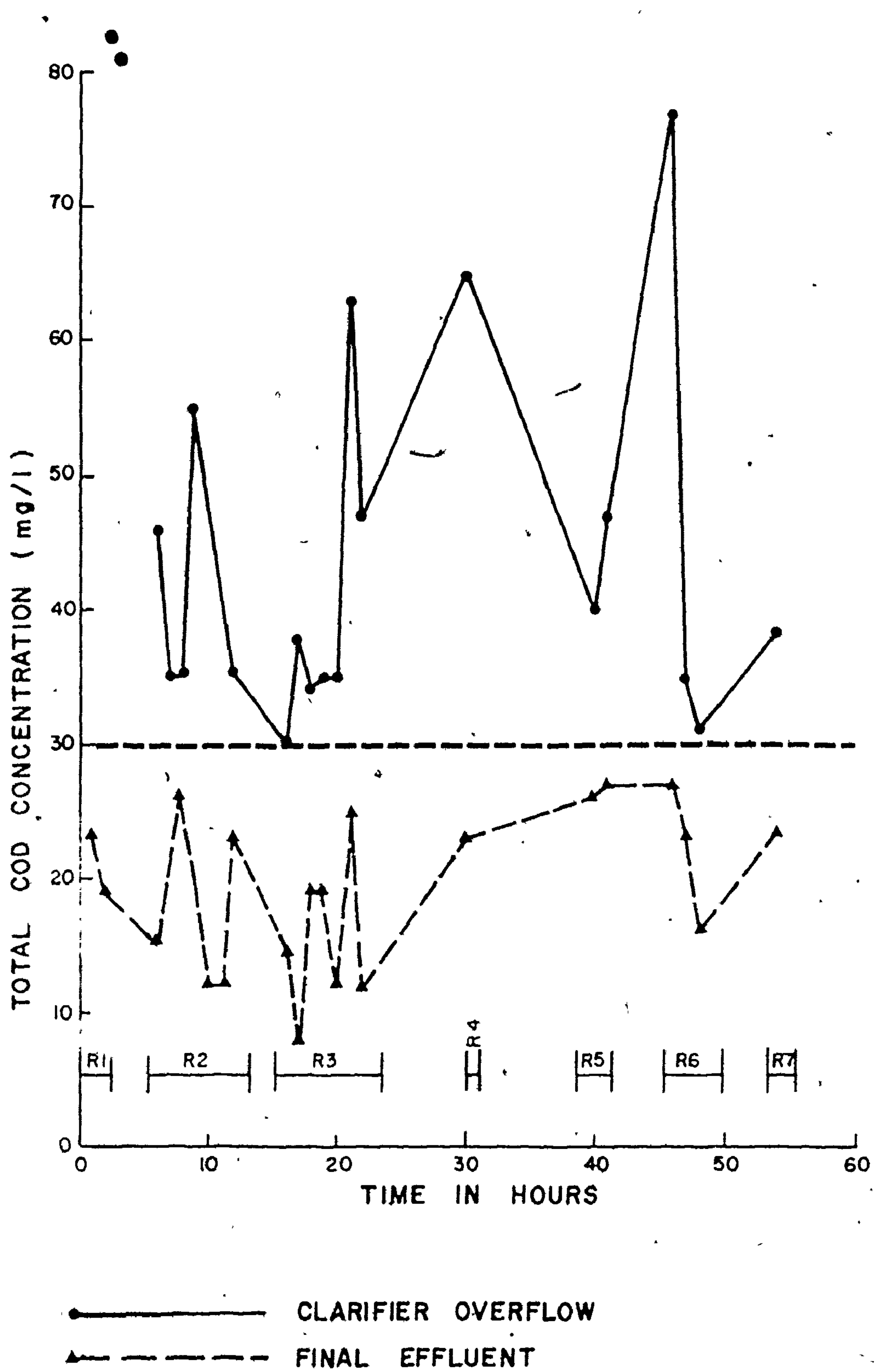


FIG. 5.7 COD - MASS REMOVED vs. MASS APPLIED (CARBON COLUMN)


TOTAL COD CONCENTRATION (mg/l)
TIME IN HOURS
0
10
20
30
40
50
60
70
80
R1
R2
R3
R4
R5
R6
R7
CLARIFIER OVERFLOW
FINAL EFFLUENT

the carbon column effluent was within the design criteria of 30 mg/l. In fact, the carbon column was performing both as a filter and an adsorber in Run 1 since, in this run, soluble COD removals in the aeration tank were quite low. This illustrates the capacity of the carbon column to moderate, and compensate for, upsets in the biological system.

Figure 5.8 also shows that the design criteria of 30 mg/l total COD in the effluent was satisfied on all days. However, during the high flow runs (4,5,6 and 7), the effluent total COD showed a tendency to approach the design limit.

Assuming that 40% of the COD removal in the carbon column was biological, then 1872 g of COD was adsorbed on the carbon surface. With a surface area of 1000 m^2/g and a total carbon mass of 55.4 kg the overall loading at the end of the experimental period was 3.4×10^{-8} of COD/cm^2 of carbon surface. The normally accepted total organic carbon (TOC) loading on activated carbon is 10^{-9} g TOC/cm^2 carbon for PCT sewage. Since the COD is normally greater than TOC it appears that the carbon was reaching the limit of its adsorptive capacity, thereby producing an effluent of diminishing quality. Further removals in the carbon column would be accomplished by biological mechanisms.

5.4.2 Suspended Solids Removal

An important function of the WatRek unit was the removal of suspended solids (SS). The purpose of this function was three-fold:

1) Direct reduction of a commonly monitored contaminant (SS);
2) Indirect reduction of the total COD in the effluent, and
3) Improvement in the clarity and aesthetics of the effluent to make it more amenable to recycle.

It may be seen from Figure 5.9 that removal efficiencies during all seven runs were high. An overall average of 97% removal was achieved over the entire experimental programme. The lowest efficiency was noted during Run 5, in which, a 95% removal rate was obtained. During this run, as shown in Figure 5.10, the effluent suspended solids level was approximately 12 mg/l. This was still well within the desired value of 30 mg/l SS in the effluent.

Figure 5.10 shows that the level of suspended solids in the effluent remained at or below 4 mg/l on all but four occasions, namely, during Runs 1, 4 and 5. In any event, the design level was met, at all times, by the system.

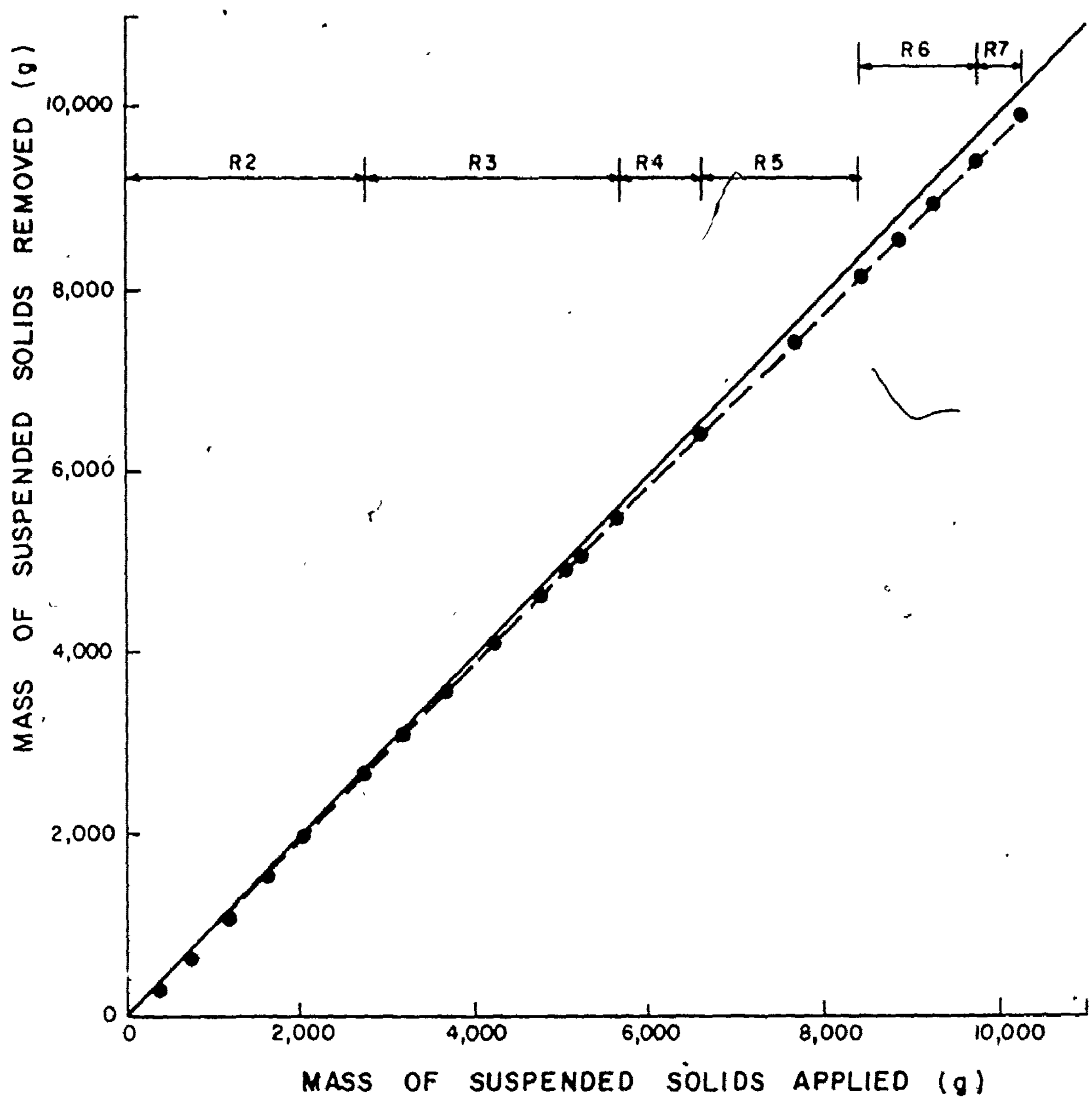


FIG. 5.9 SS - MASS [illegible] vs. MASS [illegible])

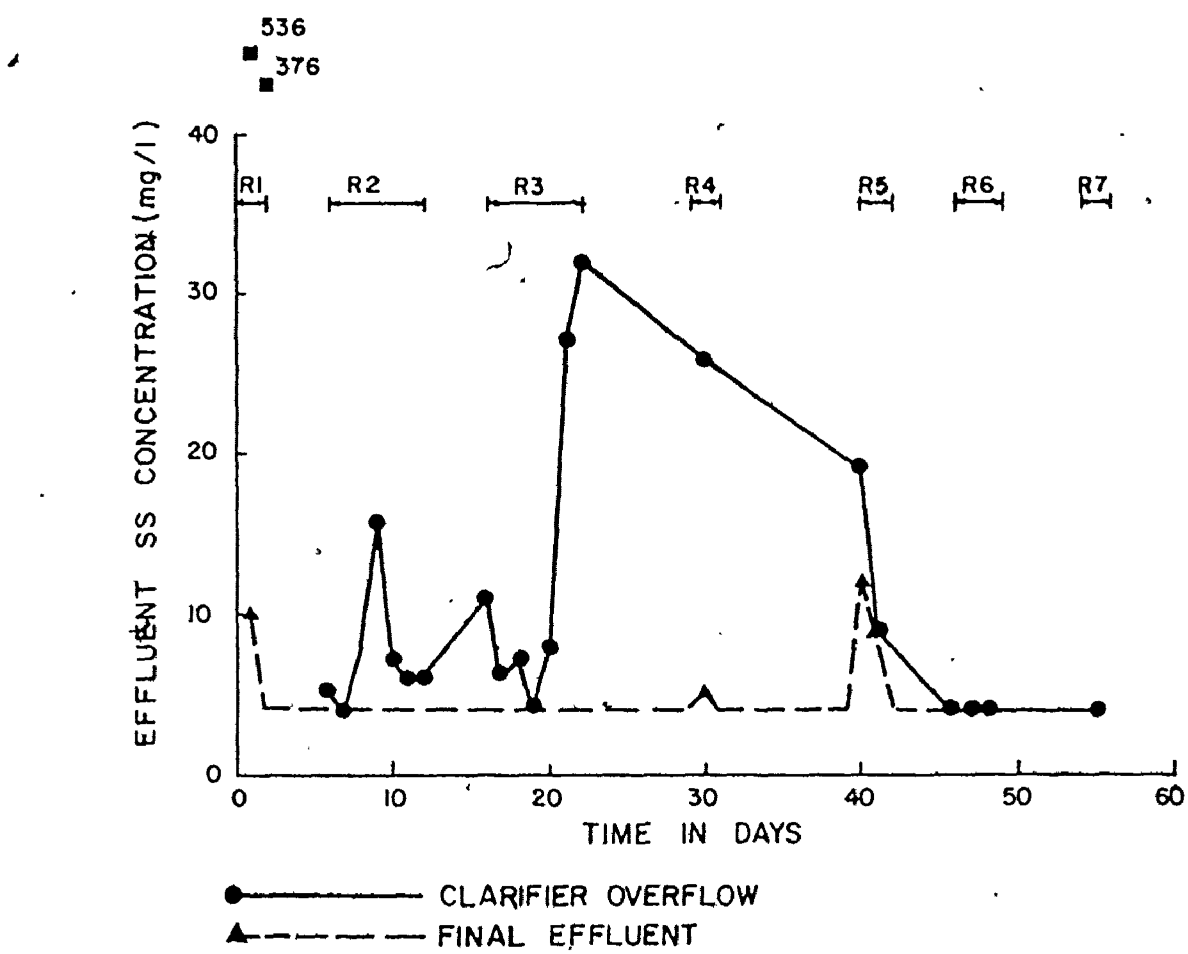


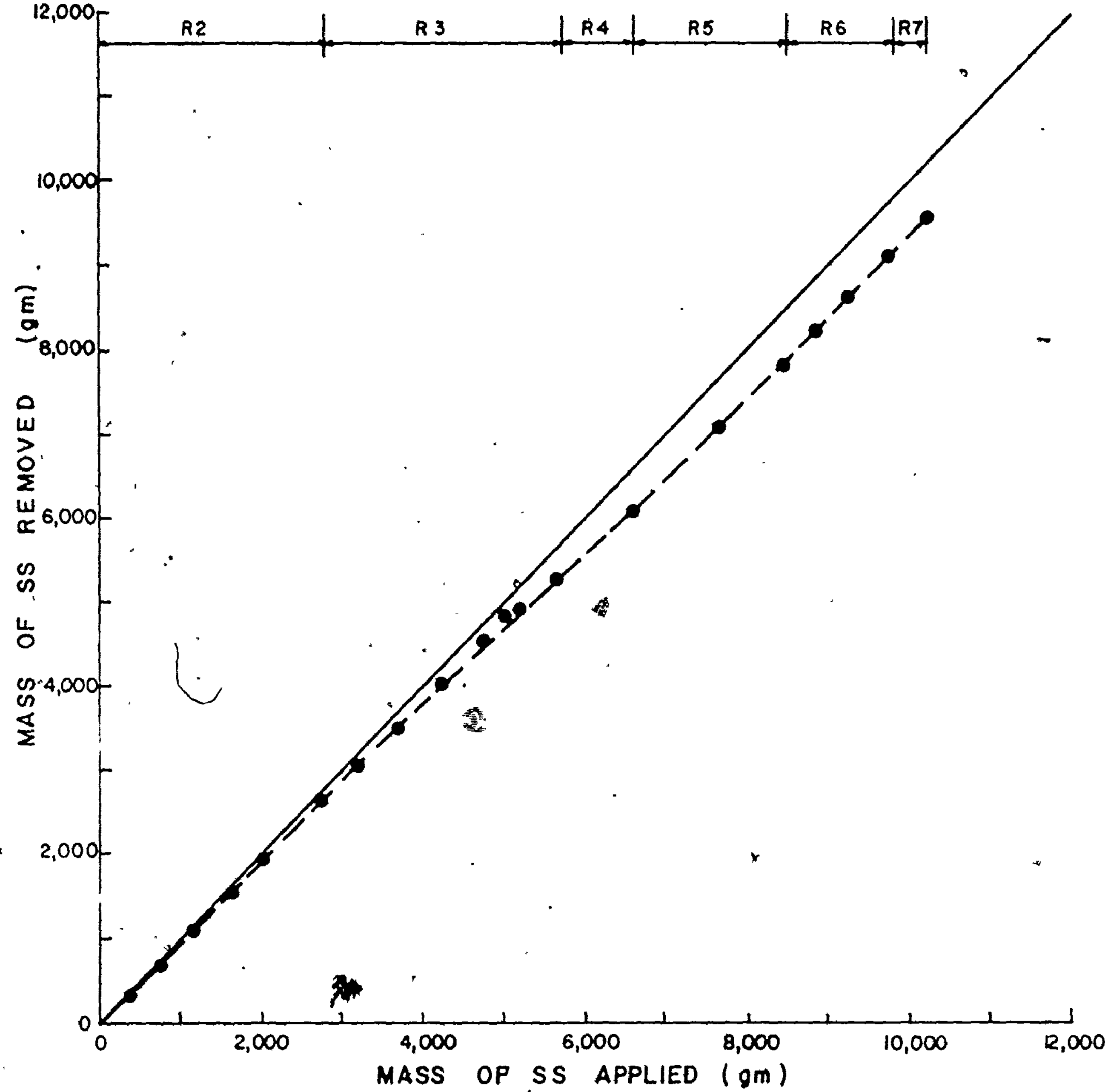
FIG. 5.10 SS – EFFLUENT CONCENTRATION

Suspended solids removal efficiencies in the clarifier are shown in Figure 5.11. The average removal in the clarifier was 94%. Lower values were obtained in Runs 3 and 4.

The low removal rate in Run 3 was due to low influent solids and substrate levels which may have caused bulking of the sludge and corresponding losses. During this run, as shown in Figure 5.10, the clarifier overflow solids level reached 32 mg/l, its highest level. Also, from Figure 5.10 it is interesting to note that the solids level in the final effluent remained at a low concentration. This demonstrates the effectiveness of the carbon filter in offsetting upsets in the aeration tank. The compensating effect of the carbon column was also shown during Run 1, when large amounts of solids escaped the clarifier but the final effluent was within design limits.

In Run 5, the compensating effect of the carbon was not as prevalent. As discussed later, there may have been some solids breakthrough in the carbon column at that time. In fact, on the second day of the run, the solids concentration entering the carbon column was less than that leaving the column.

The efficiency of the clarifier for solids removal may have been the result of its upflow operating mode. When operated in upflow a blanket of solids was maintained



R2
R3
R4
R5
R6
R7
12,000
10,000
8,000
6,000
4,000
2,000
0
MASS OF SS REMOVED (gm)
0
2,000
4,000
6,000
8,000
10,000
12,000
MASS OF SS APPLIED (gm)

within the tubes of the settler, through which the mixed liquor had to pass. As the liquid passed through this blanket, solids were removed both by sedimentation and entrainment within the blanket.

The carbon column acted as an upflow packed bed filter. Removal efficiencies are meaningless where the column is concerned because of the variation in feed solids concentration. Approximately 3% of the solids removed by the overall system during the experimental period was removed by the carbon column. Although this was a low value, the 319 g (0.7 lb) of solids it represents caused significant plugging in the carbon column during the later runs. The operational problems accompanying this plugging resulted in the brevity of Runs 4,5,6 and 7. It appears that the backwash was somewhat inefficient in removing all the solids entrained in the column, thereby, creating an accumulating solids load. This is discussed further in Section 5.5.2.

5.4.3 Sludge Digestion

Sludge production occurs within the aeration tank and within the carbon column. In a conventional system the excess biological sludge, measured as volatile solids, would require regular wasting in order to maintain the desired level of treatment. The WatRek system was designed so as to minimize the amount of sludge wastage required.

From Figure 5.12 it may be seen that over the duration of the experimental period, the mass of total solids showed an increasing trend. This was probably due to the retention of inert solids within the system.

The mass of volatile solids in the aeration and digestion tanks remained relatively constant at 9600 g (21 lb) throughout the programme.

A mass balance carried out on the volatile solids (VS) and substrate utilized within the system during the seven runs (Appendix I) shows that the overall yield value for the entire system was 0.08 g VS/g COD. This mass balance did not take into account the rest periods, during which there was no mixed liquor extraction. Since the mass of volatile solids within the system remained approximately constant over the experimental period (see Figure 5.12) any additional yield of volatile solids in the aeration tank during rest periods must have been removed and digested during the runs. This extra yield of volatile solids was not included in the mass balance and, therefore, the overall yield may be considered high.

Assuming, conservatively, that the yield obtained from the mass balance is low and that the real value is 0.1 g VS/ g COD removed, then the total volatile solids accumulated per day would be 140 g (0.30 lb). If the average concentration of volatile solids were permitted to reach 10,000 mg/l in the digestion tank,

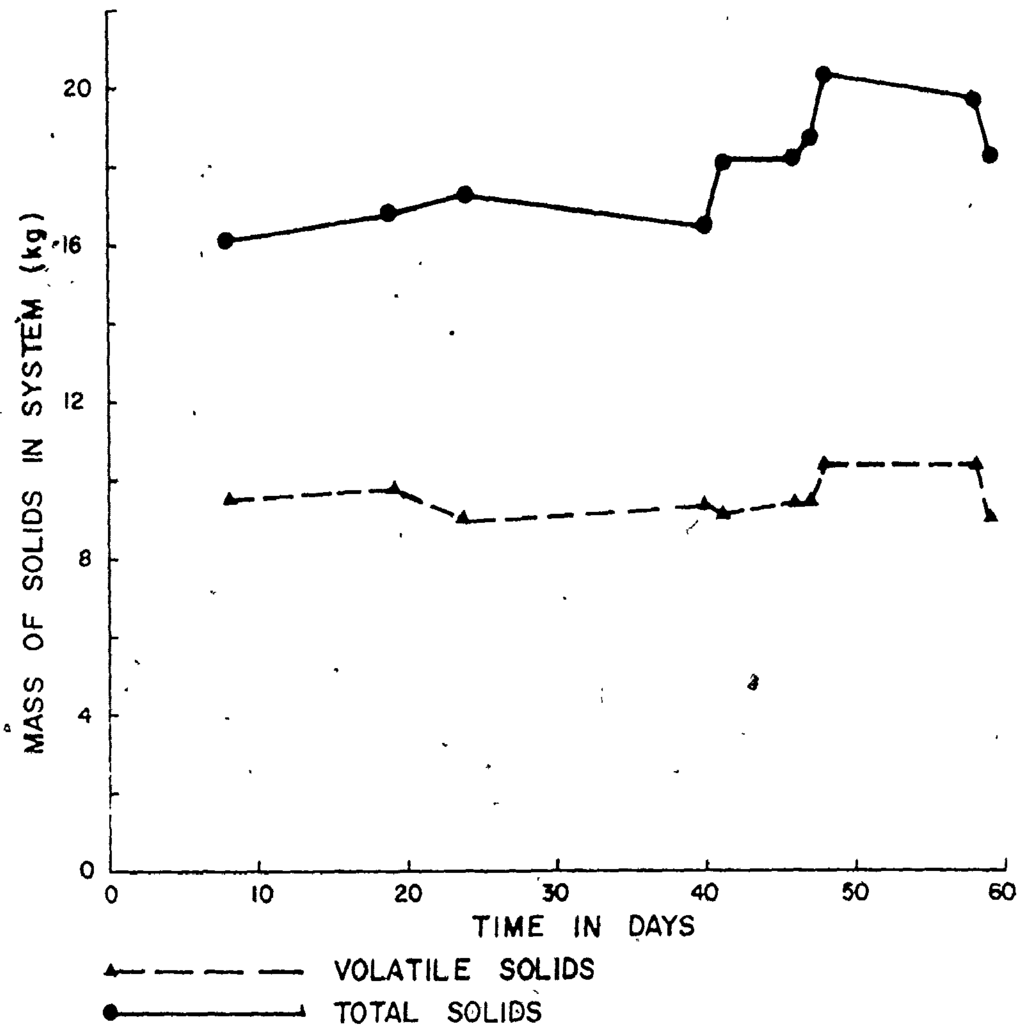


FIG. 5.12 MASS OF SOLIDS IN AERATION TANK PLUS

over a depth of 1.0 m (3.3 ft) a total mass of 16,000 g (36 lb) would be contained. This represents 4 months of feeding before any sludge removal would be required.

From this analysis, it may be concluded that the parallel aerobic digester fulfilled its stated function of reducing sludge handling requirements.

5.5 System Operation

System operation refers to the performance of the physical processes and components of the WatRek unit as distinct from actual treatment processes. The operation of the system is discussed here from a hydraulic and mechanical viewpoint.

5.5.1 Hydraulic Operation

As noted previously in Section 5.2.2.2, the floating clarifier operated in a flooded condition. There was, therefore, no flow equalization provided within the aeration tank. Storage of flow during periods of high headloss through the downstream facilities was provided, nonetheless, as during periods when the flow rate out through the carbon column fell below that of the influent, the aeration tank depth increased, thereby providing storage.

During Runs 2 and 3 a cyclical pattern of rising aeration tank level was established. Regular backwashing of the carbon column caused a decrease in the headloss through the bed and a subsequent decrease in the aeration tank level. These cycles are illustrated in Figures 5.13 and 5.14. It is apparent that the cycle in Run 2 was longer than that in Run 3. Note that in Figures 5.13 and 5.14 B/W signifies backwashing of the carbon column.

During the last two days of Run 3, a more pronounced daily rise in aeration tank level was observed. This was in conjunction with an increase in clarifier solids loss and appears to indicate that blinding of the column was occurring. It seems that at this point the backwash was not efficiently cleaning the column.

Runs 1,4,5,6 and 7 were terminated due to excessive increases in the aeration tank level. The failures of Runs 1,4 and 5 appeared to be due to the loss of solids from the clarifier. In each case, relatively high solids levels were noted in the effluent. Table 5-3 summarizes these losses; the reasons for which will be discussed in Section 5.6.

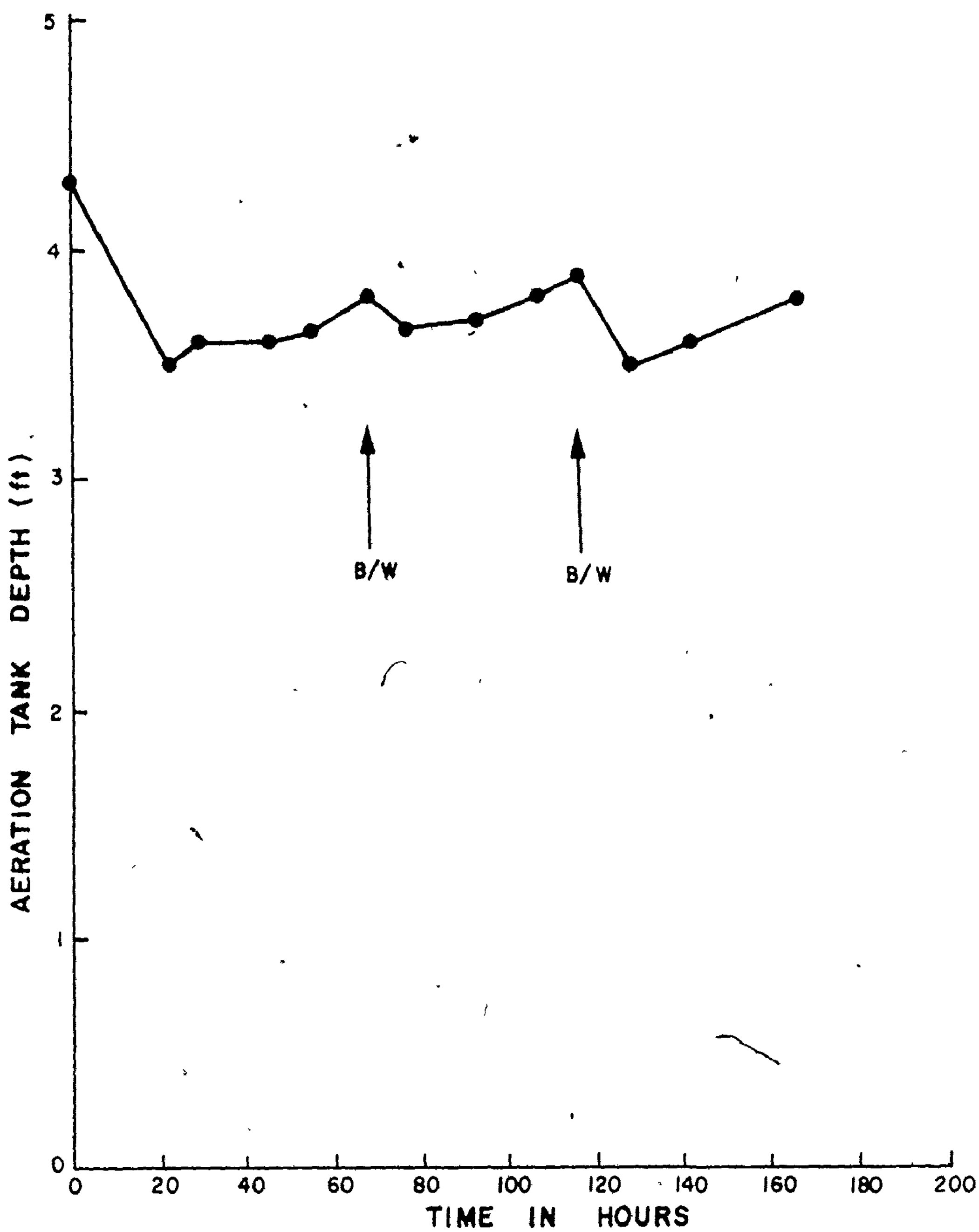
During Runs 6 and 7, the solids concentration in the clarifier effluent, as shown in Figure 5.10, was very low. Yet, both runs were terminated due to excessive aeration tank levels.

The aeration tank depth history for Run 6 shows that despite the low clarifier solids level, the level in the aeration tank rose to a high level in the first

Table 5-3

Solids Losses from Clarifier

Run/Day		Mass of solids loss
1	1	336 g
4	1	118 g
5	1	85 g
5	2	47 g



AERATION TANK DEPTH (ft)
TIME IN HOURS
B/W
B/W
0
1
2
3
4
5
0
20
40
60
80
100
120
140
160
180
200

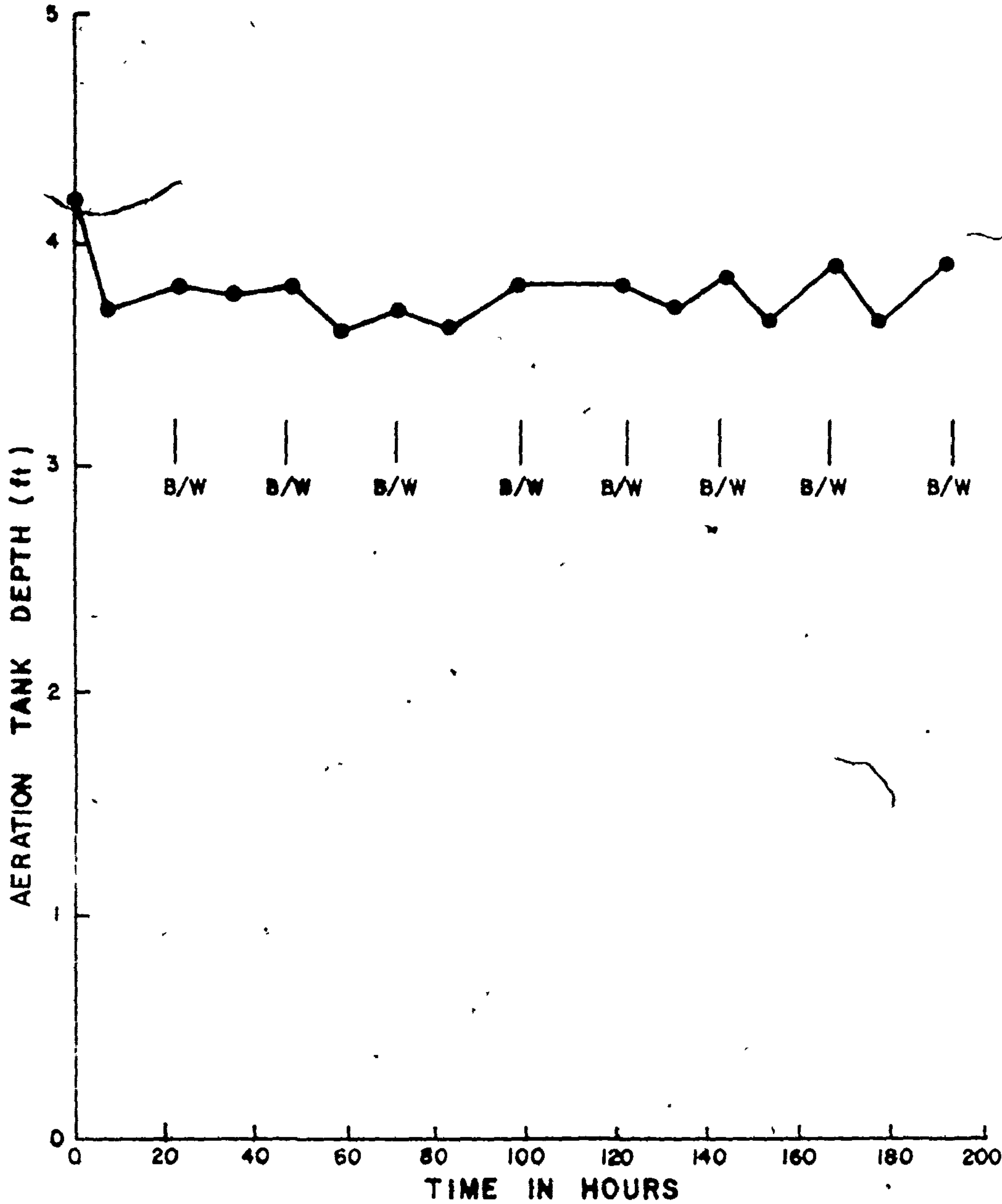


FIG. 5.14 AERATION TANK DEPTH RECORD, RUN 3

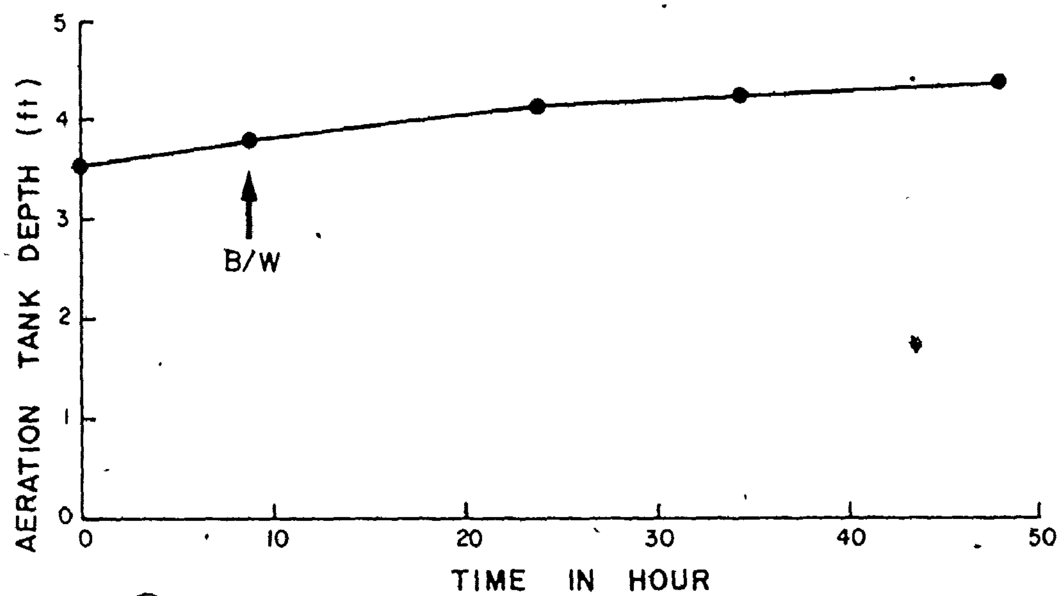
day of operation. At that time the carbon column was partially backwashed. This failed to alleviate the rise and after that there was insufficient freeboard in the tank for further backwashing. Figure 5.15, the depth history for Run 5, shows similar characteristics.

It is apparent from the curves in Figures 5.15 and 5.16 that, during backwash, sufficient solids were not being removed from the column to return the aeration tank to a normal level. The growth of bacteria within the carbon column and on the support screen may have aggrevated the situation. This led to a progressive blinding of the carbon column which caused the failures of Runs 5,6 and 7.

5.5.2 Backwash Efficiency

The frequency of backwash was dictated by the level in the aeration tank. As the tank level rose to 1.16 m (3.8 ft) the column was backwashed. The observed expansion during backwash was 43% which was somewhat less than the desired value of 50%. This small difference in expansion is unlikely to have caused the inefficient solids removal noted in the previous section.

The upflow mode of carbon column operation, coupled with the upflow backwash, may have contributed to the progressive blinding which was experienced. During operation, solids entered the column at its base. These



AERATION TANK DEPTH (ft)
5
4
3
2
1
0
B/W
0
10
20
30
40
50
TIME IN HOUR

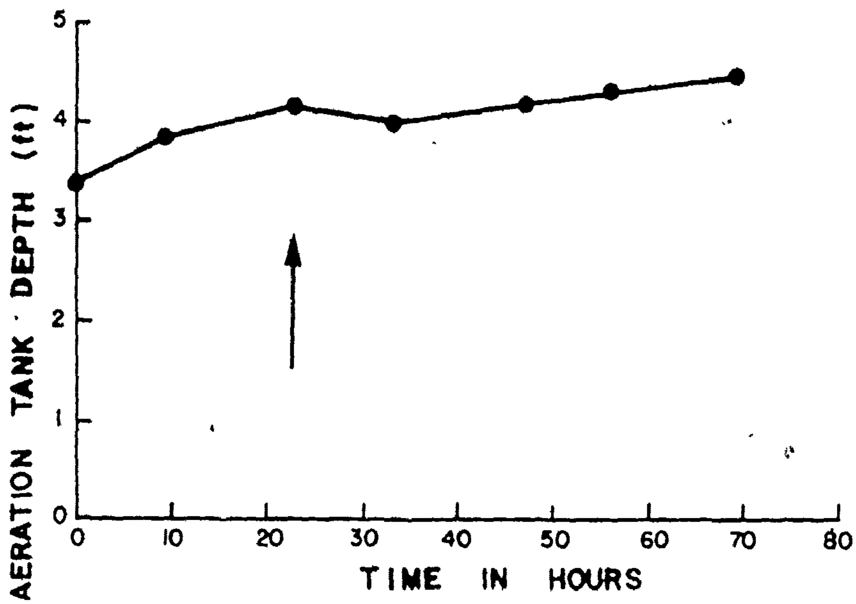


FIG. 5 ... ION TANK ...

were then propagated upward during backwash. The volume of backwash may not have been sufficient to move all the solids from the bottom to the top of the carbon bed. Some solids would then have been left in the bed and, over the course of several backwashes, would have accumulated to the point where the flow through the column was impeded.

If the carbon column were to be operated in a downflow mode, the backwash may have been more efficient because of the shorter distance of travel for the solids prior to removal from the bed. More efficient backwashing may also be achieved by increasing the volume of the backwash flow.

5.5.3 Mechanical Operation

There were two major pieces of mechanical equipment included in the WatRek unit. These were the backwash pump and the air compressor, and both performed satisfactorily.

5.6 The Effect of Process Parameters

5.6.1 Air Flow Rate

The operation of the clarifier, and hence, the entire system was affected by the rate of aeration. During the Runs operating at higher aeration rates, that is, Runs 1,2 and 4, solids mats were observed floating on the surface of the clarifier.

These floating solids were light brown in color and contained entrained gas bubbles. The character of this gas was not determined analytically, but may have been either fine air bubbles adhering to the floc, or nitrogen gas produced by denitrification in the clarifier tubes. Denitrification, however, is unlikely in view of the short liquid residence time (between 0.3 and 0.6 hr) in the clarifier.

This phenomenon was most pronounced in Run 1 when the air rate was 1.9 ~~l~~/sec (4 scfm). During this run, the mass of solids lost in the clarifier effluent reached 336 g (0.74 lb). Run 3, while deemed a successful run, showed evidence of floating solids at an air rate of 1.4 l/sec (3 scfm). The loss of solids in Run 3 appears to have been responsible for the increased daily rise in the aeration tank depth near the end of the run.

In runs during which the aeration rate was 0.9 l/sec (2 scfm), solids levels in the clarifier effluent were excellent. During these runs the power dissipation rate in the aeration tank was 0.6 HP/1000 gal. This was higher than the usual design value of 0.1 to 0.2 HP/1000 gal, but was considerably lower than the 1.2 HP/1000 gal dissipated in Run 1 at 1.0 l/sec (4 scfm). In Run 3, the energy dissipated in mixing the aeration tank contents was 0.9 HP/1000 gal.

Runs 4 and 5 appeared to fail due to plugging of the carbon column with solids passing through the clarifier. Both these runs were operated at low aeration rates and no floating solids were observed. Both were, however, run at a high feed rate and the significance of this is discussed in Section 5.6.2.

The insufficient settling in the clarifier was apparently due to the high aeration rates in Runs 1 and 3. Such high levels had a two-fold effect upon the clarifier: One, the turbulence caused by the rising bubbles imparted an agitation to the clarifier which impaired the solids removal. Two, the adherence of small air bubbles to floc particles causing them to rise to the surface of the clarifier. As the rate of aeration increases, so does the number of fine bubbles and the probability of their being carried to the clarifier inlet where they may cause sludge flotation.

While operation at a low aeration rate may appear desirable from a clarification point of view, it leaves the biomass in the aeration tank in a precarious position in relation to oxygen supply. It is illustrated in Figures 5.17 and 5.18 that the D.O. in the aeration tank was always low and sometimes as low as 0.2 mg/1.

These figures also show the cyclical nature of the D.O., corresponding to the peak influent waste concentrations. D.O. levels in this range leave very little margin of safety in the event of an extraordinary high influent waste load, or other shock loading.

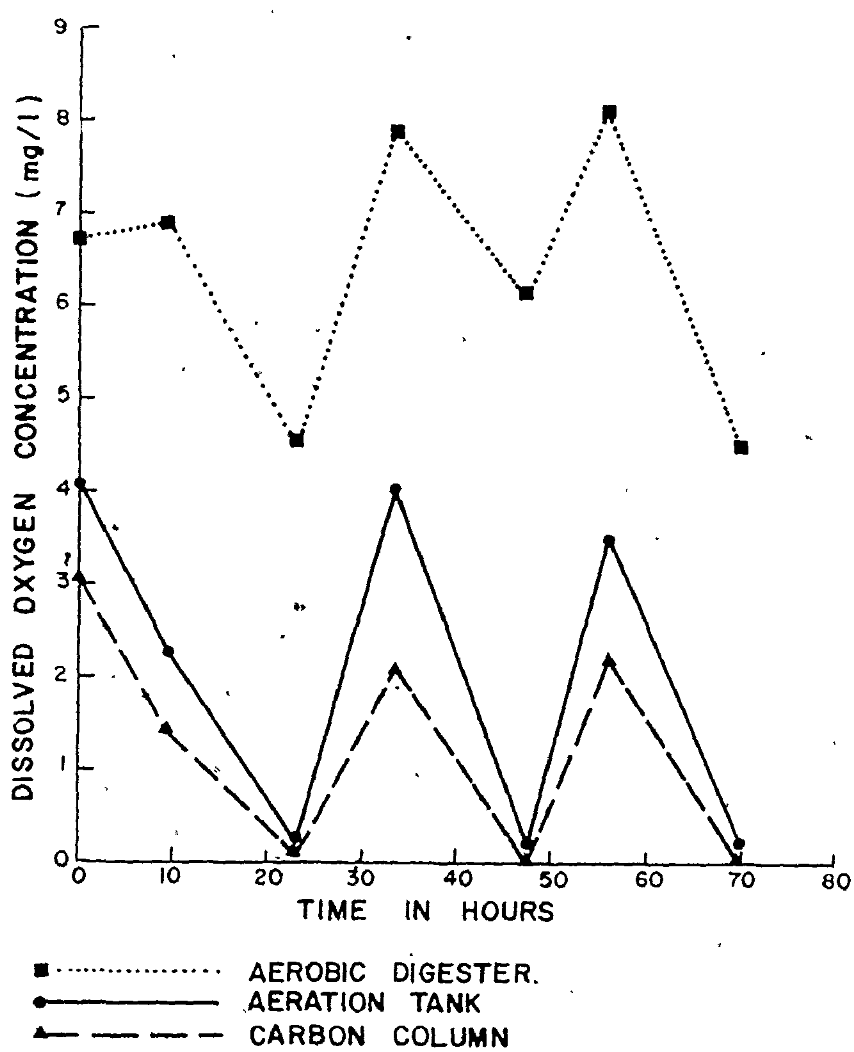


FIG. 5.17 DISSOLVED OXYGEN CONCENTRATION vs. TIME, RUN 6

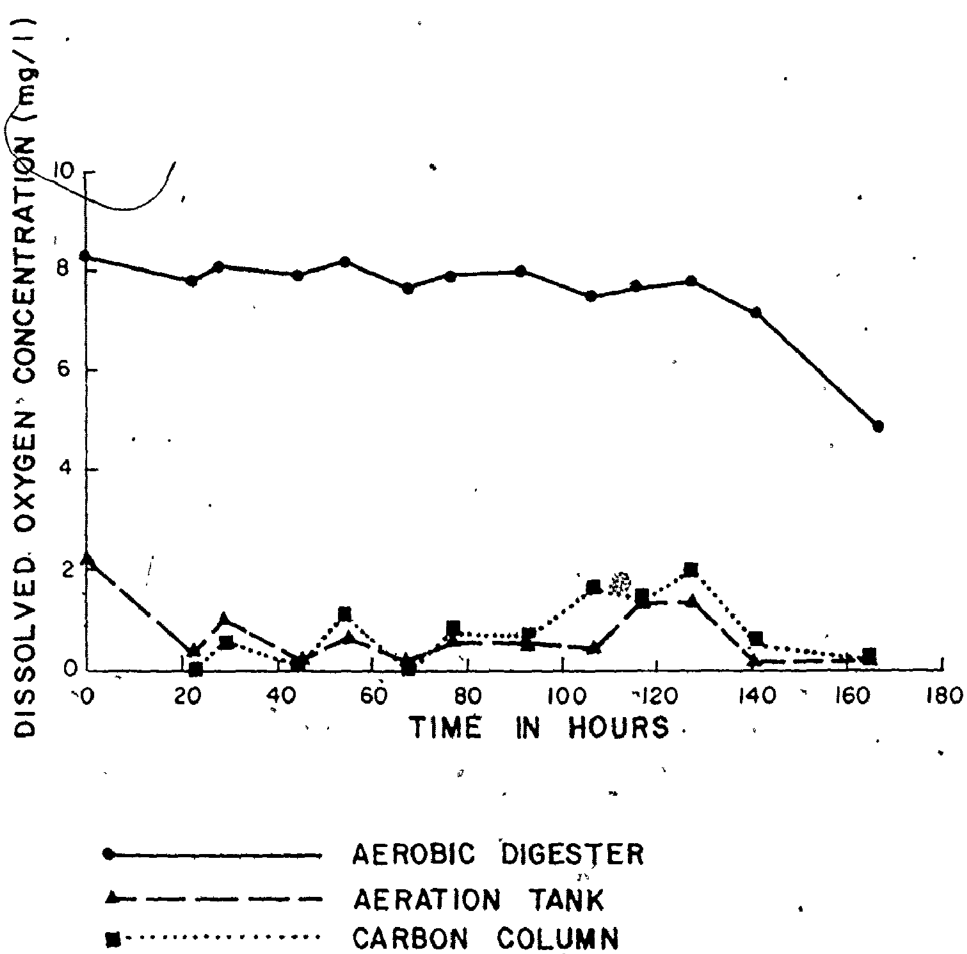


FIG. 5.18 DISSOLVED OXYGEN CONCENTRATION vs. TIME, RUN 2

It appears that if the problem of solids escaping the clarifier is to be alleviated, a more efficient method of aeration must be devised. Such a method would have efficient oxygen transfer and impart little turbulence to the clarifier.

5.6.2 Wastewater Feed Rate

Increasing the feed rate above the design rate of 2273 l/day (500 IGPD) appears to have decreased the operational efficiency of the overall system. The process efficiency remained high, as shown in Figures 5.9 and 5.11. It was apparent from the results of Runs 4 and 5 that flow rate may have had an effect on solids removal, particularly when coupled with the high aeration rate in Run 4.

The loss of solids may have been affected by the surface overflow rates in the clarifier. During Run 2, the surface overflow rate was 1.01 m/hr (500 IGPD/ft^2) while in Run 5, which had the same aeration rate, the overflow rate was twice that; 2.02 m/hr (1000 IGPD/ft^2). If the overflow rates are calculated, according to Mendis (1976), on the basis of tube area, they reduce to 0.08 m/hr and 0.04 m/hr (41.7 and 20.8 IGPD/ft^2), respectively for Runs 2 and 5. It is interesting to note that the concentration of solids in the clarifier effluent during Run 5 was twice the average concentration in Run 2, possibly due to the two-fold increase in overflow rate. However, neither overflow rate was high in comparison to literature values for tube clarifiers. This fact, coupled with the lack of correlation between overflow rate alone and SS removal suggests that the aeration rate was most important in determining clarifier effluent solids levels.

5.6.3 Mixed Liquor Extraction Rate

The mixed liquor extraction rate (MLX) cited was, a mean value. The flow through an air lift pump varies with the depth of submergence of the air inlet. Since the inlet level was fixed and the water level fluctuated, the flow through the pump varied with changes in aeration tank depth. Adjustments were made during the observation periods.

The effect of MLX on the operational and performance aspects of the system is difficult to separate from the other factors present. It appears from the mass balance noted in Section 5.4.3, that the concept of simultaneous digestion of mixed liquor solids serves to reduce the net yield in the entire system to a very low value without impairing the settling characteristics of the sludge.

Average volatile solids removal in the digestion tank appeared to be approximately 35%.

5.6.4 Specific Loading Rate

The specific loading rate or food to microorganisms ratio (F/M) is the ratio of the mass of substrate removed to the mass of biomass contained in the aeration tank. Since the digester acted as a very low rate system in which the autolysis of cells was assumed to be large in comparison to substrate removal, the digester volatile solids were not included in the F/M calculation. Inclusion of these solids would result in a lower F/M ratio. Downing *et al.* (1965) showed that, generally, as the F/M ratio increases, the process efficiency decreases.

Assuming that COD/BOD = 1.7, in the influent, and COD/BOD = 3, in the effluent from the aeration tank, the data gathered from this study may be compared with that of Downing. This shows that the present data falls on the curves shown in Figure 5.19 and, therefore, within the range of acceptable biological treatment performance.

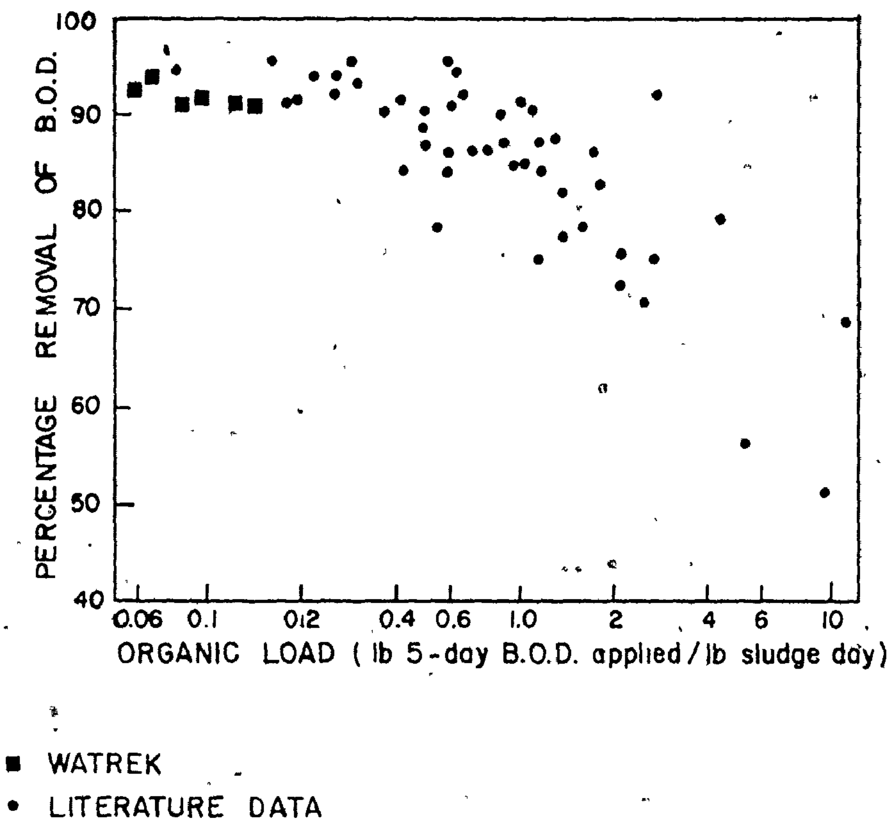


FIG. 5.19 PERCENT B.O.D. REMOVAL vs. F/M RATIO (AFTER DOWNING ET AL, 1965)

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

1) A small, portable wastewater reclamation system (WatRek) was built which could be easily transported and assembled without specific skills.
2) The flow equalization function of the system was rendered inoperative by flooding of the effluent trough of the floating clarifier.
3) Biological startup could be accomplished in 17 days without an activated sludge seed.
4) During biological startup, the activated carbon column could not be included in the system due to excessive plugging by solids escaping through the clarifier.
5) Over the course of the study period, the system achieved average removals of 92.8% chemical oxygen demand and 97% suspended solids,

thereby, meeting the effluent design criteria of 30 mg/l COD and SS.

6) The system operated best under an aeration rate of 0.9 l/sec (2 scfm). This represents an energy dissipation rate of 0.6 HP/1000 gal. Aeration rates greater than this imparted excessive agitation to the floating clarifier.

7) Flows in excess of 2273 l/day (500 IGPD) constituted a hydraulic overload to the system.

8) Over the course of the study period, the net solids production was 0.08 g MLVSS/g COD removed.

6.2 Recommendations

1) The floating clarifier requires modification in order to facilitate flow equalization. Possible modifications include:

   a) a smaller effluent trough;

   b) a lower stop on clarifier travel which will maintain an aeration tank liquid level above that of the carbon overflow;

c) if neither of the above are successful the clarifier should be fixed, with overflow rates governed by the depth of submergence of effluent orifices. As flow, and hence submergence of the orifices increases, the overflow rate increases by the relation

$$Q = C_d A \sqrt{2\ gh}$$

where: Q = flow rate (ft^3/sec);

C_d = orifice coefficient of discharge;

A = area of the orifice

g = gravitational constant;

h = submergence.

2) The mode of operation of the carbon column should be altered to be downflow with an upflow backwash. This would tend to alleviate the progressive blinding by limiting the major portion of the entrapped solids to the top of the carbon bed.

3) The wall separating the digestion tank from the clarifier should be strengthened.

4) A more efficient diffuser should be added to the aeration tank in order that maximum transfer of oxygen may be obtained with a minimum of agitation of the floating clarifier.

5) The air lift pump used to extract mixed liquor should be a more constant flow unit. This may be accomplished by suspending the air injection port from a floating platform, thereby minimizing the effect of level fluctuations in the aeration tank.

6) A quiescent zone should be provided, by means of a baffle, at the outlet end of the digestion tank in order to retain solids in the digester through settling in this zone.

7) A controlled set of experiments should be performed upon the system in order to define a mass balance for the process.

APPENDIX A

ALTER'S CRITERIA FOR NORTHERN WASTE TREATMENT

Northern Waste Treatment Design Criteria and Constraints

(After Alter, 1974)

Sewage Disposal Objectives

1) Prevent disease;

2) Achieve environmental excellence;

3) Remove and stabilize wastes for environmental excellence;

4) Provide simple, failsafe facilities;

5) Remove and stabilize wastes in an inoffensive manner;

6) Efficient service.

Engineering Constraints

1) High cost;

2) Unfavorable site conditions;

3) Repair and maintenance unavailable;

4) Knowledge of advantageous use of cold, lacking;

5) Expensive energy;

6) Freezing of system;

7) Complicated systems;

8) Poor transportation systems;

9) Inefficient systems.

APPENDIX B

PROCESS DESIGN

Aeration Tank Design

The aeration tank was designed to handle the sewage generated by ten persons. The following criteria were used in order to determine the quantity and strength of the sewage flows.

BOD 0.08 kg/person/day (0.17 lb/person/day)

$$\frac{BOD}{COD} = 0.6$$

COD 0.13 kg/person/day (0.28 lb/person/day)

Volume 227.5 lpcd (50 Igpcd)

Total volume = 2275 l/day (500 Igpd)

Load 910 g BOD/day (2 lb BOD/day)

1517 g COD/day (3.34 lb COD/day)

MLVS Assuming F/M = 0.15 and designing for a 90% removal of organics in the aeration tank yields:

$$\text{MLVS} = \frac{1517(0.9)}{0.15(2275)} = 4.0 \text{ g/l}$$

Mean Cell Residence Time

From Lawrence & McCarty (1969) it may be seen that values of θ_c for extended aeration plants vary from 14 to infinity. Metcalf & Eddy (1972) indicate that it varies from 20 to 30 days. Therefore, a value of 30 days was chosen for this application.

Reactor Volume

$$XV = \frac{Y\,Q(S_o - S_1)\,\theta_c}{1 + b\,\theta_c} \qquad \text{(Lawrence \& McCarty, 1969)}$$

where: X = biomass concentration (mg/l)

V = reactor volume (l)

Y = yield coefficient (mg/mg COD)

Q = flow (l)

S_o = influent COD concentration (mg/l)

S_1 = effluent COD concentration (mg/l)

θ_c = mean cell residence time (days)

b = decay coefficient (day^{-1})

Typical values of the coefficients were chosen from the same reference and are

$$Y = 0.67, \ b = 0.07$$

Therefore, the reactor volume:

$$V = \frac{0.67(2275)(667-67)30}{(1 + 0.07(30))4000}$$

$$= 2213 \text{ l} = 78.2 \text{ ft}^3$$

Excess Sludge Production

$$\frac{dX}{dt} = \frac{XV}{\theta_c} = \frac{4000(2213)}{30} \qquad \text{(Metcalf \& Eddy, 1972)}$$

$$= 295067 \text{ mg/day} = 295 \text{ g/day}$$

$$= 133 \text{ mg/l-day}$$

Sludge Wastage

Sludge was to be wasted from the mixed liquor at a concentration of 4000 mg/l. Therefore, a total of 74 l of mixed liquor had to be wasted each day.

Hydraulic Detention Time

$$\theta = \frac{V}{Q} = \frac{2213\ l}{2275\ l} = 23.4 \text{ hrs}$$

Oxygen Requirements

$$\frac{dCO_2}{dt} = \frac{a'dF}{dt} + b'X$$

where:

$a' = (1-1.42)Y = O_2$ required for growth

$b' = b \qquad = O_2$ required for endogenous respiration

$a' = (1-1.42)(0.67) = -0.28$

$b' = 0.07$

$$\frac{dCO_2}{dt} = -0.28(600) + 0.07(4000)$$

$$= 112 \text{ mg/l-day}$$

Mass of O_2 Required Per Day

$$\frac{112 \times 2213}{1000} = 248 \text{ g/day}$$

Aeration

Aeration provides both mixing and dissolved oxygen (D.O.) in the activated sludge process. Due to lower overall mechanical requirements and configuration constraints coarse bubble diffused aeration was chosen.

Sufficient D.O. must be added to maintain aerobic conditions in the downstream carbon column. Therefore, for the purposes of this design a minimum D.O. of 4 mg/l was chosen.

Leary *et al.* (1969) cite, for spiral flow, a k_1a or mass transfer coefficient of 47.8 day^{-1} for a fine bubble system. The mass transfer coefficients for systems using coarse bubble aeration have been found to be about 60% that of fine bubble systems. Kayser (1969) cited k_1a of 3.65 hr^{-1} for a medium bubble system. He also noted a respiration rate of 13 mg/l hr in the same system.

For conservative design a k_1a value of 1.825 hr^{-1} will be used.

Oxygen Transfer

From Fick's law:

$$\frac{dc}{dt} = k_1a\ (C_s^* - C)$$

where:

$C_s^* = C_s - r_r/k_1a$

$\frac{dc}{dt}$ = change in D.O. with time

C_s^* = operating value

C = D.O. concentration of influent

r_r = respiration rate

C_s = saturation concentration

Assuming an operating temperature of 5°C and D.O. saturation concentration of 12.8 mg/l then the amount of oxygen which can be supplied is:

$$\frac{dc}{dt} = 1.825(4-0.1) = 7.12 \text{ mg/l-hr}$$

This represents a daily supply of 389 g/day which meets the requirements of 248 g/day previously noted. Assuming the 3% of the oxygen supplied has time to be transferred during the bubble rise, then the mass of oxygen which must be delivered is:

$$248 \text{ g/day} \times \frac{1}{0.03} \times \frac{1}{24 \times 60} \ \frac{\text{day}}{\text{min}}$$

$$= 5.74 \text{ g/min}$$

Air Flow Rate

Oxygen comprises 21% of the ambient air. The approximate mole weight of which is 28 g, with a volume, at STP, of 22.4 l. Therefore, the volume of air required is:

$$5.74\left(\frac{\text{g}}{\text{min}}\right) \times \frac{1}{0.21} \times \frac{1}{28}\left(\frac{\text{moles}}{\text{g}}\right) \times 22.4 \left(\frac{\text{l}}{\text{mole}}\right) \times 0.035 \left(\frac{\text{ft}^3}{\text{l}}\right)$$

$$= 21.9 \text{ l/min } (0.77 \text{ scfm})$$

Mixing Requirements

Energy dissipation may be calculated from the isothermal expansion of the rising bubbles:

$$HP = \frac{P_A Q_A}{33000} \ln\left(\frac{H + 339}{33.9}\right)$$

where:

P_A = atmospheric pressure (lb/ft^2)

Q_A = air flow rate (scfm)

H = bubble rise (ft)

HP = power dissipated (horsepower)

The energy dissipated by the required air volume is 0.11 HP or 0.21 HP/1000 gal. Eckenfelder & Ford (1967) and Knop & Kalbskopf (1970) recommend 0.1 to 0.2 HP/1000 gal for the extended aeration process.

Therefore, the air required for aerobic processes will adequately mix the contents of the aeration tank.

Floating Clarifier Design

An overflow rate of 1.01 m/hr (500 Igpd/ft^2) was chosen for design of the clarifier. This value was well within the limits observed by other researchers. The depth of submergence of the weirs was to have controlled the overflow rate.

From Daugherty & Franzini (1965) the equation for a V-notch, knife edge weir is:

$$Q = C_d \ \frac{8}{15} \cdot \sqrt{2\ g} \ \tan \frac{\theta}{2} H^{5/2}$$

where:

θ = apex angle

C_d = drag coefficient

g = gravitational constant

H = depth of submergence

For design: $\theta = 90^{\circ}$ and $C_d = 0.59$ and $Q = 500$ Igpd.

For one weir, $H = 0.51$ inches.

For 4 x 90° weirs, $H = 0.29$ inches.

If the unit were overloaded to 750 Igpd then the required H would be 0.34 inches for 4 weirs.

Buoyancy

The specific gravity of polyvinylchloride (PVC) is 1.347. This translates to a density of 84.06 lb/ft^3.

Tube weight = 28 lb

Tube material volume = 0.33 ft^3

Flotation tray weight = 12.03 lb

Flotation tray material volume = 0.14 ft^3

Since the tubes will be completely submerged, the weight which must be overcome is the submerged weight of the tubes plus the weight of the tray:

$$\text{Buoyancy required} = 28 - 0.33(62.4) + 12.03$$

$$= 19.44 \text{ lb}$$

Volume of water which must be displaced:

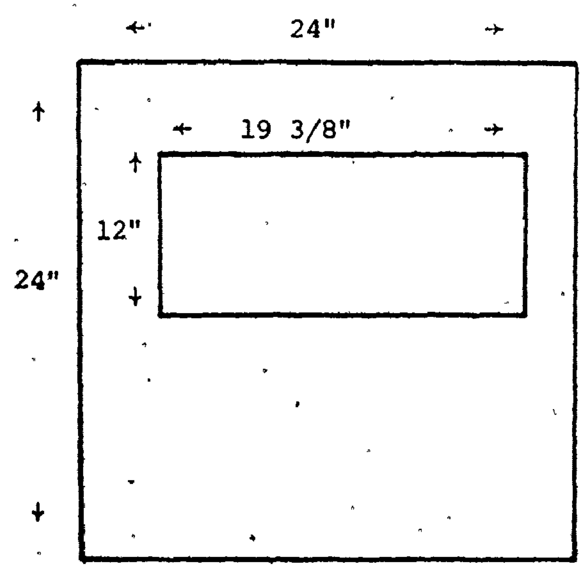
$$V = \frac{19.44}{62.4} = 0.31 \text{ ft}^3$$

Assigning tray dimensions as shown in the sketch, the height above the tray bottom at which the apex of the weirs must be set will be:

$$D = \frac{0.31}{\left(\frac{24 \times 24 - (19.375 \times 12)}{144}\right)}$$

$$= 0.14 \text{ ft}$$

$$= 1.56 \text{ inches}$$



Size of Sludge Waste Container

At a MLVSS of 4000 mg/l, the daily wastage of sludge from the aeration tank = 74 l.

If we conservatively assume no decomposition in the digester, then Mass in = accumulation.

Assuming thickening to 10000 mg/l

$$Q_i\ C_i = V\ C_A$$

where:

Q_i = influent flow

C_i = influent concentration

V = volume of container required per day

C_A = concentration in the container

$$V = \frac{74(4000)}{10000}$$

$$= 30\ \text{l/day}$$

Actual sizing will be determined according to the final unit configuration. Residence time must not be less than 30 days.

Carbon Column Design

For medium quality effluent carbon loading should be 0.6 g COD/g carbon including adsorption and bioactivity.

The feed concentration of COD to the column = 66 mg/l.

Design for the removal of 40 mg/l COD within the column.

Therefore column effluent = 26 mg/l COD.

$$SL = \frac{t_R\ Q(C_o - C_r)}{\rho_p\ (q_\infty - q_r)} \qquad \text{(Benedek, 1973)}$$

where:

t_R = time between regenerations (days)

Q = volumetric liquid flow rate (l/day)

q_r = residual adsorbate concentration after regeneration (dimensionless)

S = cross sectional area of column (cm^2)

L = length of carbon column (cm)

ρ_p = packed density (g/l)

q_∞ = final loading (dimensionless)

$C_o - C_r$ = amount of adsorbate removed (g/l)

Sufficient carbon for 8 months, unregenerated operation will be supplied, *ie.* t_R = 240 days.

$$SL = \frac{240\ (2270)\ 0.04}{420 \times 0.6} \left(\frac{\text{days}\ \frac{1}{\text{day}}\ \frac{\text{gm}}{1}}{\text{g/l}}\right)$$

$$= 86.5\ l \qquad = 86500\ cm^3$$

$$= 3.1\ ft^3 = 0.087\ m^3$$

Therefore, assuming a hydraulic loading of 0.73 m/hr (0.25 $Igpm/ft^2$)

$S = 1275\ cm^2 = 1.37\ ft^2$

$L = 68\ cm = 2.2\ ft$

Bed depth = 68 cm (2.2 ft)

Allowance for backwash = 50% = 102 cm = 3.3 ft.

Therefore, the overflow is placed 34 cm (1.12 ft) above the bed.

Backwash Requirements

Normal backwash flows for upflow columns for 8 x 30 mesh carbon are 23.3 - 29.1 m/hr (8-10 $Igpm/ft^2$).

Size of backwash tank:

$$490 \frac{l}{min\ m^2} \times 0.13\ m^2 \times 10\ min = 625\ l\ (22\ ft^3)$$

Flow = 62.5 l/min (13.7 Igpm)

Headloss in expanded bed:

$$h_f = L_e(1 - \varepsilon)\ \frac{(\rho_s - \rho_w)}{\rho_w}$$

where:

h_f = headloss in feet of water

L_e = length of expanded bed (feet)

ε = void ratio of expanded bed

where $1 - \varepsilon = \dfrac{\text{bulk density}}{\text{water density X particle density}}$

= 0.79

ρ_w = density of water (lb/ft^3)

ρ_s = particle density (lb/ft^3)

$$h_f = 3.3\ (1 - 0.79)\ \frac{(82.4 - 62.4)}{62.4}$$

$$= 0.24\ ft$$

$$= 7.4\ cm$$

Therefore, total head (H) required is:

$$H = h_s + h_f$$

where:

h_s = static head

h_f = kinetic head.

Thus, h_s = 1.67 m (5.5 ft)

H = 1.75 m (5.75 ft).

APPENDIX C

CONSTRUCTION MATERIAL DECISION

Metal

An all metal structure would have been heavy, and difficult to seal without complex gaskets or welding, when assembled in the field. PVC liners would have alleviated the sealing problem but the weight of the unit would not have been reduced. Shop-welding would have been inconsistent with the kit concept of the system.

Plastics

Plastic tanks are corrosion resistant and can be made structurally sound. In large part, the strength of such tanks results from their molded construction. This leads to circular or round cornered rectangular tanks which cannot be dismantled, therefore, volume requirements during transportation would be excessive. Using plastic sheeting as wall material would have necessitated a substantially larger steel bracing system than that finally decided upon. Common plastic materials are also significantly heavier and more costly than wood of equivalent strength.

Wood

A completely wooden structure would have been subject to rotting and water logging. The resulting degradation would have undermined the structural integrity of the tanks. To prevent this, a surface sealant would have been required. Waterproof joints would have been difficult to achieve unless wood stave construction were used. Wood stave construction was not feasible because of the retangular plan.

Laminated wood products such as plywood are strong for their weight. Even with the addition of a steel bracing system, it was anticipated that a reduction in weight over the other systems considered could be realized.

A primary concern in selecting the building material for the WatRek unit was the transportation weight. Two materials were considered in detail; plywood and steel. In choosing the material it was assumed that the steel framing system would be common to both wood and steel.

Therefore, the only differing parameter would be the unit weight of the skin material. The unit weight of 3.2 mm (1/8 inch) steel plate was 24.9 kg/m^2 (5.1 lb/ft^2). The weight of the 1.91 cm (3/4 inch) plywood was 10.9 kg/m^2 (2.23 lb/ft^2). On this basis, the plywood was chosen.

APPENDIX D

BRACING DESIGN CALCULATIONS

Steel Bracing Calculations

The bracing calculations carried out below are examples of those carried out in the design of the WatRek structure. All bracing has been designed on the basis of simple beam action in order to be conservative. Continuous beam actions and compound beam actions do exist within the structure but should serve to enhance the strength calculated below. All calculations were performed according to the procedures laid down in *The Handbook of Steel Construction*, Canadian Institute of Steel Construction, 1972.

Horizontal Bracing

Span = 3 ft = L

Unit load = w = 2.83 lb/in^2

(assuming 6 ft depth of water)

Load = W = unit load x L

= 102 lb/in

Maximum Moment = $M_{max} = \frac{WL^2}{8}$

= 16524 in-lb

Required section modulus = S_x = 0.57

From beam tables use angle 2 1/2" x 2 1/2" x 3/8"

unit weight = 5.9 lb/ft

Check deflection

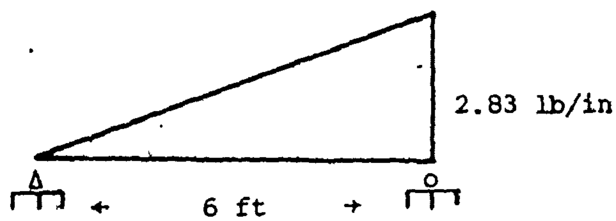
$$\Delta_{max} = \frac{5\ wL^4}{384\ EI}$$

= 0.13 in

Vertical Bracing

Member spacing = 3 ft

Beam diagram



Load = $W = \frac{wL}{2} = 3668$ lb

Maximum Moment = 0.1283 (W) = 471 in-lb

Required section modulus = 0.02

From beam tables use 2" x 2" x 1/8" angle

Check deflection

$$\Delta_{max} = \frac{0.01304\ wl^3}{EI}$$

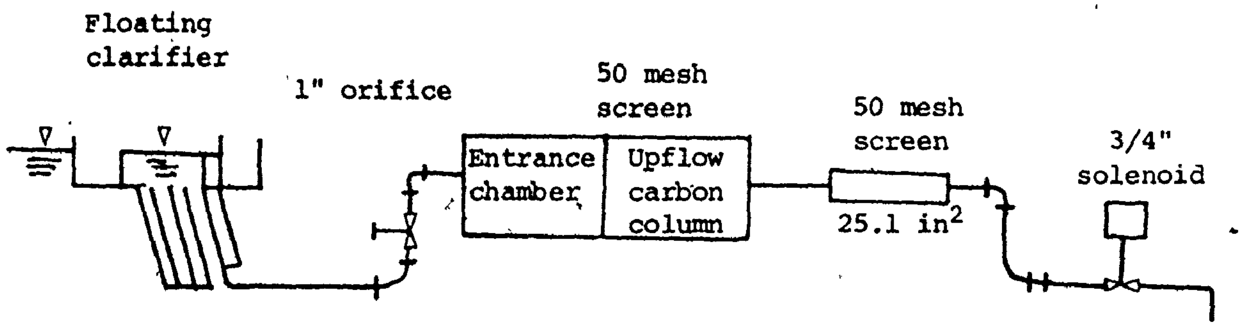
$$= 0.09 \text{ inches}$$

APPENDIX E

HYDRAULIC ANALYSIS

Hydraulic Analysis

System Diagram



Clarifier Orifices, Diameter = 1 inch

$$h_f = \frac{1}{2\,g}\left(\frac{Q}{CA}\right)^2$$

where: h_f = head loss (ft)

g = gravitational constant (ft/sec^2)

C = orifice constant

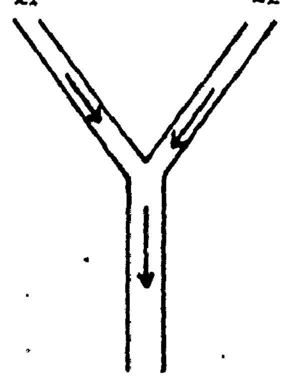
V = velocity (ft/sec)

Flow (Igpd)	Headloss/orifice (ft)	Total headloss (in)
500	3.49×10^{-4}	.008
750	8.5×10^{-4}	.02
1000	4.92×10^{-3}	.1
5300	0.042	1

Headloss at "Y" on Clarifier Outlet

Using Bernoulli:

$$\frac{P_1}{\gamma} + Z_1 + \frac{V_1^2}{2g} = \frac{P_2}{\gamma} + Z_2 + \frac{V^2}{2g}$$



Assume $P_1 = P_2 = P_3$ & $Z_1 = Z_2$, $\gamma = \gamma$ $Z_2 = 0$

$$\therefore \quad \frac{V_1^2}{2g} + \frac{V_2^2}{2g} = \frac{V_3^2}{2g} + h_L$$

$$V_1 = V_2$$

$$\therefore \quad \frac{(V_1^2)}{g} = \frac{V_3^2}{2g}$$

$$h_L = \frac{V_1^2}{g} - \frac{V_3^2}{2g}$$

Flow (Igpd)	Velocities		Headloss
	V_1	V_3	
500	0.09 ft/sec	0.17 ft/sec	1.9×10^{-4} ft
750	0.13 ft/sec	0.26 ft/sec	5.2×10^{-4} ft
1000	0.17 ft/sec	0.34 ft/sec	9.0×10^{-4} ft
5300	0.98 ft/sec	1.96 ft/sec	0.03 ft = 0.36"

Headloss Through Sudden Enlargement at Entrance to Carbon Column

$$h = \frac{(V_1 - V_2)^2}{2\,g}$$

Orifice size = 1 " = D_1

Column area = 169 in^2

Flow (Igpd)	Velocities V_1 (ft/sec)	Velocities V_2 (ft/sec)	Headloss (inches)
500	0.18	6.60×10^{-4}	0.006
750	0.28	1.03×10^{-3}	0.014
1000	0.38	1.40×10^{-3}	0.027
5300	1.83	0.007	0.62

Headloss Through Support Screen

Screen = 50 mesh

Wire diameter = 0.007 inches

Projected area of wire = 1" x 0.007"

= 0.007 in^2/in length

With 100 wires crossing in a 1 in square the projected area

= 0.7 in^2/in^2

Therefore, area available for flow = 1 - 0.7 = 0.3 in^2/in^2

Screen area = 169 in^2

Area available = 169 (0.3) = 50.7 in^2 = 0.35 ft^2

Headloss $h = \beta (w/b)^{4/3} h_v$ (Fair, Geyer & Okun, 1971)

$$h_v = \frac{V_2}{2 g}$$

Flow (Igpd)	Velocity (ft/sec)	Headloss (inches)
500	0.0027	7.5×10^{-6}
750	0.004	1.7×10^{-5}
1000	0.053	2.9×10^{-3}
5300	0.028	8.1×10^{-4}

Headloss through carbon column

$$\frac{h_f}{l} = \frac{180\,\mu\, U_s\,(1-\varepsilon)^2}{\rho\, g\, \varepsilon^3\, \Psi_s^{\,2}\, D^2}$$

where:

h_f = headloss (total)

l = length of filter = 70.3 cm = 2.31 ft

μ = viscosity, at $2^{\circ}C$ $\mu = 3.75 \times 10^{-5}$ lb ft/sec

ε = void ratio, $V_T = V_v + V_p$, $\varepsilon = \dfrac{V_v}{V_T} = 0.5$

ρ = fluid density

g = gravity

Ψ_s = particle sphericity = 0.73

D = diameter of particles = 0.8 → 0.9 mm = 2.3×10^{-3} ft

U_s = superficial liquid velocity

D_{10} = 0.7

Uniformity coefficient = 1.9

U_s at 500 Igpd = 0.0009 ft^3/sec = 6.6×10^{-4} ft/sec

750 Igpd = 0.0014 ft^3/sec = 1.03×10^{-3} ft/sec

1000 Igpd = 0.0019 ft^3/sec = 1.4×10^{-3} ft/sec

5300 Igpd = 0.01 ft^3/sec = 0.007 ft/sec

ρ = 1.94 slugs/ft^3 = 62.4 lb/ft^3

At 500 Igpd therefore,

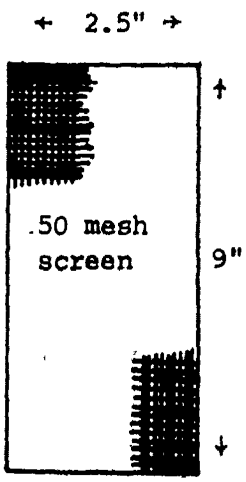
$$\frac{h_f}{l} = 2.64 \times 10^{-6} \text{ ft/ft}$$

$$h_f = 6 \times 10^{-6} \text{ ft} = 7.3 \times 10^{-5} \text{ inches}$$

Headloss through carbon column (Cont'd)

Flow (Igpd)	Bulk Velocity (ft/sec)	Headloss (inches)
500	6.6×10^{-4}	7.3×10^{-5}
750	1.03×10^{-3}	1.1×10^{-4}
1000	1.4×10^{-3}	1.6×10^{-4}
5300	0.007	3.3×10^{-4}

Headloss Through Carbon Column Exit



Area of overflow screen

$A = \pi D h$

$= \pi\ 2.5\ (9)/144$

$= 0.49\ ft^2$

Area available for flow = 0.3 (0.49)

$= 0.15\ ft^2$

$$V = \frac{Q}{A} \qquad h = \beta (w/b)^{4/3}\ h_v$$

$$= 5.5292\ \frac{V^2}{2\ g}$$

Flow rate (Igpd)	Velocity through screen (ft/sec)	Headloss (inches)
500	0.006	4.0×10^{-5}
750	0.009	8.9×10^{-5}
1000	0.012	1.6×10^{-4}
5300	0.067	0.005

Headloss Through Miscellaneous Fittings

(From Fair, Geyer & Okun, 1971)

$$h = \frac{kV^2}{2\ g}$$

For a 90^0 elbow, k = 0.5 - 0.1

For a 90^0 takeoff tee, k = 1.5

For a coupling, k = 0.3

For a gate valve, k = 0.5

Flow (Igpd)	Fitting type	Diameter (inches)	Number (coefficient)	Headloss• (inches)
500 750 1000 5300	Bulkhead fittings	1	3 (0.3)	0.005 0.014 0.018 0.6
500 750 1000 5300	Bulkhead fittings	3/4	2 (0.3)	0.010 0.023 0.042 1.2
500 750 1000 5300	90^0 elbow	1	1 (0.75)	0.004 0.009 0.016 0.54
500 750 1000 5300	90^0 takeoff tee	1	1 (1.5)	0.008 0.019 0.032 1
500 750 1000	Gate valve	1	1	0.003 0.006

Summary Table of Headlosses

Flow rate l/day (Igpd)	Headloss cm (inches)
2275 (500)	0.12 (0.05)
3410 (750)	0.28 (0.11)
4546 (1000)	0.66 (0.26)
24093 (5300)	14.5 (5.73)

APPENDIX F

CALCULATION OF TUBE SETTLING AREA

Calculation of Clarification Area in Tube Settlers

(After Mendis, 1976)

Tubes - 2 inches square

- 24 inches long

There are 9 tubes along the length and 4 in the width. For a total of 36 tubes.

Number of tubes per square foot of plan area = 36

Inclined settling surface = 2 x 24 inches

$= 48 \text{ in}^2$

Tube settling area:

$$\frac{48}{144} \times 36 \frac{\text{ft}^2}{\text{ft}^2} = 12.3 \text{ square feet}$$

$$= 1.13 \text{ m}^2$$

APPENDIX G

PRE-STARTUP

APPENDIX G. PRE-STARTUP

Structural

The 2 cm deflection noted at the rear wall was caused by the lack of longitudinal bracing at the base. The central vertical member then acted essentially as a cantilever, pinned at the top and free at the bottom.

Extra bracing was added to the wall, using two structural members. A 15.25 cm (6 inch) "H" steel section was installed at the base of the wall to take the major portion of the thrust. This member was bolted to tongues of 1.25 cm (1/2 inch) steel strip which had been welded to the corner angles. The "H" section acted as a simple beam point loaded, by the vertical wall member, at the center.

Hydraulic Considerations

At the base of the wall common between the aeration and digestion tanks, the interconnecting fitting was the source of a very persistent leak. The proximity of the shoulder of the fitting to the bottom triangular molding caused the liner material, which was somewhat oversized, to fold. In order to alleviate the leak, the wall was thickened at the orifice by a plate of 19 mm (3/4 inch) plywood on each side. These plates lifted the liner away from the wall proper and in so doing, caused it to stretch locally. A snug fit was thereby obtained, and the leak eliminated.

APPENDIX H

CALCULATION OF MASS TRANSFER COEFFICIENT AND AERATION TRANSFER EFFICIENCY

Mass Transfer Coefficient

The determination of the mass transfer coefficient was carried out using the method of Kayser (1969) and Fick's Law. The pertinent equations are used for design purposes in Appendix B. Figures H-1 to H-3 illustrate the three measurement runs with the different aerator configurations as described in Section 5.1.3.

Calculation of Aeration Transfer Efficiency

$$\frac{\text{lb } O_2}{\text{HP-Hr}} = x k_1 a \, V \, C \bigg/ \frac{P_A Q_A}{33000} \ln\left(\frac{H+339}{33.9}\right)$$

where:

k_1a = mass transfer coefficient (hr^{-1})

V = volume of water (l)

C = concentration of oxygen (mg/l)

x = 7.205×10^{-6} (lb/mg)

P_A = atmospheric pressure (psf)

Q_A = air flow rate (cfm)

H = bubble rise (ft)

HP = horsepower dissipated

$$= \frac{P_A Q_A}{33000} \ln\left(\frac{H+339}{33.9}\right)$$

Run #1: (See Figure H-1)

k_1a = 1.02 hr^{-1}

H = 3.17 ft

C = 6 mg/l

$$\frac{\text{lb } O_2}{\text{HP-Hr}} = 0.11 \frac{\text{lb } O_2}{\text{HP-hr}}$$

Q_A = 1.7 cfm

V = 2118 l

Figure H-1 Determination of k_La in Aeration with 1/4" orifices, Test No 1

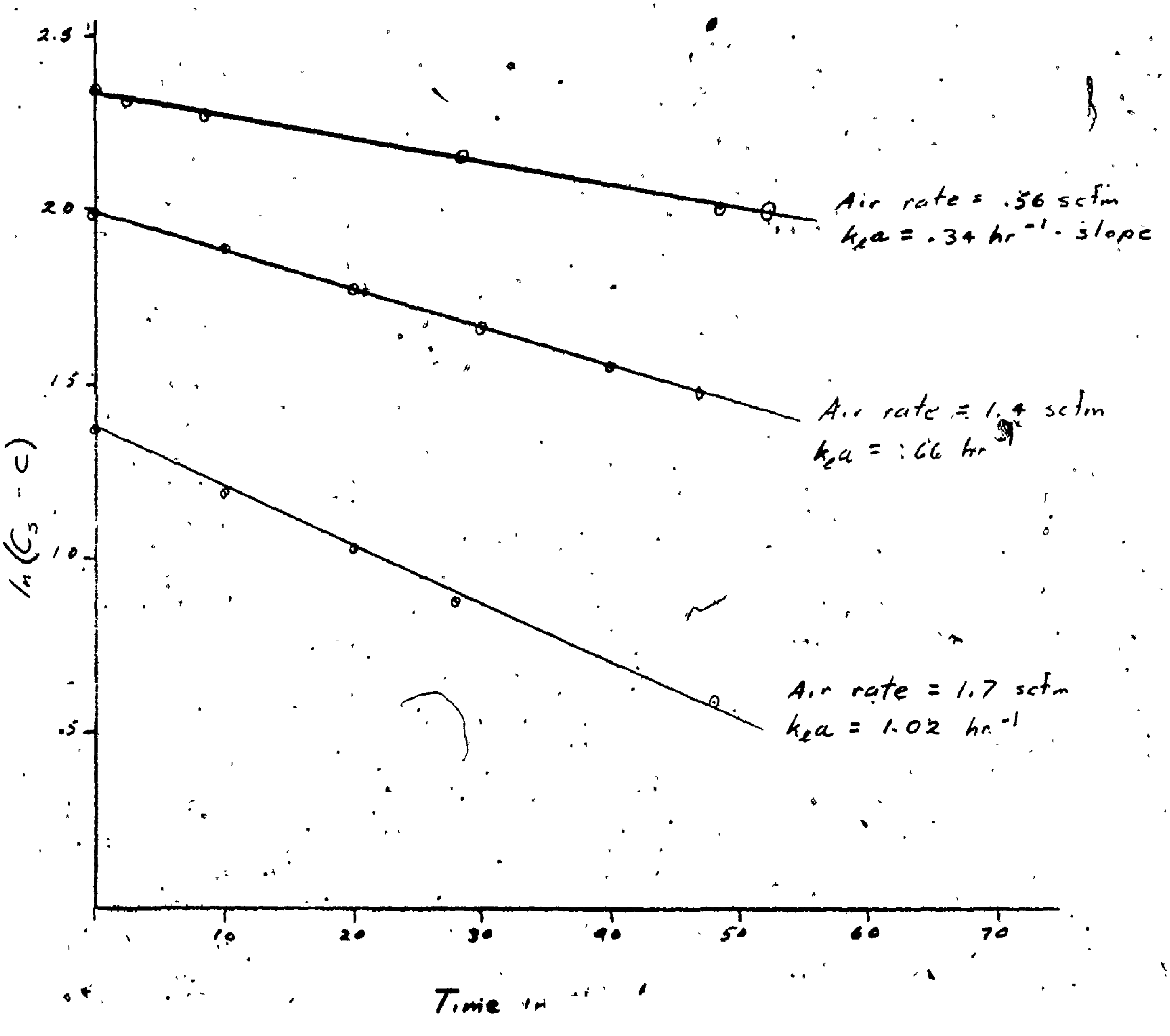


Figure H.2 Determination of k_La in Aeration Tank with 1/16" orifices Test No 2

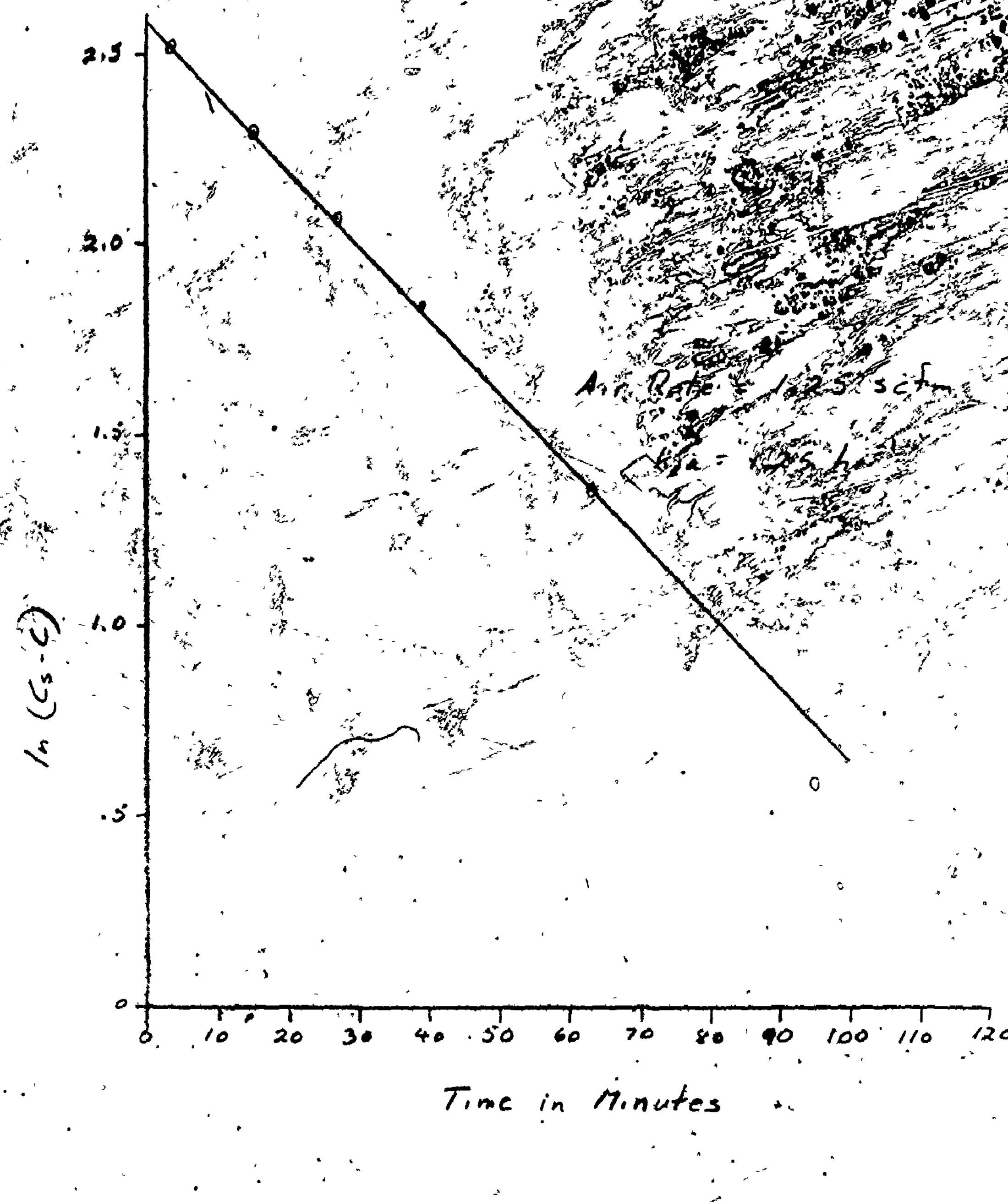
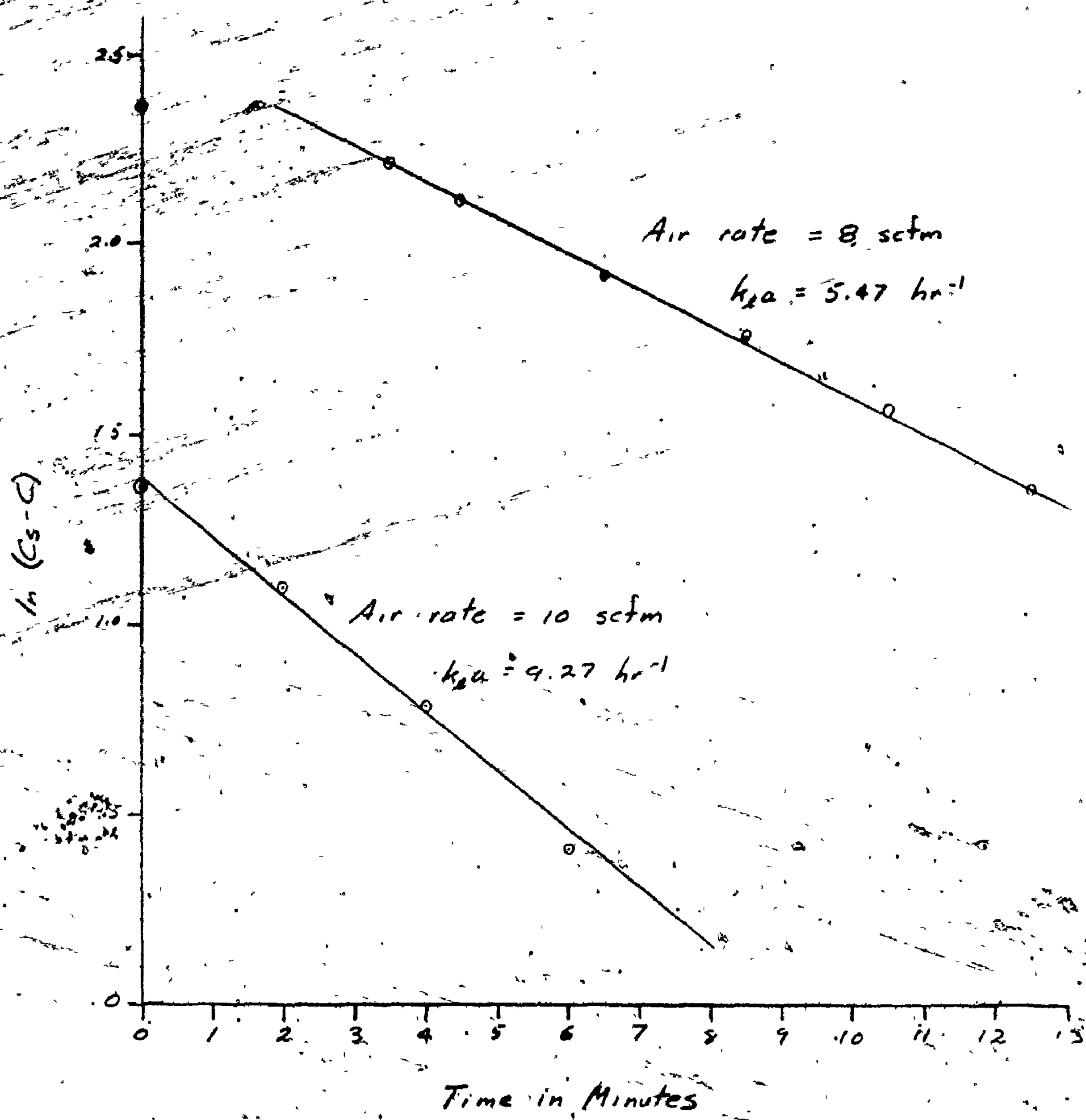


Figure H.3. Determination of k_La in Aeration Tank

Test No. 3, (90 psi air supply)



Run #2: (See Figure H-2)

$k_1a = 1.25$

$C = 8$

$Q_A = 1.25$

$V = 2339$

$P_A = 2116$

$H = 3.25$

$$\frac{\text{lb } O_2}{\text{HP-hr}} = 0.21$$

Run #3: (See Figure H-3)

$k_1a = 5.47$

$C = 6$

$Q_A = 5$

$V = 2389$

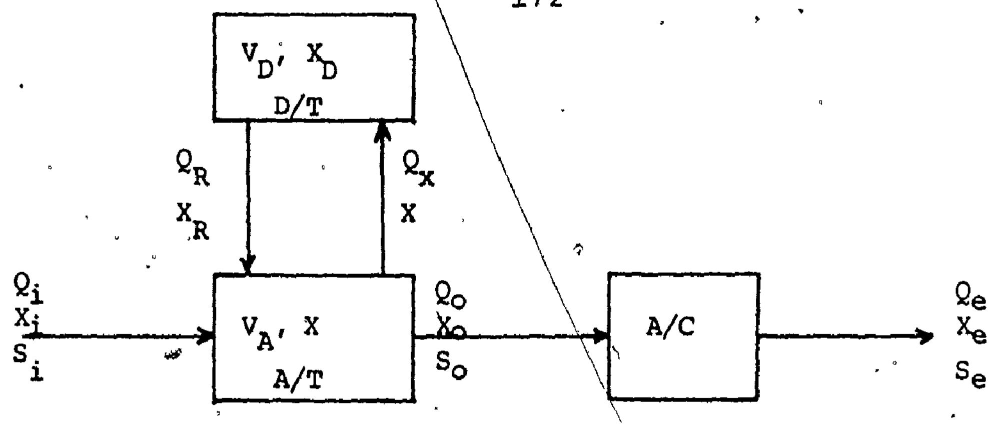
$P_A = 2116$

$H = 3.25$

$$\frac{\text{lb } O_2}{\text{HP-hr}} = 0.23$$

APPENDIX I

OVERALL YIELD CALCULATION



Substrate Mass Balance

$$\text{Food out} + \frac{ds}{dt}(A/T) + \frac{ds}{dt}(D/T) + \frac{ds}{dt}(A/C) = \text{Food in}$$

$$Q_e S_e + (Q_i S_i - Q_o S_o) + (Q_x S_x - Q_R S_R) + (Q_o S_o - Q_e S_e) = Q_i S_i$$

Biomass Mass Balance

$$Q_e X_e + \frac{dX}{dt}(A/T) - \frac{dX}{dt}(D/T) + \frac{dX}{dt}(A/C) = Q_i X_i$$

Assume $X_i = 0$

$$\therefore \frac{dX}{dt}(A/T) + \frac{dX}{dt}(A/C) = \frac{dX}{dt}(D/T) - Q_e X_e$$

Let $\frac{dX}{dt}(A/T) + \frac{dX}{dt}(A/C) = \frac{dX}{dt}$

$$\frac{dX}{dt} = \frac{dX}{dt}(D/T) - Q_e X_e$$

Assume $\frac{ds}{dt}(D/T) = 0$

$$\therefore Q_e S_e + \frac{ds}{dt}(A/T) + \frac{ds}{dt}(A/C) = Q_i S_i$$

Assume, conservatively, that all soluble substrate removal in the carbon column goes to biomass.

Therefore:

$$\frac{ds}{dt}(A/T) + \frac{ds}{dt}(A/C)_{\text{soluble}} = \frac{ds}{dt}$$

The equation for overall system yield becomes

$$\frac{dX}{dt} = Y \frac{ds}{dt}$$

$$\therefore \quad Y \frac{ds}{dt} = \frac{dX}{dt} (D/T) - Q_e X_e$$

Table I-1 shows the calculation of the overall yield coefficient for the entire system over the experimental period.

Table I-1

Calculation of Overall Yield

Run #	$\frac{ds}{dt}$ (g)	$\frac{dX}{dt}$ (D/T)	$Q_e X_e$	Y
2	4800	174	54.6	0.02
3	4965	112	62.7	0.21
4	1525	138	18.2	0.08
5	2174	160	35.6	0.06
6	2740	80	40.9	0.01
7	979	69	18.2	0.05
Overall yield	17183	1613	230.2	0.08

APPENDIX J

ASSEMBLY PROCEDURE

Assembly Procedure

NOTE: When assembling the WatRek unit, refer to Appendix N for detailed system drawings.

Structural

1) Place panel 6 in desired position.

2) Depending upon ease of further installation, place either panel 5 or 7 in position and drop in bolts.

   NOTE: a) wooden skin of panel 6 fits <u>inside</u> corner angles at corners 5 and 6.

   b) do not put nuts on any structural bolts until all structural assembly has been completed.

3) Place tie bars.

   NOTE: If panel 7 was chosen to be first side panel installed, insert tie bars through holes and install nuts and washers outside panel 7, leaving opposite end free. If panel 5 was first side panel installed, leave tie bars out until later in assembly sequence, *ie.* step 6.

4) Install rear floor section. Be sure all uprights of floor are vertical.

5) Install forward floor section. There is no need to fasten forward floor section to the rear. However, they should fit snugly.

6) Place remaining side panel (either 5 or 7) and drop in

7) Place panel 3 in position.

NOTE: a) at upper section wood skin goes inside corner 4.

b) lower section is double walled and sandwiches the corner angle.

c) place panel 3 exactly in position but do not secure.

8) Install panels 1 and 2, *ie*. place in position.

NOTE: Do not secure.

9) Spread panels 5 and 7 outward enough to place wood skin of panel 4 wholly between the corner angles.

10) Close panels 5 and 7 upon panel 4.

NOTE; a) this is a snug fit.

b) as before, the wood skin fits inside corners 5 and 7.

c) panels 1 and 2 fit on the digestion tank side of corner 1.

d) beware of panel 3 .

11) Install remaining structural bolts.

12) Install members 1, 2 and 3.

13) Put nuts on all structural bolts.

14) If not already done, install tie bars by pushing through holes in bottom angle of panel 7 and retrieving and inserting into holes in panel 5 by way of access holes in panel 5.

15) Install all stove bolts used for fastening wooden skins to the various corners.

PANEL	to	CORNER(S)
2		2 & 1
1		2 & 1
		3 & 7

NOTE: a) note recessed holes around bottom of B/W tank.

16) Install wood screws along bottom angle of panel 3.

17) Install moldings in corners 5,6 and 7.

Liners

18) Be sure all three tanks are clean and free from splinters and other sharp items which could puncture the liners.

19) Unroll aeration tank liner in the aeration tank. Noting the end tab markings for ease of positioning.

20) Beginning at corner 2, with fastener strip A-1, lift liner top into place, insert bolts with washers through fastener strip liner and top angle iron, in that order. Place nuts outside top angle and tighten moderately.

21) Proceed in a clockwise direction until corner 8 is reached.

22) The aeration tank liner folds over the dividing wall. Fold it over and secure it along the top of the wall with thumb tacks. Take care to line up holes precisely.

23) Move to digestion tank and unroll liner as done in the aeration tank.

24) Find return weir sleeve in top of dividing wall portion of the liner and slip over the return weir.

25) Starting with fastening strip D-5, proceed in a clockwise direction from the back of the digestion tank, to the point where, along the dividing wall, strips in both the aeration and digestion tanks have common bolts.

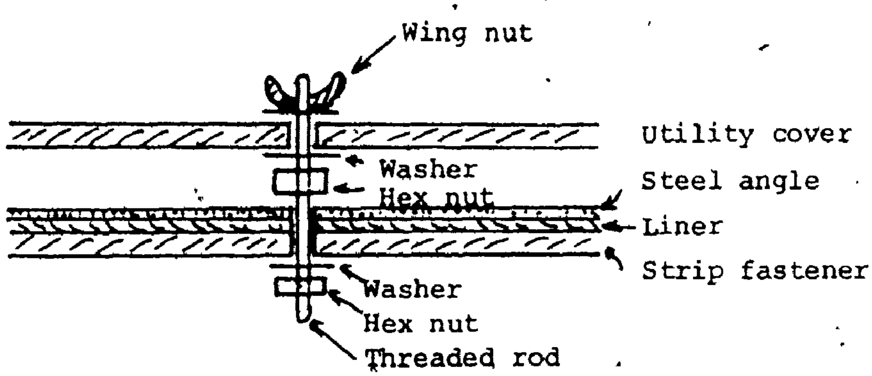
26) Install the fastening strips along the dividing wall.

27) Install the PVC gasket at the return weir.

28) Tighten all bolts holding the liners in place.

29) Place backwash tank liner in place.

30) Install holding strips as marked noting that in holes where utility cover will be fastened, the 3" threaded rod pieces are used as fasteners. See diagram.



Plumbing

31) Place gaskets on protruding nipples of bottom fittings of the carbon column.

32) Place the carbon column inside the aeration tank and carefully move into position.

NOTE: a) care must be taken not to tear the liner.

b) liner must be as smooth as possible.

33) When column is in place, apply tubing to the bulkhead fittings protruding through the panel 5 and carbon column.

34) Install clarifier outlet in panel 5 and install check valve and various other fittings for line from clarifier to carbon column.

35) Install bulkhead fittings as indicated below:

a) 2" inlet in panel 5, near corner 5.

b) carbon column overflow in panel 3.

c) backwash tank drain and overflow in panel 4.

d) digestion tank drain in panel 4.

36) Install close nipples in all the above fittings except the inlet. Install gate valves in the drain fittings in panel 4 and solenoid valve on carbon column overflow.

NOTE: Be sure solenoid is oriented in the proper direction with the "in" end toward the wall.

37) Install inter-tank fitting in dividing wall with flange on digestion tank side and install the close nipple.

38) Take the remaining gate valve and remove the spindle. Attach valve casing to the close nipple and when it is snug and in the vertical position, replace spindle.

NOTE: Be sure spindle will close the gate all the way after it has been replaced.

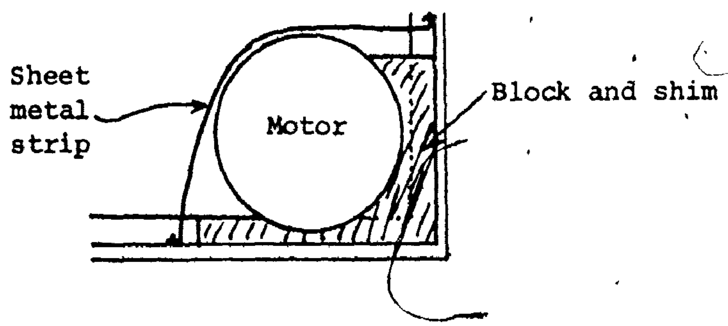
39) Place valve stem forks between spokes of valve spindle and screw bracket to liner strip fastener.

NOTE: a) be sure valve stem is in bracket when it is being screwed on.

b) test valve stem to be sure it is operating valve.

c) circular ring on valve stem fits below strip holder so that stem cannot be lifted out when plant is in operation.

40) Install backwash pump in backwash tank. See diagram.



NOTE: Be sure pump intake is resting on the floor of the tank.

41) Install level switch activators.

NOTE: Both floats must be attached.

42) Install backwash line from pump to backwash inlet of carbon column.

NOTE: Fitting sequence for entry to carbon column is included in fitting box.

43) Install 1" pipe to hose fitting inside aeration tank in clarifier outlet fitting. Put hose on fitting and clamp. Tie hose to top of the tank.

Aeration

44) Place perforated aeration tubes in aeration and digestion tanks, passing them through the loops at the bottoms of the liners.

NOTE: Place plugs in end of each tube.

45) Run air delivery lines for both aerators through the loops provided. Also, place the air lift line being sure to have

air bleed "T" just below the high point on the downcomer into the digestion tank.

46) Install the utility cover.

47) Install the compressor. Bolt it to the utility cover.

48) Run air delivery line from compressor to pressure tank.

NOTE: a) pressure tank is to be placed in a convenient position on the utility cover and need not be bolted down.

b) the outlet from the compressor is a "T" fitting. The delivery line goes from one arm and a pressure gauge is installed on the other.

49) Hang manifold plate on screws provided in panel 3.

50) Connect hoses to manifold.

NOTE: a) air lift hose goes on manifold arm equipped with a solenoid valve.

b) use hose clamps.

51) Hang backwash timer on screw provided in panel 1. Secure further with two screws through bottom rear of timer housing.

52) Hang main box on screws provided in panel 2.

53) Connect all wires to units as indicated on wire ends, using Marr connectors in handy boxes.

54) Close all handy boxes with sheet metal screws provided.

55) Secure wires to bulkheads with wire clamps.

Clarifier

56) Attach hanging brackets and guide posts to the clarifier guideway template by means of 1 1/2" x 1/4" bolts and hex nuts.

57) Hang clarifier guideway template in position near the activated carbon column and secure to the walls of the tank.

58) When water level is within a few inches of the bottom of the guideways, attach outlet hose (sitting at top of aeration tank wall, see step #43) to clarifier fitting and clamp in place.

59) Clarifier must have been previously adjusted in a separate tank for desired overflow rate using dry sand. Now place clarifier in aeration tank and position within guideways. Carefully replace sand so that clarifier floats in desired position.

NOTES

a) Activated carbon column must be charged with carbon before any liquid is introduced.

b) During filling, the inter-tank valve must be opened. It must, however, be closed during operation.

60) Check out systems, install air lift pulse timer.

APPENDIX K

DISASSEMBLY PROCEDURE

Disassembly Procedure

NOTE: When dismantling the WaTRek unit, refer to Figure N-1 for the locations of various panels and corners.

1) Drain all tanks and clean liners. Remove clarifier.

Electrical

2) Disconnect electrical feed from 110 V power supply.
3) Disconnect 2 solenoids and backwash pump at the units leaving cables attached to timer and main box.
4) Disconnect backwash timer from main box at the timer.
5) Remove all components. Leaving small air lift solenoid on the aeration manifold.

Aeration

6) Disconnect air lines from distribution manifold exit.
7) Remove air lines from all tanks.
8) Disconnect manifold inlet and compressor outlet and remove pressure tank.
9) Release compressor feet from utility cover and lift compressor and feet away.
10) Lift away the manifold plate.

Plumbing

11) Remove exterior plumbing at lower panel 5 near corner 4. Remove fitting flanges. Remove backwash pump and lines.

12) Remove solenoid and bulkhead fitting from carbon column overflow.

13) Carefully, without damaging liner, remove carbon column by sliding toward digester tank and then lifting out and over the wall. Dump the carbon out as the column tips over the wall.

14) Remove remaining bulkhead fittings from panels 5 and 2, including inter-tank valve stem.

Liners NOTE: LINERS MUST BE CLEAN AND DRY BEFORE REMOVAL

15) Remove nuts from liner fasteners in aeration tank and digestion tank. Be sure to remove inter-tank valve stem.

16) Enter aeration tank and quickly remove all holding strips from the bag, letting the bag fall inward. Do the same for the digestion tank.

17) Roll and tie bags from high to low end and mark end tab of roll.

18) Remove utility cover.

19) Remove backwash pump and assorted plumbing and bulkhead fittings in tank.

20) Remove liner holding strips and liner from backwash tank.

21) Remove backwash tank, bottom insert.

Structural

22) Remove moldings from corners 1,5,6 and 7.

23) Remove members 1,2 and 3. NOTE that member 3 fits below protrusion at corner #1.

24) Remove nuts from tie bars.

25) Remove bolts securing:
  a) wooden skin in panel 6 to corners 5 and 6;
  b) wooden skin of panel 3 to corner 4 and wood screws securing panel 3 to floor;
  c) wooden skin of panel 4 to corners 3 and 4;
  d) panels 1 and 2 to corner 1.

26) Slacken all structural bolts (1/2" diam. steel).

27) Remove structural bolts from corners 3 and 7.

28) Spread panels 5 and 7 outward at the forward end. As panel # 4 comes clear of corners 3 and 7, lift it away.

NOTE: Front face is now open giving easy access to the digestion and backwash tanks.

29) Remove bolts securing panels 1 and 2 to corner 2.

30) Remove panels 1 and 2.

31) Remove structural bolts from corner 4.

32) Remove panel 3.

33) Depending upon location remove structural bolts from either of corners 5 or 6 in order to remove panels 5 or 7, respectively.

34) Lift out forward floor panel.

35) Lift rear floor panel.

36) Retrieve tie bars.

37) Remove remaining structural bolts and carry away remaining two panels.

APPENDIX L

INVENTORY OF ITEMS

Inventory

Electrical

1 Master control box:

- 3 DPST toggle switches
- 1 fuse (15 amp)
- 1 timer case & 30-second timer
- cords & connectors

1 3/4" normally open solenoid switch

1 7-day timer with two riders

1 Small electrics box - white:

- 3-15 amp fuses
- 3 packets handy box screws
- 6 Marr connectors
- assorted cable clamps and wood screws

Plumbing

1 Sump pump

1 Box of sump pump fittings at pump end:

- 1 1" bolt (1/4") with hex nut
- 1 1/2" x 1/4" bolt
- 8-ft hose
- 1 x 1 1/4" pipe to hose fitting (pvc)
- 1 hose clamp
- 1 1 1/2" thick wooden block

- 1 1/4" pvc shim
- 1 sheet metal strap
- 1 set of floats and weights for pump level activator.

1 Box of backwash fittings:

- 1 x 1" 90° elbow (black)
- 1 x 1" gate valve } brass
- 1 x 1 1/4" check valve } brass
- 1 x 1 1/4" + 1" bushing (black)
- 2 x 1" close nipples(black)
- 1 x 1" short nipple (black)
- 1 x 1 1/4" pvc hose to pipe fitting
- 1 x 2" pvc bulkhead fitting
- 1 layout of entrance to A/C column.

1 Box of bulkhead fittings and drain valves:

- 3 x 1" gate valves (brass)
- 1 x 1/2" pvc bulkhead fitting
- 4 x 1" pvc bulkhead fittings
- 2 x 1" close nipples (black)
- 1 x 3/4" pvc bulkhead fitting
- 1 x 3/4" close nipple (black)
- 1 x 1" long nipple (black)
- assorted gaskets (rubber)
- 1 x 6 ft stem for deep valve.

Clarifier

1 Floating clarifier

1 guide template and 4 brass guides

6 ft clarifier discharge hose with appurtenances

1 Box clarifier fittings:

- 1 x 90°, 1" brass elbow
- 2 x 1" close brass nipples
- 1 x 90°, 1" nylon elbow
- 1 x 1" pipe to hose fitting (pvc)
- 3" Tygon tube, 1" bore
- 1 x 1" pvc bulkhead fitting
- 1 x 1" nylon pipe to tube union
- 1 x 1" check valve.

Aeration

Air lift

- Approx. 2 ft, 1/2" nylon tube
- Approx. 6 ft, 1/4" thickwall Tygon
- Approx. 6 ft, 1/2" Tygon
- 1 x 1/2" nylon "T"
- 1 large bore glass "T"
- 3" copper tube, 1/2"

1 Box Swagelock fittings

- 1 x 1 1/4" "T"
- 1 x 1/4" → 1/8" brass bushing

- 1 pressure gauge
- 2 x 1/4" + 3/8" unions
- 1 copper elbow, 1/4"

1 Box hose clamps and fittings:

- 2 thumbscrew hose clamps
- 7 slotted screw hose clamps
- 2 brass tube inserts
- 1, 1/4" tube to 1/4" pipe fitting

Approx. 10 ft, 3/8" ID Tygon tube

Approx. 3 ft, 1/4" copper tube

Approx. 10 ft, 3/8" ID Tygon tube

Approx. 2 ft, 1/4" copper tube

Approx. 1 ft, 1/4" copper tube

Approx. 1 x 90°, 1/2" nylon elbow

Approx. 2 ft, 1/2" nylon tube (with 1/16" holes)

Approx. 6 ft, 1/2" nylon tube (with 1/4" holes)

Approx. 2 black rubber end plugs

3 valve manifold with 1/4" solenoid

1 constant pressure tank with inlet and exit fittings and prv

- fittings are 1/4" pipe to tube

1 x 1/4 HP Gast compressor, Model #0322-P102-G18D, Serial No. 0776

Liners

1 Backwash storage tank vinyl liner

1 Aeration tank vinyl liner

1 Digestion tank vinyl liner

1 bundle - 4 strip holders for Backwash storage tank

1 bundle - 9 strip holders for Aeration tank

1 bundle - 9 strip holders for Digestion tank

1 PVC gasket set

1 box liner holder fasteners

1 package of thumb tacks

Structural

1 Utility cover

- 1 precut and fitted cover
- 1 box fasteners

1 A/C column with fittings attached

- screen holder

1 B/W tank floor insert

Panel #1 Approx. 2 ft x 8 ft, 3/4" plywood

#2 Approx. 4 ft x 8 ft, 3/4" plywood

#3 Front face of unit, steel and plywood

#4 Aeration wall, steel and plywood

#5 Side panel, steel and plywood

#6 End panel, steel and plywood

#7 Side panel, 6 ft x 8 ft, steel and plywood

2 Floor sections

3 Short members, Nos. 1,2 and 3

1 Box of wood to steel bolts

1 Box of steel to steel bolts

4 Threaded end tie bars, 1/4"

2 Boxes of odds and ends.

APPENDIX M

DETAILED COST

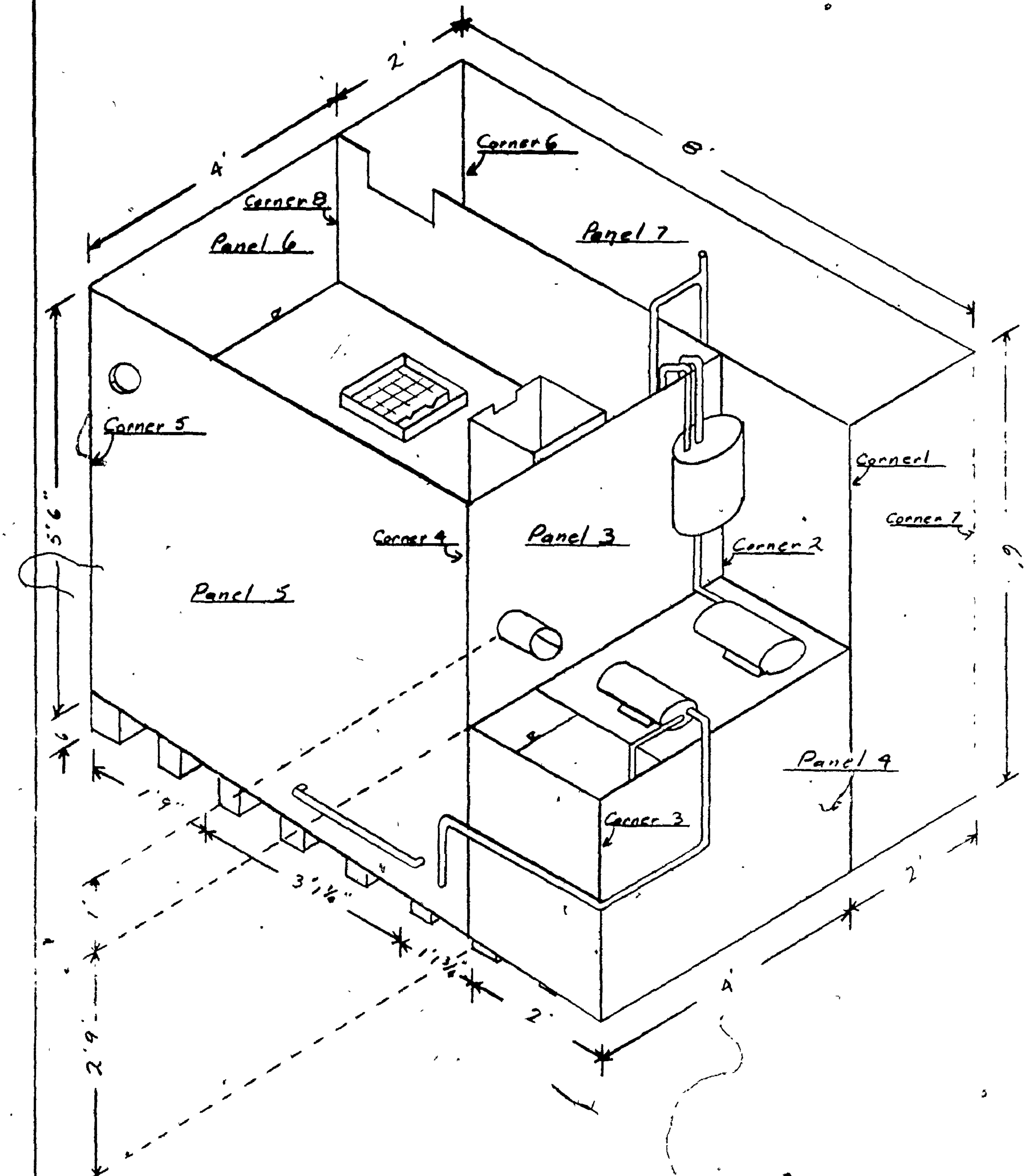
Quantity	Item	Unit price	Total cost
2	Screw jacks (Scissor)	$ 12.59	$ 25.18
5	3/8" gate valves	3.50	17.50
	Assorted Swagelok fittings	56.00	56.00
1	Air pressure gauge	5.00	5.00
1	Sump pump	54.95	54.95
10 ft.	1 1/4" hose	6.49	6.49
1	1 1/4" check valve	7.59	7.59
1 box	Nails	.29	.29
4	1" gate valves	5.29	21.16
3	1" close nipples	.52	1.56
1	1 1/4" close nipple	.49	.49
2	3/4" close nipples	.29	.58
5	1/2" close nipples	.25	1.25
1	1 1/4" PVC pipe to tube	.23	.23
8	1" PVC pipe to tube	.17	1.36
2 qt.	Epoxy paint	7.95	15.90
	Steel angle	193.05	193.05
3	PVC tank liners		150.00
1	3/4" solenoid valve (N.O.)	90.40	90.40
3.0 ft^2	Stainless steel screen		24.00
12	3/4" plywood sheets	15.50	186.00
1	1/4" plywood	6.95	6.95
			$ 865.93

Quantity	Item	Unit price	Total cost
1	Gast air compressor	$129.40	$ 129.40
1	Air pressure gauge	9.40	9.40
1	Pressure relief valve	3.10	3.10
1	Peragon 7-day timer	100.00	100.00
1	Eagle Flexopulse timer	150.00	150.00
1	Rotameter	70.00	70.00
1	1/4" solenoid (NC)	40.00	40.00
3	Handy boxes	2.00	6.00
16 ft	2 x 10" timber	.57/ft	9.12
16 ft	2 x 8" timber	.47/ft	6.72
18 ft	2 x 6" timber	.27/ft	4.86
16 ft	2 x 4" timber	.18/ft	2.88
	Assorted PVC bulkhead fittings	75.90	75.90
48 ft^2	1/8" PVC sheet	1.65/lb	79.20
16 ft^2	1/4" PVC sheet	3.30/lb	52.80
	Material costs		$1605.31
	+ 10% for unaccountables		160.53
	TOTAL COST		$1765.84

*Unaccountables include: bolts, nuts, screws, switches and miscellaneous fittings.

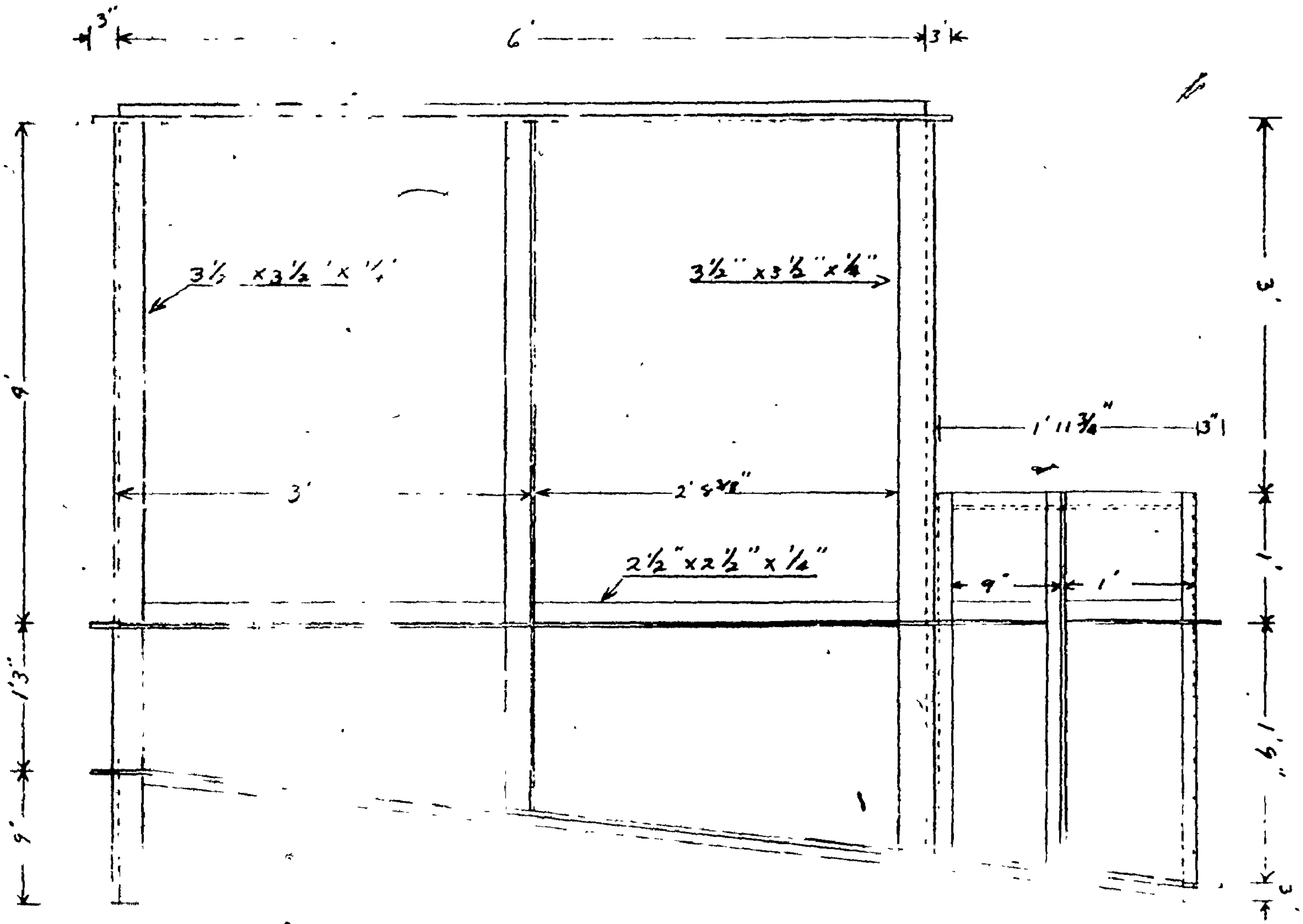
APPENDIX N

System Drawings



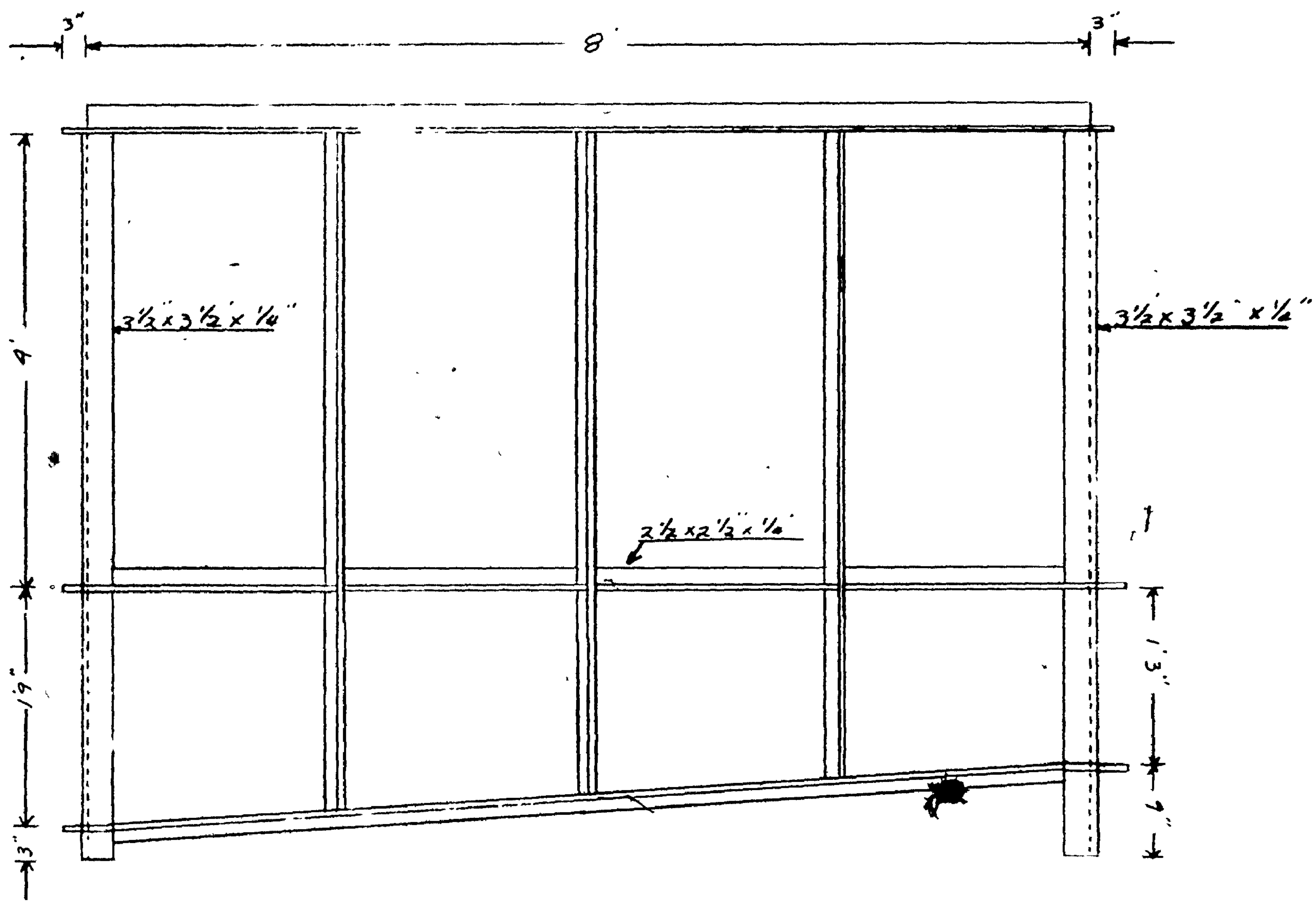
Note Not to scale

Fi



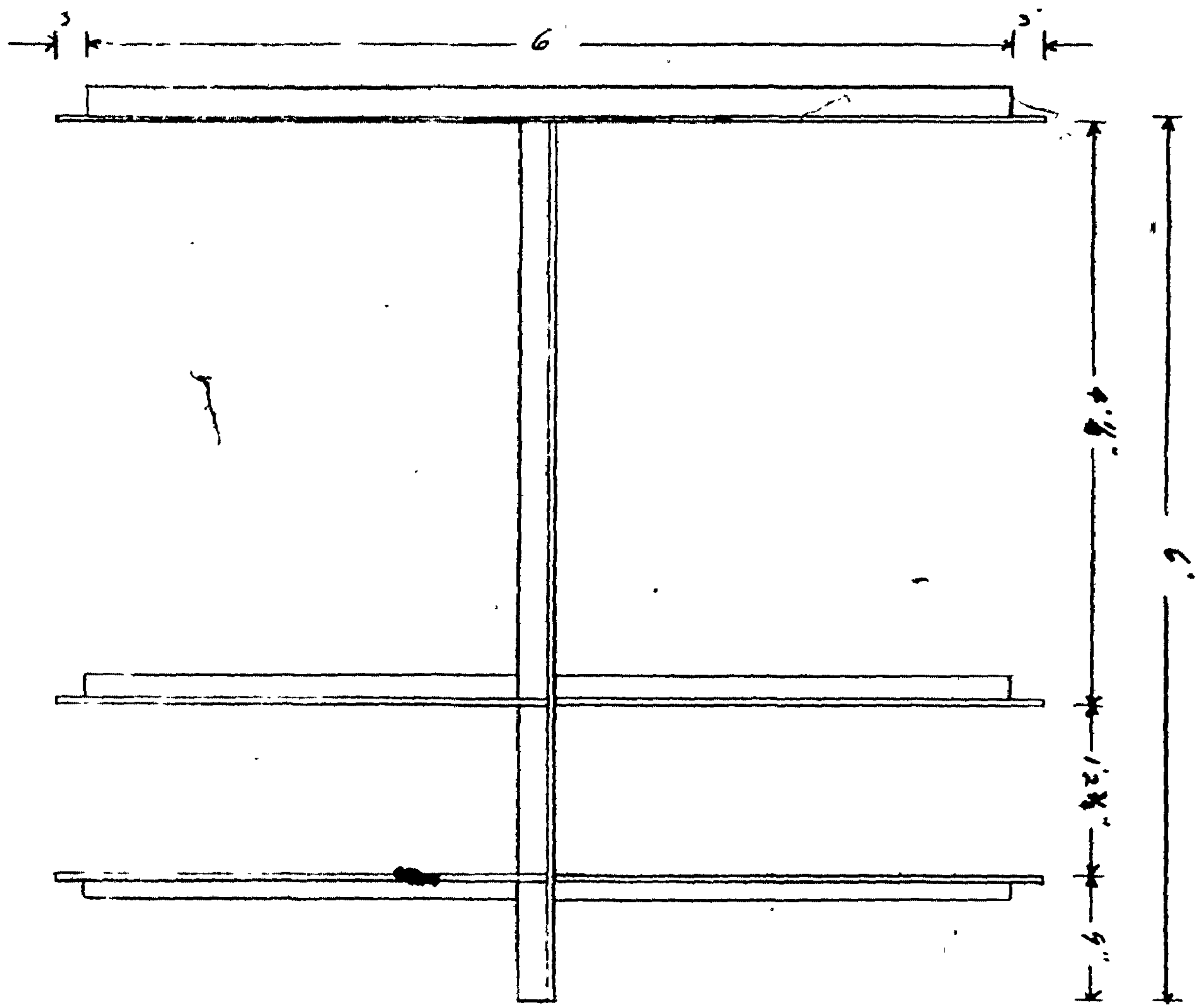
Note. All angles 2 x 2 x ¼ except as noted

Figure N.2 Panel 5



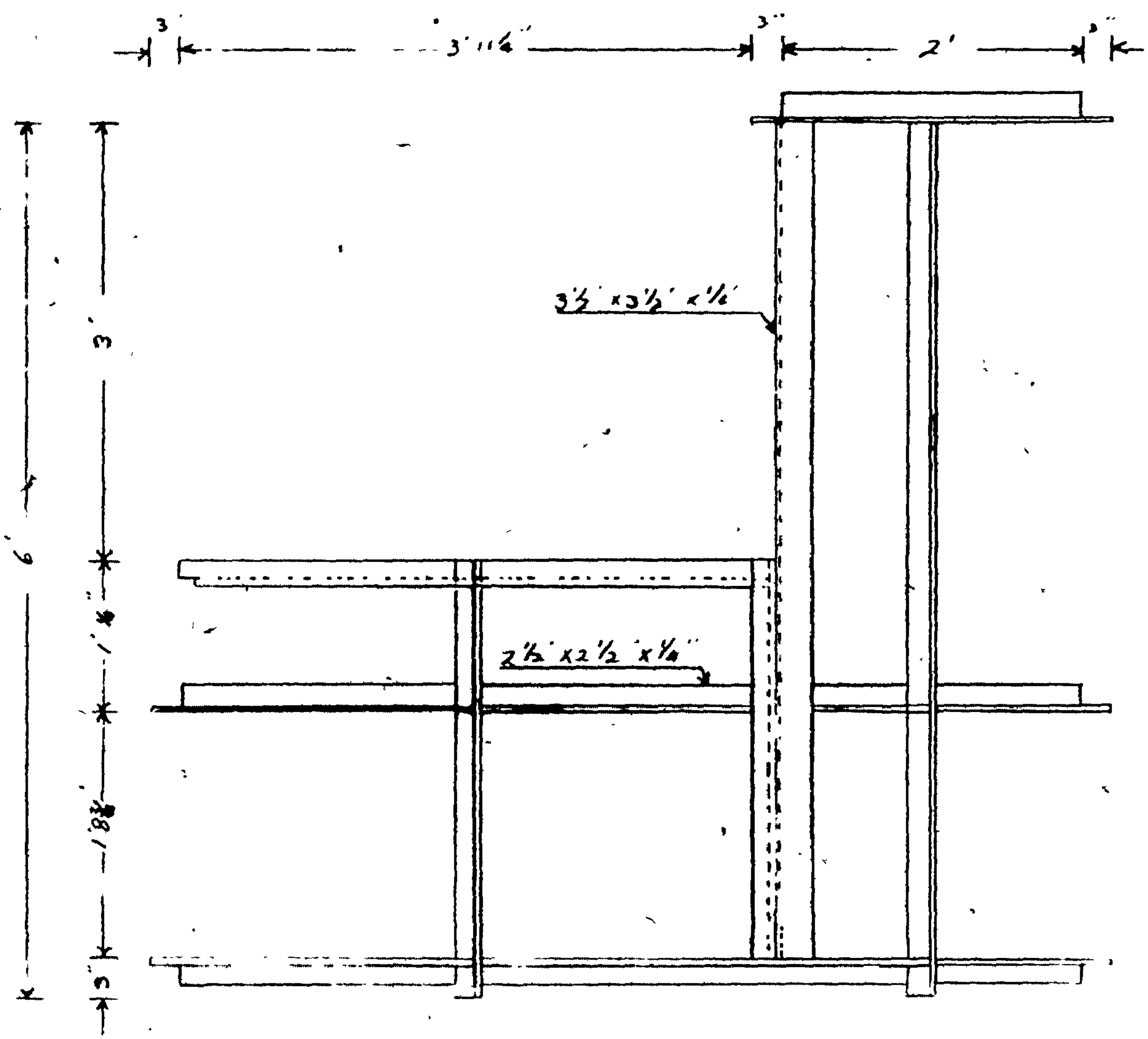
'ote All angles 2"x2"x⅛" except as noted

Figure N. 3 Panel 7



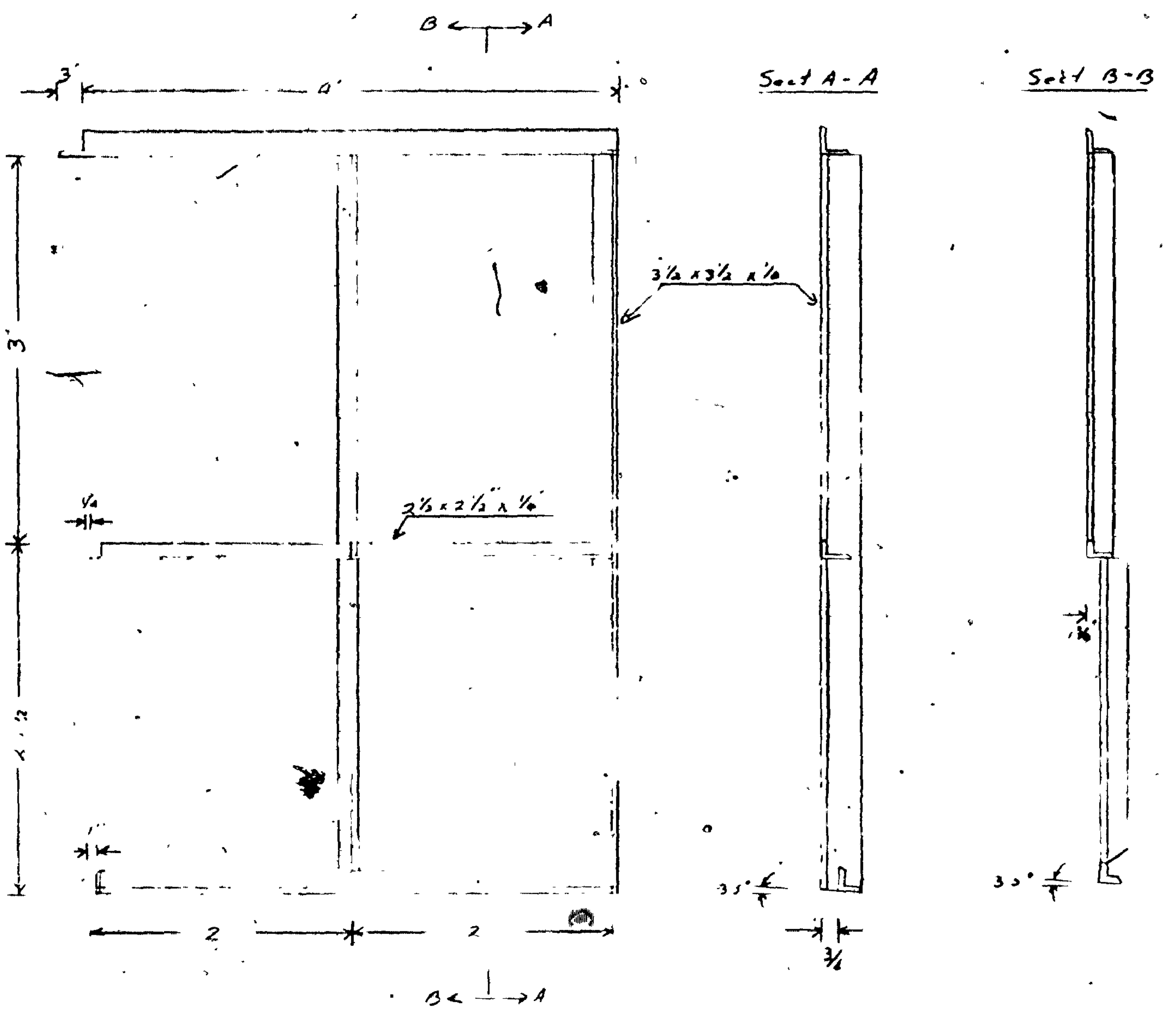
All angles 2"x2"x1/8 except as noted

Figure N.4 Panel 6



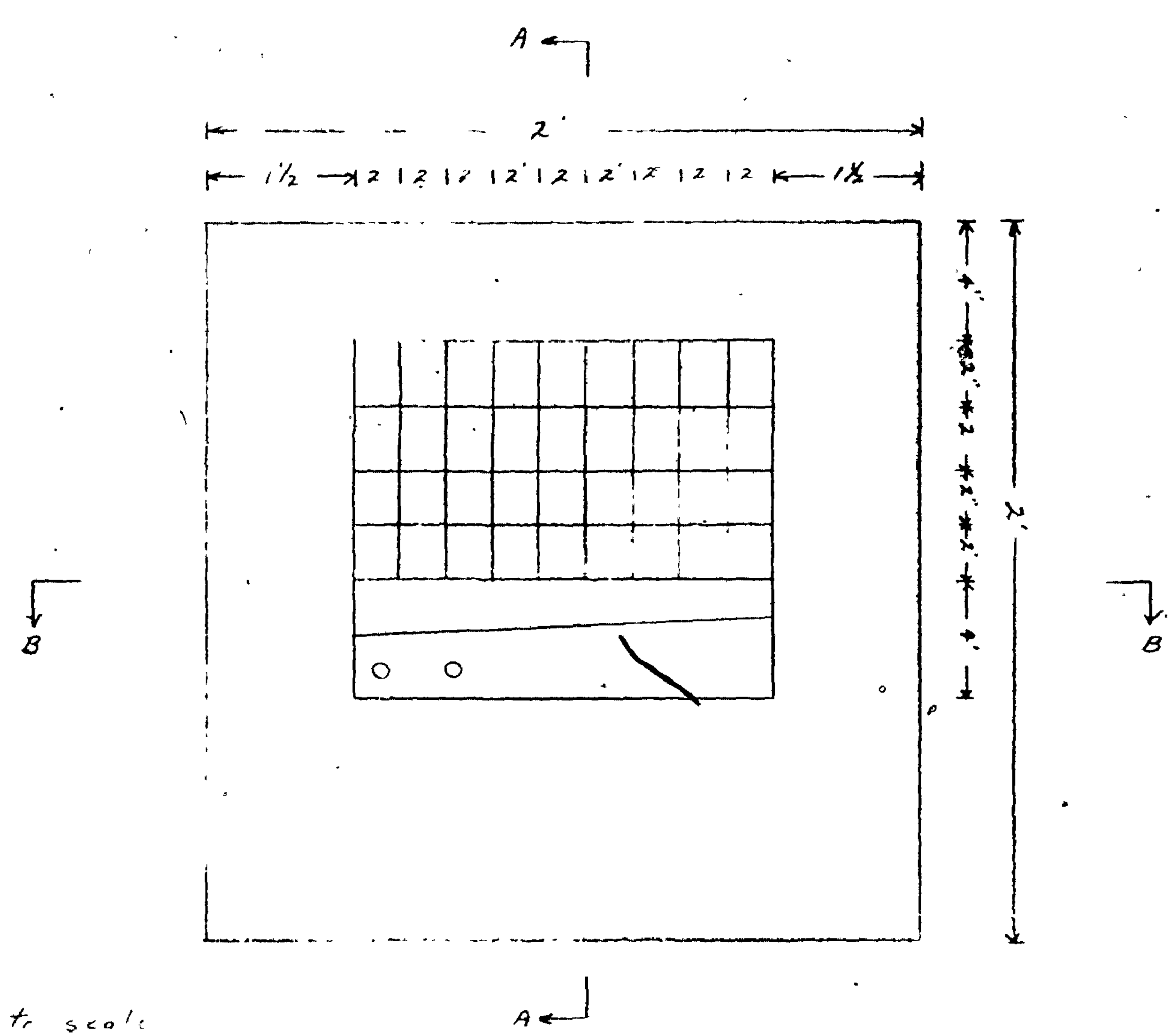
...te All angles 2" x 2" x ⅛ except as noted

Figure N.5 Panel 4



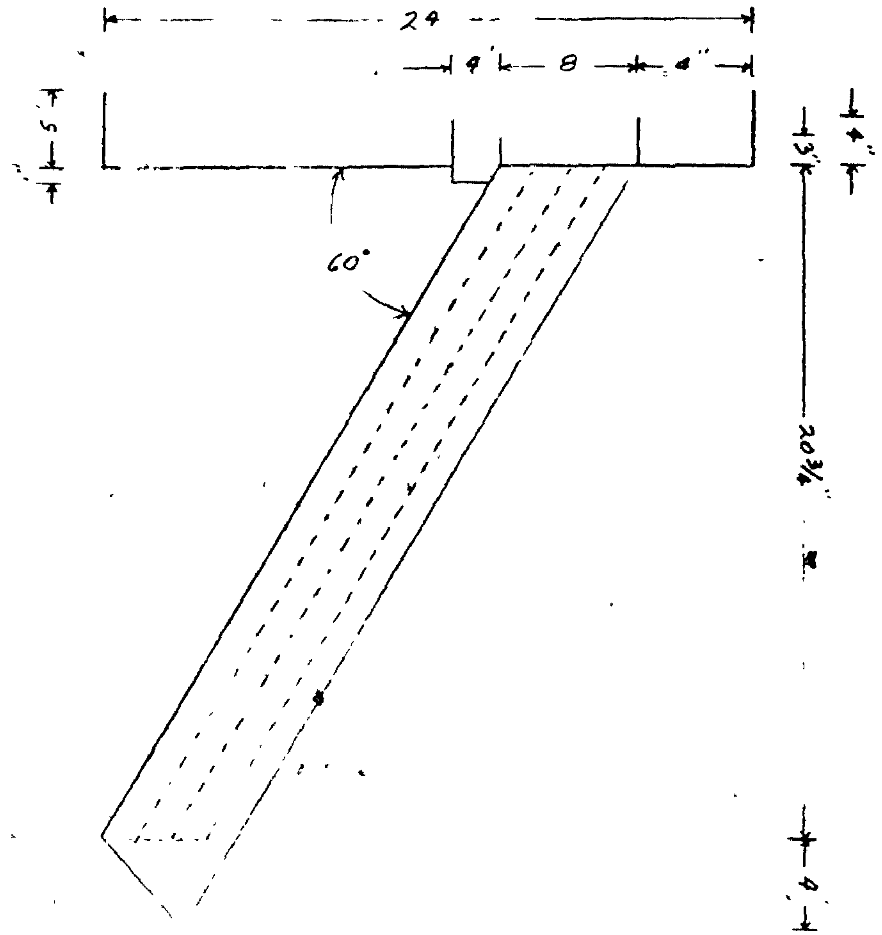
Note: All angles 2 x 2 x ⅛ except as shown

Figure N.6 Panel 3



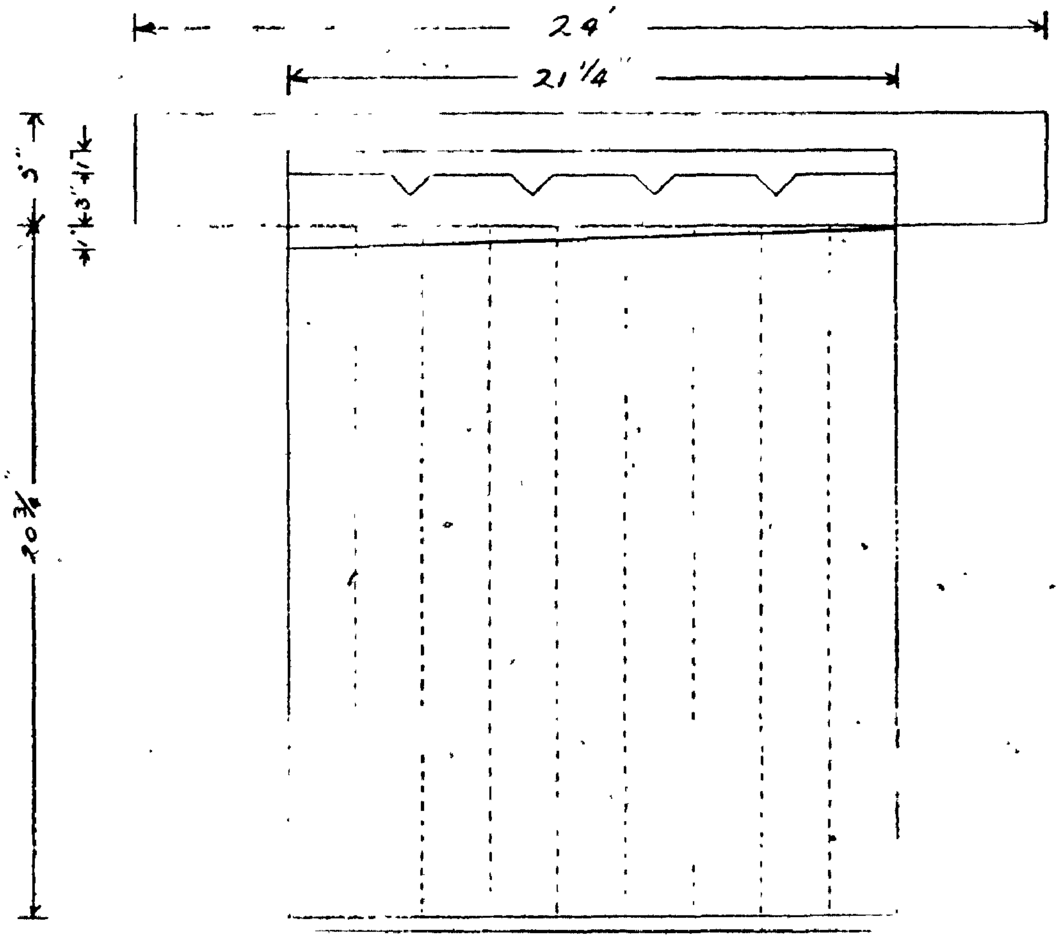
Note Not to scale

Figure No. 7 . Clarifier Plan View



Note: Not to scale

Figure No. 2 Clarifier Section A A



Notes Not to scale

Figure N. 9 Clarifier Section B-B

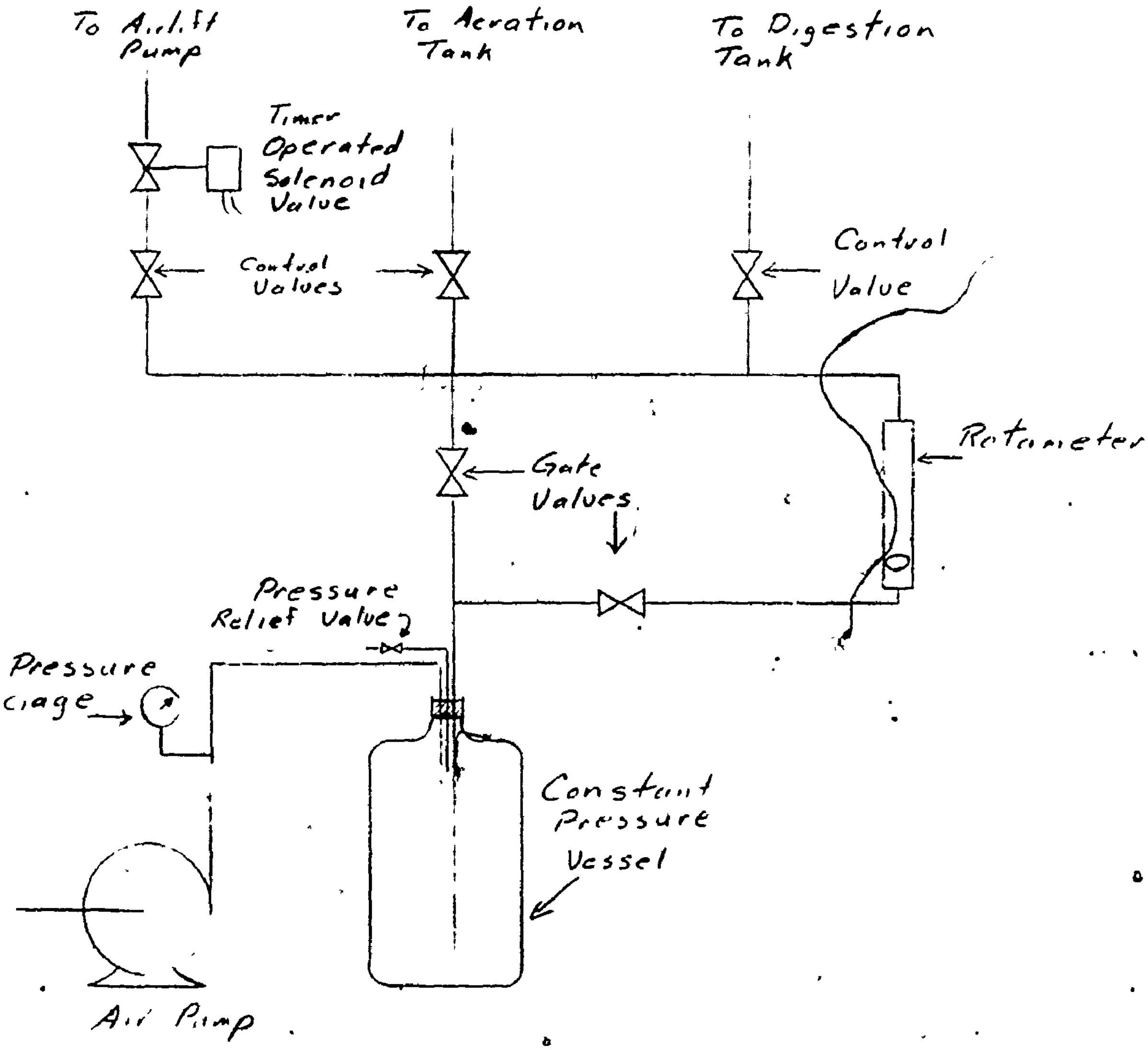


Figure N.10 : Schematic of Air Delivery System

APPENDIX O

RAW DATA

Table O-1

Run/Day		Feed			Clarifier overflow			Activated carbon overflow			
		Soluble COD (mg/l)	Total COD (mg/l)	SS (mg/l)	Soluble COD (mg/l)	Total COD (mg/l)	SS (mg/l)	Soluble COD (mg/l)	Total COD (mg/l)	SS (mg/l)	Aeration tank MLSS (mg/l)
1	1	100	449	254	65	605	536	23	27	10	2930
1	2	111	637	408	50	503	376	19	19	-	2525
2	1	68	296	160	27	35	<4	16	20	<4	3052
2	2	67	294	172	27	35	8	16	26	<4	3388
2	3	102	431	204	35	55	16	16	20	<4	3208
2	4	66	200	192	31	39	7	8	12	<4	3200
2	5	81	586	188	35	47	6	8	12	<4	3420
2	6	89	493	306	31	35	6	19	23	<4	3235
3	1	78	372	204	27	30	12	11	15	<4	3895
3	2	87	391	190	8	38	6	4	8	<4	3655
3	3	78	384	228	23	34	7	15	19	<4	3660
	4	80	380	210	24	35	<4	8	19	<4	3700
	5	42	216	126	35	35	8	4	12	<4	3620
	6	58	211	64	24	63	27	20	25	<4	3495
	7	110	370	220	20	47	32	8	12	<4	3340
	1	125	399	210	38	65	26	19	23	5	3872
	1	103	316	238	37	40	19	26	26	12	4682
5	2	78	217	170	43	47	9	23	27	10	4058

Run/Day		Feed			Clarifier overflow			Activated carbon overflow			
		Soluble COD (mg/l)	Total COD (mg/l)	SS (mg/l)	Soluble COD (mg/l)	Total COD (mg/l)	SS (mg/l)	Soluble COD (mg/l)	Total COD (mg/l)	SS (mg/l)	Aeration tank MLSS (mg/l)
6	1	131	376	120	35	77	<4	19	27	<4	4784
6	2	100	323	114	35	35	<4	15	23	<4	3856
6	3	77	227	146	27	31	<4	15	15	<4	3912
7	1	79	244	116	34	38	<4	23	23	<4	5256

Operational Parameters

Time (hr) /Run	Visit interval (hr)	Dissolved O_2			Depth (ft)		Feed flow (l/min)	Effluent flow (l/min)	Total feed (C)
		A/T	D/T	A/C	A/T	D/T			
0.0/1		2.1	7.4	0.4	3.6	3.3	1.5	1.4	
	9.5								855
9.5/1		5.6	7.7	1.0	3.75	3.3	1.6	1.3	
	13								1248
22.5/1		6.1	7.4	0.5	4.0	3.3	1.6	1.2	
	9								864
31.5/1		5.8*	7.4	1.0	4.2	3.4	1.6	1.2	
0.0/2		2.2	8.3	-	4.3	3.3	1.6	1.5	
	22.5								2160
22.5/2		0.3	7.8	0.0	3.6	3.2	1.6	1.5	
	6.5								624
29 /2		1.0	8.1	0.5	3.6	3.2	1.6	1.6	
	16								1536
45 /2		0.2	7.9	0.05	3.6	3.3	1.6	1.4	
	10								960
55 /2		0.7	8.2	1.1	3.7	3.3	1.5/1.6	1.4	
	13								1209
B/W 68/2		0.2	7.7	0.0	3.8	3.3	1.6	1.4	
	8.5								816

Operational Parameters

Time (hr) /Run	Visit interval (hr)	Dissolved O_2			Depth (ft)		Feed flow (l/min)	Effluent flow (l/min)	Total feed (C)
		A/T	D/T	A/C	A/T	D/T			
76.5/2		0.6	7.9	0.7	3.7	3.3	1.7	1.6	
	17								1632
93.5/2		0.6	8.0	0.6	3.7	3.3	1.6	1.5	
	13								1248
106.5/2		0.2	7.5	0.8	3.8	3.3	1.7/1.6	1.6	
Partial B/W	9.5								912
116 /2		0.7	7.7	0.7	3.9	3.3	1.6	1.5	
	12								1152
128/2		0.7	7.8	1.0	3.5	3.3	1.6	1.5	
	13								1248
141 /2		0.2	7.2	0.5	3.6	3.3	1.6	1.2	
	15.5								1488
166.5/2		0.2	4.9	0.2	3.8	3.4	1.5	1.6	
B/W 0.0/3		1.7	6.5	0.2	3.6/4.2	3.3	1.6		
	8								768
8.0/3		3.5	8.1	2.4	3.7	3.3	1.7/1.6	1.6	
	15.5								1535
B/W 23.5/3		2.3	5.6	0.1	3.8	3.3	1.6	1.6	

Operational Parameters

Time (hr)	Visit interval (hr)	Dissolved O_2			Depth (ft)		Feed flow (l/min)	Effluent flow (l/min)	Total feed (C)
		A/T	D/T	A/C	A/T	D/T			
	11								1056
34.5/3		3.7	7.6	2.1	3.75	3.3	1.6	1.4	
	13.5								1337
B/W 48/3		2.1	5.7	0.1	3.8	3.3	1.7/1.6	1.5	
	11								1089
59/3		3.1	7.6	2.1	3.6	3.3	1.6	1.6	
	13								1248
B/W 72/3		1.4	4.5	0.5	3.7	3.3	1.6	1.5	
	11								1056
83/3		2.0	7.1	1.7	3.6	3.3	1.6	1.5	
	15								1440
B/W 98/3		1.4	4.5	0.2	3.8	3.3	1.6	1.5	
	24								2376
B/W 122/3		3.8	5.7	2.3	3.8	3.3	1.7/1.6	1.5	
	11								1089
13.3/3		2.4	5.3	1.7	3.7	3.3	1.7/1.6	1.5	
	11								1089
B/W 144/3		2.1	4.7	0.2	3.9	3.4	1.6	1.4	
	9								837
153/3		2.4	5.1	1.7	3.7	3.4	1.5/1.6	1.4	

Operational Parameters

Time (hr) /Run	Visit interval (hr)	Dissolved O_2			Depth (ft)		Feed flow (l/min)	Effluent flow (l/min)	Total feed (C)
		A/T	D/T	A/C	A/T	D/T			
	14.5								1349
Partial B/W 167.5/3		1.6	3.4	0.25	3.9	3.4	1.6	1.4	
	9.5								912
177/3		2.1	4.8	1.0	3.7	3.4	1.6	1.4	
	15.5								1488
192.5/3		2.1	3.4	0.2	3.9	3.4	1.6	1.4	
0.0/4		2.0	4.9	0.9	4.2	3.3	3.2	4.3	
	5.5								1056
5.5/4		2.4	7.1	1.4	4.0	3.3	3.3/3.2	2.8/3.0	
	17								3264
22.5/4		0.7	4.3	0.1	4.4	3.3	3.2	2.5	
0.0/5		2.4	7.4	1.8	3.3/3.5	3.3	3.2	1.4	
	9.0								1728
9.0/5		0.1	4.9	0.0	3.8	3.3	3.2	3.1	
	15								2880
24/5		2.6	6.4	1.9	4.1	3.3	3.2	3.0	
	10.5								2016
34.5		0.1	3.9	0.0	4.3	3.3	3.2	3.0	
	13.5								2592

Operational Parameters

Time (hr) /Run	Visit interval (hr)	Dissolved O_2 A/T	Dissolved O_2 D/T	Dissolved O_2 A/C	Depth (ft) A/T	Depth (ft) D/T	Feed flow (l/min)	Effluent flow (l/min)	Total feed (C)
48/5		2.8	5.5	1.0	4.4	3.3	3.2	3.0	
0.0/6		4.1	6.7	3.1	3.4	3.5	2.4	-	
	9.5								1997
9.5/6		2.8	6.9	1.4	3.9	3.5	2.5/2.4	2.2	
	13.5								1985
B/W 23/6		0.2	4.5	0.05	4.2	3.5	2.3/2.4	2.2	
	10.5								1512
33.5/6		4.0	7.9	2.1	4.0	3.5	2.5/2.4	2.3	
	14								2016
47.5/6		0.2	6.1	0.0	4.2	3.5	2.4	2.2	
	8.5								1224
56/6		3.5	8.1	2.2	4.3	3.5	2.35/2.4	2.2	
	14								2016
70/6		0.2	4.5	0.0	4.5	3.5	2.4	1.7	
0.0/7		0.7	7.7	0.7	3.5	3.5	3.2	-	
	9.5								1824
9.5/7		0.7	8.2	0.5	4.2	3.5	3.2	-	
	14								2688
23.5		0.2	7.5	0.0	4.4	3.5	3.2	-	

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