SMALL COMMUNITY WASTEWATER TREATMENT FACILITIES– BIOLOGICAL TREATMENT SYSTEMS

by

Henry H. Benjes, Jr. [1]

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on
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[1] Chief Engineer,Culp/Wesner/Culp, Box 40, El Dorado Hills, California 95630

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SMALL COMMUNITY WASTEWATER TREATMENT FACILITIES

INTRODUCTION

The facilities provided for treatment of domestic wastewaters from small communities require some significantly different considerations than those encountered when designing large plants. The overall facility design concept of simplicity is much more important than in larger plants.

The factors which are generally prevalent and must be considered in design for small plants include:

<u>Plant Operation</u>

Available operator time will be minimal because of restrictive small community budgets.

Available operator skills will be restrictive since the skills reside with one or two individuals rather than a large staff.

Capital improvements will be a minimal or nonexistent budget item.

The plant will not be manned during night time or weekend shifts.

Preventative maintenance will be practiced as an exception rather than a rule.

<u>Wastewater Character</u>

Variations in hydraulic and organic loads will be greater.

Night time flows for very small plants may be near zero.

<u>Wastewater Process</u>

Plant operating data will be less oriented to design needs but more oriented to operational needs.

Some process alternatives may be more applicable to smaller plants than larger ones.

Process units such as sedimentation basins are smaller than those used for larger plants and the design parameters may be different than for larger plants.

The topic which this presentation reviews in detail is biological treatment systems for small community treatment plants; however other phases of the design for these plants are equally important insofar as providing acceptable design practice and economical construction and operating costs.

Other presentations included in this Small Wastewater Treatment Systems Seminar do not include the considerations given to facilities for mechanically oriented wastewater treatment systems; therefore, this presentation will include a brief discussion of the design approach of plant functional units which are adjunctive to the biological treatment process.

GENERAL DESIGN CONSIDERATIONS

For the purposes of this presentation, small community plants are considered to be those having a capacity of less than 2 mgd. The design of small plants has many considerations which are common with larger plants. Many of these considerations are outlined in EPA guidelines; however, the features required by the guidelines have been neglected in the past and bear repeating.

Flow Measurement

Every plant should provide flow measurement of the incoming wastes and a record of the flow rate. Many small plant flow meters are inaccurate because they are infrequently checked or provide little means to permit the operator to check the flow to know if the equipment requires service. For this reason, the use of an open channel flow measurement device, such as a Parshall flume is convenient, to permit the operator to zero the meter and to manually check the depth, calculate the flow, and compare it to the metered reading. The operator can also check the hourly flow and with a few calculations determine if the totalizer is working properly. The author's experience with in-channel level measurement devices is poor and stilling wells connected to the Parshall channel are preferable.

Sampling

Almost all small plants use manual sampling to obtain performance results and operational monitoring, and for this reason it is usual that only 8 hour composites are obtained. Because of the time consuming chore of collecting samples, the opportunity for error in compositing, and the lack of a total picture of the waste character, the use of automatic compositing samplers is justifiable for at least the plant influent and effluent samples. There are many compositing samplers on the market today in the cost range of from $2,000-$5,000 per sample point which, when interconnected with the flowmeter, produce an excellent composite sample. The use of these devices not only relieves the operator for other duties, but results in more accurate data than manual sampling.

Mechanical Equipment Access

There are many examples of extraordinarily poor layout design of mechanical equipment and mechanical equipment access in small community plants. It appears at times that no thought is given to removal of pumps, valves, or other equipment, let alone access by maintenance personnel.

During design the designer should keep in mind minimum aisle clearances, adequate spacing between equipment, and other passage access space required for personnel.

It is also important to work out procedures which would be employed to remove equipment from structures or basins in the event replacement is required. The life of the structure probably exceeds the equipment life by 4 times and future plant expansions may require upsizing equipment and retaining the use of the structure.

Buildings

Recently more attention by designers has been directed to building layout and design; however, it is worthwhile to reiterate certain of the more salient features which should be included.

Laboratory. Many past designs of laboratories for wastewater utilities were perfunctory. The lab design should be based on establishing work areas for the various analyses; counting the numbers of tests, bottles and equipment required to establish space and lab utility requirements; and, placing equipment in logical groupings to prevent the operator from wandering from one end to the other to perform one analysis. Good lighting and ventilation also are necessary. Even in small labs, safety equipment should be provided, such as fire extinguishers, eyewashes, emergency showers, etc.

A great percentage of the operator's time at a small community plant is required for performing lab abalyses and a thoughtfully planned lab will contribute significantly to the savings in time spent for this operator function.

Maintenance Shop. A place for repair of equipment should be provided commensurate with the organizational setup of the utility. If maintenance and repair of small parts is to be performed at the plant, a workshop area should be provided.

Office/Lunchroom/Records. A room, even though it may be small should be provided to permit storage of records and a place to make out reports. This space also provides a location where the operator(s) may have lunch and coffee breaks away from the lab. A bacteriological/chemical laboratory is no place for lunch.

Plant Site and Landscape

The planning of the plant site and landscaping also provides the designer an opportunity to minimize maintenance and operation labor and facilitate future expansion of plant facilities.

The plant site should be as compact as possible, but retaining access by cranes or other lifting equipment between structures and retaining access to buried piping for future expansions. A compact plant layout will cost less for connecting piping, sidewalks, driveways and will be more convenient during operation.

Roadways into the plant and to unloading facilities (such as chlorine cylinders) and to loading facilities (such as grit and screenings containers) should be based on the appropriate truck and turning radius. This may seem to be obvious; however, the numbers of small community plants with inadequate vehicle access provisions are legion.

The plant site and associated yard work are poorly planned as a general rule. It is typical for small community plant designs to fence the entire property and plant grass in the enclosed area. The size of the yard and the maintenance required either results in a hit and miss maintenance program or a considerable amount of maintenance labor to retain a presentable site. As a rule-of-thumb, it takes about 30 MH/year/acre to maintain a lawn. A 5 acre site will require a man-month/year. Therefore, it is thoughtful to minimize the portion of the site which is maintained in lawn. One alternative to a lawn may be ground covers which do not require mowing. Automatic irrigation systems in many climates are also a labor saving device.

TREATMENT UNITS

Grit & Screening

There are many approaches to grit and screening of wastewater, most of which are applicable to small plants. Light duty equipment has been used in many small plants with limited success. It must be remembered that even though the smaller plants are subjected to less severe conditions that larger plants, almost everything can, and does, come down the sewer. The use of articulating arm grit channel collectors, or grit channels not preceded by screening devices cause the operators many problems during peak wet weather flow conditions. It is important to precede the grit collecting device with a screening device and to provide heavy duty grit removal equipment that will not bog down when slugs of gravel come to the plant during peak wet weather flows. Consideration should be given to screening the wastes and collection of the debris, rather than subsequent grinding and placing the shredded debris back in the flow. The small amount of debris and grit at smaller plants permits direct burial rather than maintaining a shredder and coping with the problems of debris with downstream equipment. Of course two sets of grit and screening equipment are required, even though the standby set may require manual cleaning.

Primary Treatment

The use of primary treatment facilities in small community wastewater treatment plants is prevalent. Primary treatment affords a means of removing settleable solids, some of which are biodegradable, providing downstream protection from solids pluggage and reducing the size of secondary treatment facilities. Since the advantages afforded to the secondary process may be offset by the added preceding facilities (primary sedimentation basins, scum wells, sludge pumping, solids treatment and disposal), elimination of primary treatment facilities may be advantageous. Small community wastewater treatment plants, however, have smaller pumps and piping and potential pluggage of these units should be carefully considered.

Attached growth processes require primary treatment to prevent pluggage of small openings in the media. Primary treatment may be provided in the conventional manner, using a basin, or alternatively using a fine mesh hydroscreen, as shown on Figure 1. The hydroscreen generally requires less capital expenditure and provides the necessary protection required for downstream processes. The solids removed from the hydroscreen are much more concentrated than from settling basins and either must be diluted for subsequent treatment, unless dewatering and composting or chemical stabilization is provided.

Anaerobic Digestion

The one area of design, sorely lacking, for most small community plants is sludge treatment and disposal. This area is also the most costly unit function at most plants and requires the greatest operator effort in terms of labor and skill. A successful small community plant design will result in the simplest and most direct means of sludge treatment and disposal.

Anaerobic digestion has been used extensively in small community wastewater treatment plants. The use of anaerobic digesters is usually associated with primary treatment facilities. The general lack of success of this process at small plants is partially caused by gross overdesign, lack of proper mixing, lack of monitoring, and lack of control facilities for upset.

A recent review of plant facilities for a small California wastewater treatment plant revealed the anaerobic digester had over 400 days storage capacity for solids. It is not unusual to find anaerobic digesters designed for 40 to 60 days detention. Many digesters are provided with small gas recirculation systems predesigned by manufacturers. The usual design basis is about 5 to 10 CFM/1,000 cubic feet. This level of mixing is generally adequate to maintain some of the solids in suspension

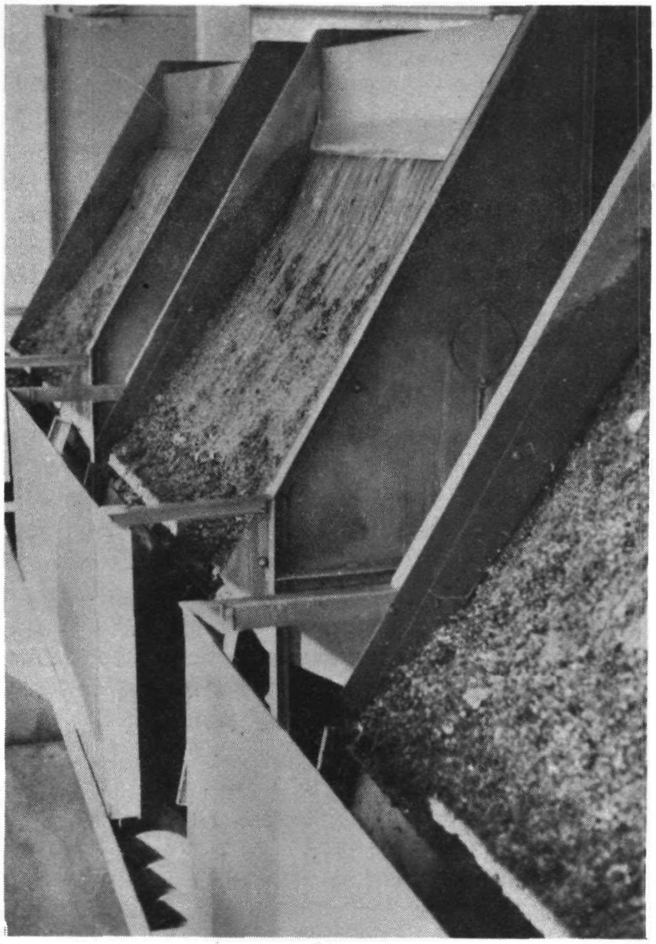


FIGURE 1 – HYDROSCREEN PRIMARY TREATMENT

and control the scum blanket but is insufficient to provide proper mixing for a complex biological reactor. In aerobic systems, at least 20 CFM/1,000 cubic feet is considered necessary to approach complete mixing. The anaerobic system, to perform as well as aerobic systems, should be mixed equally well. Anaerobic bacteria gain about 1/5 the energy from a unit of organics as compared to aerobic bacteria. Anaerobic digesters designed at about 0.1 lb of volatile solids/cubic feet per day are loaded at about 50 pounds of BOD/1,000 cubic feet. This is similar to loadings used for aerobic processes. Therefore, considerably less bacteria are present in an anaerobic digester as compared to an aerobic system. The methane bacteria are very sensitive to changes in pH. A sudden addition of organic matter first causes formation of volatile acids and can result in retardation of the methane bacteria if pH is depressed.

If used, anaerobic digesters should be completely mixed, continuously fed, or fed frequently at small doses, loaded higher than convention to result in greater concentrations of bacteria, and routinely monitored for volatile acids and alkalinity.

Many plants recycle waste activated sludge to the primary or add the waste activated sludge to the anaerobic digester. Either of these actions frequently proves to be unsatisfactory. Placing a large quantity of bacteria with a large quantity of organics for cosettling in the primary basin can only lead to the lower efficiency of this unit process. The bacteria will liquefy the organics; the added organic material will tend to disperse the bacteria; and it is not unusual to find the primary effluent BOD equal to or greater than the influent BOD.

Addition of waste activated sludge to an anaerobic digester results in poor supernatent separation for the same reasons. Waste activated sludge and raw sludge codigested in an anaerobic digester will typically result in very high solids and BOD in the supernatant return to the plant.

On the other hand, the cosettling or the codigestion of primary waste and waste sludge from attached growth biological processes has a higher degree of success. The lower activity and more stable sludge, from attached growth systems often will permit cotreatment of these sludges although there are instances where poor results have been obtained.

Even properly designed anaerobic systems applied only to raw wastewater solids result in supernatant return to the liquid process which has a high BOD and is odorous. Preaeration or judicial selection of the return point for this liquor is necessary to prevent odorous conditions or effluent quality deterioration.

It is the author's opinion that anaerobic digestion, although an extremely efficient process, should be applied to small plants only when operator skills are suitable; the process will be routinely monitored; external means are provided for chemical (sodium bicarbonate or lime) additions for pH control; the process is provided with means to continuously feed organics, or frequently fed in small doses; complete mixing is provided; and initial organic loads are above 75 pounds per 1,000 cu ft/day in the primary digester. Otherwise, other means of sludge treatment which require less precise operator control should be used.

Aerobic Digestion

Aerobic digestion of sludges when properly designed is an extremely simple process which is applicable to all biolgocial or organic sludges and in any combination. Unfortunately, it is a poorly understood process and inadequate designs are prevalent. Aerobic digestion will require more energy than anaerobic digestion because of the energy requirements for oxygen transfer. Whereas a proper design for anaerobic digestion may require 0.4 kwh/lb BOD_5 and produce energy in the form of methane gas, an aerobic digester will generally require from 1.1 to 1.6 kwh/lb BOD_5.

The aerobic digestion process provides a stable supernatant and a stable sludge. A combination of the anaerobic and aerobic process for digestion has been successfully applied. Using the anaerobic digester for raw sludge stabilization and the aerobic digester for waste activated sludge and anaerobic digester supernatant stabilization as shown on Figure 2 provides the advantage of separate treatment of incompatible sludges, stabilization of anaerobic digester supernatant, which is often a problem when returned to the liquid process, and compromises on the energy savings by using the anaerobic digester for the greatest share of the organic sludge.

The proper design of an aerobic digester is not complex; however, many of the approaches used to date have erroneously been based on solids destruction or solids loadings from which empirical factors are applied. Solids destruction, although a desireable goal is limited in biological unit processes. It is fundamental that only the biodegradable fraction can enter into the biological reaction. The biodegradable fraction is measured by the BOD test. The stabilization in an aerobic digester can be predicted by the following equation:

$$\frac{\text{BOD out}}{\text{BOD in}} = \frac{1}{K_T KeT+1}$$

Where Ke is the endogenous respiration rate

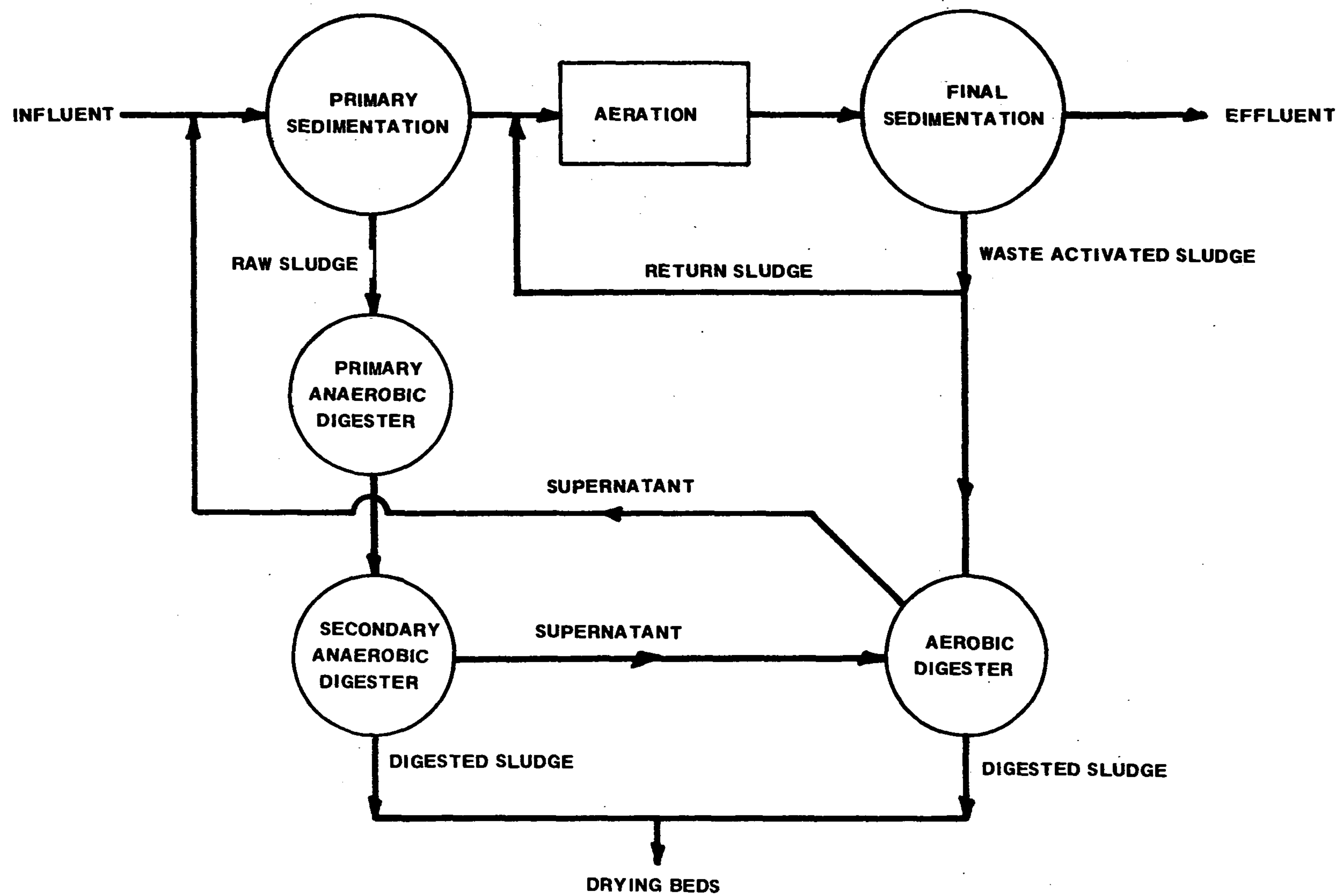
K_T is the temperature effect modifier

T is the sludge detention time in days

The "constants" in fact are not constant and are determined from the following equations:

$$K_T = 1.072^{(20-T)}$$

$$Ke = 0.5\ (0.66^{\ln T})$$



SLUDGE TREATMENT SCHEMATIC
USING ANAEROBIC & AEROBIC DIGESTION

FIGURE 2

The oxygen uptake rate in the aerobic digester may be determined by the following equations:

$$\frac{do}{dt} = \frac{1.3\ (BOD\ in - BOD\ out)}{T \times 23}$$

Where do/dt = oxygen uptake rate mg/l/hr

BOD = mg/l

T = aerobic digester detention, days

The above equations and the effect of raw sludge or anaerobic digester supernatant have been developed previously[1].

Sludge Disposal

The disposal of the stabilized sludge is an onerous problem. There are many combinations of processes available which may be used. However, especially for small community wastewater treatment plants, most of the available alternatives may be eliminated from consideration upon cursory review.

Disposal of sludge to natural watercourses may be eliminated as a viable alternative for almost any plant because of regulatory resistance to this alternative. Disposal of sludge by incineration for small plants is uneconomical. For large community plants these disposal alternatives may be economically attractive and worthy of further pursuit, but for small community plants they may be eliminated summarily.

The sludge for small community plants will, with few exception, be discharged to the land.

Liquid Sludge Disposal
- Transport
  - Pipeline
  - Truck
- Application
  - Injection
  - Spraying
- Storage

Dewatered Sludge Disposal
- Dewatering
  - Sand Drying Beds
  - Mechanical Dewatering
- Removal/Transport
  - Truck
  - Community Removal
- Disposal
  - Landfill
  - Land Spreading

The selection of the method of disposal is primarily one of economics. For small community plants, the considerations of operational simplicity, flexibility and reduction of the number of times the sludge is handled will generally result in the most economical solution.

It is not intended to explore the advantages and disadvantages or design approach of the above alternatives. A few out-of-the-ordinary successful sludge disposal practices are presented below as examples of good practice for small community plants.

Trinity River Authority, Texas. The 6 mgd TRA Ten Mile Creek plant was originally equipped with centrifuge dewatering for the anaerobically digested sludge. The large chemical demands and mechanical operating problems associated with the centrifuge led the authority to pursue land disposal of the sludge on an adjacent site. The land being uniformly and gently sloped led the authority to use a land spreading application method. Figure 3 shows the land disposal site.

Corpus Christi, Texas. Corpus Christi has 6 plants many of which are located in developed areas. At their Westside plant a program of sludge disposal by shallow injection was piloted to ascertain adverse environmental effects from this procedure. The impetus to investigate this procedure came from the previous practice of the City to heat dry their sludge in rotary driers, a costly procedure requiring large amounts of fuel.

Figure 4 shows the tractor and sludge supply hose in operation. The equipment has the capacity to inject liquid sludge at 400 gpm at a depth of 4 inches below the soil. At this rate, one week's accumulation of sludge is disposed of in 20 minutes for the 1.5 mgd plant. The sludge is pumped directly from the secondary digester. The lack of odors and savings in energy and mechanical equipment resulted in a successful pilot operation.

Placerville, California. Placerville, California operates a 0.75 mgd plant and anaerobically digests the sludge. The digested sludge is dewatered on sand drying beds. See Figure 5. The demand by private individuals for the dried sludge is so great, that the City permits only those individuals who are willing to remove the sludge from the beds to use the sludge. For the past several years, the City has had no requirements for labor to remove the sludge from the beds and no sludge haul requirements.

The above examples present sludge disposal practices at small communities which represent a practice which logically evolved in each community to find an easier and more economical method for sludge disposal.

Design practice should emphasize simplicity in disposal procedures and avoid complex processes that may be abandoned by the operating agency in favor of simpler procedures.

BIOLOGICAL TREATMENT

This section will review performance problems and costs of selected biological treatment processes. The processes selected for presentation are those that appear particularly well suited for application to small

FIGURE 3 – LAND SPREADING OF SLUDGE

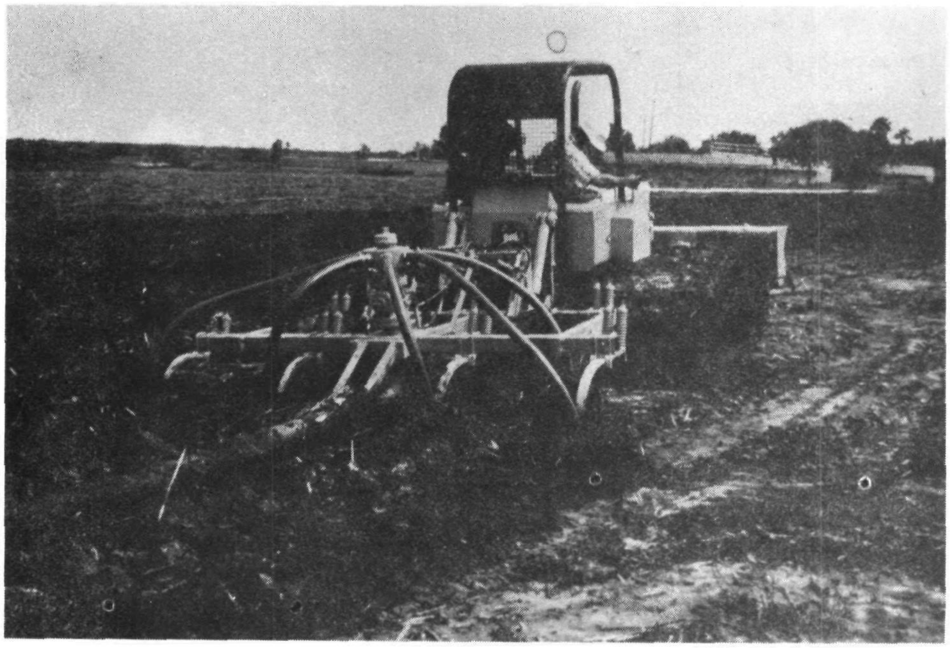


FIGURE 4 – HOSE DELIVERY SYSTEM FOR SLUDGE INJECTOR

FIGURE 5 – SAND DRYING BEDS

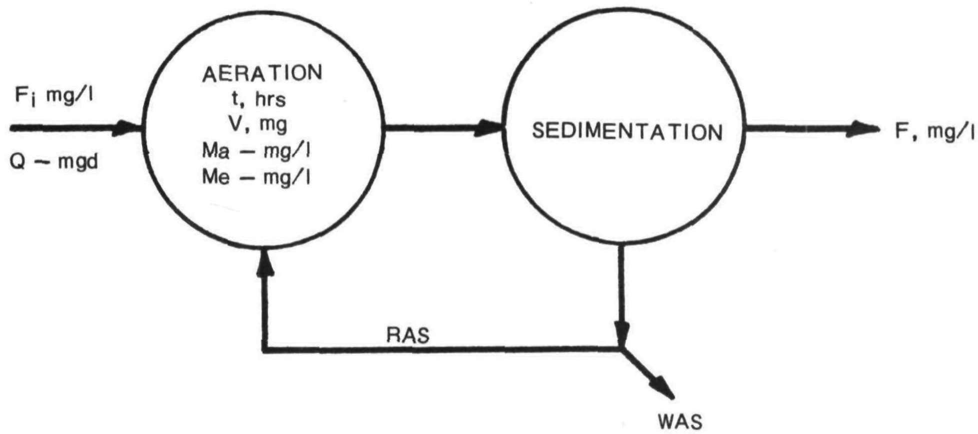


FIGURE 6 – ACTIVATED SLUDGE PROCESS

community wastewater treatment processes and include:

- Suspended Growth Biological Treatment
  - Conventional Activated Sludge
  - Extended Aeration Activated Sludge
  - Oxidation Ditch Activated Sludge
- Attached Growth Biological Treatment
  - Rock Media Trickling Filters
  - Plastic or Redwood Media Trickling Filters
  - Rotating Biological Media

Suspended Growth Biological Treatment

Design Approach. The design approach to suspended growth biological treatment systems has been outlined by several researchers[2,3,4] and presented in a unified model by Goodman[5,6]. The proper usage of any of these procedures will result in sufficiently accurate designs.

It will be helpful to review one of these models from which certain observations may be made concerning the differences in design of the selected alternative suspended growth systems. The McKinney model[3] is used extensively in the midwest and is presented below. Refer to Figure 6 for a schematic of the process which parallels the following model.

$$F = \frac{Fi}{Kmt+1}$$

$$ts = SRT = \frac{\text{\# solids in aeration and sedimentation basins}}{\text{\# solids wasted and lost in effluent per day}}$$

$$Ma = \frac{Ks\ F}{Ke+1/ts}$$

$$Me = 0.2\ Ke\ Ma\ ts$$

$$do/dt = \frac{1.5(Fi-F)}{t} - \frac{1.42(Ma+Me)}{ts \times 24}$$

Where Fi = influent BOD_5, mg/l

F = effluent unmetabolized BOD_5, mg/l

t = aeration, detention time, hours

Ma = active cell mass in MLSS, mg/l

Me = endogenous mass in MLSS, mg/l

ts = SRT, days

do/dt = oxygen demand in aeration basin, mg/l hr

Km = metabolism rate constant = 7.2/hr @ 20C

Ks = synthesis rate constant = 120/day @ 20C

Ke = endogenous rate constant = 0.48/day @ 20C

The constants may be adjusted for temperature variations from the 20C base by:

$$K_T = K_{20} x 1.072^{(T-20)}$$

Where T = temperature, degrees Celcius

The MLSS may be estimated by the sum of the bacterial mass (Ma+Me) plus the buildup of the inert solids which are in the incoming sewage. The inert solids include the nonvolatile solids plus the fraction of the volatile solids which are not biodegradable. The inert sewage solids accumulate in the MLSS in proportion to the SRT.

A portion of cell mass generated is nonvolatile. This fraction may be estimated as being 0.1 (Ma+Me). Therefore, the total MLSS may be calculated as follows:

Solids Source	MLVSS	MLSS
Ma	Ma	Ma
Me	Me	Me
Inert nonvolatile sewage solids, ISS	0	$\frac{ISS x t_s x 24}{t}$
Inert volatile sewage solids, IVS	$\frac{IVS x t_s x 24}{t}$	$\frac{IVS x t_s x 24}{t}$
Nonvolatile cell mass	0	0.1(Ma+Me)
SUM	$Ma+Me+\frac{IVS x t_s x 24}{t}$	$1.1(Ma+Me)+\frac{(IVS+ISS) t_s x 24}{t}$

The sludge production is calculated by the pounds of solids in the system divided by the SRT or:

$$\text{Sludge Production} = \frac{MLSS x V x 8.33}{t_s}$$

Where V = aeration basin volume, mg

Based on the above, several general relationships may be derived to review process differences between the three suspended growth systems selected.

The key design parameters for sizing basins and equipment are:

Oxygen Requirements. An adequate oxygen supply must be provided for conversion of the organics to bacterial cells and the stabilization of the bacterial cells to result in a readily settleable sludge. The oxygen demand rate also influences the size of the aeration basin. Large demand rates (in excess of 60-70 mg/l/hr) are usually beyond the capability of conventional aeration devices. Therefore, peak demand rates should be maintained below this level. One means of controlling the peak rate during design is to increase the aeration basin size.

MLSS. The concentration of MLSS in the aeration basin is determined by the food supplied and the SRT. The physical limitation on the MLSS concentration is the compactability of the sludge in the final sedimentation basin. For a conventional design based on 50 percent recycle and a settled sludge or return sludge concentration of 10,000 mg/l (SVI = 100), the solids balance for the final sedimentation basin may be stated:

Solids In		=	Solids Out
$(Qi+Q_R)\mathrm{xMLSS}$		=	$Q_R C_R$
If:	Q_R	=	$0.5\ Q_i$
Then:	$1.5\ Q_i\mathrm{MLSS}$	=	$0.5\ Q_i C_R$
And:	MLSS	=	$\frac{1}{3}\ C_R$
If:	CR	=	10,000 mg/l
Then:	MLSS	=	3,333 mg/l

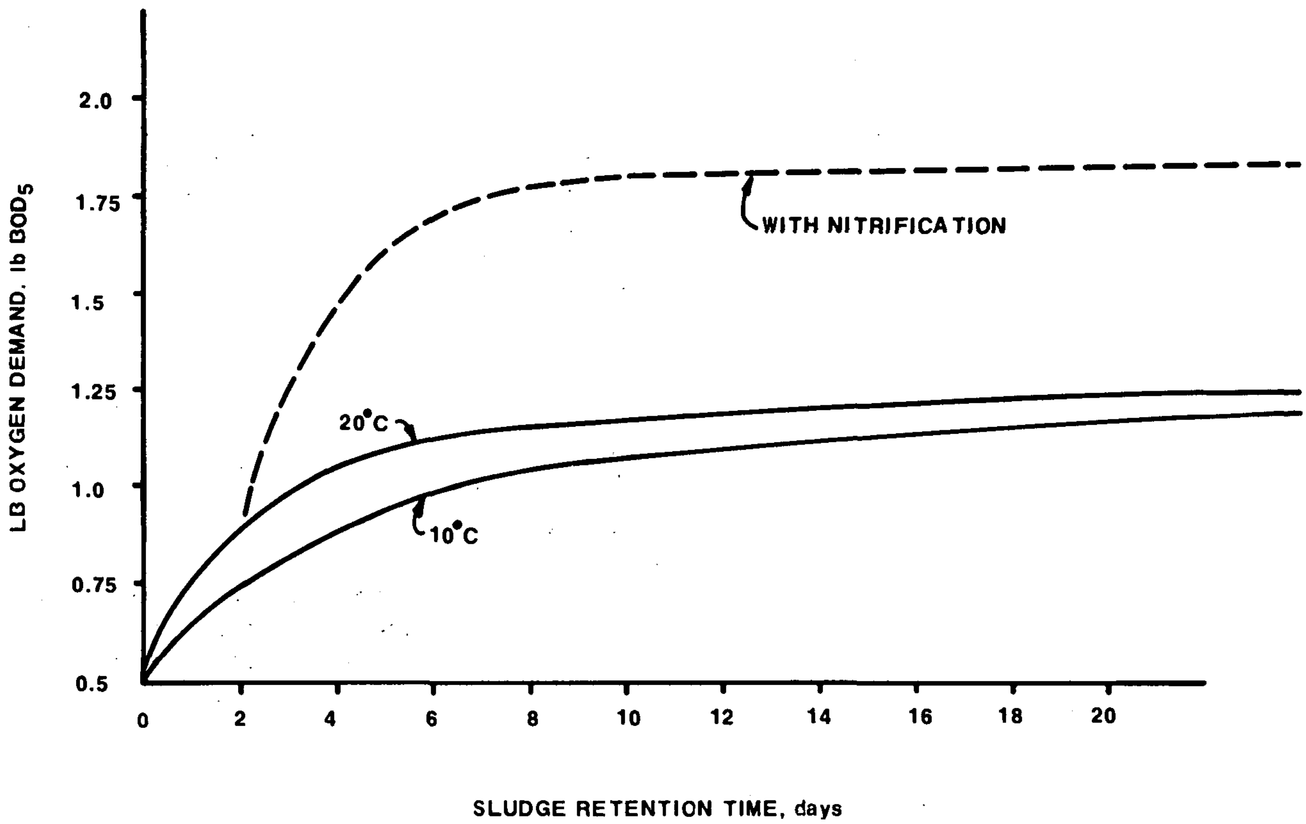
Therefore, conventional design indicates a practical design upper limit on the MLSS of about 3,000 mg/l. Higher MLSS concentrations may be attained in operation; however, to provide a design for a stable operating system, the author recommends a design based on a MLSS not to exceed 3,500 mg/l at the desired SRT.

Sludge Production. The quantification of the sludge produced from the process is a key parameter for subsequent design of sludge handling/treatment/disposal facilities.

Nitrification. Where effluent requirements include nitrification, the operating characteristics and oxygen requirements to achieve nitrification must be considered.

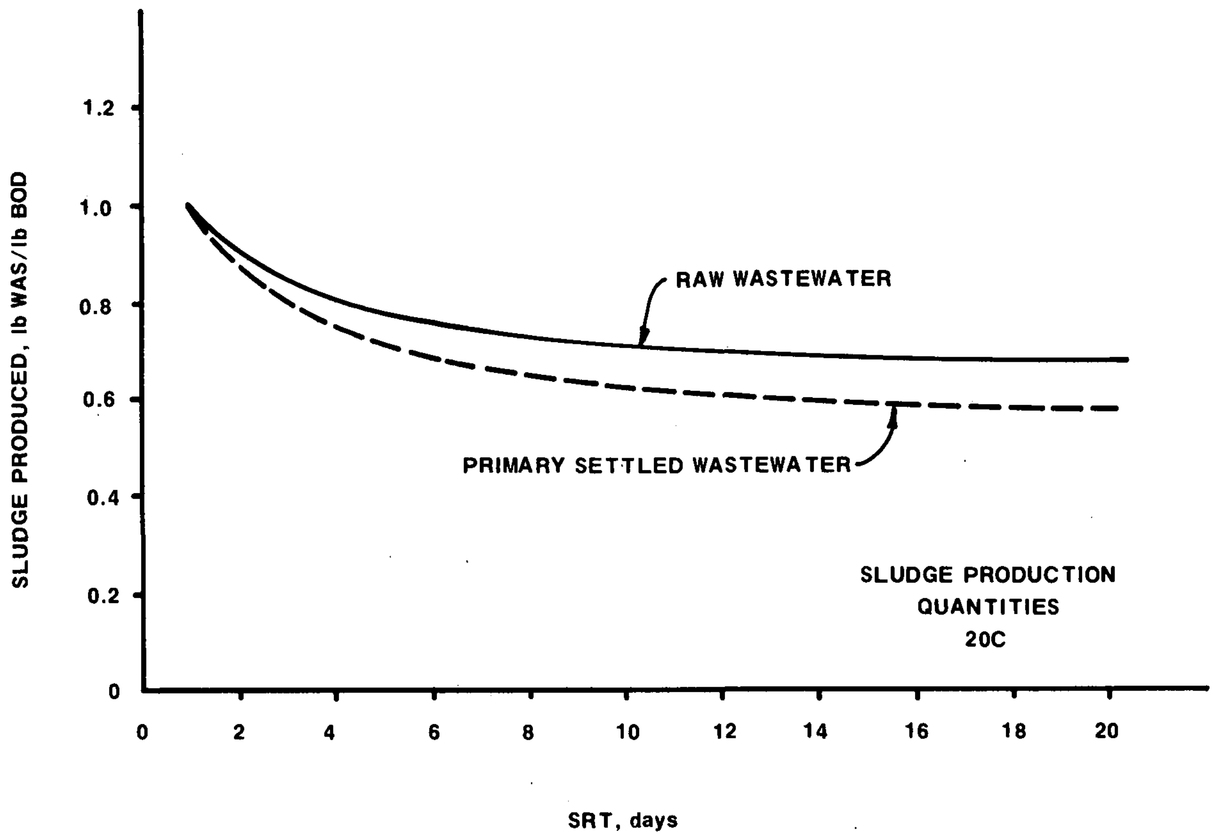
The key design parameters described above have been generalized on Figures 7, 8, and 9 based upon an influent waste having the characteristics defined in Table 1. The cases presented include:

1. Raw wastewater directly to aeration
2. Primary treatment preceding aeration
3. Nitrification required



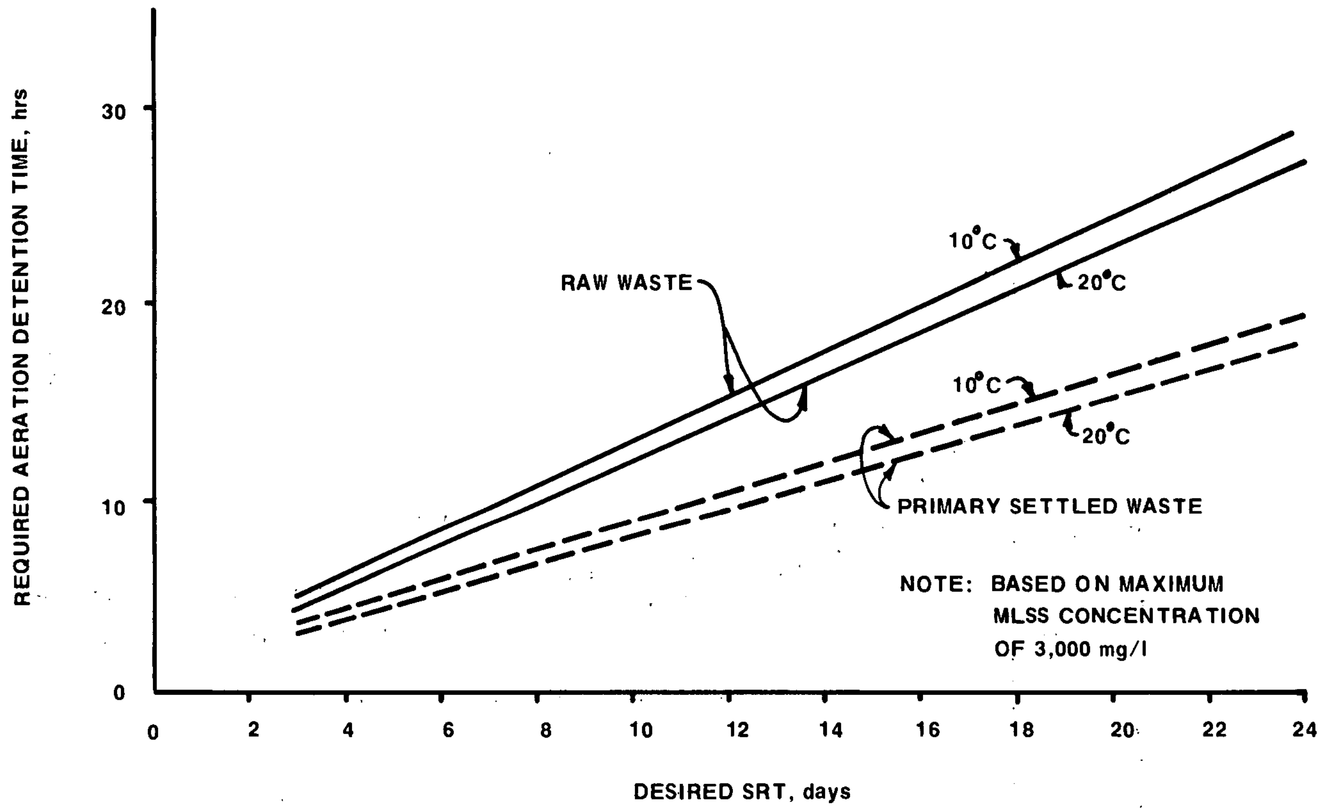
OXYGEN REQUIREMENTS

FIGURE 7

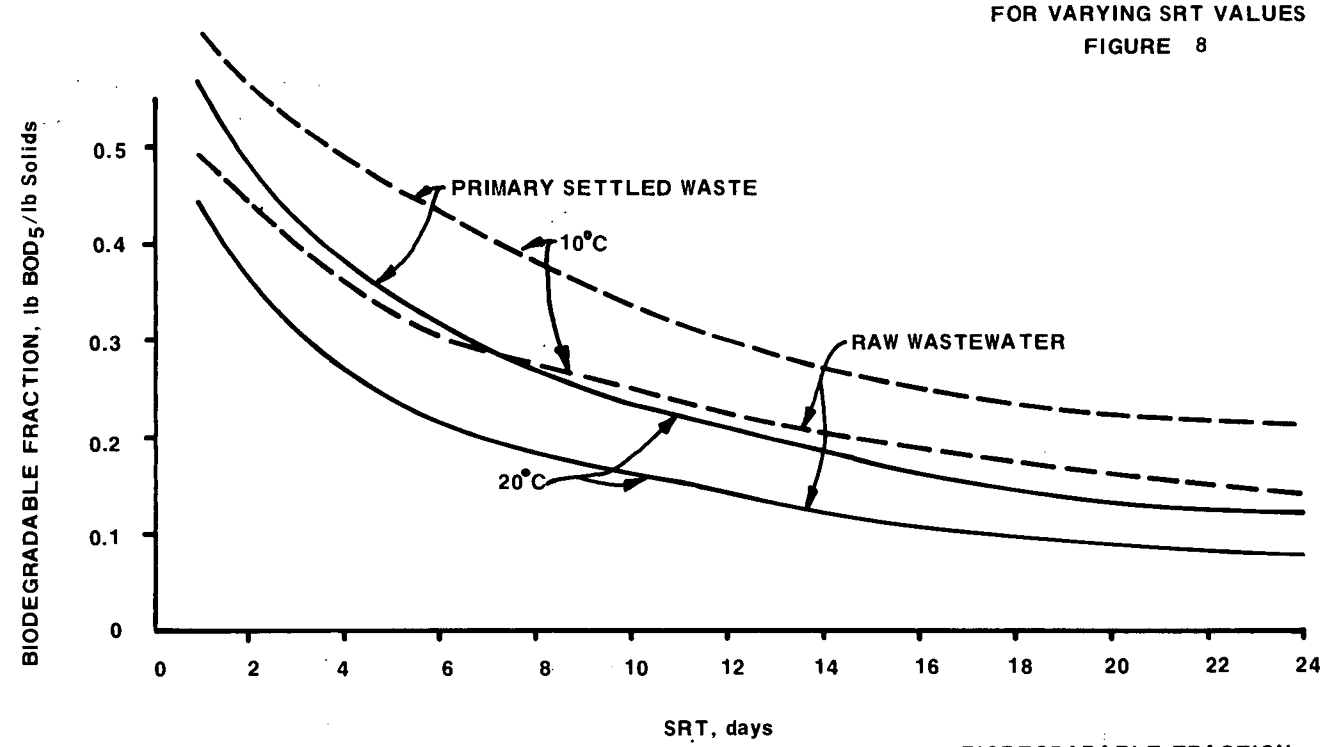


SUSPENDED GROWTH
TYPICAL SLUDGE PRODUCTION

FIGURE 9



REQUIRED AERATION TIME
FOR VARYING SRT VALUES
FIGURE 8



BIODEGRADABLE FRACTION
OF WASTE SLUDGE

FIGURE 10

TABLE 1. EXAMPLE WASTEWATER CHARACTERISTICS

	Raw	Settled
BOD_5 mg/l	200	133
TSS mg/l	200	100
NH_3-N mg/l	30	30
Inert nonvolatile solids, ISS, mg/l	40	20
Inert volatile solids, IVS, mg/l	60	30

The general relationships presented on the figures clearly depict the practical design basis of the various suspended growth systems.

Conventional Activated Sludge

The conventional activated sludge system, which operates at an average detention time of 6 hours has a practical design limitation of an SRT of 4-5 days when treating raw wastewater (Figure 8). The limitation is based on the presumption of not exceeding 3,000 mg/l in the MLSS. The average oxygen demand of 1.1 lb/lb of BOD_5 (Figure 7) represents an uptake rate of 37 mg/l/hour at the 6 hour detention. For small flow plants, it is typical to provide 2 times the average oxygen demand rate to enable meeting peak demand rates. The oxygen transfer capacity would need to be nearly 74 mg/l/hour. This will exceed the capacity of many conventional aeration devices. Mechanical aeration will meet this demand; however, the power requirement will be about 3 HP/1,000 cu ft which will cause spray and mist problems.

Using conservative design, if a 6 hour detention period, conventional activated sludge process is to work properly on normal domestic waste, primary sedimentation will be required. At 6 hours on settled wastewater input, a 7 day SRT can be attained at average conditions (Figure 8) assuming the same design limitations previously expressed. An oxygen supply of 1.13 lb of oxygen/lb BOD_5 (Figure 7) at average loadings is required which represents an average oxygen uptake rate of 22 mg/l/hour and a peak oxygen uptake rate of 44 mg/l/hour. This is within the capability of conventional aeration devices.

If nitrification is required, an average oxygen uptake rate of 39 mg/l/hour is indicated, and a peak oxygen uptake rate of 78 mg/l would be required assuming the 7 day SRT is adequate for the nitrifiers. Therefore, the conventional 6 hour aeration period would necessarily be extended to be suitable for conventional aeration devices.

Figure 10 presents a relationship showing the fraction of biodegradable solids in the waste activated sludge (WAS) for various SRT values. The conventional activated sludge process treating primary settled wastewater results in a biological sludge having about 30 percent biodegradable material. From experience at many plants, this sludge when placed on drying beds or on the land, is odorous and dewaters poorly. Sludge digestion or other means of treatment is necessary to produce a sludge suitable for dispsoal.

Extended Aeration Activated Sludge

Extended aeration plants have been used for treatment of small flows to overcome some of the limitations associated with the conventional activated sludge. The design formulae for extended aeration plants are the same as for conventional activated sludge.

Extended aeration activated sludge plants are typically based on 24 hours aeration, and rarely are accompanied by primary sedimentation. From Figure 8, a 24 hour detention period and 3,000 mg/l of MLSS results in an SRT of 20-22 days. The carbonaceous oxygen demand (Figure 7) is nearly 1.25 lb/lb BOD_5. Nitrification will occur if sufficient oxygen is available. In fact, if sufficient oxygen is not available, partial nitrification will occur to the limits of the available oxygen, depleting the dissolved oxygen to less than 1 mg/l. If sufficient oxygen is not made available at the high SRT values, there may be problems in operation associated with filimentous growths. Therefore, even when not required by effluent criteria, it is important to provide sufficient oxygen transfer capability in extended aeration plants to meet nitrification reuqirements. Therefore, at average conditions, the oxygen requirements are about 1.85 lb/lb BOD_5, assuming the 30 mg/l NH_3-N used in the example waste.

Oxygen requirements for nitrification are about 4.5 lbs oxygen/lb NH_3-N. The raw wastewater will contain organic nitrogen and ammonia nitrogen. Only in the coldest climates does domestic wastewater contain nitrite or nitrate nitrogen. The organic nitrogen is mostly in an insoluble form. Conventional activated sludge plants operated at moderate to low SRT's result in the insoluble organic nitrogen being enmeshed in the activated sludge floc and to the greatest part removed with the waste sludge. However, in extended aeration plants, the high SRT's afford the bacteria time to convert organic nitrogen to the ammonia form. The design of the oxygen resources for extended aeration should include the organic nitrogen as well as the ammonia nitrogen. A typical domestic wastewater will have about 10 mg/l of organic nitrogen.

Also, the nitrogen in the activated sludge must be considered in evaluating nitrification requirements. About 9 percent of bacterial cell mass is nitrogen. The bacteria will convert the raw wastewater nitrogen to cell mass and, upon endogenous respiration, will release the nitrogen. At an SRT approaching zero, about 0.7 lb of bacterial cells are formed per pound of BOD_5 stabilized. At an SRT approaching infinity, the residual cell mass approaches 0.15 lb/lb BOD_5 stabilized. The associated nitrogen in cell mass is 0.07 lb/lb BOD_5 at zero SRT and 0.015 lb/lb BOD_5 at an infinite SRT. For typical extended aeration plants, the nitrogen in bacterial cell mass will be about 0.02 lb/lb BOD_5 stabilized. For the example waste, the percent nitrogen associated with bacterial cell mass, and that available for nitrification is as follows:

	Influent lb per mgd	Effluent lb per mgd	
BOD, 200 mg/l	1,670	50	
NH_3-N, 30 mg/l	250	300	
OrgN, 10 mg/l	83	33	(Cell Mass)
Total N, 40 mg/l	333	333	

Therefore, the nitrogen which may be nitrified in this example exceeds the influent ammonia by 20%. This example is somewhat overstated because a fraction of the organic nitrogen is not degradable (1 to 3 mg/l).

In the example, the oxygen required for nitrification will be 1,350 lb/mg (4.5 x 300 lb). The oxygen required for carbonaceous BOD_5 stabilization will be 2,100 lb/mg (1.25 x 1,670 lb). The peak oxygen demand for small plants is about 2 times the average demand. The peak daily demand is about 1.5 to 1.6 times the average demand based on loading variations. Measured hourly variations in oxygen demand are less than the variations in raw waste loadings and a peak hour demand of about 1.25 x times the average during the peak day is appropriate for small plants.

Therefore, the average oxygen supply, for the example, would be 3,450 lb/day/mgd, and the peak oxygen supply would be 6,900 lb/day/mgd. The corresponding oxygen uptake rates for a 24 hour extended aeration plant would be 17 mg/l/hour and 34 mg/l/hour.

From Figure 9, the sludge production for the extended aeration plant approaches 0.68 lb/lb BOD_5. Of this quantity, 0.5 lb/lb BOD_5 are associated with nonbiodegradable raw sewage solids. Therefore, only 0.18 lb/lb BOD are associated with bacterial solids. From Figure 10, 8 to 15 percent of the solids are biodegradable. The low percentage of biodegradable solids is indicative of a sludge which will dewater readily if placed on drying beds or the land, and will not be malodorous. If the plant effluent solids were 25 mg/l or 210 lb/mg, the waste sludge quantity would be 930 lb/mg (solids production = 1,140 lb/mg).

Nitrification causes a reduction in alkalinity and potentially may depress the pH. In the conversion of 1 pound of ammonia nitrogen to 1 pound of nitrate nitrogen, about 7 pounds of alkalinity are destroyed. Thirty mg/l of ammonia nitrogen reduced to 1 mg/l of ammonia nitrogen will destroy 203 mg/l of alkalinity. For low alkalinity wastewaters, the pH will drop to harmful levels and chemical addition will be necessary.

Oxidation Ditch Activated Sludge

Whereas, a typical extended aeration plant is usually a prefabricated tank using diffused aeration, the oxidation ditch is an extended aeration process using a long narrow continuous, typically oval or circular, channel and paddlewheel type mechanical aerator.

The long, narrow, continuous aeration basin associated with the oxidation ditch may lead some to believe the process is "plug flow", however, the minimum velocity of 1 fps will result in a cycle time of less than 15 minutes, even in the longest channels used. Compared to the typical 24 hour detention time, the cycle time becomes insignificant. Therefore, the oxidation ditch may be considered to be an extended aeration, completely mixed, activated sludge process.

The extended aeration process design example presented above is applicable for determining the oxygen supply for the oxidation ditch. The peak oxygen supply was determined to be 6,900 pounds of oxygen per million gallons.

Assuming the following design conditions were established for peak conditions:

Minimum Basin Dissolved Oxygen	0.5 mg/l
Elevation	500 ft
Alpha α	0.9
Beta β	0.95
T	20C

The oxygen transfer capability to pure water at standard conditions (20C, sea level), which is the normal rating condition for aeration devices, would need to be approximately 8,500 pounds of oxygen per million gallons.

Mechanical aeration devices are generally rated in pure water at standard conditions at about 3 to 3.5 lb/hp hr. One hundred hp of mechanical aeration is indicated, having a transfer capability of about 7,200 to 8,400.

The above design approach for oxidation ditches is typical. Oxygen is provided for both carbonaceous BOD removal and nitrification at peak demands. A combination of using a conservative design and a basically simple process for small communities is vindicated by the excellent results obtained by operating oxidation ditch plants.

Comparison of Extended Aeration & Conventional Activated Sludge

The extended aeration process and conventional activated sludge plants differ in aspects of particular significance in small plants such as:

1. Process stability
2. Stability of waste sludge

Stability is achieved by providing a sufficiently large aeration basin to dampen variations in oxygen demand and unusual shifts in solids inventory between the aeration basin and sedimentation basin.

For small plants receiving less than 2 mgd, variations in organic and hydraulic loading are more extreme than for larger plants. A comparison between a conventionally designed 6 hour detention aeration basin and a 24 hour detention aeration basin is shown on Figure 11 for an average and a short term peak load condition. The short term peak load imposed represents a sudden doubling of BOD_5 and ammonia mass loading.

The 24 hour detention basin (at average flow) experiences a 63 percent increase in oxygen demand from 16 mg/l/hr to 26 mg/l/hr. If the oxygen concentration in the basin was 4 mg/l, it would eventually drop to 1 mg/l at the higher uptake rate and have a 3 mg/l (4-1) buffer, or at least 18 minutes at the increased uptake rate from the excess basin dissolved oxygen to absorb the added load.

The 6 hour detention basin (at average flow) experiences a 70 percent increase in oxygen demand from 56 mg/l/hr to 95 mg/l/hr. If the oxygen concentration in the basin was 4 mg/l, it would eventually drop to 0.2 mg/l; and would have 3.8 mg/l buffer (4-0.2) which only represents 6 minute buffer at the increased uptake rate to absorb the added load.

So, it can be concluded that a greater detention period will result in a slightly more stable system for variations in organic load.

Many plant upsets are caused by loss of solids from the final clarifier either by poor solids inventory management or marginal designs. The use of longer detention periods provides significant advantages in maintaining good quality under variations in hydraulic load. An example is shown on Figure 12.

The comparison shown in between an aeration basin having 24 hours detention and a conventionally designed sedimentation basin versus an aeration basin having 6 hours detention and a conventionally designed sedimentation basin.

Most small plants will operate with a set, or fixed, recycle flow rate. At night, when inflow rates are low, the system solids will tend to shift to the aeration basin since the solids flux to the sedimentation basin is low and the recycle rate is constant. When the daily peak flows occur, the solids will shift to the sedimentation basin. The critical consideration is preventing the solids to fill the final basin and spilling over into the effluent. The example on Figure 12 depicts the percentage of the sedimentation basin which is used for solids storage.

The 24 hour detention aeration basin under typical operating conditions will result in only 18 percent of the volume of the final basin occupied by sludge. A sudden increase in flow (2 times average) will cause a greater influx of solids to the final basin and a dilution of

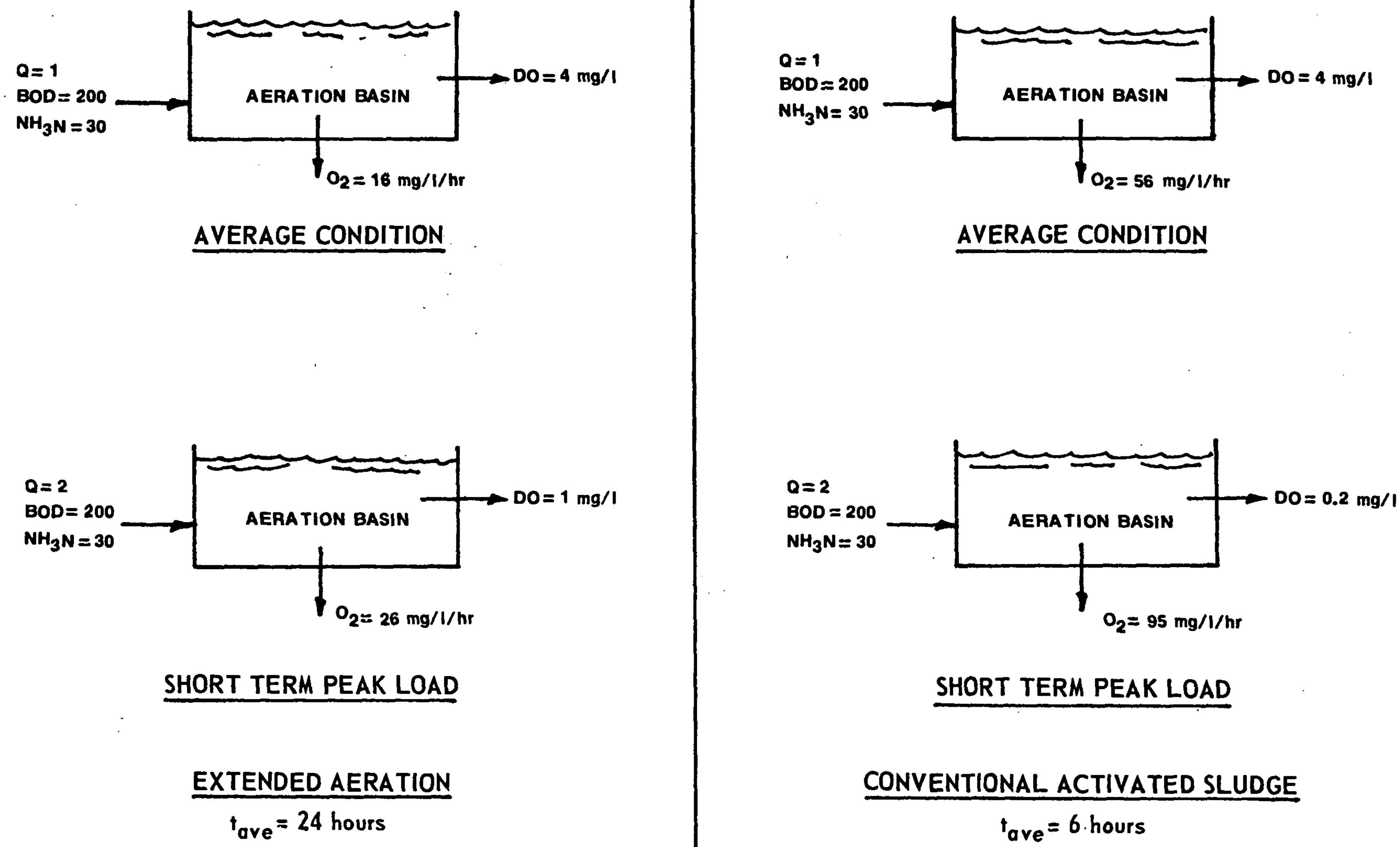
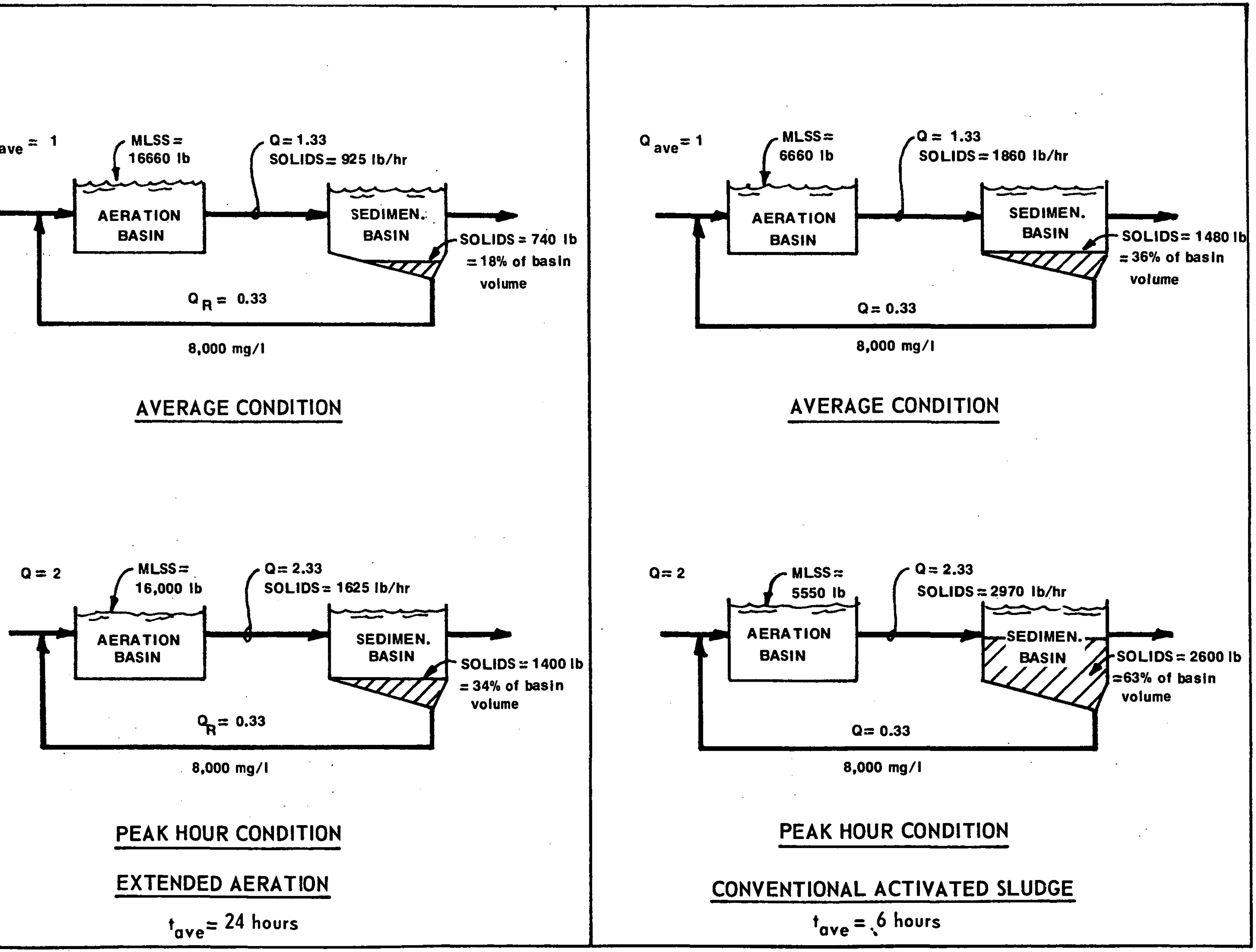


FIGURE 11

EFFECT OF ORGANIC LOAD VARIATIONS
CONVENTIONAL & EXTENDED AERATION ACTIVATED SLUDGE



EFFECT OF HYDRAULIC VARIATIONS
CONVENTIONAL & EXTENDED AERATION ACTIVATED SLUDGE

FIGURE 12

the solids concentration in the aeration basin. After about 1 hour under this condition the amount of solids in the aeration basin for the example will decrease from 16,660 pounds to 16,000 pounds.

The loss of solids from the aeration basin, of course, will be added to the sedimentation basin, increasing the inventory from 740 pounds to 1,400 pounds. The volume occupied by the solids will approach 34 percent of the sedimentation basin volume. The solids flux rate will increase from 13 lb/day/sq ft to 24 lb/day/sq ft.

In the conventionally designed plant, the same circumstances will cause the volume occupied by the sludge in the final basin to increase from 36 percent to 63 percent. The solids flux rate increases from 27 lb/day/sq ft to 43 lb/day/sq ft.

Therefore, as the extended aeration plant remains within reasonable operating parameters for high quality treatment, the conventionally designed plant approaches marginally acceptable conditions. In effect the conventionally designed plant would require operational procedures to adjust for the change in hydraulic load, such as increasing the recycle rate.

The shift in solids inventory is actually more pronounced than the example depicts since the peak daily hydraulic load does not occur upon onset of equilibrium conditions dictated by the average hydraulic load, but occurs after the night-time minimum hydraulic conditions which cause the solids inventory to shift to the aeration basin. The greater solids concentration in the aeration basin at onset of peak hydraulic load causes higher sedimentation basin solids influx than depicted. The management of solids inventory for the conventionally designed plant is as important during the minimum flows as during the maximum flows to compensate for this effect.

Any biological design is concerned with the amount of sludge production and disposal procedures. The extended aeration activated sludge process has certain inherent advantages. The long SRT's at which these plants operate (20-30 days) results in a well stabilized, aerobically digested sludge. In a conventional plant having an SRT of from 4 to 10 days, the sludge, if placed on drying beds or on the land, will be odorous and objectionable because of the relatively high biodegradable organic content of the sludge. Aerobic digestion of the sludge for 7 to 15 days will result in a stable product suitable for disposal on drying beds or the land. The total sludge age prior to disposal will be from 15 to 20 days. In effect then, the extended aeration process itself provides a sludge stability comparable to that from conventional activated sludge and separate aerobic digestion.

Attached Growth Biological Treatment

Processes which may be categorized under the general heading of attached growth biological treatment include:

Trickling Filters - or biofilters wherein stationary media is arranged over an underdrain system and the wastewater is distributed over the media. Various media used include rock, plastic, and redwood.

Rock media trickling filters flow schematics have been highly variable insofar as staging of filters, the presence or absence of intermediate clarification, and the source and quantity of recycle water.

The array of alternative flow schemes which the rock media trickling filter system may be applied, reflects the uncertainty of the critical parameters which determine the trickling filter performance.

Plastic media trickling filters are most commonly a single stage process. The media is piled or stacked to a greater depth than rock media and recirculation is commonly taken directly from the trickling filter underflow, but in instances is taken from the clarifier underflow. Plastic media is manufactured in various forms. Plastic media manufacturers strive to obtain large surface areas per cubic foot on the premise that media surface area is a prime performance parameter. Another substitute for rock media in trickling filters is redwood media. The redwood media is manufactured in the form of slats which are fabricated in the form of pallets which are stacked in the trickling filter.

Rotating Biological Media - where the media is rotated slowly through a bath of the wastewater. The media is almost universally constructed of synthetic materials and is available in the form of discs or a structural lattice.

Rotating biological media systems were developed in Europe and recently have been applied in the United States. There are several domestic manufacturers. The process differs from the concept used for trickling filters by moving the media through the waste (in a bath) instead of passing the waste through the media. The media rotates slowly through the bath exposing the attached growth to the wastes, and through the atmosphere for oxygen supply. Recirculation of liquid around the rotating media unit process is not practiced.

The media originally introduced into the United States was a series of closely spaced, parallel, flat discs. This media is still commonly used in Europe. The major manufacturers in the United States currently offer a lattice structured media, made of thinner plastic sheets, but structurally supported by closely spaced intermediate bracing. The current design offers about 50 percent more available surface area per unit volume.

Full scale installations to date (1976) use mechanical rotational drives; however, one manufacturer offers an "air drive" system which has been tested in pilot and bench scale units. Several projects currently under design are reportedly intended to incorporate the "air drive". Air is injected below the media causing a combination of an off-center byoyancy of the media and an air lifting of the liquid which effects media rotation. The air also provides added oxygenation. Design and operating data for this type of rotating biological media systems are not established sufficiently to include in this report.

The media is almost always externally protected by constructing a superstructure over the rotating biological media system or by covering each shaft with an individual cover specially constructed and provided by the manufacturer. Media construction by one manufacturer is offered with larger specific surface areas (square feet of exposed media per cubic foot) intended for use in second stage systems or nitrification where solid pluggage is less likely.

The application of attached growth systems generally requires pretreatment, including screening of debris from the waste stream and primary sedimentation. The attached growth system as applied to organic removal, requires subsequent sedimentation to remove synthesized bacteria and accumulated inhert sewage solids.

Rational Design Basis for Attached Growth Systems

A sound design for attached growth biological systems requires the designer to be familiar with the basis of the design procedures employed, the adequacy of these procedures to predict performance, and the differences between real data and procedural predictions.

The rational design of attached growth biological systems has been elusive. Empirical curve fitting has been substituted for a rational design basis with limited success requiring the design for a specific effluent condition to be conservative.

Traditionally the concept of attached growth systems has been visualized as a decreasing concentration of organics passing over a film of attached bacterial growth. The organics move from the carriage water to the growth in proportion to the organic concentration. Likewise oxygen in the air is transferred to the carriage water and then to the bacterial growth. Theoretically then, the surface area of the media should have a major effect on performance. The greater surface area per unit volume will support more bacterial growth, cause a thinner film of carriage water per unit flow of water, thus increasing oxygen transfer and slow the rate of carriage water over the bacterial growth.

The predictive techniques used for design of attached growth systems may be categorized into empirical models and rational models. Empirical models comprise the vast majority of techniques available for attached growth system design and are the procedures used by almost all design engineers. These procedures are based on statistical curve fitting of plant data to variations in plant operating conditions and physical facilities. Since the many available models tend to give varying results, it is likely that they do not express the true removal phenomena.

Recently many investigators[7,8,9] have attempted rational development of attached growth design conditions. The Williamson and McCarty biofilm model[7] is a well presented sample representing the rational approach. This model considerd many factors which describe substrate utilization by biofilms but may be too complex for general usage by design engineers. Basically, the model predicts soluble substrate removal from limitations of diffusion of oxygen and substrate through the liquid and the biofilm to the bacteria and the simultaneous effects of biochemical reactions. The surface area of biofilm becomes a key design parameter.

The limitations on the usage of the model may include the absence of the effect of suspended biological growths in the bulk liquid, the absorption/adsorption of substrate, and the physical removal of insoluble substrate. The degree of influence of these potential limitations is not know; however, there is indication that the influence is significant. Culp[10] in comparing two similar trickling filter systems, one recycling plant (secondary clarifier) effluent and the other recycling trickling filter underflow directly showed that the improved treatment resulted from recycling directly. It may be hypothesized that improved treatment resulted from recycling suspended biological growth. Slechta[11] reported on pilot studies where comparative parallel tests were conducted. One system used a trickling filter with direct recycle (trickling filter underflow) and the second system used final clarifier underflow. The system using final clarifier underflow showed almost twice the removal capability as the direct recycle system. The conclusion is that the amount of suspended biological growth in the bulk liquid will significantly affect the performance of the attached growth system.

The "rational approach" exemplified by the Williamson-McCarty biofilm model may be limited in its predictive capability for real systems; however, the investigators do make observations from their model which are useful to a better understanding of the removal phenomena in an attached growth system.

1. "Any change in environmental conditions that encourages biofilm growth such as an increase in k (Monod maximum utilization rate), Dc (diffusion coefficient in biofilm), Xc (bacterial concentration within biofilm), or So (bulk liquid substrate concentration) will not result in as large an increase in the substrate removal rate. The k value would have to be increased by a factor of 2...One implication is the under adverse environmental conditions, the substrate removal rate is not decreased as drastically for biofilms as it is for dispersed growth systems."

2. "On the basis of...(the model and certain rate assumptions) substrate utilization in these two reactors (trickling filters and rotating biological media) are predicted to be dependent on D.O. concentrations for all cases in which the soluble BOD exceeds approximately 40 mg/l."

3. "...The D.O. concentration required to avoid oxygen flux limitation would have to be 2.7 times the ammonia-N concentration (for nitrification in attached growth systems)."

These conclusions represent a portion of the removal phenomena because they relate only to attached growth. If significant suspended biological growth is carried in the bulk liquid, the limitation imposed by oxygen concentration is lower than in attached growth systems. Also significant suspended growth in the bulk liquid will reduce the soluble substrate concentration and reduce the level of effort by the attached biofilm.

Although the biofilm kinetic models are enlightening, insofar as the removal phenomena of the attached growth is concerned, the use of these models may be limited to conditions wherein suspended growth is dispersed or is not significant. For real systems this confines the evaluation of attached growth systems to previously developed empirical relationships.

There is a large school of thought that the surface area of media is the primary criteria for trickling filter sizing. That is, a media having more surface area per unit volume may permit a smaller volume than a media having less surface area per unit volume. The complicating multiple conditions which occur in an attached growth system makes such a simple premise doubtful. From the previous discussion, it was stated that the specific surface area will have less effect on the design when greater concentrations of suspended growths are carried in the bulk liquid. On the other hand when suspended growth concentrations are minimal in the bulk liquid, specific surface area may have greater effect on the design.

A later section reviews available data on various media to ascertain the difference in treatment capability associated with greater unit specific surface area.

Empirical predictive techniques for the attached growth biological process have been presented by several investigators. The more generally used formulae are presented in this section. More complete reviews of attached growth biological system models are presented elsewhere[12,13,14]

Of the more commonly used formulae, the earliest was developed by the National Research Council (NRC), where:

$$E = \frac{1}{1 + 0.0561 \left(\frac{W}{VF}\right)^{1/2}} \quad \text{for first stage} \qquad (1)$$

Where E = fraction of BOD removed

W/V = lb BOD_5/day/1,000 ft^3

$F = (1+R)/(1+0.1R)^2$

R = ratio of recirculation to influent flow

Following several formulae based on estimation of fluid travel time through attached growth systems, Eckenfelder[15] presented the formulae:

$$\frac{Le}{Lo} = \frac{1}{1 + 2.5D^{2/3}/(Q/A)^{1/2}} \qquad (2)$$

Where: Le = BOD_5 out

Lo = (Li + RLe)/(1 + R)

Li = BOD_5 in

D = filter depth, ft

Q = hydraulic flow to filter, mgd

A = filter area, acres

R = recycle ratio

Galler and Gotaas[16] later proposed a formula incorporating more variables and fitted by regression analysis to existing trickling filter plants:

$$Le = \frac{0.464\ Lo^{1.19}\ (1 + R)^{0.28} \left(\frac{Q}{A}\right)^{0.13}}{(1 + D)^{0.67}\ T^{0.15}} \qquad (3)$$

Where: T = temperature, C

Manufacturers of plastic media trickling filters increased the general usage of the Velz equation in the following form:

$$\frac{Le}{Li} = e^{-kD/q^{\frac{1}{2}}} \qquad (4)$$

Where: q = flow rate gpm/sq ft excluding recycle flows

A similar equation form has been developed during this study for general usage with all attached growth media systems. This equation was developed primarily to assess the removal phenomena as a function of hydraulic loading rate per unit volume.

$$\frac{Le}{Lo} = e^{-K\left(\frac{V}{Q}\right)^{\frac{1}{2}}} \qquad (5)$$

Rock Media Trickling Filters

Considerable data are available to judge the accuracy of design formulae. Most data reported represent averages and certain of the parameters must be assumed in order to calculate values from the various models. A summary of data is shown on Table 2. Using physical description and operating parameters given, the predicted values for the several more frequently used empirical formulae have been calculated.

Equations (4) and (5) are generally not applied to rock media trickling filters. Because these equations are in general usage for media other

TABLE 2

COMPARISON OF TRICKLING FILTER MODELS WITH DATA

Plant Location	D (ft)	R	Q/A (mgd/acre)	W/V (lb/BOD/1,000 ft)	Le (mg/l)	Predicted Effluent BOD (mg/l)		
						NRC	Eckenf.	Galler/ Gotaas
Aurora, Ill.	6	-	2.1	4.4	14	7	11	14
Dayton, Ohio	7.5	-	3.5	12	33	22	22	30
Durham, N.C.	7	-	1.9	13	68	44	34	61
Madison, Wisc.	10	-	2.4	6.4	33	17	16	24
Richardson, Tx.	6.5	-	3.9	13.3	20	20	22	27
Plainfield, N.J.	6	0.6	2.4	25	13	14	8	12
Great Neck, N.Y.	4	1.0	7.8	20	20	19	21	26
Oklahoma City, Ok.	6	1.0	16.3	78	66	83	59	71
Freemont, Ohio	3.3	1.5	19.0	41	21	20	23	23
Storm Lake, Iowa	8	2.1	21.5	62	61	88	50	61
Richland, Wa.	4.5	2.8	19.6	44	20	23	17	19
Alisal, Ca.	3.2	3.1	20.8	53	24	39	31	46
Chapel Hill, N.C.	4.25	2.0	16.3	19	44	11	13	14
Dallas, Texas	7.5	0.5	5.6	21.4	37	41	32	43
Bridgeport, Mi.	6	1.2	20.6	29	42	24	25	19
Cass City, Mi.	6	1.3	10	23	33	34	31	28
Charlotte, Mi.	6	-	7.7	29	63	34	36	33
Hillsdale, Mi.	6	-	3.6	10	32	19	22	22
Lapler, Mi.	5.8	0.3	13.5	22	23	16	20	16
State Prison, Mi.	8	0.1	3.8	13	17	25	23	30
Vassar, Mi.	5.6	1.7	9.2	6	29	9	11	8
				Average	34	29	26	29
Standard Error of Estimate						17	16	14

than rock, the applicability has been reviewed for data from rock media biofilters as shown in Table 3. The value of k (Equation 4) and K (Equation 5) is dependent upon the surface wetting rate as shown on Figure 13. Also the effect of depth does not seem to affect the results more so than volume.

Plastic and Redwood Media Trickling Filters

The several forms of fabricated media available include:

Plastic media - stacked
Plastic media - random dumped
Redwood Media - stacked

The data on Table 4 indicate that both k and K are variable, implying factors other than flow will influence the predictability of the degree of treatment. However, the domestic waste treatment as represented by Chipperfield[18] imply media volume is as representative of a treatment parameter as is depth. The application of formulae developed for rock media trickling filters to the plastic media trickling filters will not produce successful predictions. For instance, as a general rule, the Galler/Gotaas equation (Equation 3) which is successful with rock media trickling filters, will predict a much lower effluent BOD from plastic media trickling filters than is experienced.

The data on Table 4 imply that the capability of the plastic media with an effective surface area of 25-30 square feet/cubic foot is about the same as the redwood media having an effective surface area of about 14 square feet/cubic foot. A more direct comparison of the capability of the two media was made in Salem, Oregon[19] and is shown on Figure 14.

These data indicate little difference in capability of the two media, and in this presentation no differentiation will be made in the design procedures. Furthermore, when compared to performance of rock media having a surface wetting rate above 0.35 gpm/sq ft, the plastic or redwood media trickling filters appear to provide equal treatment per unit volume.

Rotating Biological Media

The design approach for rotating biological media has been a graphical relationship between the effective surface area of the media and the percent removal efficiency as shown on Figure 15. Basically this relationship implies beneficial results for higher specific unit surface areas. Media is manufactured in the form of discs which have a specific unit surface area of 20-25 sq ft per cubic feet and in the form of lattice structure which has a specific unit surface area of 30-35 sq ft per cubic foot. A higher specific unit surface area is available (45-50 sq ft/cu ft) for use in the latter stages of the system which purportedly reduces the overall volume of the media. The usage of the high specific surface area media in early stages of the rotating biological media system often results in clogging due to the smaller clearances and is not recommended. As mentioned in previous sections of this report, the disc type media is no longer available from the two major domestic manufacturers; however, a

TABLE 3

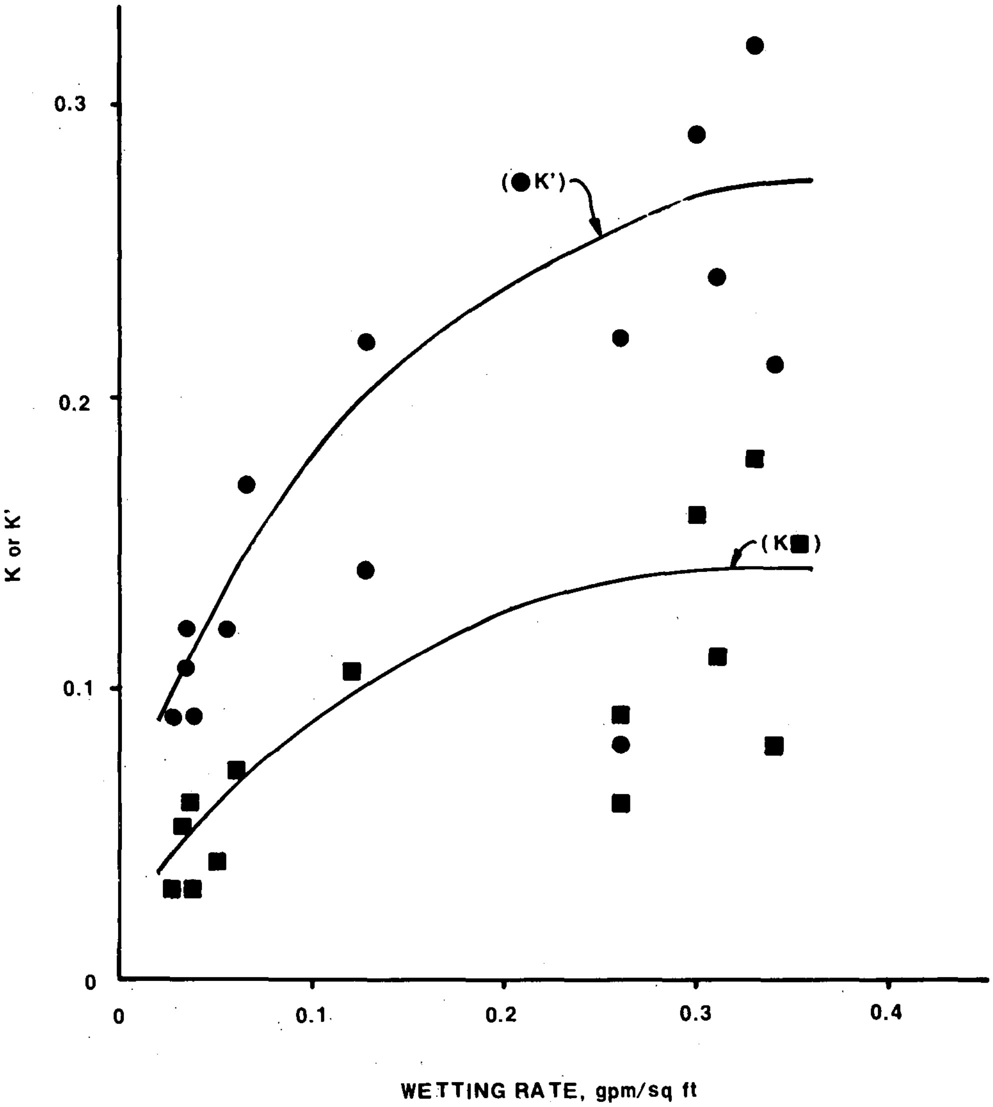
ROCK MEDIA BIOFILTERS
DATA EVALUATION FOR DEPTH AND VOLUME EFFECTS

Plant Location	Depth (ft)	q (gpm/sqft)	Wetting Rate (gpm/sqft)	BOD in (mg/l)	BOD out (mg/l)	Depth k	Volume K
Aurora, Ill.	6	0.034	0.034	70	14	0.05	0.12
Dayton, Ohio	7.5	0.056	0.056	137	33	0.04	0.12
Durham, N.C.	7	0.030	0.030	261	68	0.03	0.09
Madison, Wisc.	10	0.038	0.038	138	33	0.03	0.09
Richardson, Texas	6.5	0.062	0.062	118	20	0.07	0.17
Plainfield, N.J.	6	0.024	0.038	76	13	0.06	0.11
Great Neck, N.Y.	4	0.062	0.125	117	20	0.11	0.22
Oklahoma City, Okla.	6	0.130	0.260	300	66	0.09	0.22
Freemont, Ohio	3.3	0.121	0.30	95	21	0.16	0.29
Storm Lake, Iowa	8	0.111	0.34	381	61	0.08	0.21
Richland, Washington	4.5	0.082	0.31	118	20	0.11	0.24
Alisal, Calif.	3.2	0.081	0.33	185	24	0.18	0.32
Chapel Hill, N.C.	4.25	0.087	0.26	77	44	0.04	0.09
Dallas, Texas	7.5	0.090	0.13	130	37	0.05	0.14
Bridgeport, Mich.	6	0.15	0.329	99	42	0.07	0.16
Cass City, Mich.	6	0.07	0.160	152	33	0.09	0.17
Charlotte, Mich.	6	0.214	0.214	119	63	0.06	0.21
Hillsdale, Mich.	6	0.057	0.057	91	32	0.052	0.13
Lapler, Mich.	5.8	0.160	0.214	65	23	0.09	0.16
State Prison, Mich.	8	0.050	0.060	153	17	0.06	0.17
Vassar, Mich.	5.6	0.090	0.231	59	29	0.05	0.13
				Average		0.07	0.17

TABLE 4

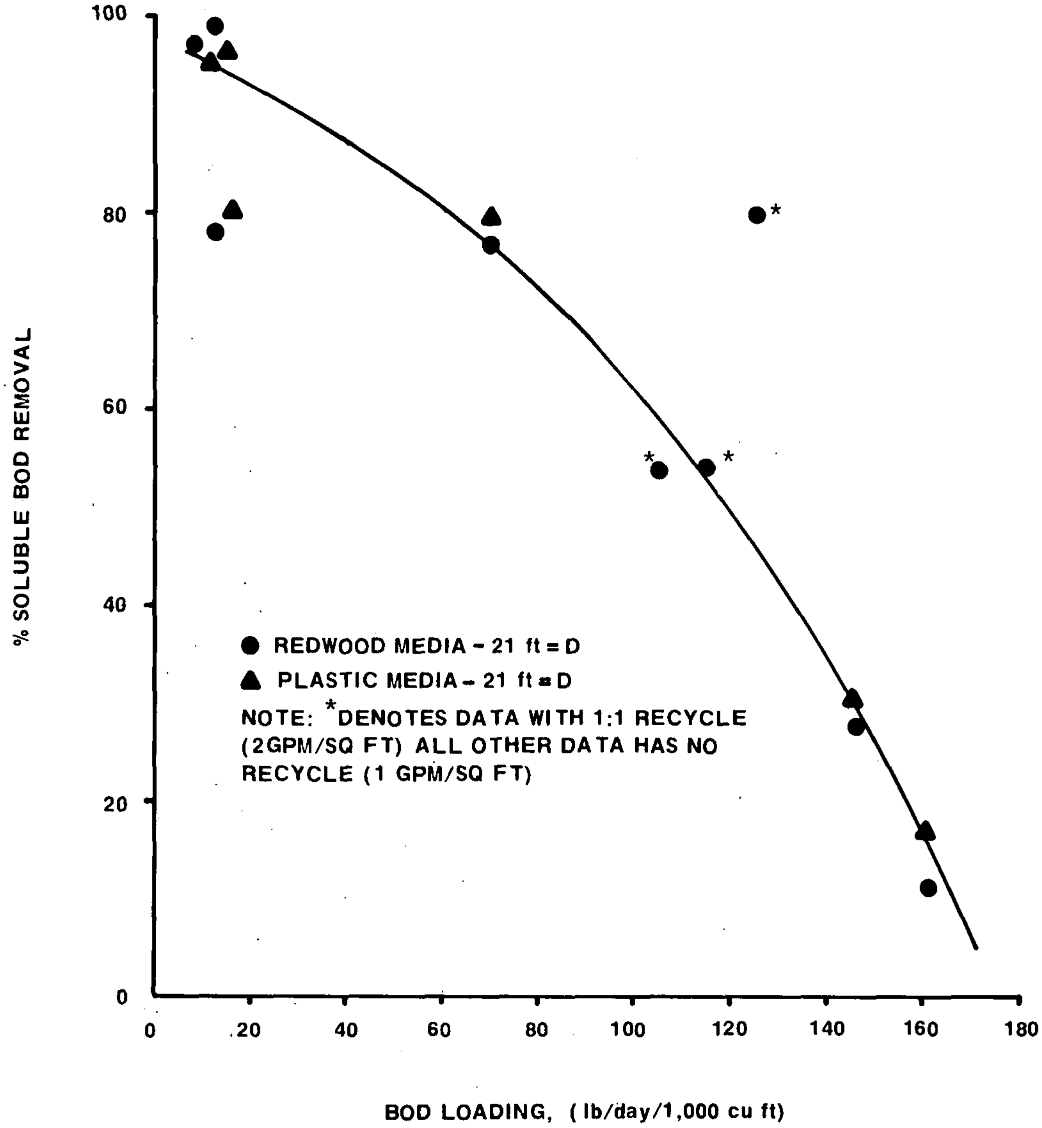
PLASTIC AND REDWOOD MEDIA BIOFILTERS
DATA EVALUATION OF EQUATIONS (4) AND (5)

Location	Media	Depth (ft)	q (gpm/sqft)	Wetting Rate (gpm/sqft)	BODIn (mg/l)	BODout (mg/l)	Depth k	Volume K
Indianapolis, IN[13]	Plastic	21.5	2.0	2.0	112	57	0.04	0.20
Stockton, CA[14]	Plastic	21.5	0.28	0.71	240	40	0.04	0.20
Wiskeywaste[15]	Plastic	34	NA	NA	950	65	0.04	0.20
Domestic[15]	Plastic	6	0.2-0.8	0.2-0.8	a	a	0.11	0.28
Domestic[15]	Plastic	18	0.6-2.3	0.6-2.3	a	a	0.06	0.25
Corvallis, OR[16]	Redwood	14	0.94	3.3	100	24	0.10	0.37
Corvallis, OR[16]	Redwood	14	1.12	4.3	192	72	0.07	0.28
Idaho Falls, Idaho[17]	Redwood	21.5	0.34	1.0	60	9	0.05	0.24
Madera, Calif.[17]	Redwood	12	0.20	3.2	220	25	0.08	0.28
Akron, Ohio[18]	Plastic Dumped	25.5	0.36	0.75	120	20	0.48	0.22
Buena Vista, Mich.[19]	Plastic	20	0.46	1.20	54	21	0.03	0.14
Bay City, Mich.[19]	Plastic	21.5	0.90	1.1	79	18	0.005	0.31
Essexville, Mich.[19]	Plastic	21.5	0.75	1.50	23	11	0.03	0.15
Greenville, Mich.[19]	Plastic	21.5	0.46	0.50	62	15	0.05	0.21
Rockwood, Mich.[19]	Plastic	22	0.32	0.97	61	23	0.03	0.12



ROCK MEDIA TRICKLING FILTERS
EFFECT OF WETTING RATE
ON EVALUATION CONSTANTS

FIGURE 13

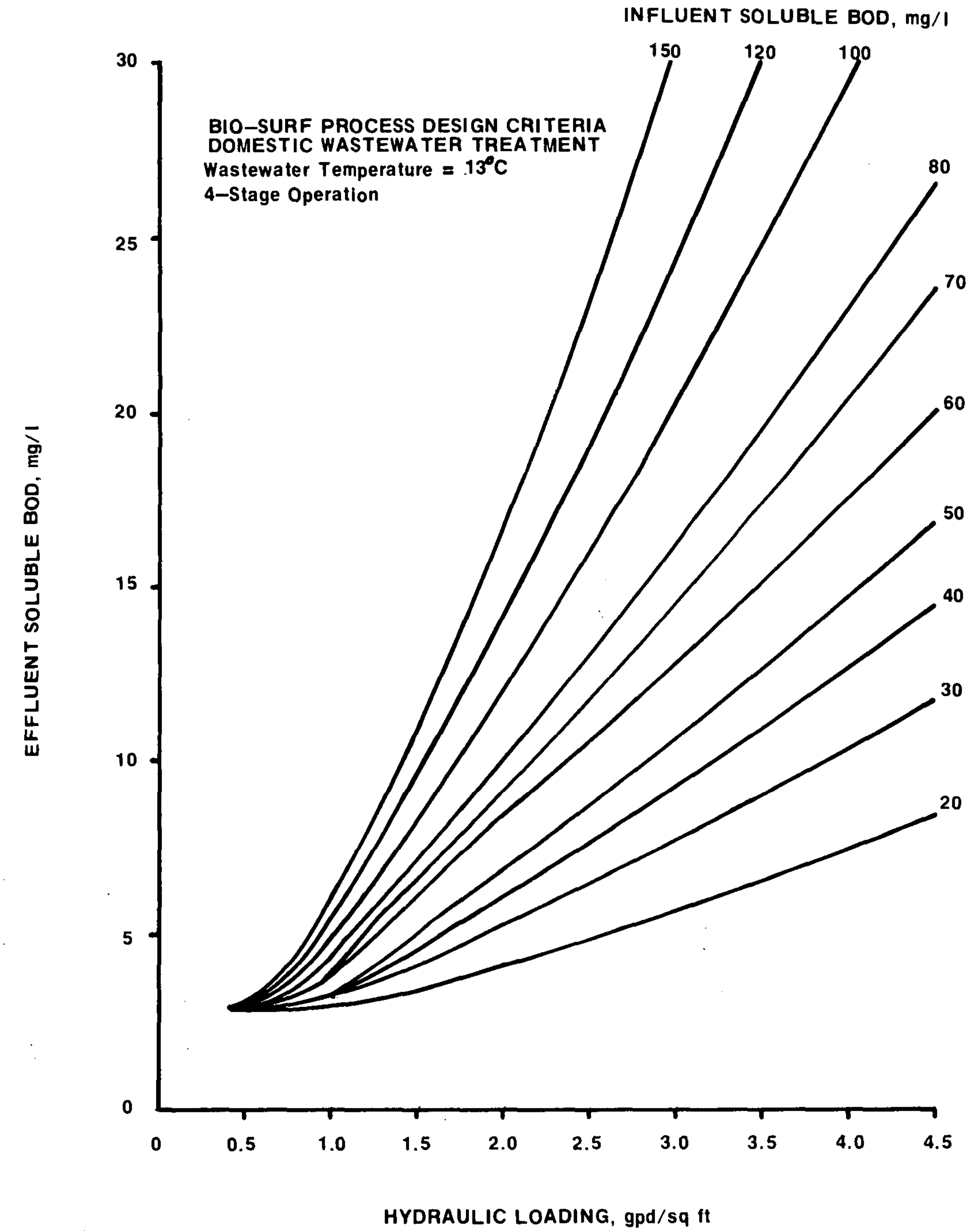


REDWOOD & PLASTIC MEDIA

TRICKLING FILTERS

SOLUBLE BOD REMOVAL EFFICIENCY

FIGURE 14



ROTATING BIOLOGICAL MEDIA

MANUFACTURER'S DESIGN APPROACH

FIGURE 15

review of data from existing installations is helpful to assess the effects of varying specific surface areas.

One manufacturer's (Autotrol) design approach is based on soluble BOD_5 in the influent and effluent. Much of the existing data indicates the effluent BOD_5 from the RBM final clarifier will be 50 percent soluble and 50 percent suspended material. This is consistent with the limited data from other attached growth systems. However, the influent soluble BOD_5 portion is highly variable. For example, the following results have been reported for primary effluent at various locations.

Plant	Percent Soluble BOD_5
Pewaukee, Wisconsin	66
Seattle, Washington	31-50 (41 average)
Tucson, Arizona	50-75 (67 average)

The use of soluble influent BOD as a critical design parameter, if applicable, will be unwieldy because of the general lack of data for this parameter and the variability even at a single plant location.

To provide a more consistent design approach with other attached growth systems and to enable realistic data evaluation, equation (5) has been applied to the RBM systems. It is impractical to attempt to evaluate the manufacturer's design approach unless data are generated for soluble influent BOD_5.

The data which are available are evaluated and summarized on Table 5. These data are from discs and lattice type RBM systems and represent full scale and pilot plant installations. Individual data have been shown to indicate the range of calculated K values.

The conclusions which may tentatively be made from the available RBM data are:

1. The Pewaukee pilot plant data and full scale data, as well as the Gladstone pilot plant data and full scale data may be correlated reasonably well by use of equation (5).

2. The K_{20} value evaluated for discs and lattice media do not indicate that higher unit specific surface area is a factor in BOD_5 removal.

3. For BOD_5 removal, a design value for K_{20} of 0.30 appears appropriate.

Field data from other RBM installations are limited and are presented below:

TABLE 5

ROTATING BIOLOGICAL MEDIA
PERFORMANCE DATA

Plant	Media	Volume (cu ft)	Q (gpm)	BODin (mg/l)	BODout (mg/l)	K_T	T (F)	K_{20}	
Pewaukee,[10] Wisconsin	5.75 ft diam disk	197	8.06	205	37	0.35	54	0.36	
			6.90	183	34	0.31	50	0.36	
			3.40	175	22	0.27	42	0.39	
			3.38	192	30	0.24	42	0.34	
			1.50	111	17	0.16	44	0.21	
			1.77	170	19	0.21	39	0.33	
			0.83	112	12	0.15	45	0.20	
			0.83	134	10	0.17	40	0.26	
			4.95	104	22	0.25	55	0.25	
			8.50	139	30	0.32	58	0.32	
			15.30	128	44	0.30	61	0.30	0.30(AVE)
Pewaukee,[11] Wisconsin	10 ft diam disk	10,450	133	150	24	0.21	47	0.26	
			132	148	16	0.25	45	0.33	
			199	129	27	0.22	46	0.28	
			202	100	22	0.21	49	0.24	
			242	100	18	0.26	55	0.26	
			157	110	14	0.25	61	0.25	
			184	110	14	0.27	65	0.27	
			195	108	21	0.22	66	0.22	
			302	158	23	0.33	65	0.33	
			239	90	18	0.24	61	0.24	
			242	109	23	0.24	56	0.24	0.27(AVE)
Edgewater,[12] New Jersey	12 ft diam lattice	6,110	275	166	42	0.29	72	0.29	
			340	177	49	0.30	65	0.30	
			388	132	27	0.40	58	0.40	
			432	89	20	0.40	54	0.42	
			419	113	31	0.35	52	0.37	
			409	133*	34	0.35	54	0.37	
			393	92*	22	0.36	58	0.36	
			405	154	32	0.40	65	0.40	
			329	171	39	0.34	72	0.34	
			223	208	75	0.19	76	**	
			273	164	43	0.28	78	**	0.36(AVE)

*Estimated temperature correction
**System biological growth predominated by Beggiota.

TABLE 5

(continued)

Plant	Media	Volume (cu ft)	Q (gpm)	BODin (mg/l)	BODout (mg/l)	K_T	T (F)	K_{20}	
Gladstone, Michigan[13]	4 ft diam disks	196	10.4	100	32	0.26	56	0.26	
			6.9	85	13	0.35	56	0.35	
			3.5	62	9	0.26	62	0.26	
			5.2	111	21	0.27	50	0.31	0.30(AVE)
Gladstone, Michigan[14]	12 ft diam lattice	16,300	508	117	24	0.28	52	0.30	
			543	99	17	0.32	58	0.32	
			608	105	19	0.33	62	0.33	
			539	102	20	0.30	65	0.30	0.31(AVE)

Location	Plant Design Flow (mgd)	Current Flow (mgd)	Volume of Media (cu ft)	Media Description (lattice)	BOD Raw (mg/l)	BOD Primary (mg/l)	BOD Final (mg/l)	K_T
Woodland, Washington	0.45	0.15	2,413	12 ft diam	270	175*	28	0.38
Kirksville, Missouri	5.0	1.30	63,100	12 ft diam	252	164*	15	0.29
Georgetown, Kentucky	3.0	1.10	25,240	12 ft diam	230	150*	21	0.34

*Estimated; BOD_5 is not measured on primary effluent.

These data generally confirm the conclusions reached concerning BOD removal relationships for the RBM system.

There are many European manufacturers of rotating biological media systems (primarily discs). The design relationships presented by Schuler/Stengelin[19] have been evaluated in terms of equation (5) and an average K value of 0.30 was obtained.

Temperature

The temperature effects on effluent quality and system design requirements for attached growth systems are usually critical for cold weather conditions. For a year-around effluent quality criteria, the cold weather conditions will determine the size of the attached growth reactor because the lower biological reaction rate. An extensive evaluation of data which assesses temperature effects was made by Galler/Gotaas. In their formulae, temperature affects on effluent quality may be stated:

$$\frac{Le_T}{Le_{20}} = \frac{20}{T}^{0.15}$$

Where: T = temperature, celcius
Le_T = effluent BOD mg/l at temperature T
Le_{20} = effluent BOD mg/l at temperature 20C

For example: To obtain an effluent BOD at 30 mg/l at a temperature of 10C, the effluent BOD at 20C would need to be 27 mg/l.

Eckenfelder[15] states the effect of temperature as:

$$E_T = E_{20} \times \theta^{-(T-20)} \qquad \text{Where: } \theta = 1.035 \text{ to } 1.040$$

In a presentation of actual data, Benzie, et al[21] provided a basis to evaluate θ.

Of the 17 plants reported, 6 plants had a value for θ exceeding that predicted by Galler/Gotaas (1.011). Of these 6 plants, 5 plants employed recirculation, whereas, of the eleven plants having a calculated θ value below 1.01, only two plants employeed a 1:1 recirculation.

A comparison of plants employing recirculation from different sources by Culp[10] indicates that the location of the source of recirculation effects the results. The calculated θ value are as follows:

	Warm Weather		Cold Weather		
	T-C	E	T-C	E	θ
Direct Filter Recirculation	18.3°	60.5	10.4	56.2	1.009
Recirculation from Final Effluent	18.6	51.4	9.4	38.6	1.032

The conclusions which may be derived from these data support the conclusions reached by Williamson/McCarty[17] that diffusion of the organic through the bulk liquid and biofilm are limiting rather than biological reaction rates under specific circumstances. Where high recirculation rates are employed, or final effluent is recirculated, a larger temperature effect relationship is likely applicable where θ may be as much as 1.035. This may be caused by the cooling tower effect.

The temperature effect in plastic media biofilters has been calculated from plastic media manufacturer's literature on a common basis and a θ value of 1.018 appears to have been used widely.

RBM data evaluated in this report are shown on Figure 16 and indicate temperature has no measurable effect above 13C. Below 13C, the relationship shown on Figure 16 would be appropriate.

Nitrification

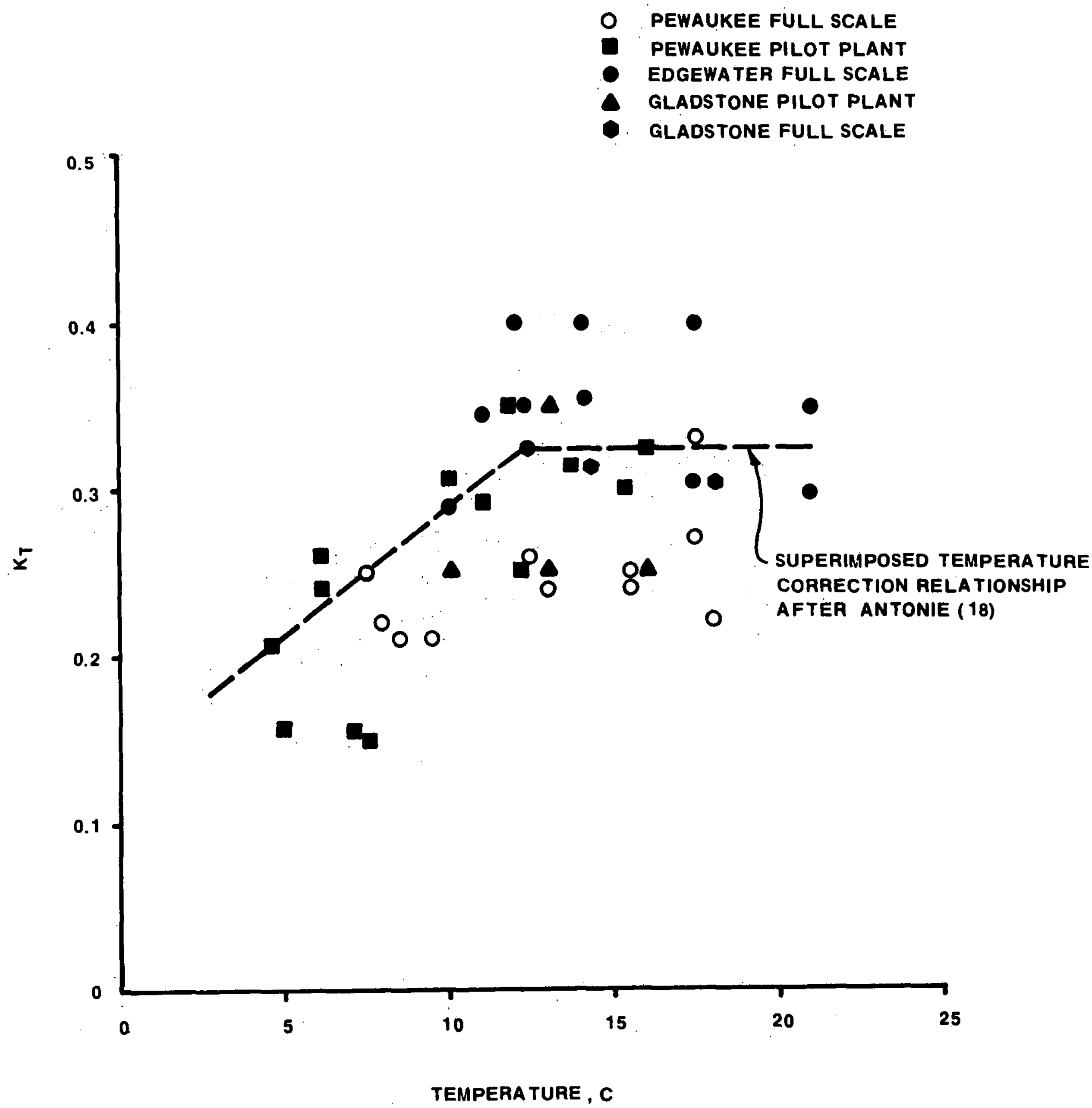
The conventional design of an attached growth biological system for nitrification has also been based on experience and empirical relationships. The EPA, Technology Transfer Process Design Manual for Nitrogen Control[22] reports that in rock media trickling filters, the organic load must be limited to 10-12 pounds per day per 1,000 cubic feet to obtain efficient nitrification.

From the same reference[22] data were collected from the literature relating nitrification efficiency to organic loading (lb/day/1,000 cu ft). The relationship shown is reprinted as Figure 17.

The reported data from second stage trickling filters shows mixed results which defy confident prediction of results from one plant to the next. Data which have been reported are shown below.

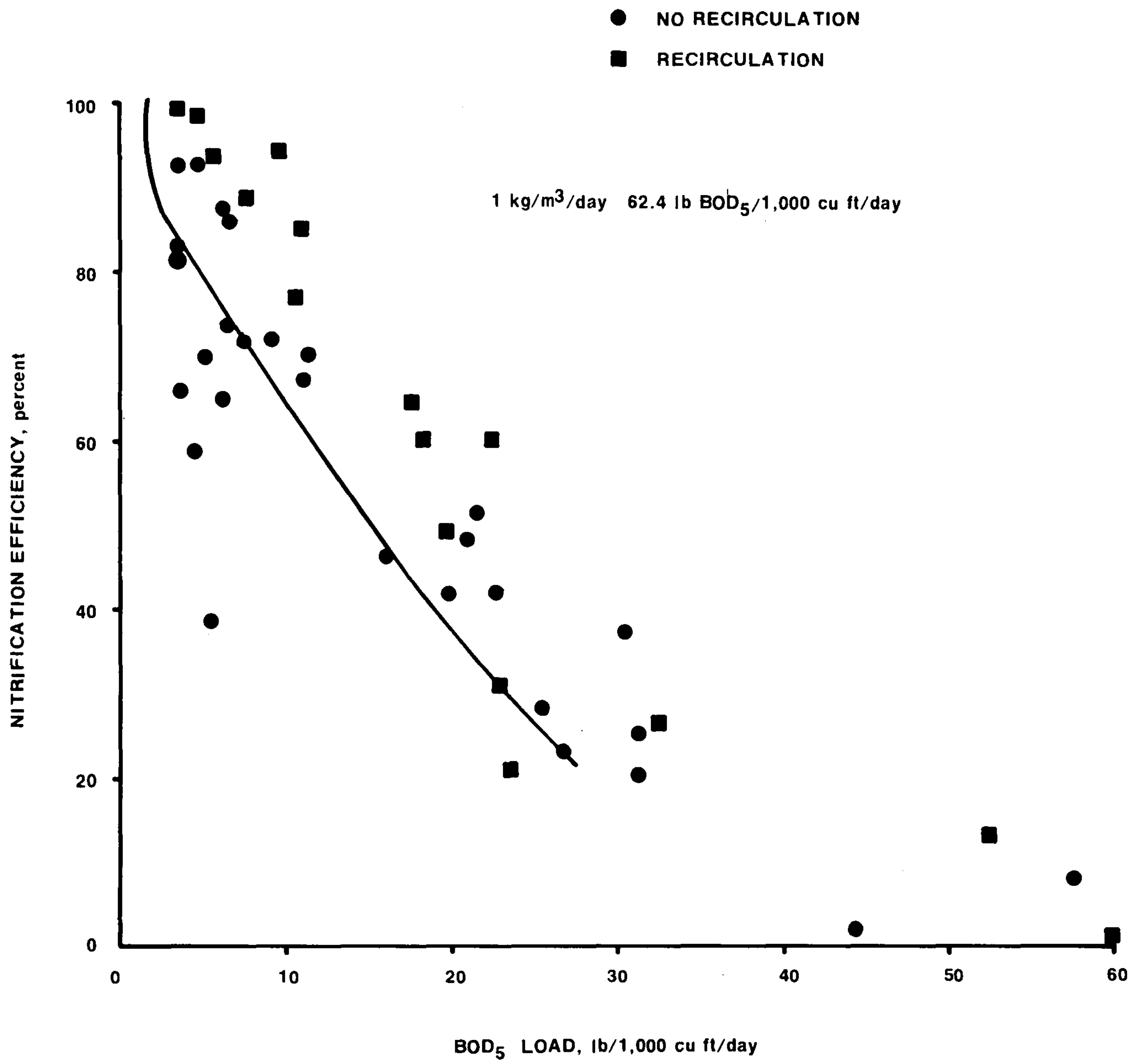
Second Stage Filter Nitrification Efficiency

Location	BOD Load	Effluent NH_3-N	NH_3-N Removal	
	(lb/day/1,000 cu ft)	(mg/l)	(mg/l)	(%)
Johannesburg, SA	3.4	4.4	20.8	83
" "	4.3	9.1	12.9	59
" "	6.3	8.3	15.6	65
Northhampton, E	3.7	11.2	21.8	66



ROTATING BIOLOGICAL MEDIA
TEMPERATURE EFFECTS

FIGURE 16



EFFECT OF ORGANIC LOAD ON NITRIFICATION EFFICIENCY OF ROCK MEDIA TRICKLING FILTERS

FIGURE 17

Organic nitrogen removals in rock media trickling filters are also unpredictable. The organic nitrogen in biological waste treatment plant effluents typically consists of 1-3 mg/l of soluble refractory organic nitrogen. Also, about 10 percent of the effluent suspended solids are organic nitrogen. Raw waste organic nitrogen sources may cause an additional effluent organic nitrogen in attached growth processes. To attain an organic nitrogen concentration of less than 3 mg/l, effluent filtration is probably required.

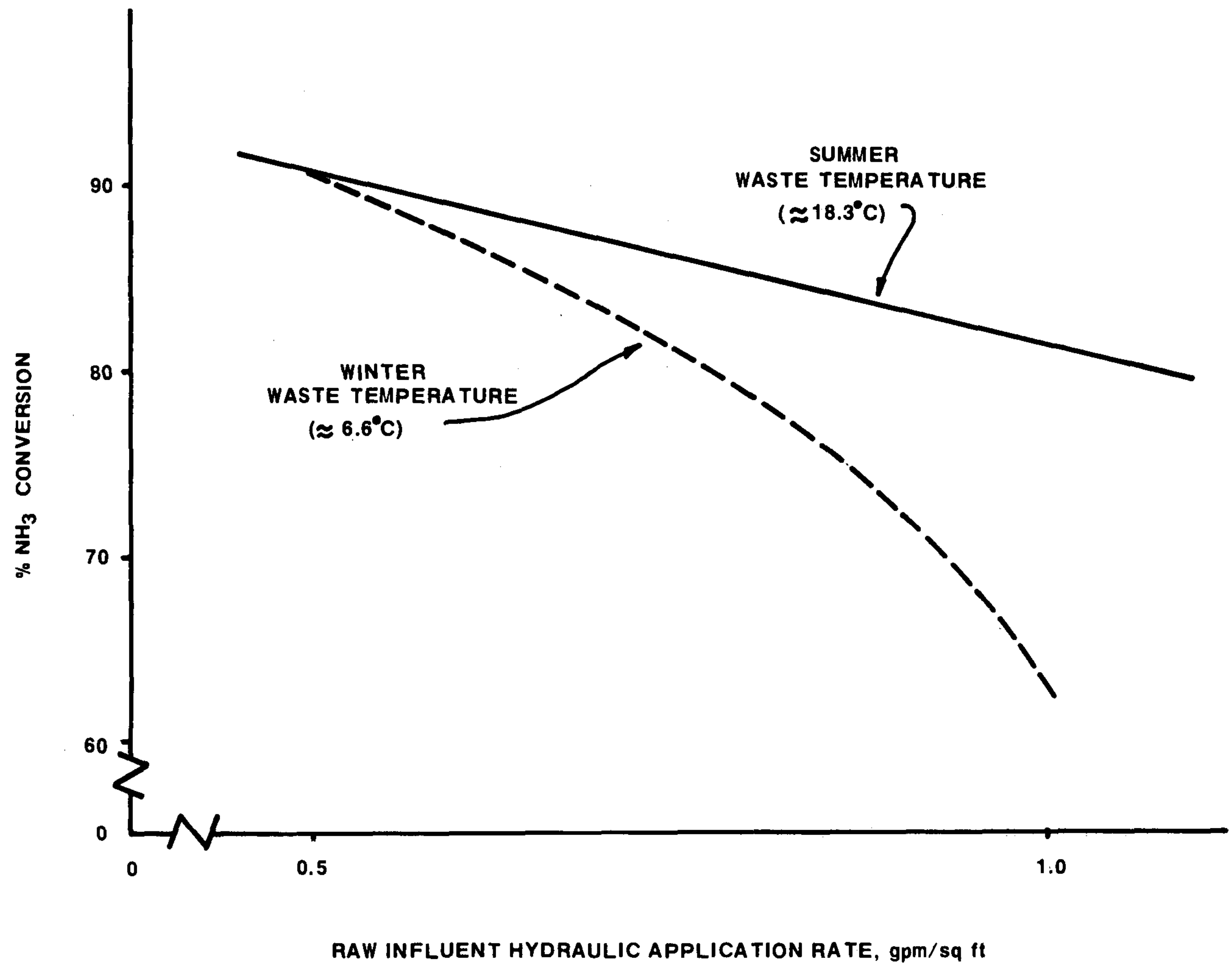
Plastic media trickling filters have been proposed for nitrification. Duddles, et al[23] reported on a second stage plastic media trickling filter with a loading rate of 0.5 gpm/sq ft treated waste flow. The typical influent BOD_5 was reported to be 20 mg/l. It can be calculated that the BOD_5 loading was 11 lb/1,000 cubic feet per day for the 0.5 gpm/square foot loading. Ammonia removals of 90 percent were achieved. Temperature effects at these loadings were not influential as shown on Figure 18. Stenquist, et al[24] reported that a single combined carbonaceous/nitrification trickling filter at 14 lb/1,000 cubic feet per day, attained average effluent ammonia concentrations of 1 mg/l at a pilot plant in Stockton. Raw waste flow application rates were 0.15 to 0.20 gpm/sq ft. Temperatures were always in excess of 20C during the pilot work. Both plants used a 21.5 feet deep medium. Current reports of the full scale Stockton plant indicate that at 14 pounds of BOD_5/1,000 cubic feet, effluent ammonia concentrations are 4-5 mg/l.

The above studies show effluent organic nitrogen concentrations to be 0.9-2.7 mg/l[23] and 7.2-12.7[24]. Filtered effluents from these studies showed the soluble effluent organic nitrogen to be 0.8-2.0 mg/l[23] and 2.1-3.0 mg/l[24]. The Stockton plant receives canning wastes containing higher than normal organic nitrogen concentrations; therefore, it is likely that this organic nitrogen data are not typical.

It appears that to attain 90 percent, plus, nitrification efficiency, BOD_5 loadings must be maintained below 10 pounds per 1,000 cu ft in a single stage plastic media trickling filter, or below 10 pounds per 1,000 cu ft and 0.5 gpm/sq ft in a second stage plastic media trickling filter. At these low loading rates, temperatures above 10C do not appear to influence the degree of nitrification.

Rotating biological media systems have also been proposed for nitrification. The Gladstone, Michigan plant data indicate that flow application rates of 1.0 to 2.0 gpd/sq ft and BOD loadings of from 24-76 pounds/1,000 cubic feet resulted in ammonia removals of from 0-96 percent with an average of 66 percent. Temperatures varied from 8C to 20C. Effluent pH varied from 6.5-7.4 and is influenced by alum feed of about 60 mg/l as well as the nitrification effect.

Pilot plant studies were conducted at Belmont, Indiana[26] using the RBM unit as a nitrification unit preceded by a carbonaceous waste treatment process. With BOD loadings of 5-14 lb/day/1,000 cubic feet and hydraulic loadings of 1.8-3.0 gpd/sq ft ammonia removals ranged from 60-94 percent.



PLASTIC MEDIA TRICKLING FILTER
LOADING – TEMPERATURE – PERFORMANCE RELATIONSHIP
OF A NITRIFYING TRICKLING FILTER (22)

FIGURE 18

Date	T	$^V/Q$ (cu ft/gpm)	g-gpd/ sq ft	BOD in-mg/l	NH_3-N in mg/l	NH_3-N out mg/l
3/23-3/27	14.3C	17.5	2.6	8	11	1.4
3/28-4/30	16.4	15.1	3.0	17	14	5.7
5/1-5/13	19.1	23.9	1.9	18	12	1.9
5/17-5/26	20.0	25.2	1.8	16	8	0.5
5/27-6/17	21.8	15.6	2.9	18	12	1.9

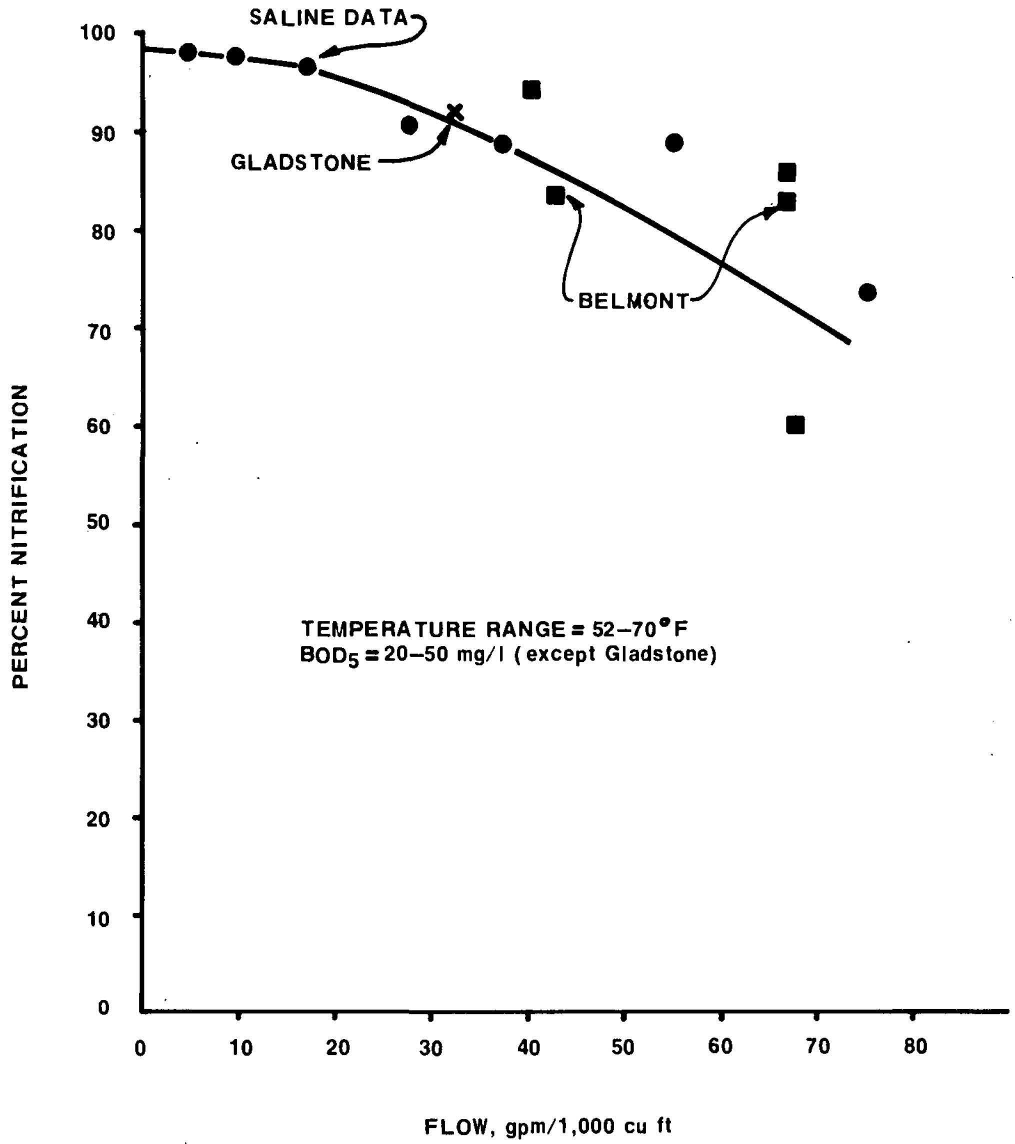
At Saline, Michigan, pilot plant studies of disc type RBM have been conducted to determine nitrification capabilities. These data are shown on Figure 19. Also shown are the Belmont data. From these data, the hydraulic loading must be below 35 cu ft/gpm or 24,300 cu ft/million gallons to obtain 90 percent nitrification. This corresponds to a unit hydraulic loading of about 2.0 gpd/sq ft of effective surface area for lattice type RBM media and about 3.0 gpd/sq ft of effective surface area for disc type RBM media. The data from the disc media used at Saline, Michigan, and the lattice media used at Gladstone, Michigan and Belmont, Indiana indicate that unit surface area has a little effect on the nitrification results. It appears that hydraulic loads, even with low influent BOD concentrations influence the nitrification efficiency. Sufficient data to assess the temperature effects are available only from Gladstone. Figure 20 presents the relationship developed by Antonie to fit the data available from Gladstone[21]. The Saline data indicate that at lower loadings than those experienced at Gladstone, temperature has less effect on nitrification efficiency.

Solids Production

Field data for solids production are always subject to errors in sampling, measurement and system storage complications. Solids production is an important design consideration for all wastewater treatment schemes. The wastewater applied to the attached growth biological system will be composed of biodegradable organics which will be in the solid and soluble form and non-biodegradable volatile and nonvolatile solids. The portion of influent settled sewage non-biodegradable solids were presented previously and for the typical waste represent about 0.38 lb/lb BOD_5.

The theoretical range of solids production from organic synthesis is from 0.15-0.75 pound per pound of BOD_5. The "normal" value of biological cell production is about 0.3 pound per pound BOD_5. Therefore, a typical total solids production (including solids lost in the effluent) would be 0.68 pound per pound BOD_5. If the effluent solids were 30 mg/l, the waste solids production would be about 0.45 pound per pound BOD_5 for a typical domestic waste.

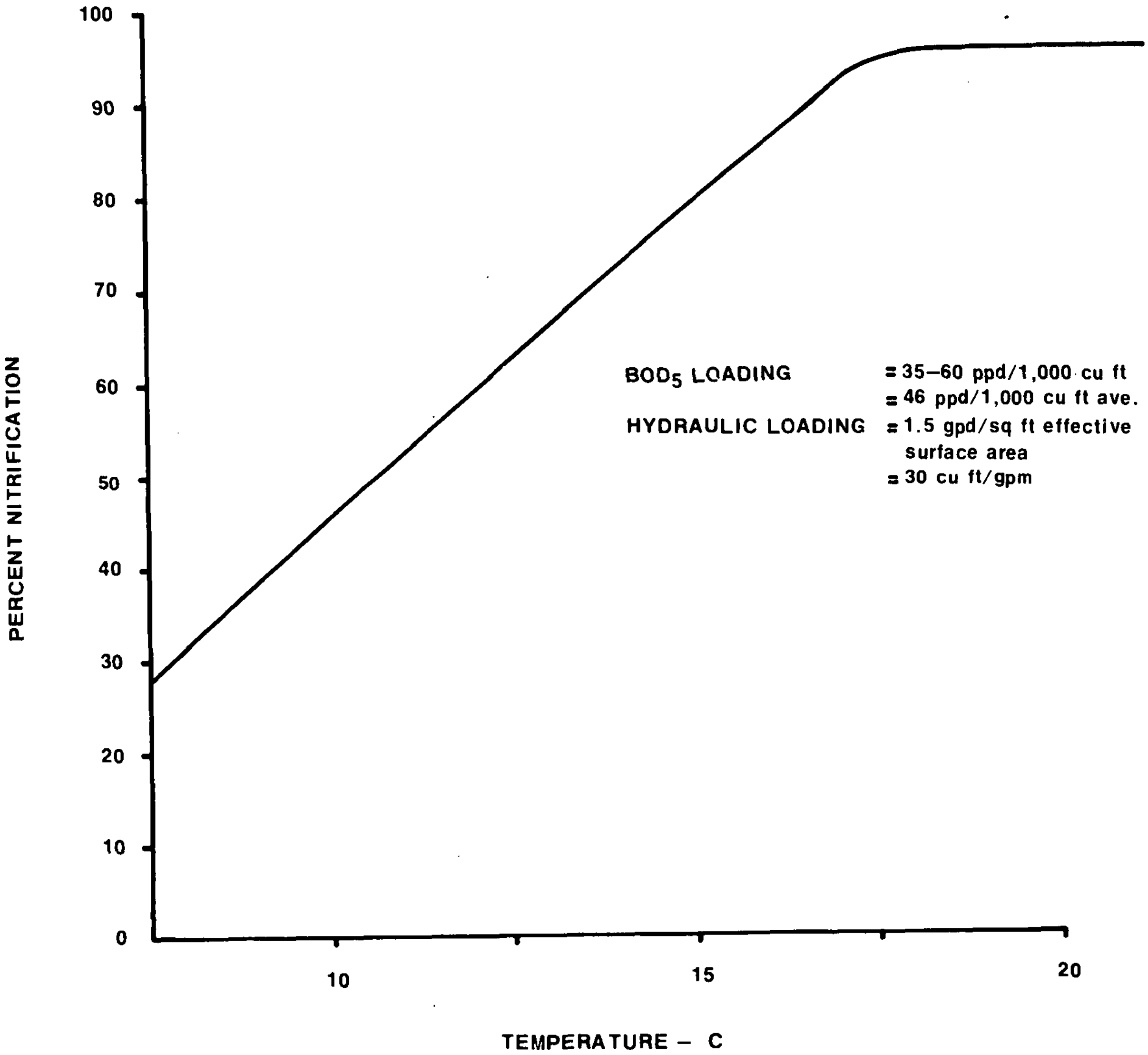
Data are shown in the following tabulation for sludge production from attached growth plants. The variability of the sludge production figures are typical. However, note that three of the total solids production values are near the typical solids production value. The waste sludge production values are calculated based on reported solids production and the solids leaving the system.



RBM PROCESS

NITRIFICATION – HYDRAULIC LOAD RELATIONSHIP

FIGURE 19



RBM PROCESS

NITRIFICATION – TEMPERATURE RELATIONSHIP

FIGURE 20

Process	Location	Total Solids Production (lb solids/lb BOD_5 Applied	Waste Sludge Production (lb solids/lb BOD_5 Applied	Effluent Solids (mg/l)
Rock Media	Dallas, Tx. North Plant	0.42	0.22	40
Rock Media	Dallas, Tx. South Plant	0.65	0.33	43
RBM	Pewaukee, Wisc.	0.62	0.43	30
ABF	Corvallis, Ore.	0.67*	0.39	34
ABF	Rochester, Minn.	0.47*	0.39	17

*Estimated from volatile solids data.

For nitrification, solids production values are very low. The theoretical solids production is 15 percent of the dry weight of ammonia nitrogen nitrified. For example, a waste having an influent ammonia nitrogen concentration of 20 mg/l will produce 3 mg/l of solids. This is a small quantity and is lost in the significance of the carbonaceous solids production values.

PROCESS PERFORMANCE

The characteristic capability and reliability of various processes is an important consideration in meeting effluent criteria. Not only is the average effluent quality important, the extremes must be considered to assure meeting the criteria imposed on most all plants. This section will review reported data for the various processes discussed.

Extended Aeration and Conventional Activated Sludge

The activated sludge process has the capability of converting essentially all influent soluble organic matter to solids. It is necessary to efficiently remove the solids in order to attain high quality effluents in terms of organics. Unfortunately, plain sedimentation of flocculant solids is not easily predicted. When dealing with large input solids quantities, density currents, and thickening considerations, careful operational consideration of solids balances is necessary to attain good effluent quality consistently.

The data from activated sludge processes reflect the problems in attaining consistently good effluent quality. The Deeds and Data section of the JWPCF reports data from 20 plants during the period from 1960 to 1965. Plant BOD loadings ranging from 18 to 74 pounds BOD_5/1,000 cubic feet resulted in average effluent BOD_5 values of 3 to 86 mg/l with 8 of the 20 plants reporting average BOD_5 values of less than 20 mg/l.

Data are shown on Figure 21. Data has been selected to exemplify representative experience and potential process capability. The data in each case represent daily data for an entire year. The plants selected experience a range of loadings. Also, shown on Figure 21 are typical data for oxidation ditch plants which will be discussed later. The conclusions which may be made from these data are:

1. Two plants shown have significant industrial waste flows. The High Point, North Carolina Eastside plant receives textile dye wastes and the Grand Island, Nebraska plant received slaughter-house wastes. Both plants perform as well as the domestic waste plants.

2. The loadings on the plants range from 20 to 80 pounds of BOD_5 per 1,000 cubic feet of aeration capacity. The performance of the plants are not related to unit organic loading to the aeration basin.

3. The Grand Island plant data are presented for the best one year of data (1968) and the worst one year of data (1965) from the same plant. A long period of operator training by the consulting engineer and continual data monitoring on this plant is part of the reason for the excellent improvement in effluent quality.

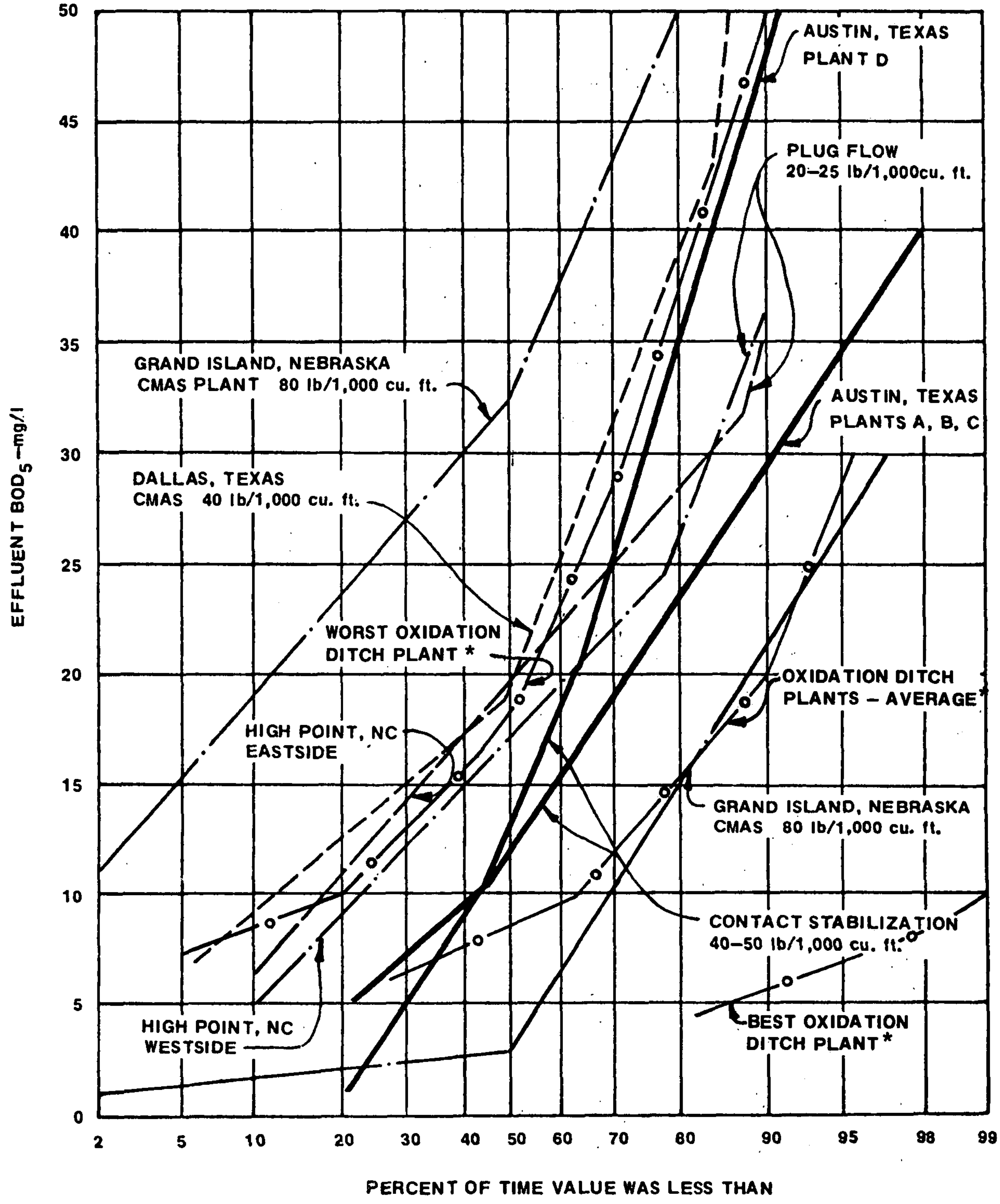
4. Whereas all of the plants shown are considered to have good operational control and design, the Grand Island plant, for one year, produced an effluent BOD significantly better than 10 mg/l, 70 percent of the time. Four of the plants produced an effluent better than 35 mg/l, 90 percent of the time. This level of treatment is a fair representation of current activated sludge process capability and reliability under typical conditions.

Many extended aeration plants do not practice good sludge inventory and wasting management and periodic discharges of high solids concentrations are experienced. Extended aeration plants typically will "burp" the solids upon high flows to the plant. The results of a plant study by Morris, et al[28] are shown on Table 6 which emphasize poor solids management.

The <u>potential</u> for the activated sludge process is better exemplified by the Grand Island plant producing a quality better than 5 mg/l, 50 percent of the time and 20 mg/l, 90 percent of the time.

Biological nitrification of ammonia to nitrate is a well established phenomenon and several bench scale processes and demonstration processes have shown virtually complete conversion is possible if sufficient oxygen transfer is available. Several activated sludge plants having excess oxygen transfer capability do nitrify; however, until the past few years, few plants routinely monitored effluent ammonia.

A source of good data suitable for probability analysis on activated sludge nitrification is available from the Dallas demonstration pilot plant. The plant was a constant flow (150 gpm) plant receiving trickling filter effluent having an average BOD_5 of 60 mg/l. The aeration basin was loaded at 20 pounds/1,000 cubic feet and had an average hydraulic detention



* OXIDATION DITCH PLANT DATA BASED ON 17 PLANTS.

ACTIVATED SLUDGE
EFFLUENT QUALITY

FIGURE 21

TABLE 6

EXTENDED AERATION PERFORMANCE[28]
(Reference: Morris, et. al)

Date	Flow, gpd	MLSS, mg/l	Effluent BOD, mg/l	Effluent Suspended Solids, mg/l	Effluent NH_3-N, mg/l
Aug. '61					
8	20,400		10		0.48
9	18,400	6,580	9	17	0.46
10	18,000	5,480	9	30	0.42
11	18,900	6,000	10	20	0.48
12	22,800	5,910	6	12	0.62
13	26,600	6,090	8	12	0.44
14	21,200	6,440	11	14	0.44
Dec. '61					
12	27,800	6,380	14	14	0.54
13	23,900	6,580	10	15	0.42
14	24,000	7,240	8	69	0.54
15	22,300	6,260	8	20	0.48
16	47,300	6,220	>71	1500	3.00
17	36,200	6,600	24	20	1.06
18	42,800	6,480	34	190	0.54
Mar. '62					
6	32,000	4,640	21	29	1.06
7	39,900	4,440	100	180	2.72
8	59,100	5,360	34	45	1.30
9	71,500	5,340	210	490	1.74
10	46,800	5,180	34	32	2.04
11	58,100	5,180	43	110	1.16
12	45,300	5,380	50	58	2.48
May '62					
14	28,000	8,000	26	15	7.06
15	23,900	7,860	27	12	6.70
16	21,300	8,320	28	16	5.96
17	22,000	7,980	34	25	6.00
18	22,400	7,900	27	21	3.20
19	23,600	8,220	18	14	2.50
20	22,300	7,960	19	12	2.70

time of 4 hours. Sludge retention time (SRT) varied from 7 to 20 days.

The activated sludge effluent BOD and ammonia nitrogen are shown on Figure 22.

The effluent BOD median value was less than 20 mg/l and 50 percent of the time a zero ammonia nitrogen value was obtained. Seventy percent of the time an effluent ammonia value of less than 2 mg/l was obtained. Poorer results were obtained when SRT's in excess of 15 days occurred. Clarifier solids buildup associated with attempting to thicken sludge in the clarifier resulted in denitrification and poorer quality. The pilot plant was monitored continually and the operators were highly skilled individuals who reacted quickly to ill effects.

The data for this study show that the activated sludge process may produce an effluent quality of 2 mg/l NH_3-N seventy percent of the time.

Oxidation Ditch

The oxidation ditch extended aeration process has enjoyed consistently good results insofar as reliability and performance are concerned. Table 7 shows results of performance from several plants. Data is presented on Figure 21 representing a recent survey of operating data from 17 plants.

The results show consistently low average effluent values, with peak values which are typical of other activated sludge plants, but lower than the poorly managed extended aeration or conventional activated sludge plants. The one Texas plant, on Table 7, shows peak effluent BOD and TSS values indicating the need for good solids management, which if not practiced, will result in poorer effluent quality.

Trickling Filters

Selected trickling filter plant effluent data are presented in Figure 23 to indicate process reliability. Process capability has been presented in detail in earlier sections of this presentation. A guideline summary is presented in Figure 24 relating approximate effluent quality to organic loading. The data on Figure 23 indicate that the effluent quality variation is probably no more than the influent quality variation.

Rotating Biological Media

Rotating biological media, as a secondary treatment alternative, is relatively new and only a few plants have been in operation for more than one year.

Very few full scale data are available.

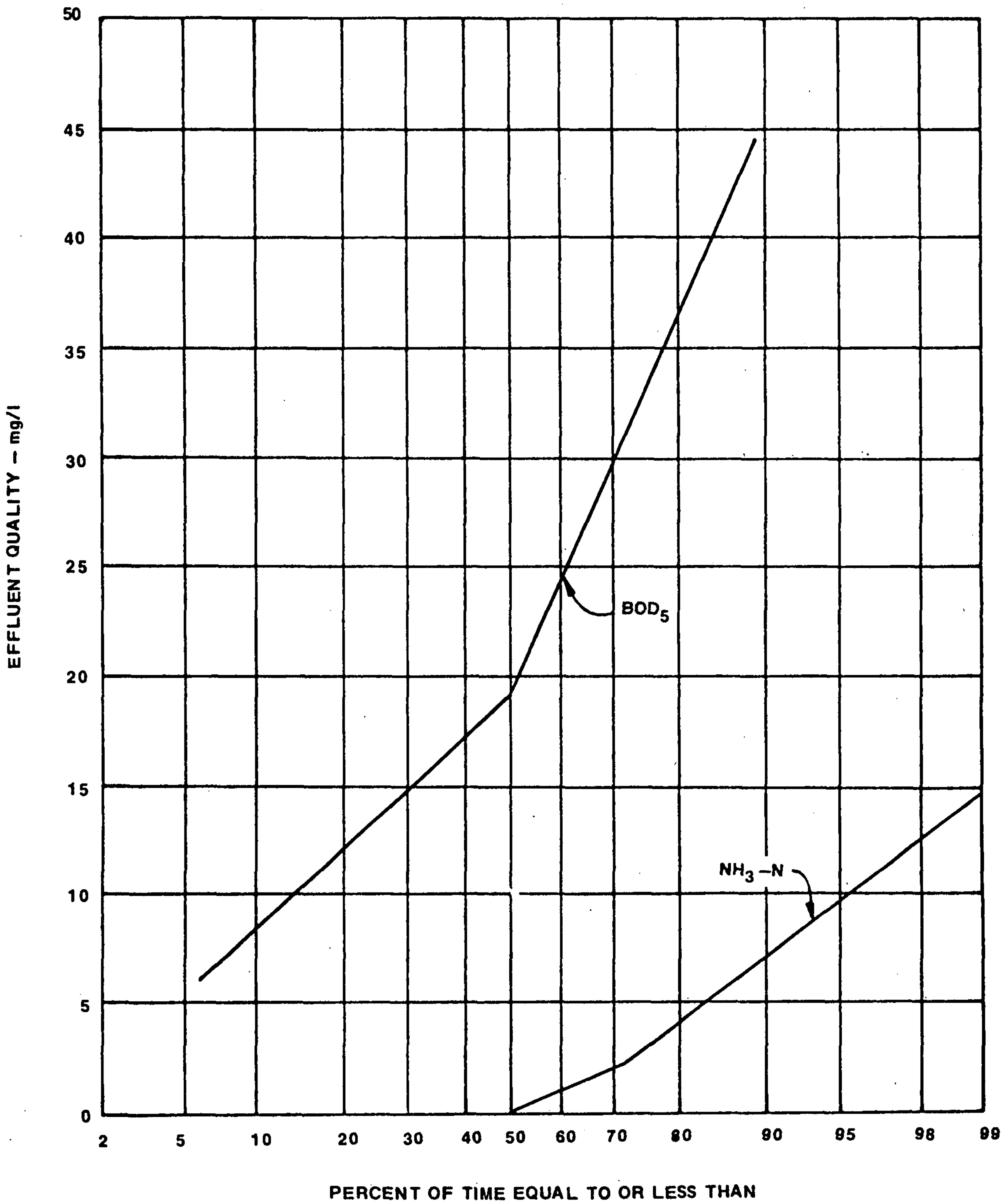
Recently, the data from the Gladstone, Michigan plant have become available affording a detailed analysis of the RBM process capability at one plant. Return frequency data for the Gladstone plant are shown on Figure 25.

TABLE 7

OXIDATION DITCH PERFORMANCE

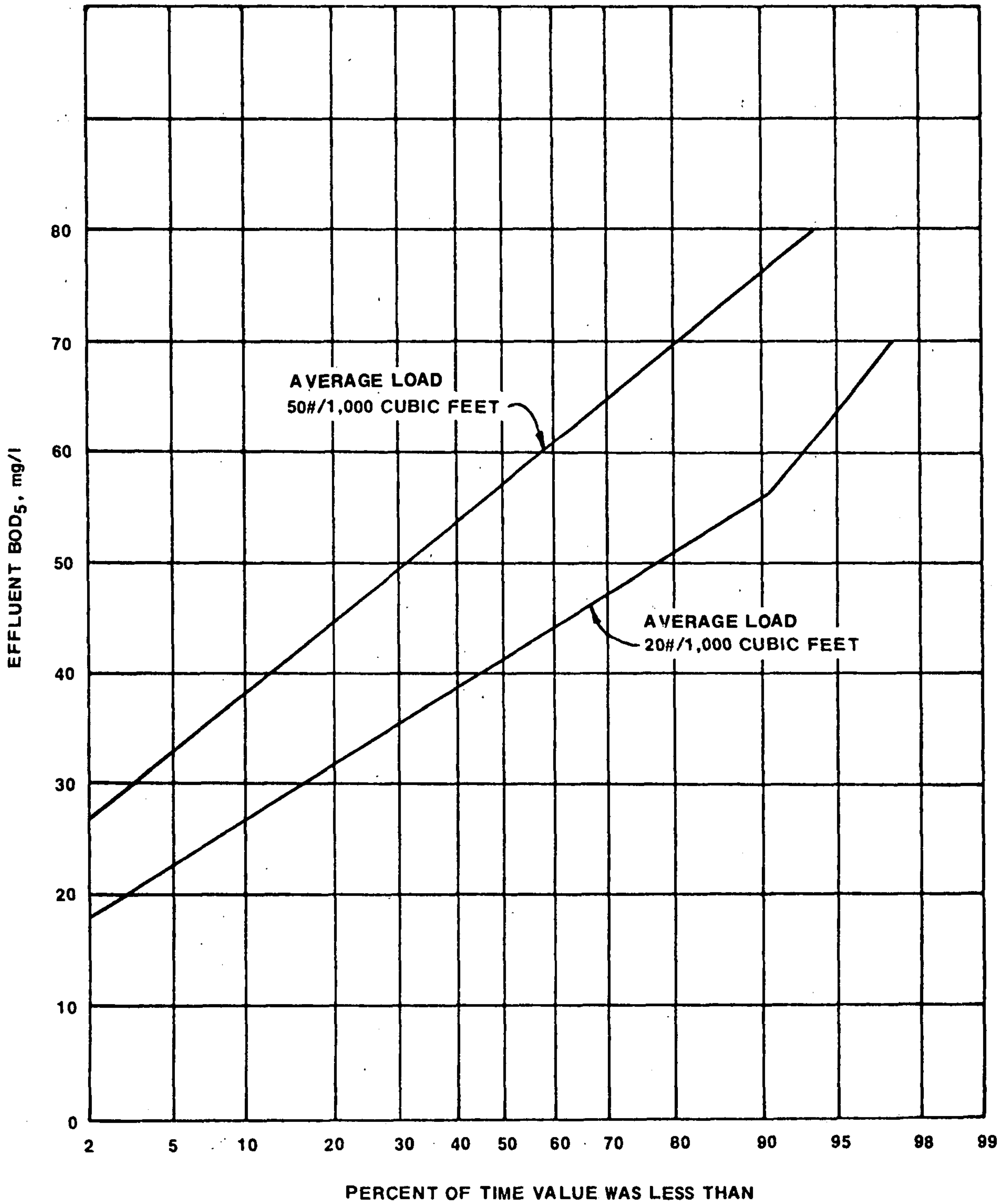
	Period of Record months	Q (mgd)	Ave. Effluent Quality-mg/l BOD	TSS	NH_3	Org N	Peak Effluent Values-mg/l BOD_5	TSS	NH_3
Glenwood, Minn.	2	0.34	7	13	8.2	2.3	18	34	19
Somerset, Ohio	9	0.10	7	15	0.1	-	19	35	0.7
W. Liberty, Ohio	12	0.20	2	2	-	-	3*	6*	-
Lucasville, Ohio	12	0.20	3	8	-	-	7*	10*	-
Sugar Creek, Ohio	2	0.8	12	8	-	-	14	9	-
Brookston, Ind.	1	0.20	7	6	-	-	12	20	-
Clayton Co., Ga.	12	0.44	5	10	-	-	15	40	-
Paris, Texas	18	3.90	17	14	-	-	60	60	-

*Peak Month



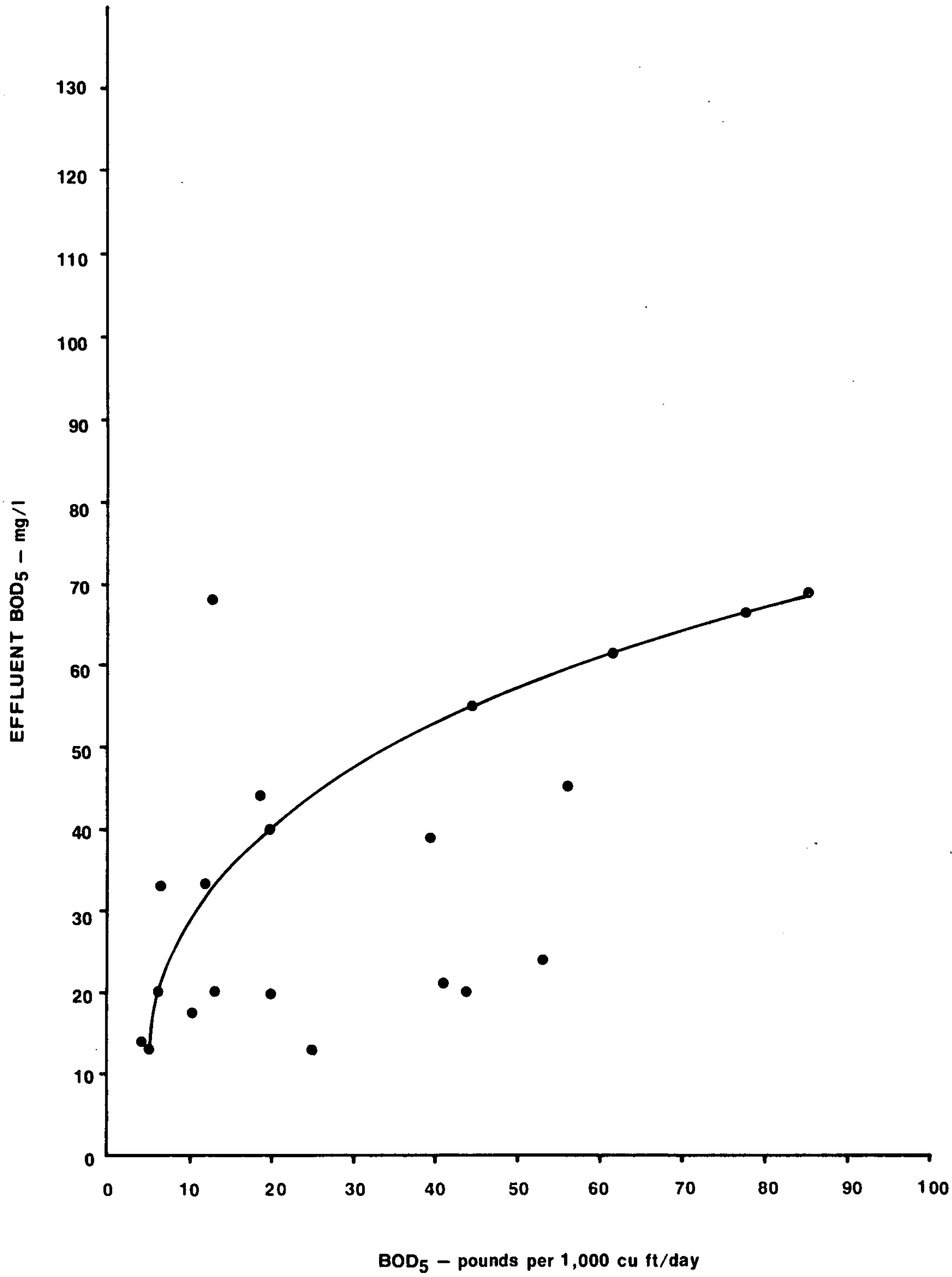
ACTIVATED SLUDGE EFFLUENT QUALITY, DALLAS, TEXAS NITRIFICATION PILOT PLANT

FIGURE 22



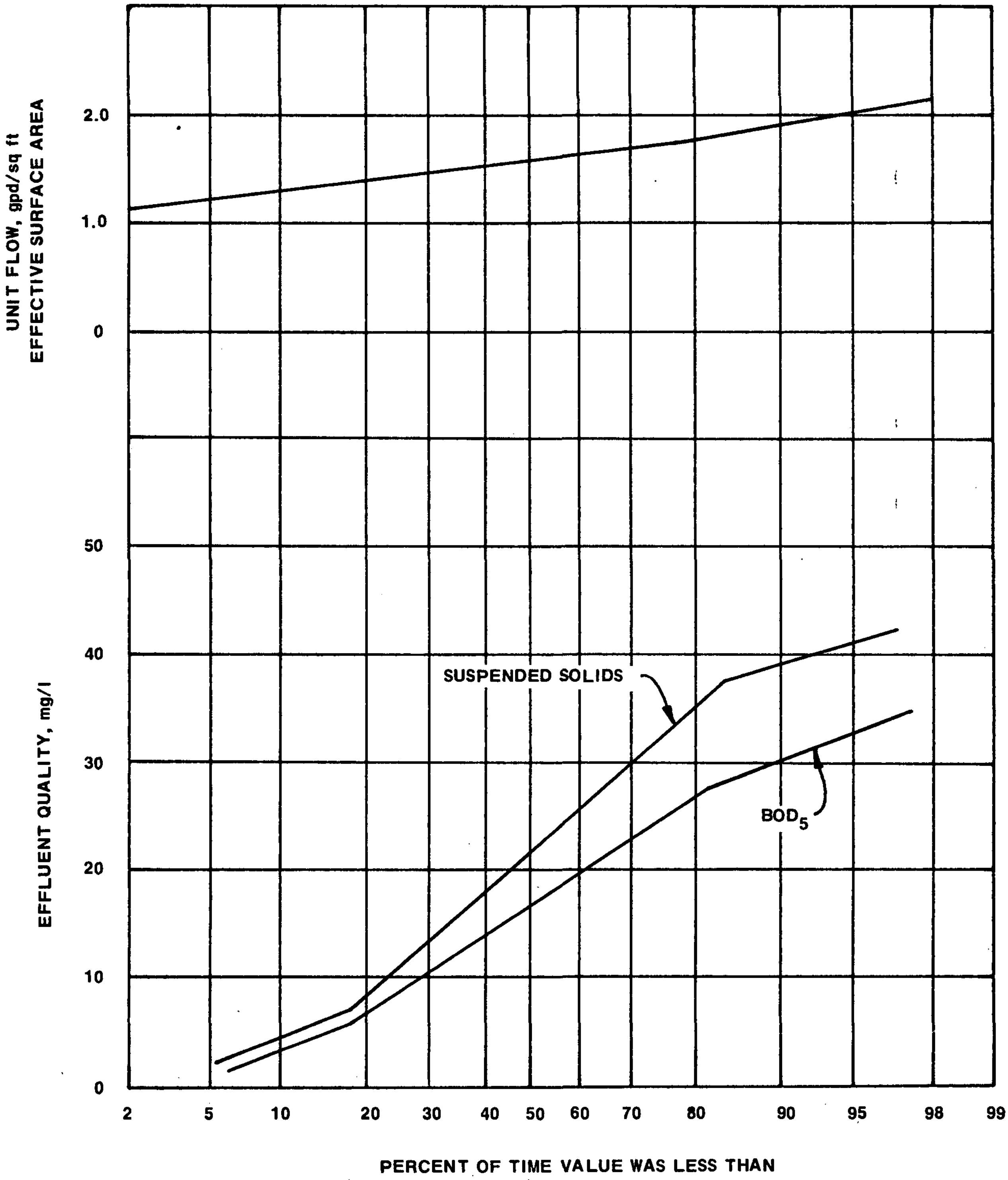
TRICKLING FILTER
EFFLUENT QUALITY
TWO TEXAS PLANTS

FIGURE 23



EFFLUENT QUALITY
TRICKLING FILTERS

FIGURE 24



RBM EFFLUENT QUALITY
GLADSTONE, MICHIGAN

FIGURE 25

The Gladstone, Michigan plant is a 1 mgd plant and consists of primary sedimentation, RBM's designed for 1.94 gpd/sq ft of effective surface area, chemical addition, and final sedimentation. The plant started in March of 1974 and reached stable operation by June of 1974. The manufacturer's literature would predict the following effluent quality based on the operating data when chemicals were not added.

Month	BOD_5 in mg/l	Q gpd/sq ft	Predicted Removal BOD_5 %	Predicted Removal NH_3-N %	Predicted Effl. Quality BOD_5 mg/l	Predicted Effl. Quality NH_3-N mg/l
June, 1974	99	1.5	97.5	99	7(17)*	-
July, 1974	105	1.7	92	97	8(19)	0.6(<1.0)
Aug., 1974	102	1.5	92.5	99	8(12)	0.2(<1.0)

(*) Actual values

The actual results are shown in parenthesis. For the three months of operation when chemicals were not added, the effluent BOD_5 averaged 16, whereas a BOD_5 of 8 mg/l would be predicted by the manufacturer's literature.

The conclusions which may be reached based on the Gladstone, Michigan data are as follows:

At low unit flow rates (1.0-2.0 gpd/sq ft) effluent BOD_5 values from the RBM, will be comparable to activated sludge processes.

Ammonia nitrogen concentrations in the Gladstone, Michigan effluent exceeded 2 mg/l consistently; however, good nitrification was experienced during the warmer summer months.

A review of effluent data fro various biological waste treatment processes indicates that capability to achieve year around effluent BOD_5 and NH_3-N criteria for well designed and operated plants may generally be assigned as follows. Specific plants designed for unusual temperature and/or industrial wastes may be assessed differently.

	Effluent BOD_5 or Suspended Solids 50% of Time	Effluent BOD_5 or Suspended Solids 90% of Time	Ammonia N*-mg/l 50% of Time
Conventional Activated Sludge	20	40	1
Extended Aeration	10	30	1
Oxidation Ditch	10	30	1
Trickling Filter	30	40	3
RBM	20	40	3

*If system is designed for nitrification

The key area of alternative comparison is equitable cost comparisons. In the facility planning stage of a project, the cost estimation is necessarily based on generally defined facility components. To make comparisons of costs of several alternatives, it is impractical to make detailed lists of material and equipment components for each alternative; therefore, the use of general cost estimating guides for the process functional units are relied upon.

This section presents procedures which may be used to develop construction costs and operating and maintenance requirements of the alternative processes previously described. Estimates are presented for construction costs as a function of appropriate capacity parameters for the major plant components. The total initial investment, which includes engineering, fiscal, administrative and land costs are not shown but may be developed on the basis of these relationships.

To make planning cost estimates for a project, several techniques are used. For conventional facilities, or often used unit processes, the results of previously developed detailed cost estimates may be extrapolated to the project at hand. Extrapolation of costs requires consideration of different unit size, local variations in labor and material costs, differences in site requirements, inflation, and added or reduced ancillary systems. Although each consideration may be quantified, considerable judgement on the part of the estimator is required offering potential error in the estimate.

Where extensive cost data are not available, other techniques must be employed. Alternative procedures include a thorough takeoff of a specific component and relating the cost of the facility to the component by a factor. A procedure commonly used in chemical industry is to add the costs of all major purchased equipment and multiply an appropriate experience factor times the equipment purchase cost to determine the overall facility cost; typically used factors range from 2.0 to 3.0, depending on the equipment intensity. For example, experienced ratios of equipment purchase cost to installed facility cost for vacuum filters range from 2.2 to 2.7 based upon detailed estimates of cost of several projects. Again this method is subject to considerable judgement and may afford opportunity for significant error.

The most frequently used approach to estimate costs for facilities which do not have significant historical cost background is to:

a. Define the facilities by dimensions, construction material, equipment piping and valve requirements. A general plan of the facility is drawn defining walls, overall dimensions, and structural requirements.

b. Estimate quantities of major cost components: Rules of thumb are applied to derive quantities, e.g., concrete walls - 8 inch minimum, or 1 inch per foot of height. Concrete footings - two thirds the quantity of wall concrete.

c. Estimate costs of major cost components including: concrete, equipment, piping and valves, excavation, housing.

d. Add 10 to 20 percent of sum of cost for miscellaneous minor cost components, which are not detected in the major cost items.

The use of any method of cost estimating requires careful consideration of inflation. This has been especially true for the last 5 years since inflation of construction costs have averaged about 9 percent per year. The rapid change in costs effects both the use of previous cost estimates to predict project costs and the planning for project cost which may be 6 months to one year away from time the planning estimate is prepared.

Many planners and engineers are accustomed to using cost indices which track costs of specific items and proportion these costs in a predetermined mixture. Unfortunately, there is all too much evidence that these time honored cost indices are not understood by the user, and/or are inadequate for many specific applications.

The basis for all cost indices used in the construction industry is to monitor the costs of specific construction material and labor costs, proportion these costs by a predetermined factor and thereby derive an index. The most frequently used indices are probably the Engineering News Record's (ENR) Construction Cost Index and Building Cost Index.

The ENR indices were started in 1921 and intended for general construction cost monitoring. The large amount of labor included in the construction cost index was appropriate prior to World War II; however, on most all contemporary construction, the labor component is far in excess of current labor usage. In fact, there should be little, if any, application of the construction cost index to water utility plant projects. This index does not include mechanical equipment, pipes and valves, which are normally associated with water utility plant construction, and the proportional mix of materials and labor are not specific to water utility construction.

To provide a more specific index the Environmental Protection Agency developed a Sewage Treatment Cost Index. This index was based on the cost components of a hypothetical 1 mgd trickling filter plant. The quantities of labor, materials, construction equipment and contractor's overhead and profit remain constant and the unit prices and price changes as derived from the U.S. Bureau of Labor Statistics and Engineering News Record are applied to the constant quantities to derive the index. Because this index was specific to a single process and because more activated sludge plants are being constructed currently, the EPA has developed a new index based on the components of a hypothetical 5 mgd activated sludge plant and 50 mgd activated sludge plant followed by chemical clarification and filtration.

Obviously, the more specific an index is, the more accurately it will track cost change. The variation in inflation of various cost components cannot be monitored by a single component index. If an index is based on an improper mixture of several single component indices, it also will fail. It is necessary for the planner to recognize the shortcomings of cost inflation and use judgement and the best data at hand in deriving budget comparative estimates.

The cost estimating techniques used for the various unit processes involved in this study are varied. Where historical cost data are available, these have been used. Where little or no historical cost data are available, costs are developed by identifying costs of major components and adding experience factors for miscellaneous unaccounted for features. The basis of estimating each functional unit is described in the following paragraphs. The cost relationships are shown graphically in Appendix A. The costs presented include electrical work associated with the unit function and a 15 percent contingency.

Raw Wastewater Pumping (Figure A-1)

Raw wastewater pumping stations are often incorporated into other structures at small community wastewater treatment plants. When inappropriate to incorporate the pumping station into other structures at the plant site, the use of package pumping stations is common. The construction costs for the raw wastewater pumping station reflect construction costs of both prefabricated and custom designed pumping stations with a separate concrete wetwell and the use of manually cleaned basket screens for pump protection.

Preliminary Treatment (Figure A-2)

Preliminary treatment includes screening, grit removal and flow measurement. The provisions for screening are based on comminutors for flows less than 0.5 mgd, and mechanically cleaned screens without shredders for flows in excess of 0.5 mgd. A manually cleaned screen in a bypass channel is provided. Grit removal is based on an aerated grit basin with grit pumping to a grit washer. Flow measurement is based upon a Parshall flume.

The design basis for these facilities is peak flow rate.

Sedimentation Basins (Figure A-3)

Costs for construction of plain sedimentation basins with sludge collection equipment have been presented in earlier cost studies by Black & Veatch[29]. These cost estimates were made on the basis of plants larger than 1 mgd. For plants smaller than 1 mgd, estimates of quantities have been prepared during this study for selected sedimentation basin sizes. To provide updating of the previous information, the cost data from the Black & Veatch study were used as well as quantity takeoff information from several selected sedimentation basin sizes.

The cost data are presented as a function of the surface area provided, as was done in the earlier study. Costs are based on the use of two basins. The basin depth will affect the cost of the sedimentation basin; albeit, minor variations will not exceed the accuracy of the estimate. The cost data presented have been based on a basin having 15 feet side water depth and a 1.5 feet freeboard. Cost components are presented on the basis of steel launders and weirs. The costs for basin surface areas in excess of 1,500 sq ft are applicable to sedimentation basins using circular sludge collection equipment in circular basins. Cost data for basins less than 1,500 sq ft in surface area are applicable to straight line sludge collection equipment in rectangular basins.

Waste Sludge Pumping Stations (Figure A-4)

Waste sludge pumping equipment is selected based on the sludge concentration to be pumped and the operation intended. Sludge pumping units which operate continuously may be centrifugal pumps, so long as one avoids high solids concentrations and large suction head losses. Normally better control is established using intermittent sludge pumping and use of positive displacement pumps.

Positive displacement pumping units are more expensive than equal capacity centrifugal pumping units.

The cost data presented in the earlier study by Black & Veatch were based on positive displacement pumping units. This study updates those costs. A practical limitation is imposed as to the minimum size of pumping unit and sludge piping which can be used. This limitation is reflected in the cost estimate by 10 gpm.

The station is based on an underground structure which houses pumping units and piping, constructed adjacent to and having common walls with the solids separation unit process. A superstructure is included to access the station from the ground level and to house electrical control equipment.

Prefabricated Extended Aeration Plants (Including Aeration) (Figure A-5)

Prefabricated extended aeration plants are typically used for extremely small flows. Estimates for capacities from 10,000 to 90,000 gpd were made. Costs are presented for shop fabricated units. At some point the economics shift in favor of field fabricated units and the designer should investigate this for each application. Air requirements are based on 2,100 cubic feet per pound of BOD removed (2 lbs BOD/1,000 gallons). Aeration using positive displacement blowers with 100 percent standby are provided. Prefabricated extended aeration plants include a sedimentation zone, return sludge pumping, waste sludge storage, and chlorine contact basin, but not chlorine feed equipment. The prefabricated plant is estimated on the basis of an above ground unit installed on a concrete pad. Freight costs are included at $15 per cwt. A contingency allowance of 15 percent was added to the manufacturer's estimate of the equipment and erection costs. In addition, percentages of equipment costs were used for electrical (15 percent) and contractor's overhead and profit (25 percent).

Prefabricated "Contact Stabilization" Plants (Including Aeration) (Figure A-5)

Construction costs have been developed for prefabricated contact stabilization plants although the specific design approach has not been presented. The prefabricated plant for contact stabilization is more closely akin to conventional activated sludge and is normally used without primary sedimentation. Single stage systems are not normally adaptable to situations requiring nitrification for the same reasons explained for typical conventional activated sludge systems.

The prefabricated contact stabilization plant normally has a 3 hour contact zone and a reaeration zone. Although the flow path is identical to the true contact stabilization process, the contact zone is about 6 times larger. True contact stabilization relies on adsorption/absorption of organics in the contact zone with little or no real stabilization. A reaeration zone is provided to condition the return activated sludge to provide a suitable SRT. The prefabricated plant provides relatively short term stabilization in the contact zone and further stabilization in the stabilization zone.

Prefabricated contact stabilization plants are normally provided with return sludge and waste sludge pumping, aerobic digestion of waste activated sludge and a chlorine contact basin. The estimated prices shown include blowers and blower housing.

Custom Designed Extended Aeration Basins (Figure A-6)

For plants larger than 100,000 pd, the use of prefabricated construction becomes marginally economical. The use of either concrete structures, steel basins, or concrete lined, earthen basins becomes more desireable. The construction costs estimates presented for custom designed aeration basins are based on construction with structural concrete and concrete lined earthen basins. Provisions are included for walkways, supports, and handrails for the structural concrete basin. The estimated costs reflect a square or circular geometry associated with a completely mixed aeration basin in contrast to the long narrow basins sometimes associated with plug flow.

Oxidation Ditch Aeration Basins (Figure A-7)

Oxidation ditch aeration basins have been estimated using vertical structural walls and sloped concrete side walls. The costs for these alternative construction systems are very close. The construction cost estimates are shown for either construction system. Aeration equipment is not included in the oxidation ditch basin costs.

Mechanical Aeration Equipment (Figure A-8)

Aeration equipment estimated construction costs include purchase cost as quoted by manufacturers, installation, manufacturer's installation check, and contractor's overhead and profit. Costs are based on fixed platform mounted surface aerators and paddle wheel type aerators.

Diffused Aeration Equipment (Figure A-9)

Diffused aeration equipment is based on the use of centrifugal blowers, wherein two blowers are provided, one serving as a standby. The blowers having inlet filter silencers are housed in a superstructure. Air piping and sparger type diffusers are included.

Recirculation Pumping Stations (Figure A-10)

Recirculation pumping stations include the facilities for return activated sludge pumping stations and similar uncomplicated pumping stations. The basis of the cost estimates shown are of the type of station employing vertical diffusion vane pumping units with attendent valves, piping and control facilities. The pump is suspended in the wetwell and motors and motor control centers are housed in a superstructure. The cost data base for recycle pumping stations is limited because these facilities are normally constructed as part of other facilities.

The Black & Veatch report, "Estimating Costs and Manpower Requirements for Conventional Wastewater Treatment Facilities", presents cost relationships for recycle pumping stations. The few data for recent recycle pumping station costs have been reviewed in relationship to the earlier Black & Veatch cost data. The recent cost data indicate the influences on costs have approximately doubled the cost of recycle pumping stations. These influences include inflation, OSHA regulations, and EPA regulations on reliability which have been instituted since the earlier B & V work.

Trickling Filters (Figure A-11)

Costs for trickling filters were estimated on the basis of rock at $12/cubic yard, redwood media at $2.75/cubic foot, and plastic media at $2.75/cubic foot of media. Rock media trickling filters are based upon a filter depth of 6 to 8 feet and plastic and redwood media filters are based on a depth of 21 feet. Rotating distribution equipment costs were obtained from manufacturers. The cost curves include the facilities within the confines of the biocell foundation and do not include piping to and from other functional units.

Rotating Biological Disks (Figure A-12)

Cost development procedures and unit costs for rotating biological disks have been derived from Autotrol and from limited quantity take-off information provided from recent construction projects.

The manufacturer's estimating cost for 100,000 sq ft (effective area) have been used plus the estimated time associated with installation and tankage as provided by the manufacturer.

Sludge Treatment (Figures A-13 and A-14)

Estimated costs for sludge treatment facilities are presented for anaerobic digestion and sludge drying beds. Aerobic digestion costs, where applicable, may be derived from the construction cost estimates for aeration basins and aeration equipment. The aerobic digester costs derived would represent continuous flow designs, or designs which incorporate decanting provisions but may not represent batch operated systems.

Construction costs for anaerobic digestion have been derived, in part, from the costs presented by Black & Veatch[29]. The costs have been updated by using limited number of costs experienced for recently bid construction projects and inflating the cost relationship based on these more recent costs. Anaerobic digesters represent two stage digestion volume and include provisions for heating to 95F and mixing of the primary digester and include an unheated, unmixed secondary digester of equal size as the primary digester.

Sludge drying beds are based on jobs constructed during the past year (1976) and estimates of intermediate sized installations. The estimated costs include influent distribution piping and valves and perforated underdrains.

Disinfection (Figures A-15 & A-16)

Feed Equipment & Storage. The most prevalent form of disinfection is chlorine gas. The equipment and storage facilities requirements are well known and commercial equipment is readily available. Construction costs for chlorine feed equipment have been presented previously[29]. The previous work cites the difficulty in isolating costs for the chlorine feed and storage facilities. Most often, the chlorine feed and storage facilities are combined with other sturctures, making analysis difficult. Ton cylinders are shown; however, for less than 1,000 pounds per day feed rate, 150 pound cylinders were used as a basis of storage requirements.

Several quantity take-offs of similar chlorine feed and storage facilities were reviewed. Of seven installations, the installed chlorination system facility was estimated to cost from 2.5 to 3.5 times the purchase price of the chlorinators. The average estimated installed cost of the seven installations was 3.0 times the quoted purchase price of the chlorinators above.

The total installed cost includes distribution panels, cylinder chocks, installation, manufacturer's preparation of shop drawings, installation check and startup, and contractor's overhead and profit. Chlorinator costs include one standby chlorinator.

Miscellaneous piping varies significantly depending on the layout. Piping costs will vary from 5 to 10 percent of the installed chlorination equipment cost.

Hoist equipment will be essentially constant for electrically operated, monorail trolley hoists. For large storage areas having long rails and extensive duct-o-bar electrical systems, the costs will approach 30,000 dollars for a 30 cylinder storage system or 1,000 dollars per cylinder. Manually operated hoists systems are less expensive (about half) but require more labor for loading and unloading. For the purposes of this analysis, hoisting equipment is estimated at 0.50 dollars per pound of cylinder storage capacity.

Chlorine Contact Tanks. Contemporary chlorine contact tanks are constructed to provide a serpentine flow path to enable maximum use of the chlorine fed. The construction costs of these structures are much more than single or double pass basins constructed in the past. The costs for the multi-pass contact tanks are presented in this report to reflect current practice. The cost estimates presented are based on 2 basins, and structural concrete construction.

OPERATION & MAINTENANCE REQUIREMENTS

Operation and maintenance requirements include:

Administration
Labor
Power Costs
Chemical Costs
Miscellaneous Supply Costs

For small plants the segregation of these total operation and maintenance costs into the above categories is difficult. Small communities often do not have detailed budgets and in many cases do not maintain records of the total cost of wastewater treatment. Many large utilities have extended their recordkeeping to the costs associated for the above categories by each unit process. Therefore, there are available data to reasonably predict operation and maintenance requirements for larger plants, but any attempt to accurately predict operation and maintenance requirements for small plants is subject to potentially large errors.

The information presented in this section is based on distributing experienced requirements for small community plants on the basis of published information for operation and maintenance requirements from individual process units for larger plants.

The labor requirements are presented on the basis of manhours required. Miscellaneous supply costs are presented on the basis of annual cost.

Labor requirements represent both operation and maintenance labor. Most of the plants are not operated full time, and the plants are unattended at night and on weekends. In these instances, it is necessary to provide alarm monitoring to a continuously manned site, such as the police dispatcher.

Power and chemical requirements are not shown since these may be readily calculated from the system connected and operating equipment and on typical chemical dosage rates. Appropriate unit costs may be applied to values determined.

Miscellaneous supply costs are variable and difficult to assign to individual unit functions. These costs have been assigned in proportion to the distribution found at larger utilities where more detailed records are maintained.

Requirements for site work and laboratory work are a function of plant site size and number of analysis made, respectively, and are presented as such in the following operating and maintenance requirement relationships.

The numbers of samples and laboratory analyses presumed to be performed are outlined below. The unit time required for each analysis and sample are obtained from information derived from the laboratory director of Metropolitan Denver Sewer District No. 1 and from information presented in EPA's Handbook for Analytical Quality Control in Water and Wastewater Laboratories"[30]

PARAMETER	UNIT TIME* (Hours)
BOD	0.24
TSS	0.36
COD	0.36
TKN	0.36
NO_3-NO_2	0.18
NH_3	0.18
PO_4	0.18
Dissolved Oxygen	0.12
pH	0.07
Conductivity	0.07
Turbidity	0.10
Alkalinity	0.18
Color	0.12
Automatic Sample Obtained	0.24
Manual Sample Obtained	0.60
Coliform	0.40
Cl_2 Residual	0.20

*Based on 10 percent nonproductive time plus 5 percent standardization and reagent preparation time plus 5 percent reporting time.

The laboratory and sampling requirements for various numbers of samples and assuming one sample per sampling point per day of operation are summarized below based on automatic samplers and the following analysis per sample:

BOD, TSS, NH_3, pH, Coliform, Cl_2 Residual

LABORATORY MANHOURS REQUIRED PER YEAR

Number of Sampling Points	No. of Days Analyses are Performed Per Year 40	60	80	100	200
2	130	194	260	324	648
4	260	387	520	648	1296
6	387	580	774	970	1940
8	520	774	1040	1300	2600
10	650	970	1300	1630	3240

The cost for laboratory supplies presented in the Black & Veatch study[29] were about 0.70 to 3.00 dollars per manhour required in the laboratory per year. The larger plants required greater supply costs than the smaller plants. The supply costs for small community plants will likely be in the range of 1.00 dollar per manhour.

Yard Maintenance. If the land upon which the facilities are located are landscaped and grassed, the labor and supplies associated with maintenance and care of the yard may be a significant budget item. The requirements for the care of the yardwork is dependent upon climate, types of plantings and area of site. Therefore, the requirements for yard maintenance are basically independent of the flow capacity of the plant. Guidelines are presented in the Dodge Guide[31] which relate yard maintenance to area and these are repeated here to arrive at a basis for estimating yard maintenance.

	Average Frequency/ Year	Labor (Hours /Year 1,000 sq ft)	Materials (Dollars/Year 1,000 sq ft)	Equipment* (Dollars)
Mowing	30	0.5	0.50	160
Fertilization	2	0.1	3.0	5
Crabgrass Control	1/3	0.05	1.50	-
		0.65	5	165

Area of Plantsite	Maintenance/ Labor (Hours)	Material & Equipment Costs (Dollars)
50,000 sq ft	32.5	415
100,000 sq ft	65.0	665
150,000 sq ft	97.5	915
250,000 sq ft	162.5	1415
500,000 sq ft	325.0	2665
1,000,000 sq ft	650.0	5165

*Amortized over 5 years at 8 percent and independent of area.

Comparison of Alternative Processes

The primary purpose of this evaluation is to show examples of the use of the cost data and to generally determine the relative economics of alternative processes most likely to be used for small community wastewater treatment. For secondary levels of treatment, the costs of the following competitive processes were evaluated.

Capacity, mgd	Process
0.01, 0.1	Prefabricated extended aeration plants
0.1, 0.5, 2.0	Custom built extended aeration plants, conventional activated sludge, trickling filters, rotating biological media systems and prefabricated contact stabilization plants.

The applicability of individual processes for specific design flows is not fixed nor representated to imply typical applicability. These examples are merely presented to guide the reader through examples of the use of information in this presentation.

The design conditions for the processes are as follows:

Raw Wastewater:	
Suspended Solids	200 mg/l
Volatile Content	75 percent
BOD_5	200 mg/l
NH_3-N	30 mg/l
Temperature	20C
Peaking Factor (dry weather)	1.5
Peaking Factor (wet weather)	4.0

Effluent Quality	Case I	Case II
BOD_5	25	25
TSS	25	25
NH_3-N	-	3

The secondary process design bases were developed as shown on Table 8. The schematic process diagrams and unit processes are shown on Figures 26 through 33. The examples shown may superficially appear to represent conservative aeration capacities for the plant sizes shown. The peaking capacity required for small plants and the author's opinion that aeration capacity should be provided for peak hour conditions is reflected in these values.

The Case II (nitrification requirement) requires increasing the biological treatment capabilities of all processes except the extended aeration alternatives. Detention and oxygen supply have been included in the extended aeration alternatives (Case I) to assure adequate dissolved oxygen concentrations at normal operating conditions. The modifications which are required to Tables 9-15 to provide for nitrification are as follows: (Go to page 93)

TABLE 8

EXAMPLE PROCESS DESIGN BASIS SUMMARY

Unit Process	Activated Sludge: Extended Aeration	Activated Sludge: Conventional	Attached Growth Systems: Rock Media	Attached Growth Systems: Plastic Media	Attached Growth Systems: Rot. Biological Media
Primary Sedimentation					
Average Overflow Rate (gpd/sq ft)	-	800	800	800	800
Suspended Growth Biological Treatment					
Detention (Hours)	24				
SRT (Days)	20				
Waste Sludge (#/#BOD)	0.6	0.6	0.5	0.5	0.5
Oxygen Supply (#/#BOD)	2.0	1.0	-	-	-
Trickling Filter Design					
K Value	-	-	0.2	0.2	0.3
Recirculation Rate (Q_R/Q_i)	-	-	1.0	2.0	0
Final Sedimentation					
Average Overflow Rate (gpd/sq ft)	600	600	600	600	600
Chlorination					
Contact Detention Time @ Peak Flow (Hours)	0.5	0.5	0.5	0.5	0.5
Dosage Rate @ Peak Flow (mg/l)	10	10	10	10	10
Dosage Rate @ Average (mg/l)	3	3	3	3	3
Aerobic Digestion					
Detention Time, days	-	15	-	-	-
Sludge Concentration, percent	-	2	-	-	-
Anaerobic Digestion					
Primary Detention Time, days	-	15	15	15	15
Secondary Detention Time, days	-	15	15	15	15
Sludge Drying Beds					
Anaerobically Digested sq ft/lb/day dry solids	-	10	10	10	10
Aerobically Digested sq ft/lb/day dry solids	20	20	-	-	-

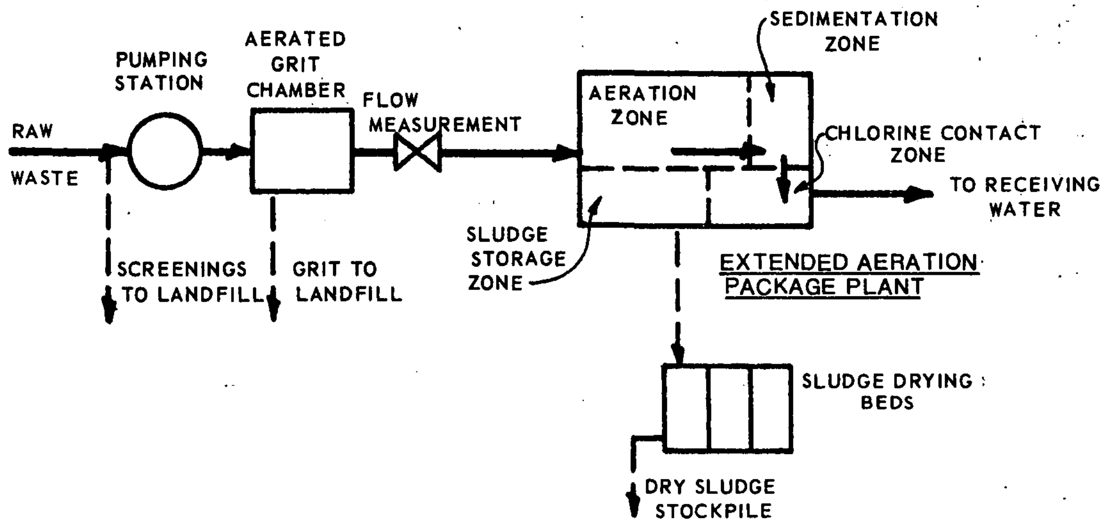


FIGURE 26 – PROCESS SCHEMATIC – EXTENDED AERATION PROCESS (0.01 to 0.1 mgd)

For cases requiring nitrification or not requiring nitrification

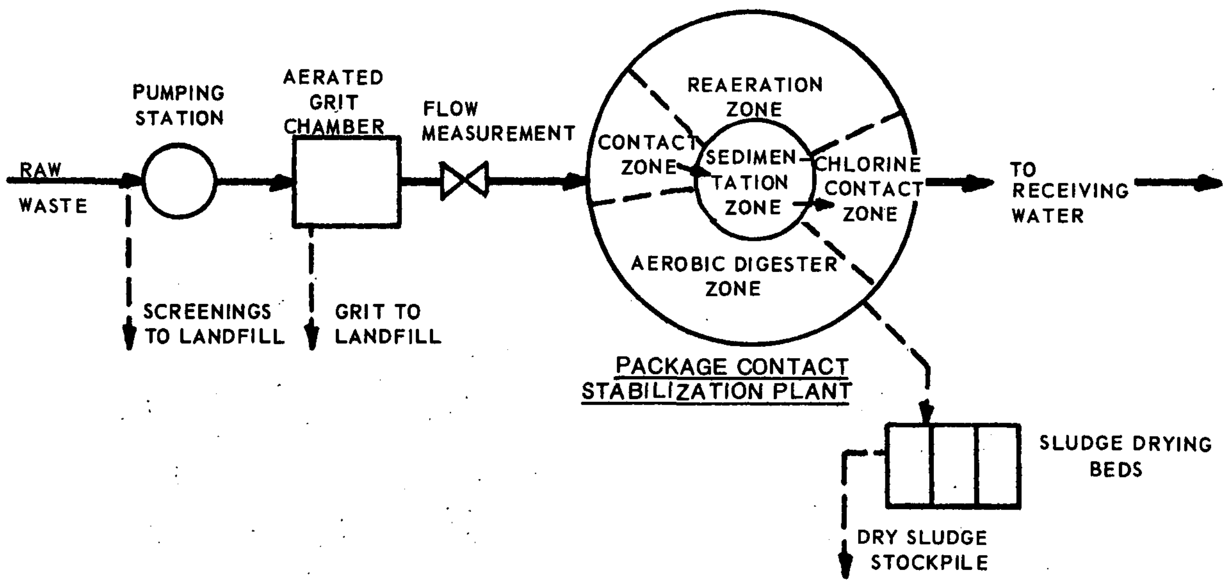


FIGURE 27 – PROCESS SCHEMATIC – PREFABRICATED CONTACT STABILIZATION PLANTS (0.1 to 1.0 mgd)

For cases not requiring nitrification

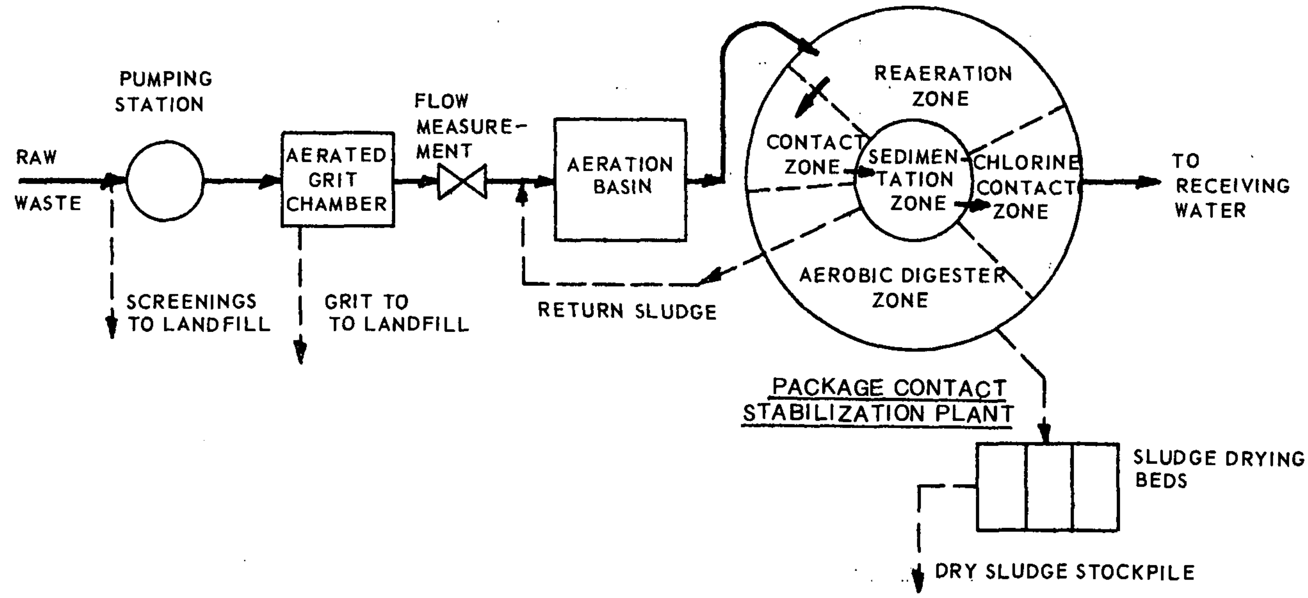


FIGURE 28-PROCESS SCHEMATIC – PREFABRICATED CONTACT STABILIZATION PLANT (0.1 to 1.0 mgd)

For cases requiring nitrification

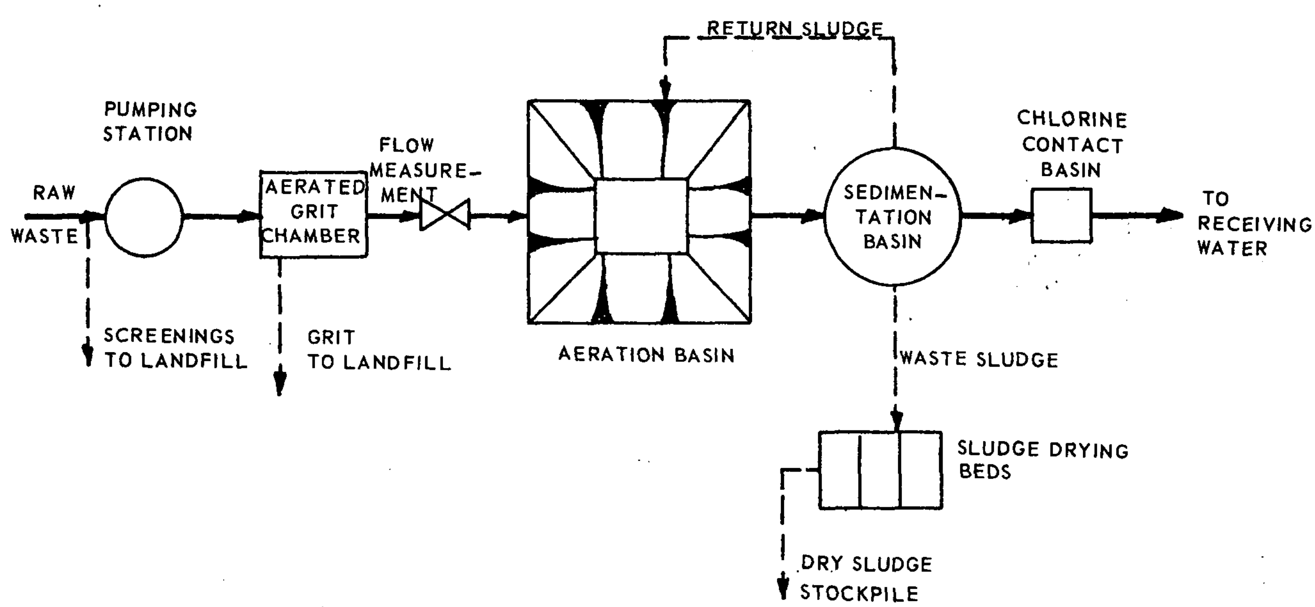


FIGURE 29 – PROCESS SCHEMATIC – CUSTOM DESIGNED EXTENDED AERATION PLANTS (0.1 to 2.0 mgd)

For cases requiring nitrification or not requiring nitrification

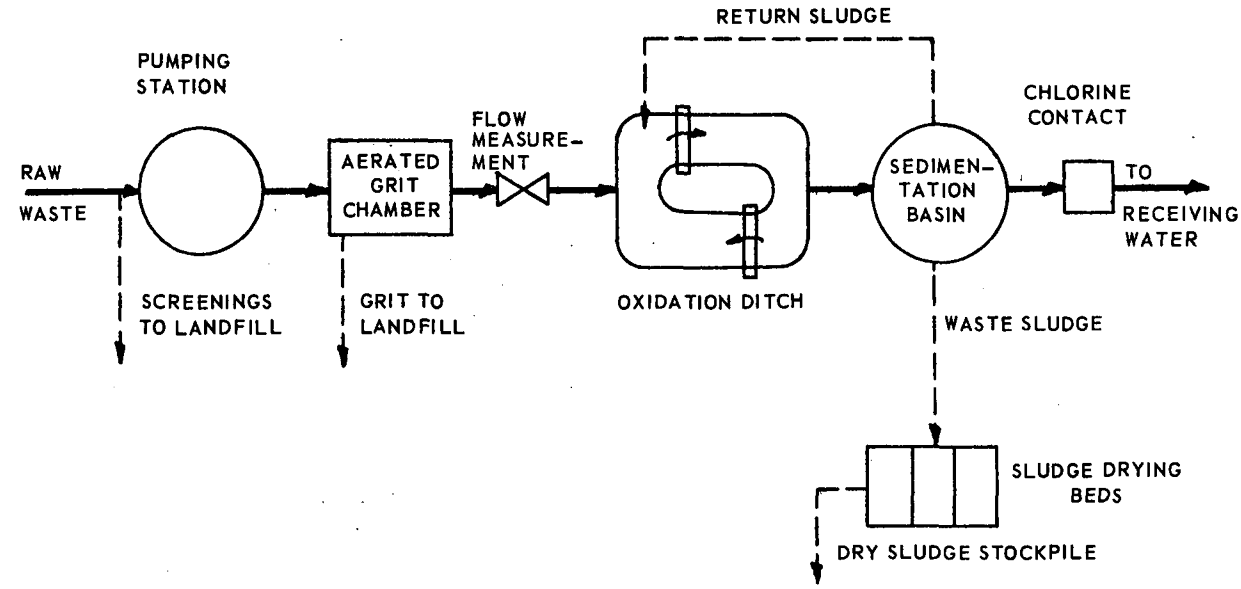


FIGURE 30 – **PROCESS SCHEMATIC – OXIDATION DITCH EXTENDED AERATION PLANT (0.1 to 2.0 mgd)**

For cases requiring nitrification or not requiring nitrification

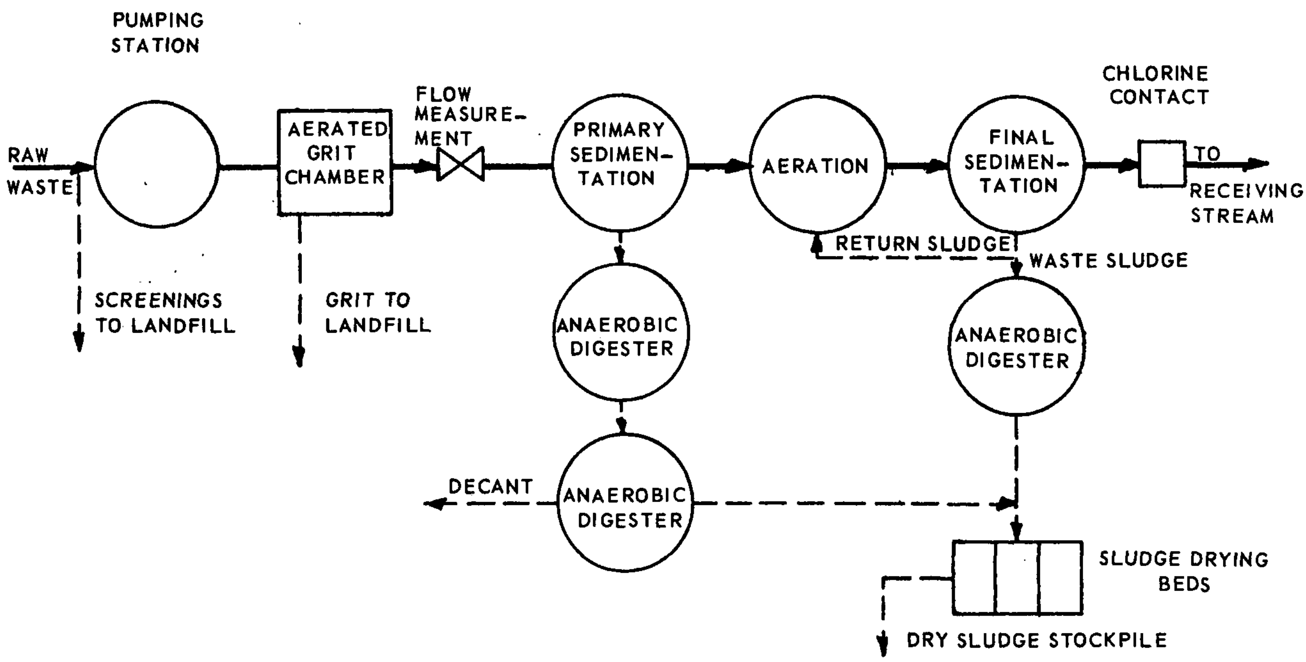


FIGURE 31 – **PROCESS SCHEMATIC – CONVENTIONAL ACTIVATED SLUDGE (0.1 to 2.0 mgd)**

For cases requiring nitrification or not requiring nitrification

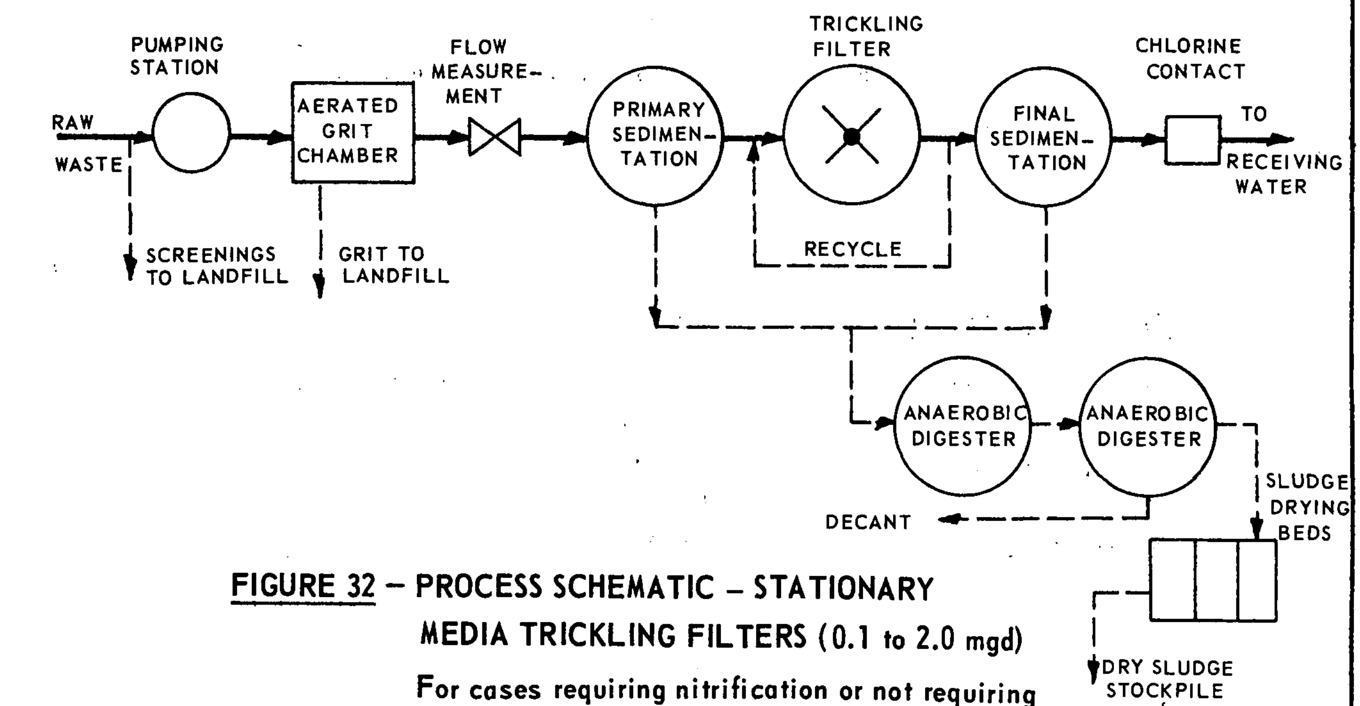


FIGURE 32 – PROCESS SCHEMATIC – STATIONARY MEDIA TRICKLING FILTERS (0.1 to 2.0 mgd)

For cases requiring nitrification or not requiring nitrification

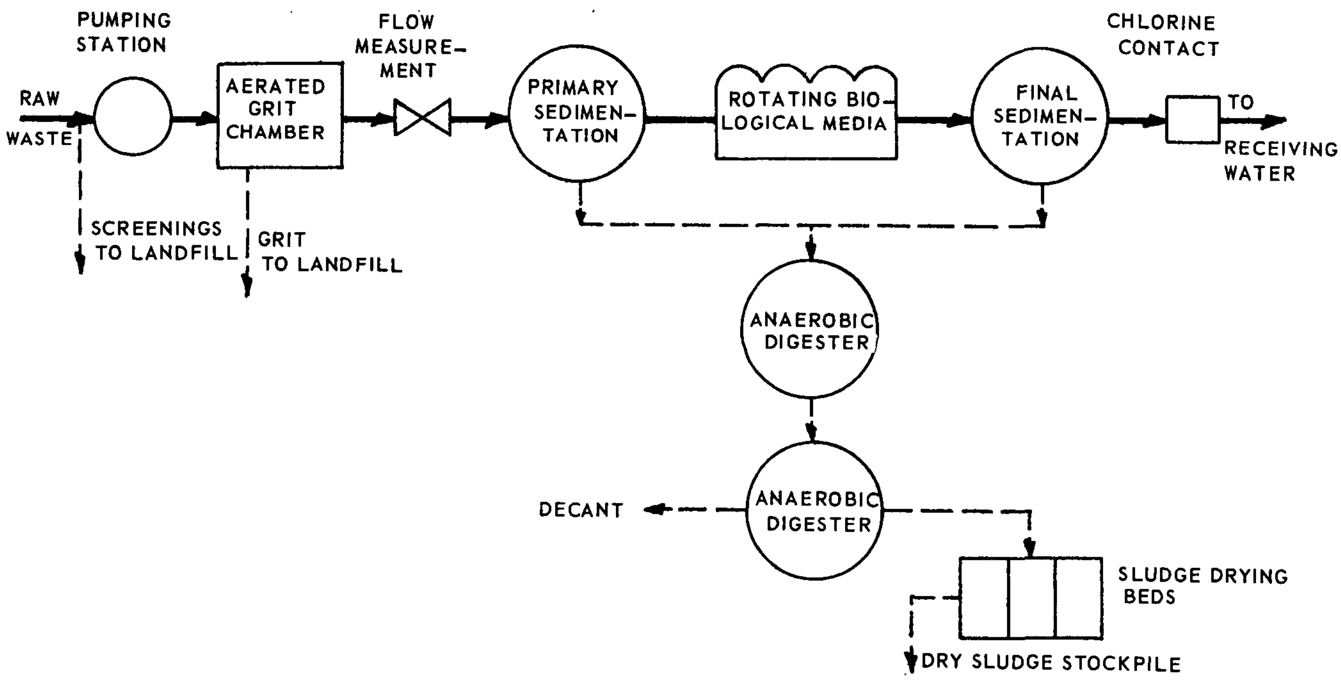


FIGURE 33 – PROCESS SCHEMATIC – ROTATING BIOLOGICAL MEDIA SYSTEM (0.1 to 2.0 mgd)

For cases requiring nitrification or not requiring nitrification

TABLE 9

PREFABRICATED EXTENDED AERATION PLANT

Unit Process/Function	Unit	Plant Design Capacity - MGD 0.01	0.05	0.10
Raw Sewage Pumping Station	mgd capacity	0.04	0.20	0.40
Chlorine Contact	cu ft volume	120	560	1200
Chlorination	ppd capacity	10	25	50
	ppd feed ave.	1	1	3
Drying Beds	(sq ft)	400	1000	2000
Site Area	acres	0.5	0.7	1.0
Lab Analysis	sampling points	2	2	2
	days per year	40	40	80

TABLE 10

PREFABRICATED CONTACT STABILIZATION PLANTS

Unit Process/Function	Unit	Plant Design Capacity - MGD 0.10	0.5	1.0
Raw Sewage Pumping Station	mgd capacity	0.40	2.0	4.0
Preliminary Treatment	mgd capacity	0.40	2.0	4.0
Chlorination	ppd capacity	50	250	500
	ppd feed ave.	3	15	30
Drying Beds	(sq ft)	2000	10,000	20,000
Site Area	acres	1.0	2.0	3.0
Lab Analysis	sampling points	2	2	2
	days per year	80	80	120

TABLE 11

CONVENTIONAL ACTIVATED SLUDGE

		Plant Design Capacity - MGD			
Unit Process/Function	Unit	0.10	0.50	1.0	2.0
Raw Sewage Pumping Sta.	mgd capacity	0.40	2.0	4.0	8.0
Preliminary Treatment	mgd capacity	0.40	2.0	4.0	8.0
Primary Sedimentation	sq ft area	170	850	1700	3400
Sludge Pumping	gpm capacity	10	15	25	50
Aeration Basin	cu ft volume	3000	16,700	33,300	66,700
Aeration Basin	CFM AIR	180	900	1800	3600
Aeration Basin (Alternative)	HP aerators	10	30	60	120
Secondary Sedimentation	sq ft area	170	850	1700	3400
Sludge Pumping	gpm capacity	15	25	50	100
Recirculation Pumping	mgd capacity	0.05	0.25	0.5	1.0
Chlorine Contact	cu ft volume	1200	5600	11,000	22,000
Chlorination	ppd capacity	50	250	500	1000
	ppd feed ave.	3	15	25	50
Aerobic Digester	cu ft volume	800	4000	8000	16,000
	CFM AIR	20	90	180	360
(Alternative)	HP aerators	1	5	10	20
Anaerobic Digester (Primary)	cu ft volume	400	2000	4000	8000
(Secondary)	cu ft volume	400	2000	4000	8000
Drying Beds	sq ft	1800	9000	18,000	36,000
Site Area	acres	1.0	2.5	4.0	6.0
Lab Analysis	sampling points	3	3	3	3
	days per year	80	80	120	200

TABLE 12

CUSTOM BUILT EXTENDED AERATION

Unit Process/Function	Unit	Plant Design Capacity - MGD			
		0.10	0.50	1.0	2.0
Raw Sewage Pumping Sta.	mgd capacity	0.40	2.0	4.0	8.0
Preliminary Treatment	mgd capacity	0.40	2.0	4.0	8.0
Aeration Basin	cu ft volume	13,300	66,700	133,300	266,700
Aeration Basin	CFM AIR	600	3000	5700	11,400
Aeration Basin (Alternative)	HP aerators	20	100	200	400
Secondary Sedimentation	sq ft area	170	850	1700	3400
Sludge Pumping	gpm capacity	10	30	60	120
Recirculation Pumping	mgd capacity	0.05	0.25	0.5	1.0
Chlorine Contact	cu ft volume	1200	5600	11,100	22,200
Chlorination	ppd capacity	50	250	500	1000
	ppd feed ave.	3	15	25	50
Drying Beds	sq ft	2000	10,000	20,000	40,000
Site Area	acres	1.0	2.5	4.0	6.0
Lab Analysis	sampling points	2	2	2	2
	days per year	80	80	120	200

TABLE 13

EXTENDED AERATION OXIDATION DITCH PLANT

Unit Process/Function	Unit	Plant Design Capacity - MGD 0.10	0.50	1.0	2.0
Raw Sewage Pumping Sta.	mgd capacity	0.40	2.0	4.0	8.0
Preliminary Treatment	mgd capacity	0.40	2.0	4.0	8.0
Aeration Basin	cu ft volume	13,300	66,700	133,300	266,700
Aeration Basin	HP aerators	20	100	200	400
Secondary Sedimentation	sq ft area	170	850	1700	3400
Sludge Pumping	gpm capacity	10	30	60	120
Recirculation Pumping	mgd capacity	0.05	0.25	0.50	1.0
Chlorine Contact	cu ft volume	1200	5600	11,100	22,200
Chlorination	ppd capacity	50	250	500	1000
	ppd feed ave.	3	15	25	50
Drying Beds	sq ft	2000	10,000	20,000	40,000
Site Area	acres	1.0	2.5	4.0	6.0
Lab Analysis	sampling points	2	2	2	2
	days per year	80	80	120	200

TABLE 14

ROCK MEDIA TRICKLING FILTERS

Unit Process/Function	Unit	Plant Design Capacity - MGD			
		0.10	0.50	1.0	2.0
Raw Sewage Pumping Sta.	mgd capacity	0.40	2.0	4.0	8.0
Preliminary Treatment	mgd capacity	0.40	2.0	4.0	8.0
Primary Sedimentation	sq ft area	170	850	1700	3400
Sludge Pumping	gpm capacity	10	15	25	50
Secondary Sedimentation	sq ft area	170	850	1700	3400
Sludge Pumping	gpm capacity	10	25	50	100
Recirculation Pumping	mgd capacity	0.10	0.50	1.0	2.0
Trickling Filter	cu ft volume	7150	35,750	71,500	143,000
Chlorine Contact	cu ft volume	1200	5600	11,100	22,200
Chlorination	ppd capacity	50	250	500	1000
	ppd feed ave.	3	15	25	50
Anaerobic Digester (Primary)	cu ft volume	700	3400	6700	13,400
(Secondary)	cu ft volume	700	3400	6700	13,400
Drying Beds	sq ft	1400	7000	14,000	28,000
Site Area Rock Media	acres	1.5	3	5	7
Plastic Media	acres	1.0	2.5	4	6
Lab Analysis	sampling points	3	3	3	3
	days per year	80	80	120	200

TABLE 15

ROTATING BIOLOGICAL MEDIA

Unit Process/Function	Unit	Plant Design Capacity - MGD 0.10	0.50	1.0	2.0
Raw Sewage Pumping Sta.	mgd capacity	0.40	2.0	4.0	8.0
Preliminary Treatment	mgd capacity	0.40	2.0	4.0	8.0
Primary Sedimentation	sq ft area	170	850	1700	3400
Sludge Pumping	gpm capacity	10	15	35	50
Secondary Sedimentation	sq ft area	170	850	1700	3400
Sludge Pumping	gpm capacity	10	30	60	120
RBM System	cu ft volume	3700	18,000	32,000	74,000
Chlorine Contact	cu ft volume	1200	5600	11,000	22,200
Chlorination	ppd capacity	50	250	500	1000
	ppd feed ave.	3	15	25	50
Anaerobic Digester (Primary)	cu ft volume	700	3400	6700	13,400
(Secondary)	cu ft volume	700	3400	6700	13,400
Drying Beds	sq ft	1400	7000	14,000	28,000
Site Area	acres	1.5	3	5	7
Lab Analysis	sampling points	3	3	3	3
	days per year	80	80	120	200

CHANGES TO CASE I CONDITIONS FOR NITRIFICATION

ALTERNATIVE	PLANT DESIGN CAPACITY - MGD 0.1	0.5	1.0	2.0
Prefabricated Extended Aeration	NO CHANGE			
Prefabricated Contact Stabilization				
Add: Preceding Aeration Basin (cu ft)	3,000	15,000	30,000	
Surface Aerators (hp)	10	40	80	
Recycle Pumping Station mgd	0.05	0.25	0.5	
Conventional Activated Sludge				
Increase: Aeration Basin Size to (cu ft)	5,700	28,500	57,000	114,000
Aeration Capacity to (CFM)	380	1,900	3,750	7,500
or (HP)	15	60	120	240
Custom Built Extended Aeration	NO CHANGE			
Oxidation Ditch	NO CHANGE			
Rock Media Trickling Filters, Increase Media Volume To	11,300	56,500	113,000	226,000
Rotating Biological Media Increase Media Volume To	7,400	37,000	74,000	148,000

Other factors, besides economics, which affect the selection of alternative processes include the ease of operation, process reliability, process and mechanical reliability and the effect of sludge treatment process alternatives on the overall process. Table 16 presents general advantages and disadvantages of the alternative secondary treatment processes which may or may not be reflected in the economic analysis.

Construction costs and operating and maintenance requirements were developed from the relationships shown in Appendix A. In addition to the construction costs for the unit processes for which the cost curves include electrical work and contingencies, costs are provided for the following:

Site Improvements	15 percent of subtotal of unit process costs
Engineering, Legal, Administrative	25 percent of construction costs
Interest During Construction	5 percent of subtotal of projections

TABLE 16

PROCESS ADVANTAGES AND DISADVANTAGES OF BIOLOGICAL TREATMENT ALTERNATIVES FOR SMALL COMMUNITY APPLICATIONS

	Advantages	Disadvantages
Prefabricated Extended Aeration Plants:	1. Stable process when proper sludge management is performed. 2. Standardized design and components readily available. 3. Package design permits relocation, if necessary, for growing metro areas. 4. High quality effluent. 5. Predictable process.	1. Small air lift pumps clog often, at specific plants. 2. Requires good operator skills and routine monitoring to assure continuing high quality effluents. 3. Sufficient oxygen supply should be provided for nitrification and pH may need to be controlled.
Prefabricated Contact Stabilization Plants:	1. Two basins of active sludge provide opportunity for fast recovery after upsets caused by hydraulic peak loads or toxic loads. 2. Standardized design and components readily available. 3. Can be re-erected at other sites, with difficulty for growing metro areas. 4. High quality effluent. 5. Predictable process.	1. Small air lift pumps clog often, at specific plants. 2. Requires good operator skills and routine monitoring.
Custom Designed Extended Aeration Using Low Speed Surface Aerators:	1. Stable process when proper sludge management is performed. 2. High quality effluent 3. Many types of alternative aeration devices may be considered. 4. Predictable process.	1. Icing in cold weather climates must be considered. 2. Major maintenance requires crane to remove equipment. 3. Drive units afford higher mechanical maintenance. 4. Requires good operator skills and routine monitoring. 5. Sufficient oxygen supply should be provided for nitrification and pH may need to be controlled.

TABLE 16
(continued)

	Advantages	Disadvantages
Oxidation Ditches:	1. Stable process when proper sludge management is performed. 2. High quality effluent. 3. Predictable process.	1. Icing of aerator supports and nearby area must be considered. 2. Major maintenance requires crane to remove equipment. 3. Drive units require higher maintenance frequency. 4. Requires good operator skills and routine monitoring. 5. Sufficient oxygen supply should be provided for nitrification and pH may need to be controlled. 6. Only one type of aeration device is applicable.
Conventional Activated Sludge:	1. High quality effluent. 2. Predictable process. 3. Many types of aeration devices may be considered.	1. Requires good operator skills. 2. Requires frequent monitoring. 3. Daily variation in flows cause significant shift in sludge inventory. 4. Mechanical aeration may cause spray and mist problems. 5. More subprocesses complicate overall plant.
Trickling Filter:	1. Stable process. 2. Operator skills and monitoring requirements less than suspended growth systems. 3. Energy requirements less than suspended growth systems.	1. Effluent quality is not as predictable as suspended growth processes. 2. Filter flys, snails are problem at some locations. 3. High quality effluents are difficult to achieve. 4. More space required than suspended growth systems.

TABLE 16
(continued)

	Advantages	Disadvantages
Rotating Biological Media	1. Stable process. 2. Good quality effluent. 3. Simple operation. 4. Low maintenance, as a general rule.	1. Effluent quality is not as predictable as suspended growth process. 2. Heavy load on first cell may cause odors. 3. Multiple drives at larger plants affords proportionally higher maintenance requirements. 4. Shaft and drive failures have been experienced and require major maintenance. 5. Oil leaks from drive units are common. 6. Larger plants require more space than equal size suspended growth systems.

Inflation Allowance	8 percent of subtotal of project costs
Land	Not included

Based on the unit process sizes, cost relationships, and operating and maintenance requirements shown, the construction and operating costs for Case I and Case II conditions are developed on Figures 34 and 35. The project costs have been amortized at 6.5 percent interest and 25 years to develop a total annual cost which has been converted to a unit cost on the basis of cost/1,000 gallons treated at the design flow condition. The relationships comparing unit costs for the various processes are shown on Figures 36 and 37. Labor has been charged at $9/hour, power at $0.03/kwh, and chlorine at $250/ton.

A summary of capital costs and operating costs are tabulated in Appendix B.

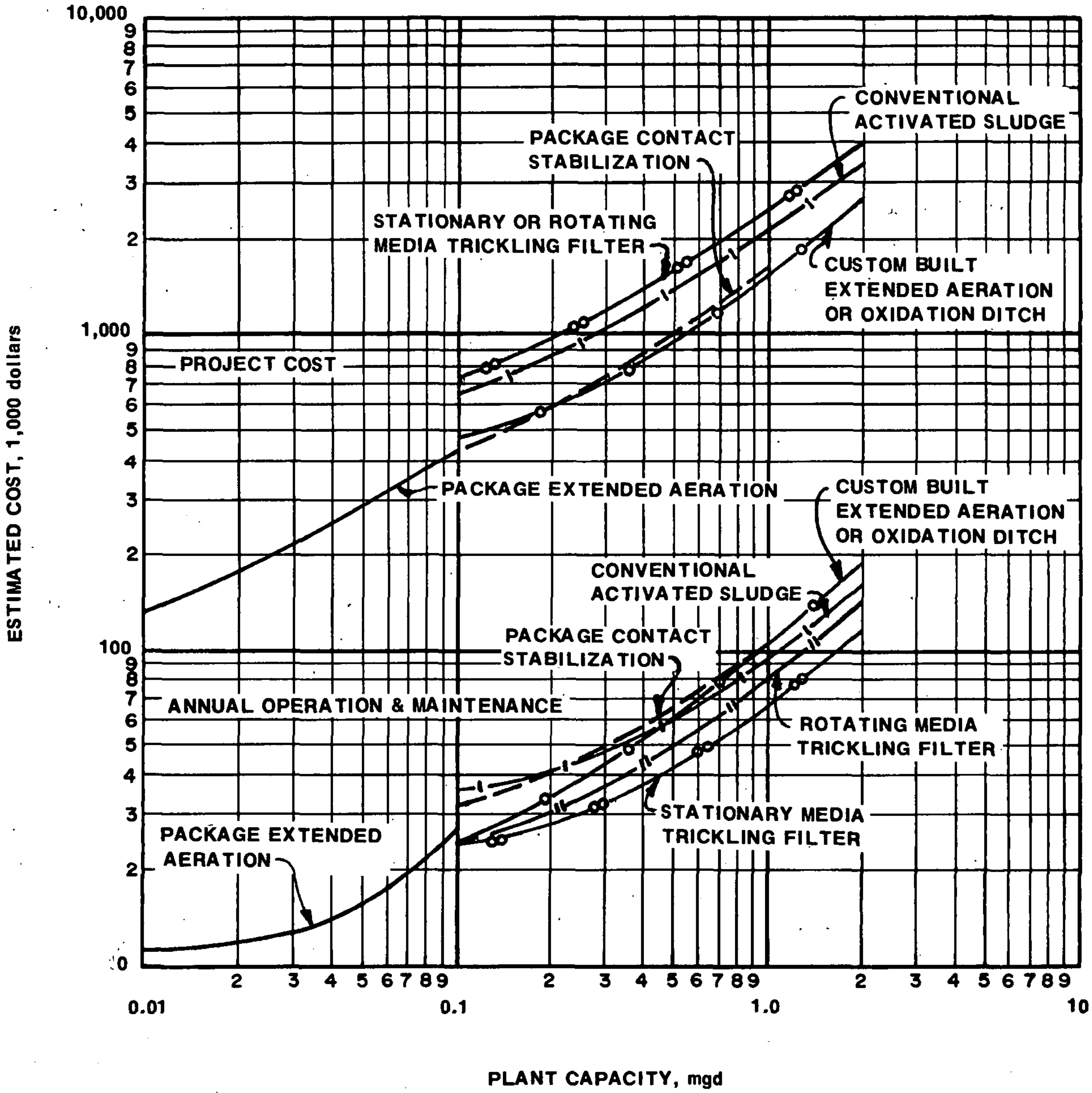
The cost comparison of the various alternatives as mentioned, are not the "bottom line". When cost estimates for facility planning purposes are within 10 percent, the accuracy of the estimate may not permit a clear cut advantage. It may be necessary to eliminate costs of common functions and reflect upon the costs of dissimilar functions.

For situations where dissimilar functions for alternative processes are estimated to cause less than 10 percent difference in costs (during the facility planning stage), the best answer might be more reasonably chosen from considerations other than the cost analysis.

The cost value is not absolute. The methods used in arriving at the costs are general. They are not intended for precision but are intended to fairly and conservatively arrive at a project cost. The estimated costs derived in the manner presented should not arbitrarily be reduced unless detailed layout, quantity and unit price tabulation and more rigorous analysis indicate reduction in cost is appropriate.

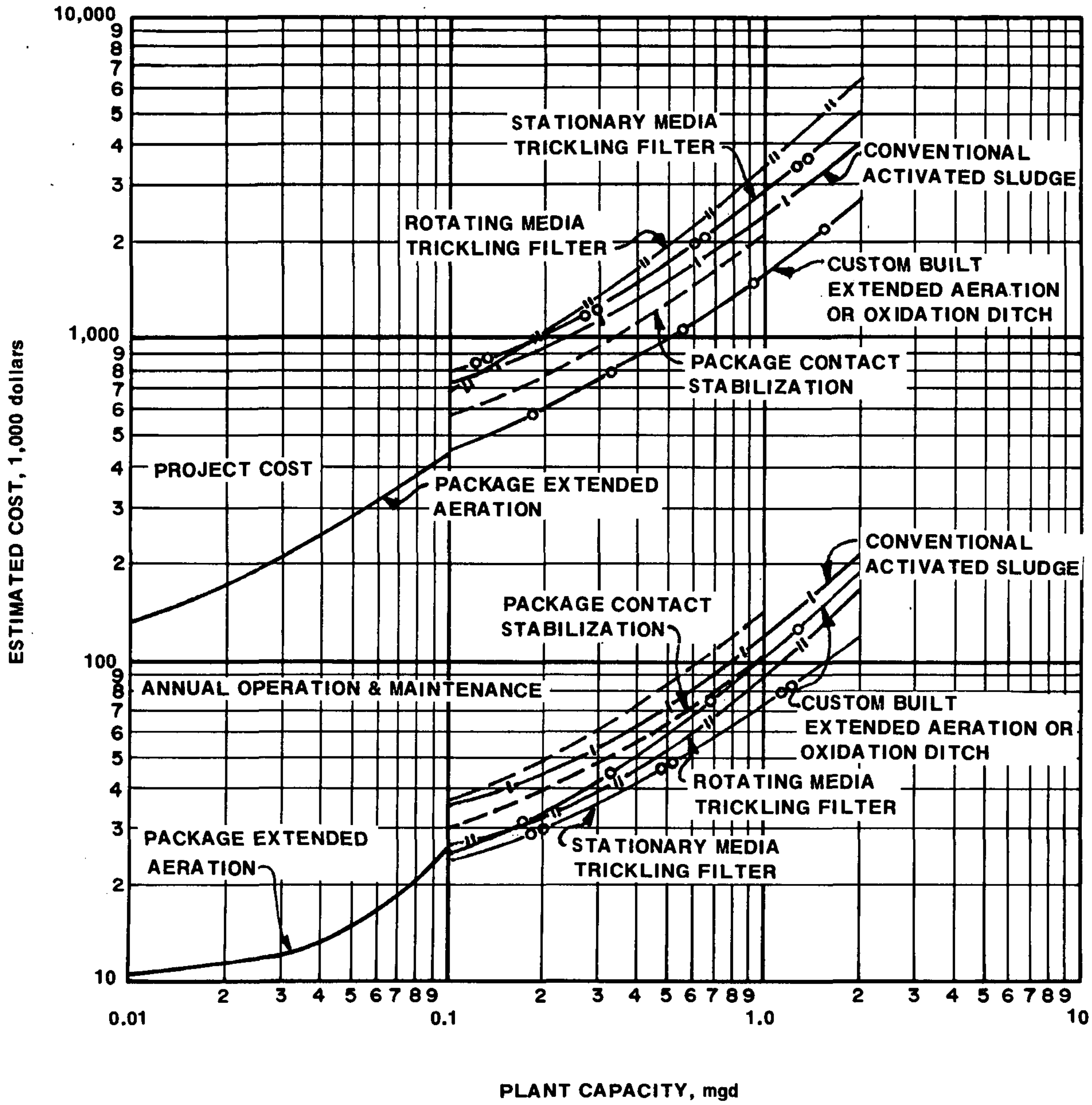
Operating and maintenance requirements for small community plants developed in the example cost analysis, are typical but the unit costs used for the labor ($9/hour) are not typical. For instance, in a recent survey by the author's firm, principal labor costs (including fringe benefits) were found to be $3.50-$5.00 per hour at small communities and $8.00-$11.00 for larger communities. Because small community plants have a higher proportionate cost associated with labor costs, the unit cost used will heavily influence the total operation and maintenance costs.

The relationship of equivalent unit costs shown on Figures 36 and 37 emphasises that for small community wastewater treatment plants, less complex and fewer unit processes provide a facility that is not only less difficult to operate, they generally provide a higher reliability in effluent quality and are more economical.



CASE 1 – ESTIMATED COST COMPARISON
NITRIFICATION NOT REQUIRED

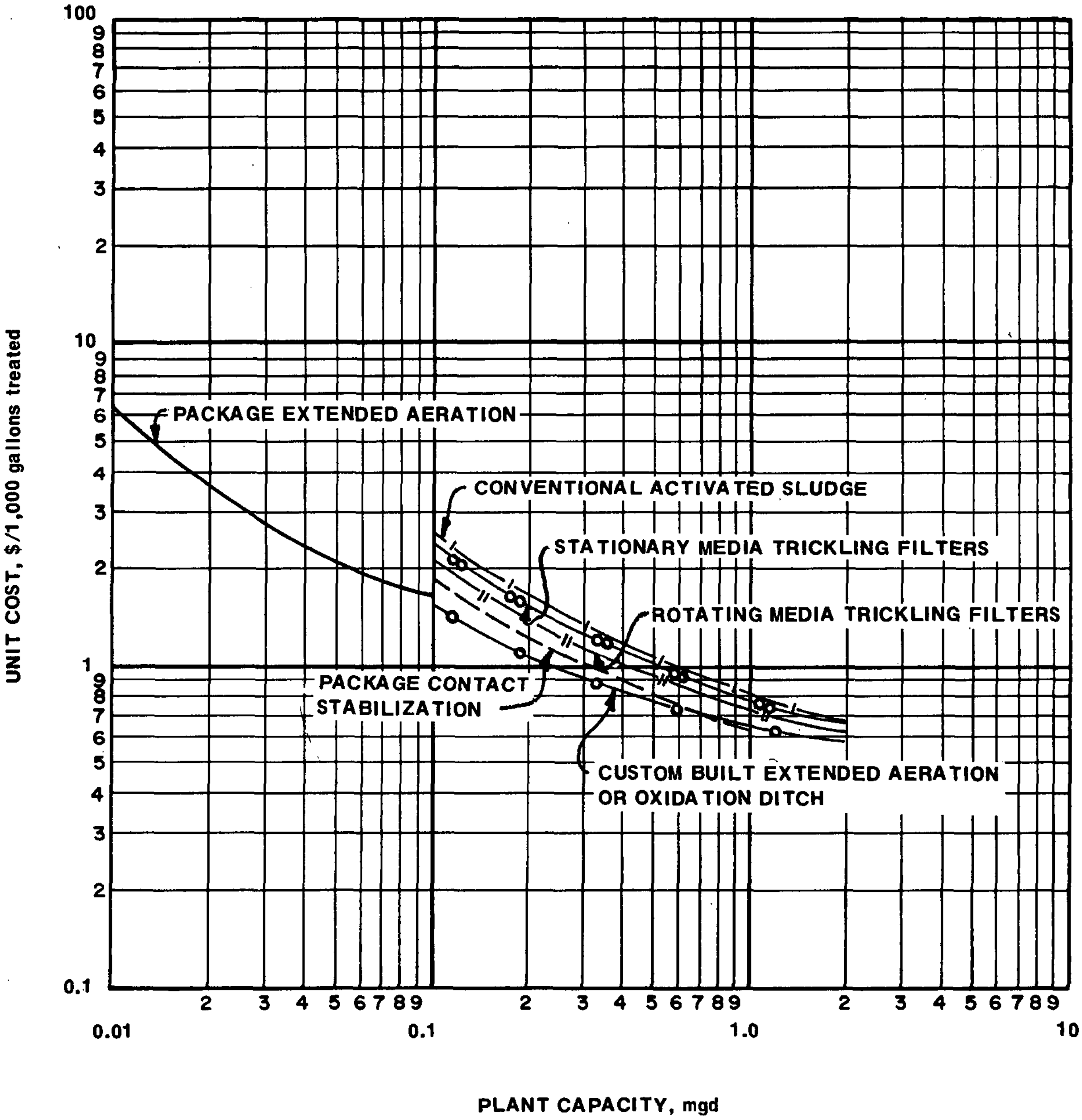
FIGURE 34



CASE II – ESTIMATED COST COMPARISON

NITRIFICATION REQUIRED

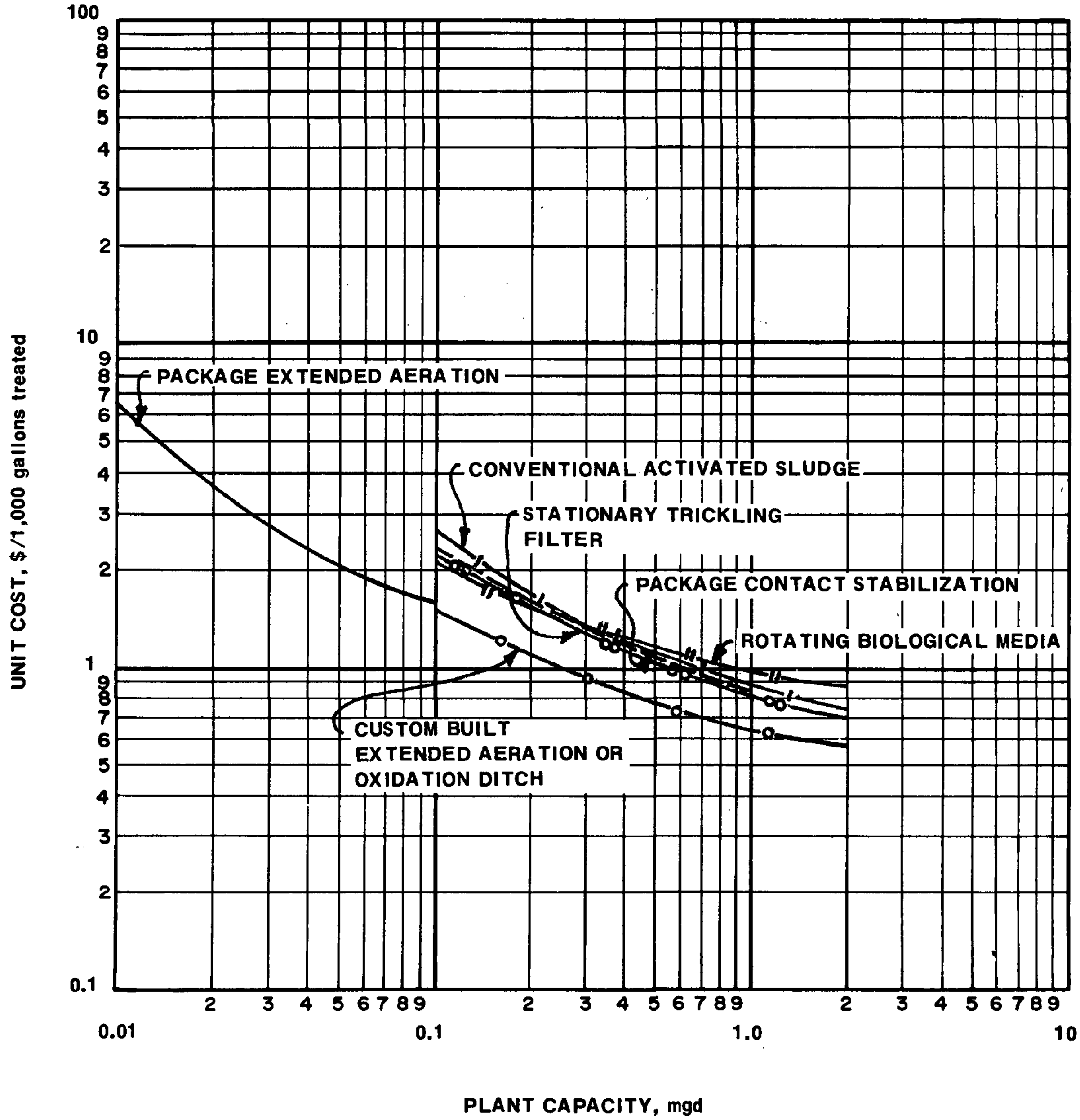
FIGURE 35



CASE I – ALTERNATIVE COMPARISON

NITRIFICATION NOT REQUIRED

FIGURE 36

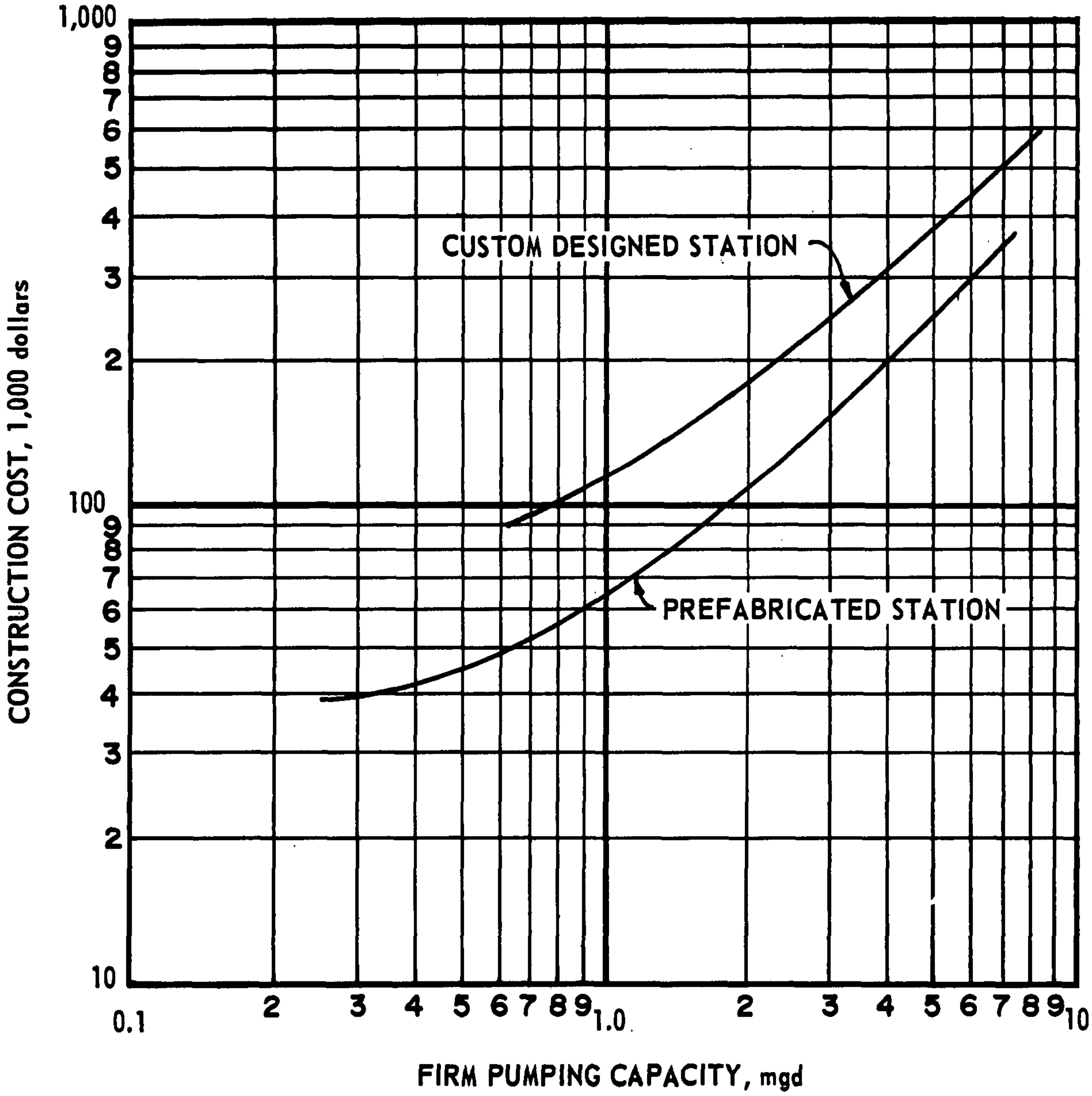


CASE II – ALTERNATIVE COMPARISON
NITRIFICATION REQUIRED

FIGURE 37

APPENDIX A

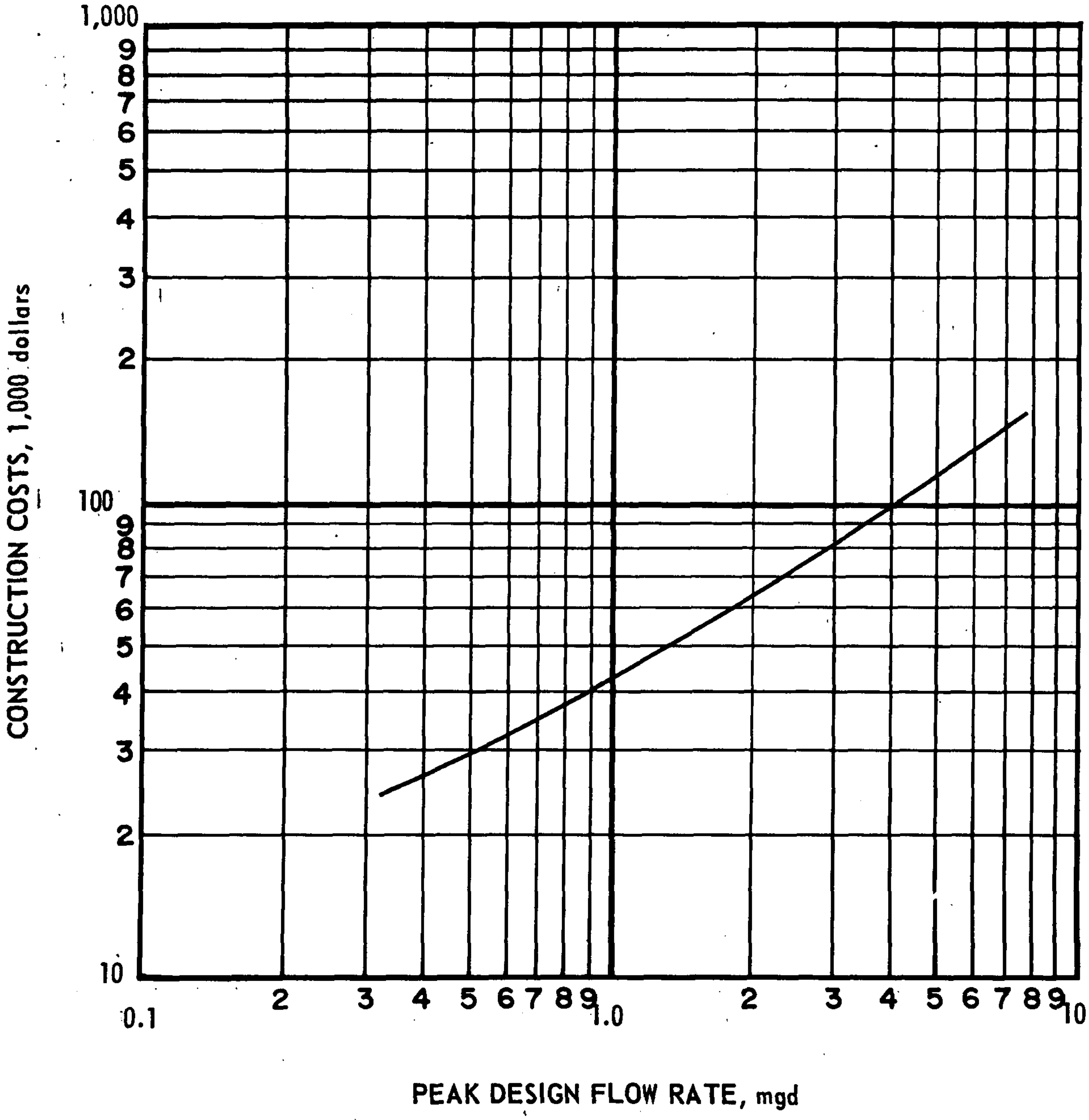
CONSTRUCTION COST RELATIONSHIPS
OPERATING & MAINTENANCE REQUIREMENTS



ESTIMATED CONSTRUCTION COST

RAW WASTEWATER PUMPING

FIGURE A-1

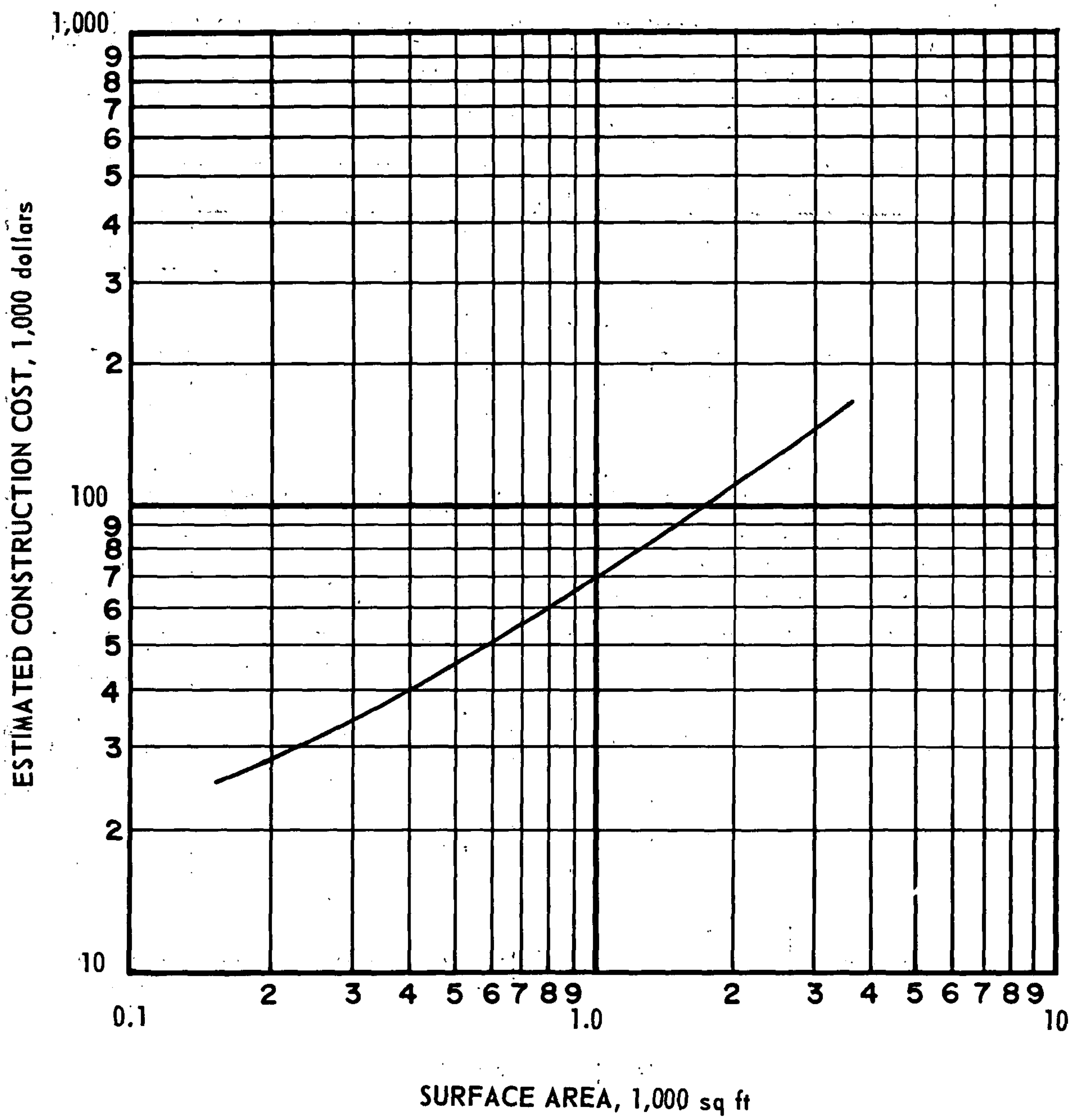


ESTIMATED CONSTRUCTION COSTS

PRELIMINARY TREATMENT

(Screening, Grit removal & Flow measurement)

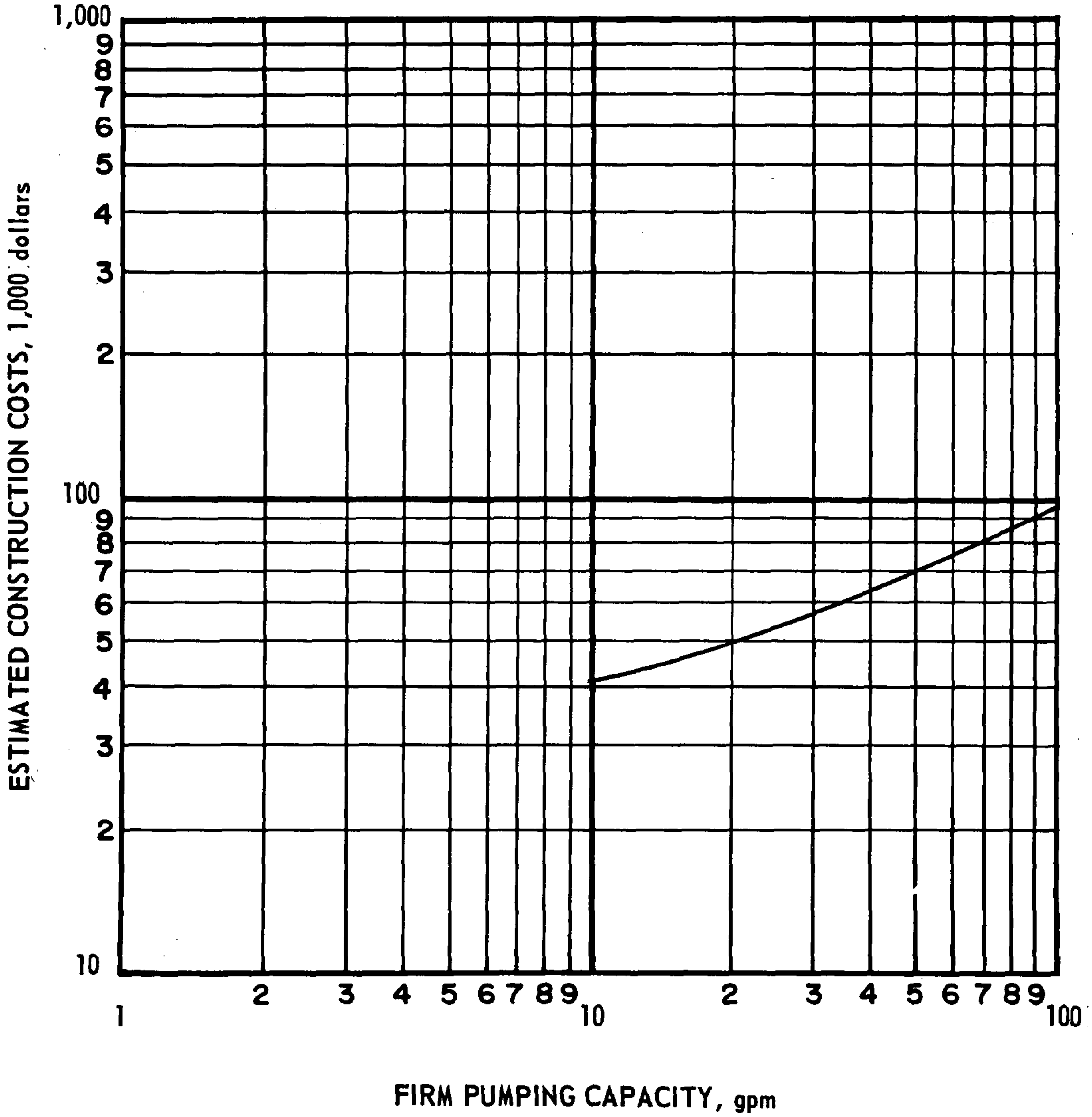
FIGURE A-2



ESTIMATED CONSTRUCTION COST

SEDIMENTATION BASINS

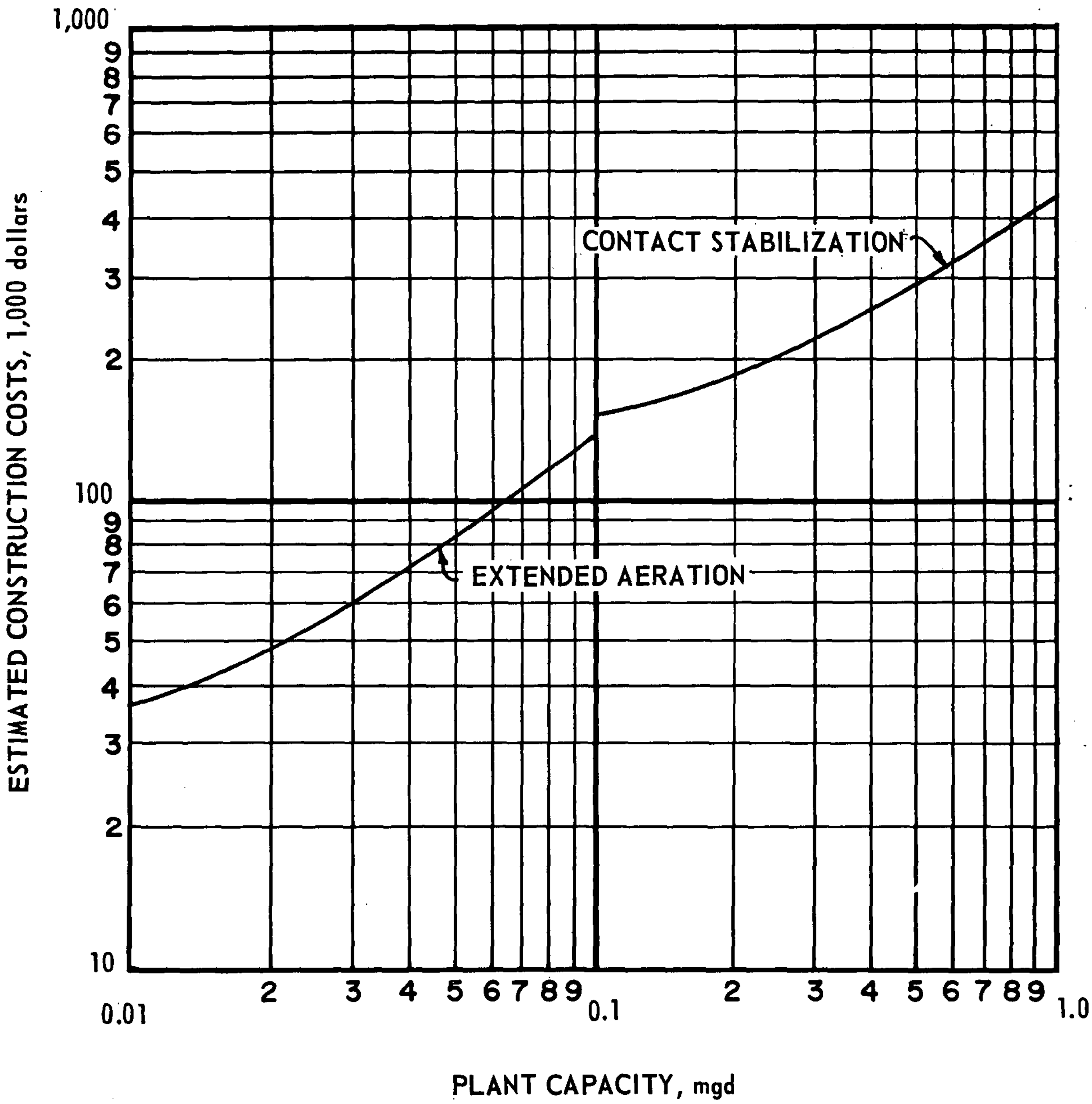
FIGURE A-3



ESTIMATED CONSTRUCTION COSTS

SLUDGE PUMPING STATIONS

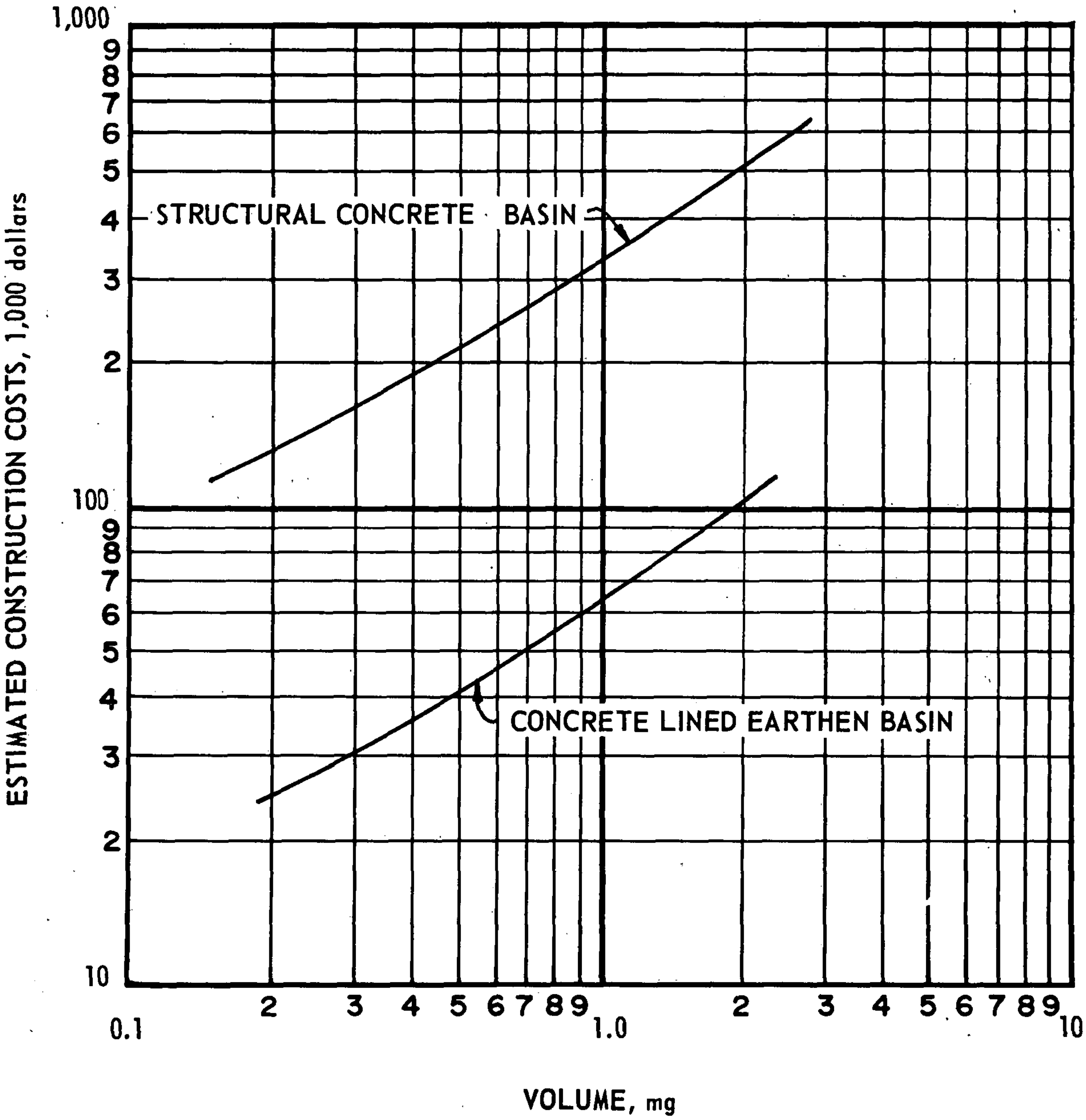
FIGURE A–4



ESTIMATED CONSTRUCTION COSTS

PREFABRICATED ACTIVATED SLUDGE PLANTS

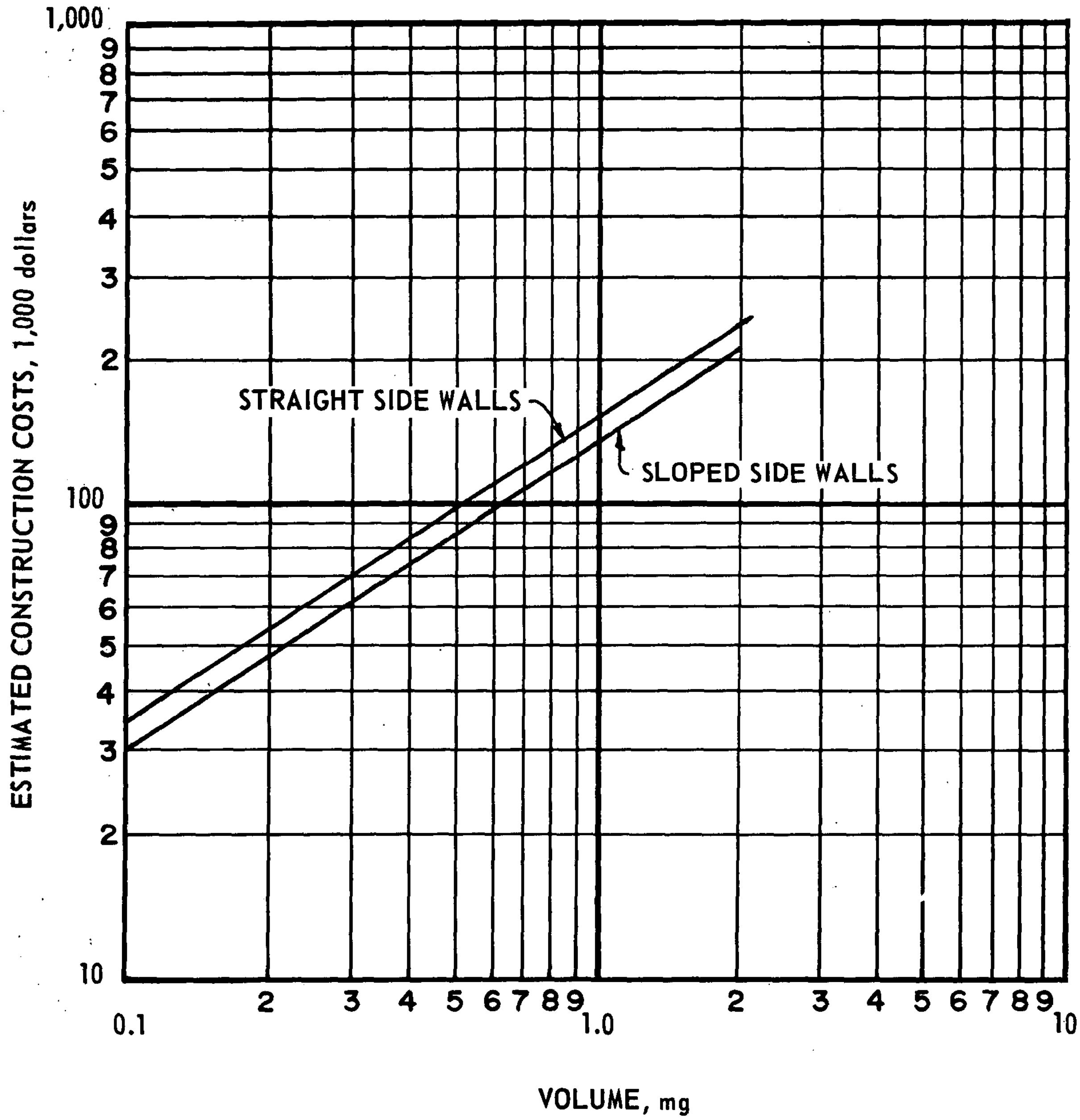
FIGURE A–5



ESTIMATED CONSTUCTION COSTS

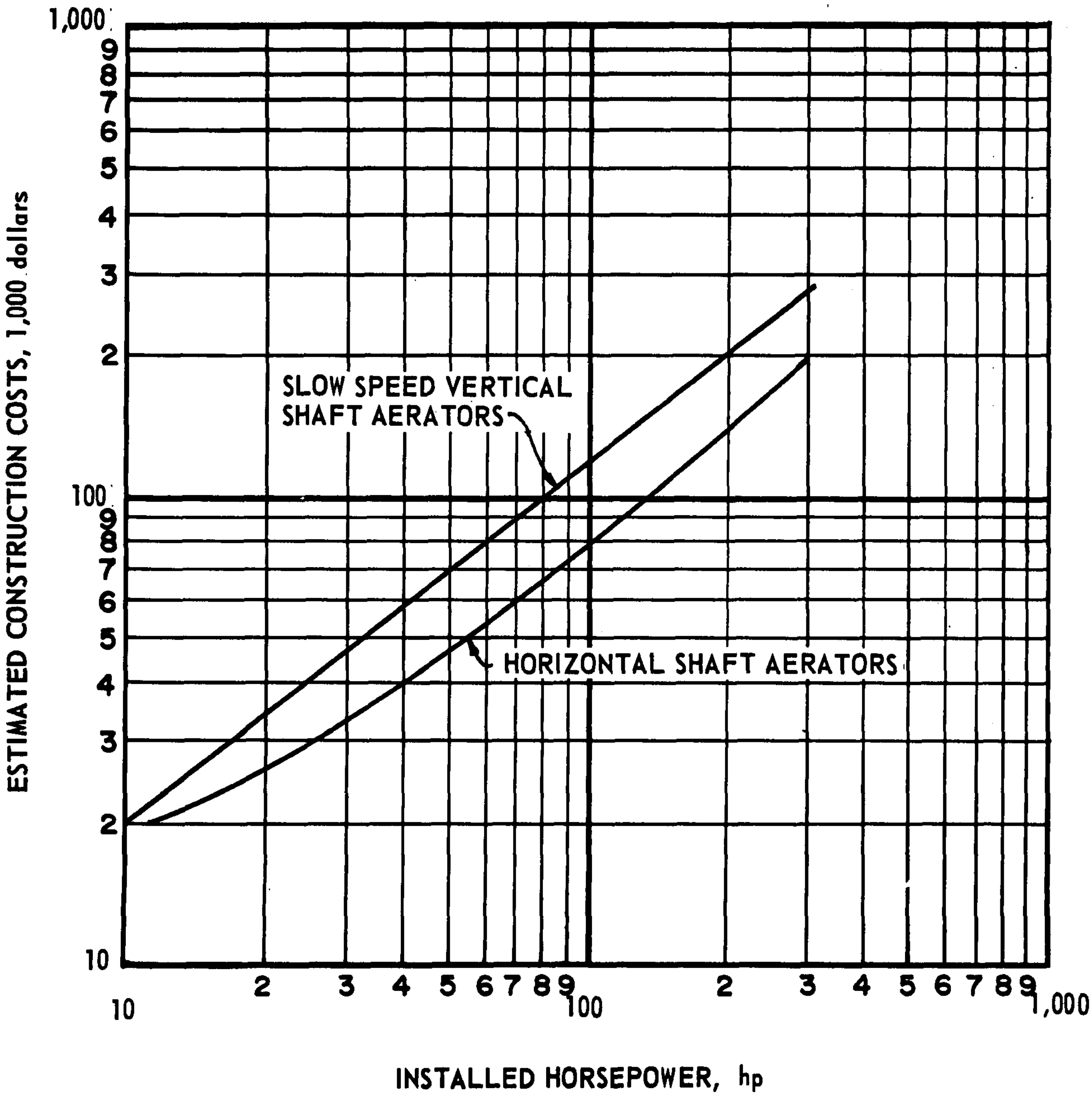
CUSTOM BUILT AERATION BASINS

FIGURE A–6



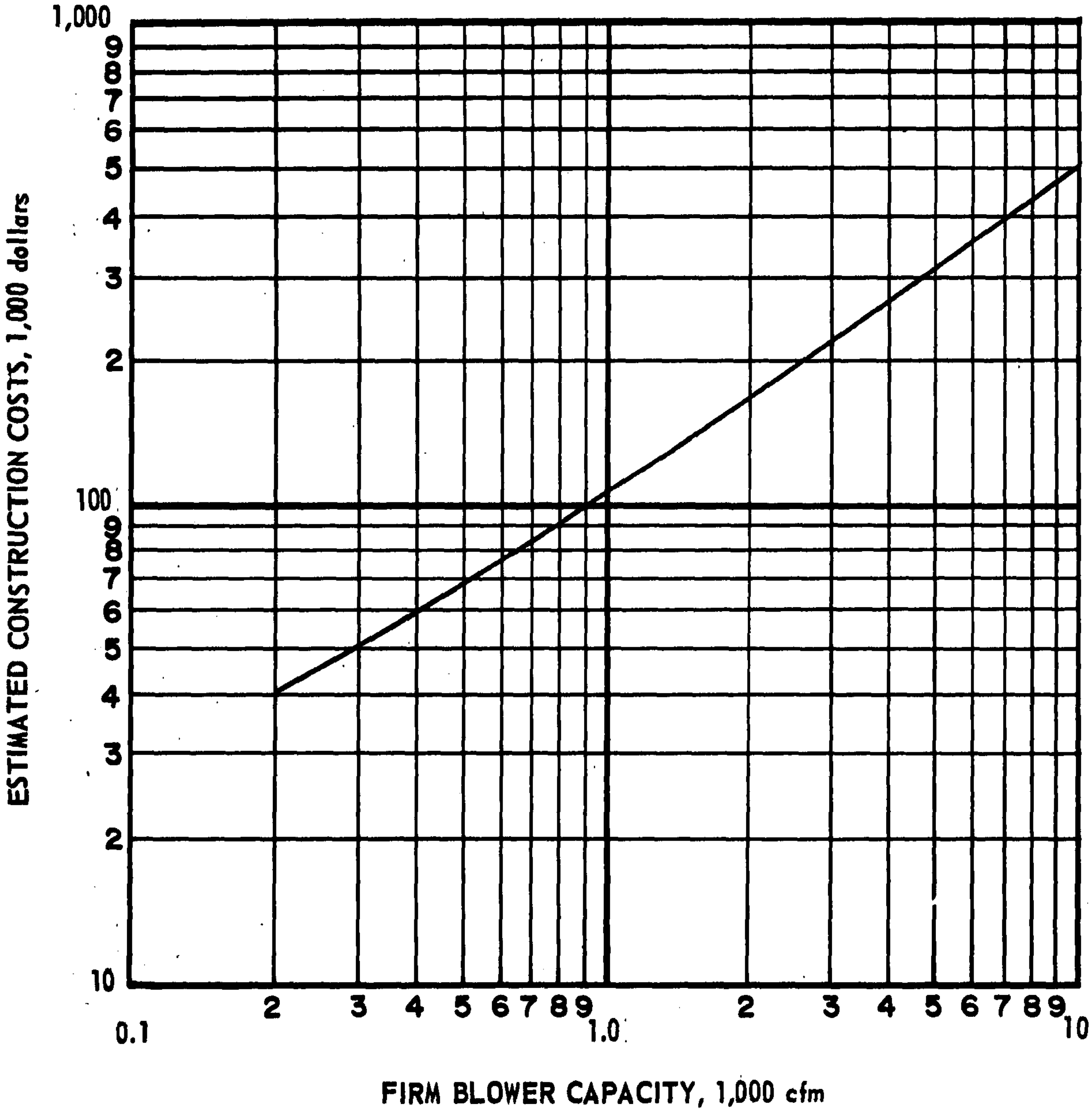
ESTIMATED CONSTRUCTION COSTS
OXIDATION DITCH AERATION BASINS

FIGURE A-7



ESTIMATED CONSTRUCTION COSTS
MECHANICAL AERATION

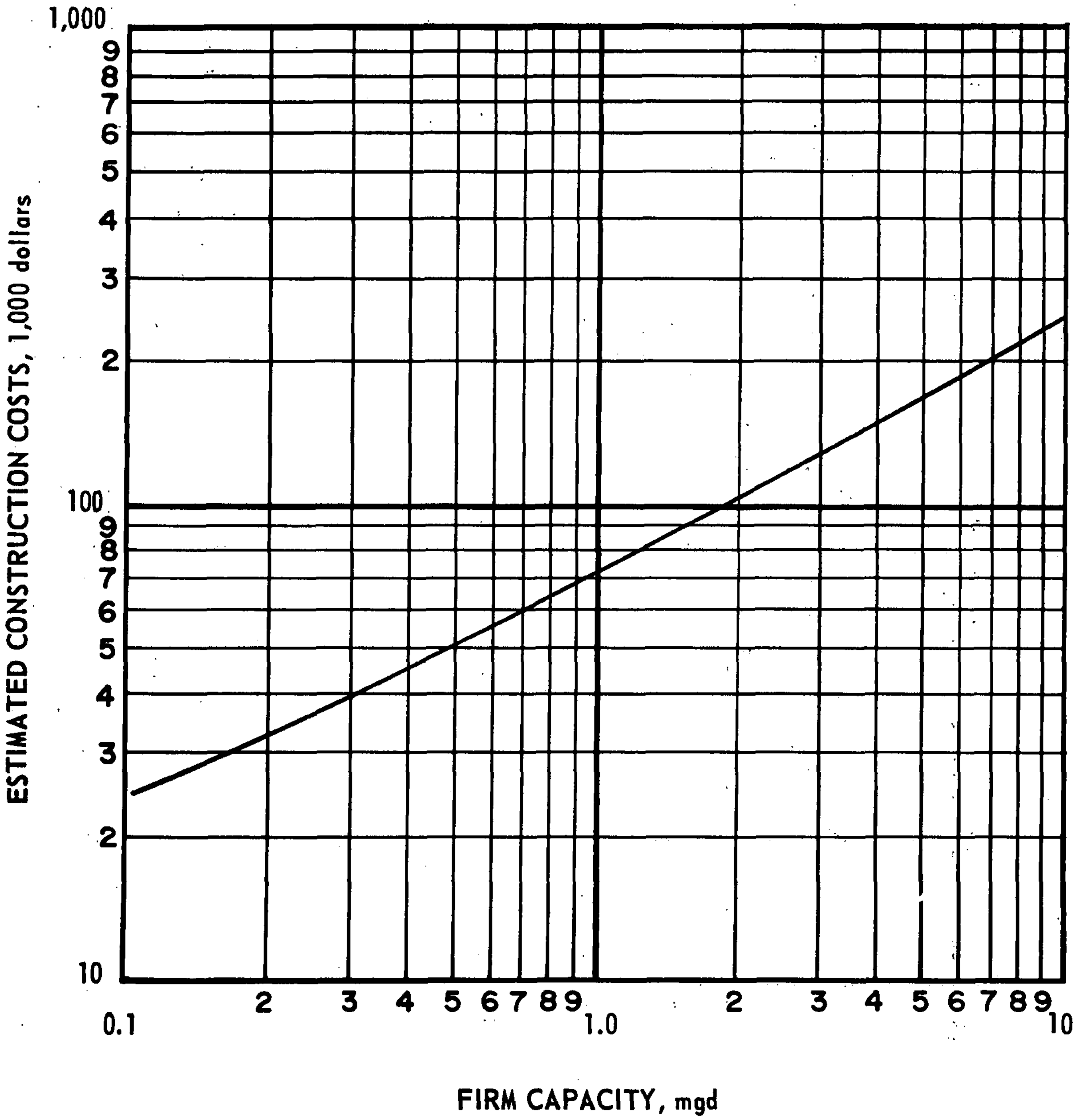
FIGURE A-8



ESTIMATED CONSTRUCTION COSTS

DIFFUSED AERATION

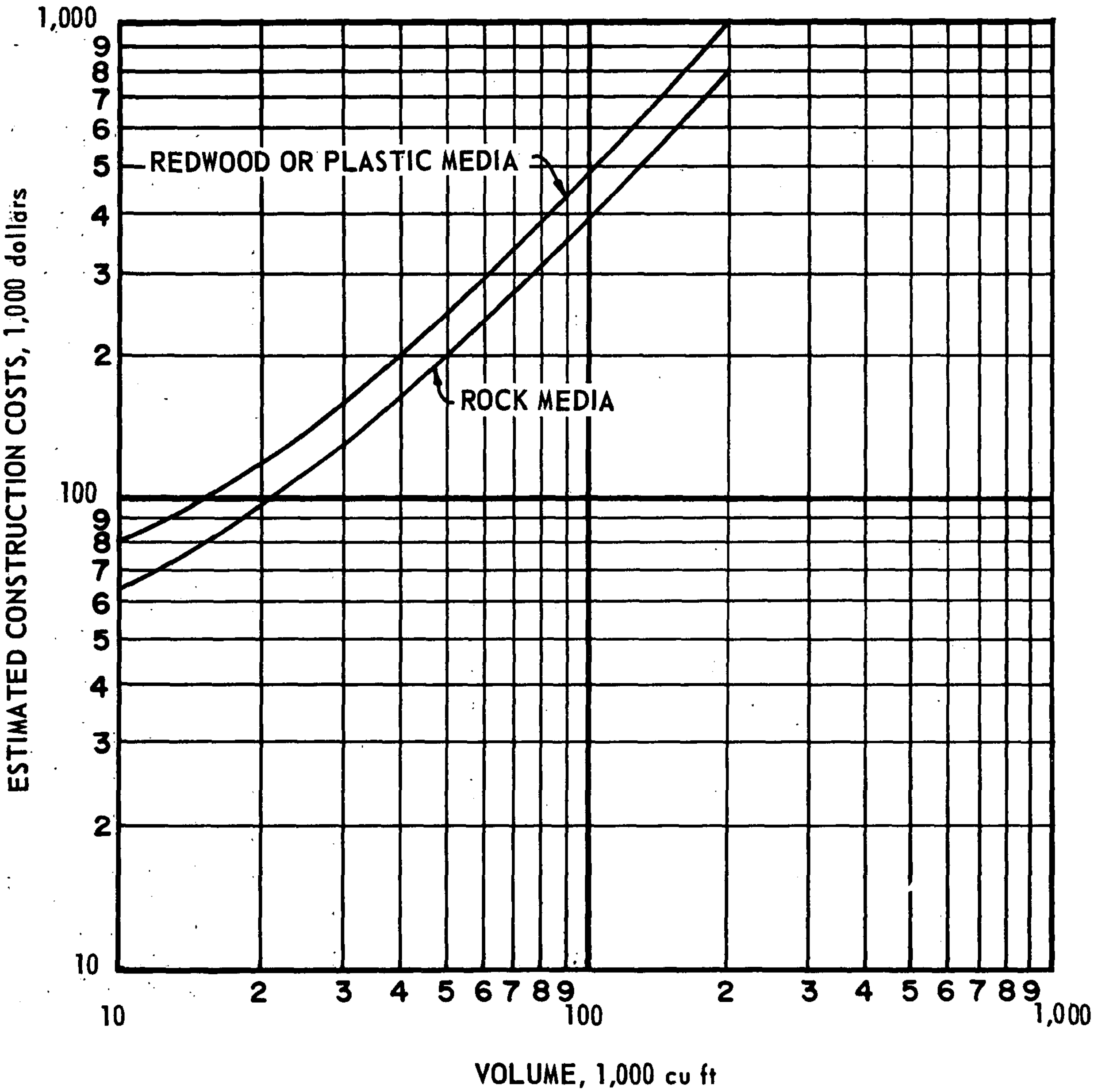
FIGURE A-9



ESTIMATED CONSTRUCTION COSTS

RECIRCULATION PUMPING

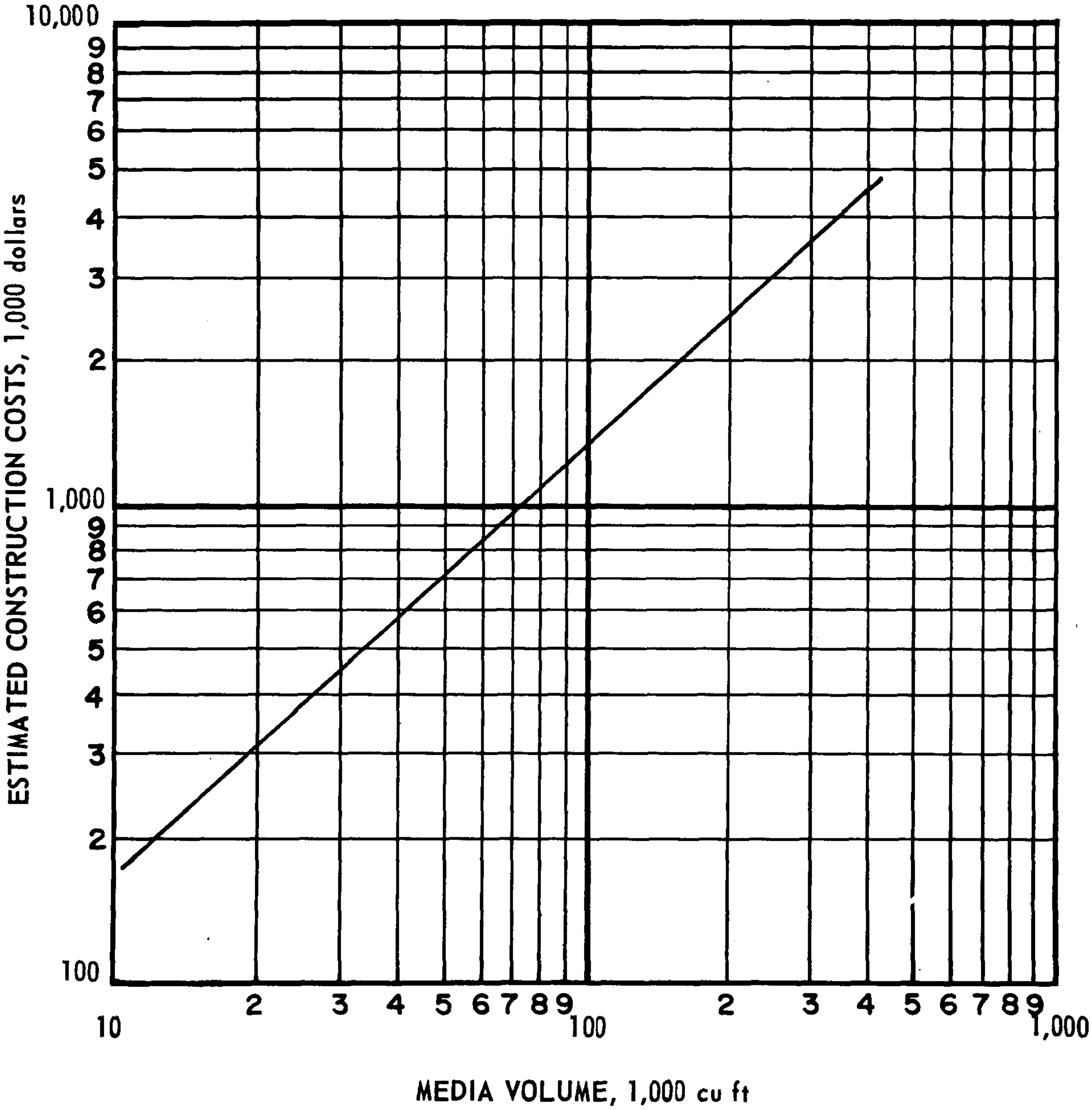
FIGURE A–10



ESTIMATED CONSTRUCTION COSTS

TRICKLING FILTERS

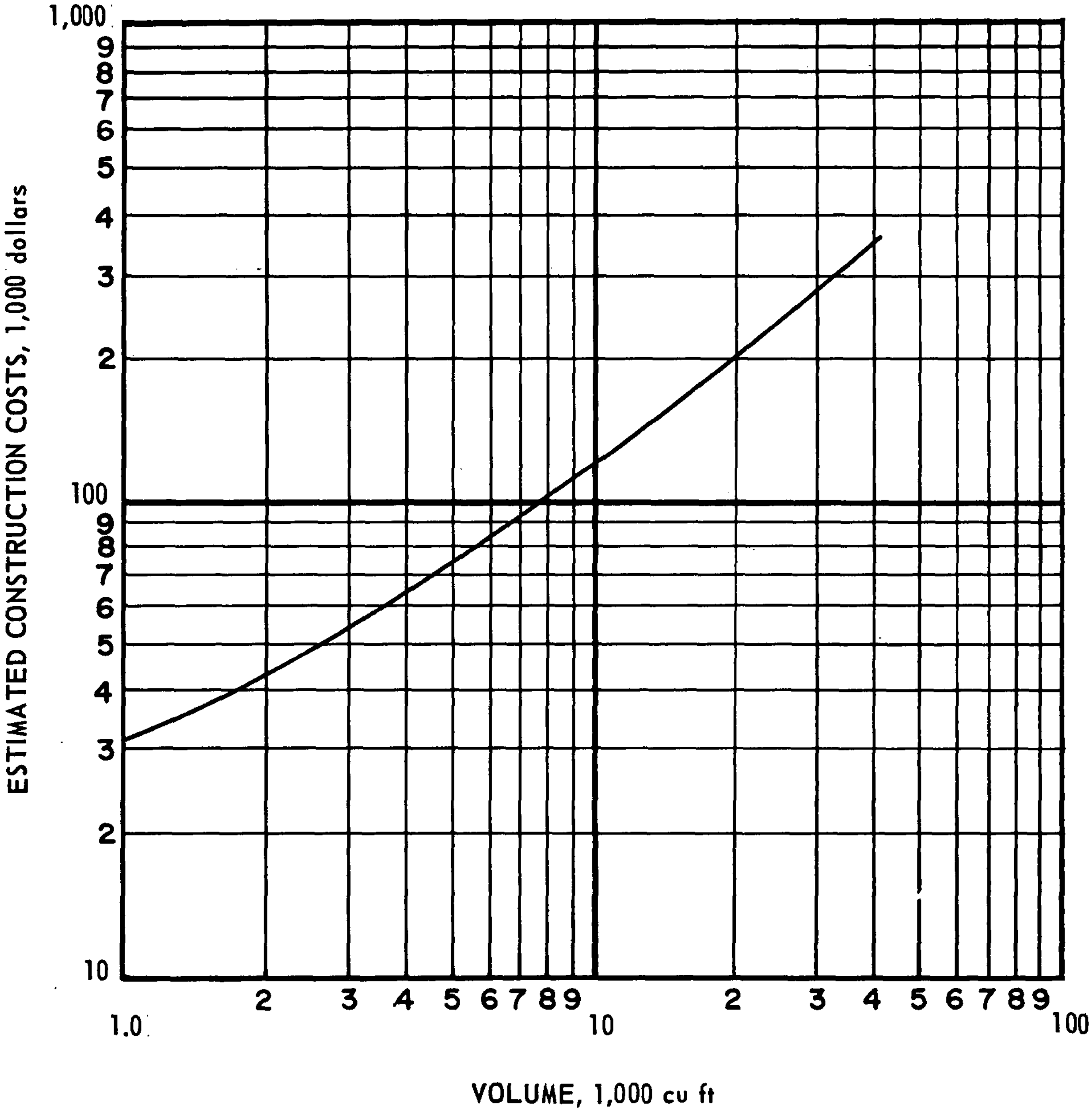
FIGURE A–11



ESTIMATED CONSTRUCTION COSTS

ROTATING BIOLOGICAL MEDIA

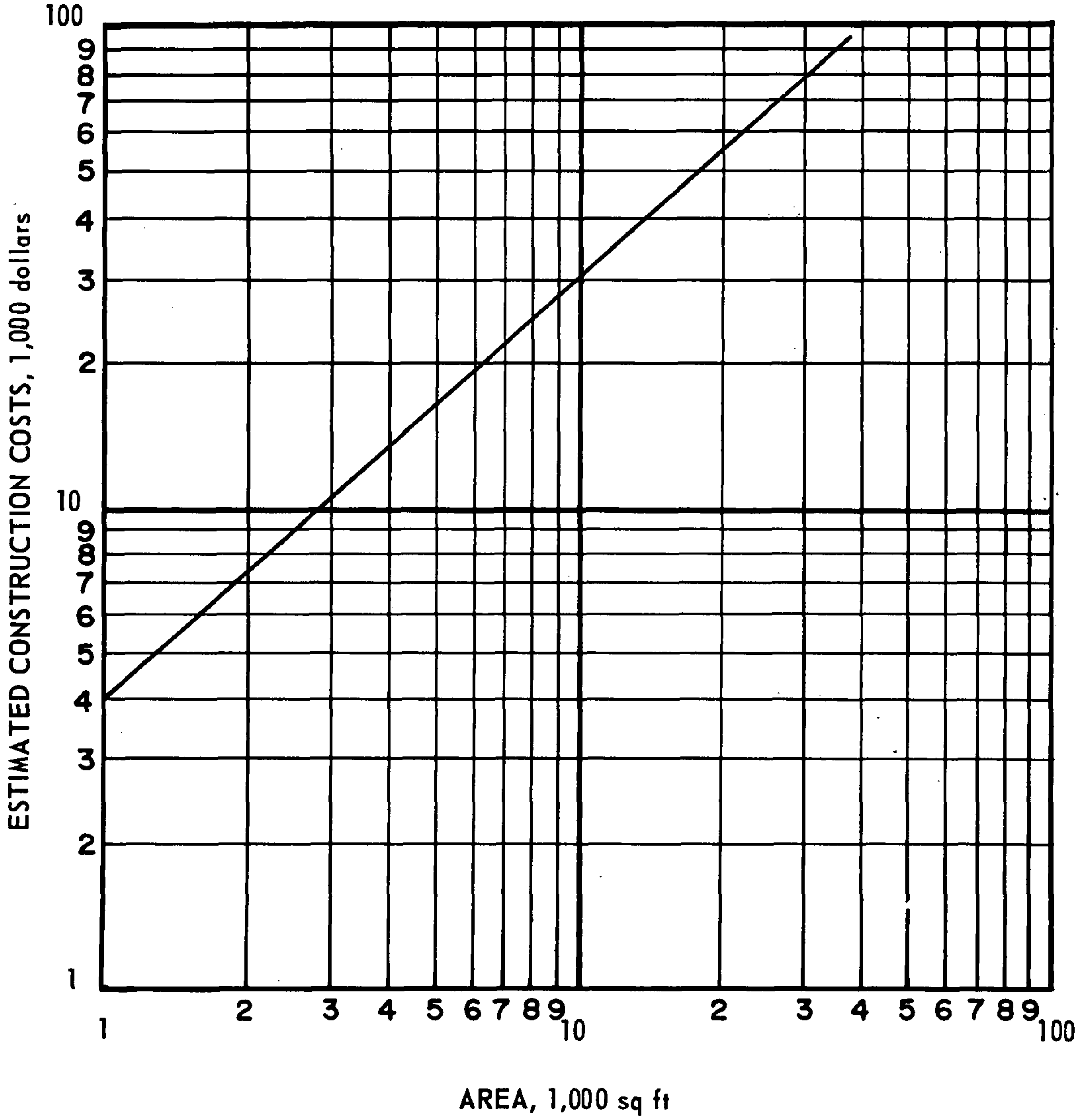
FIGURE A–12



ESTIMATED CONSTRUCTION COSTS

ANAEROBIC DIGESTION

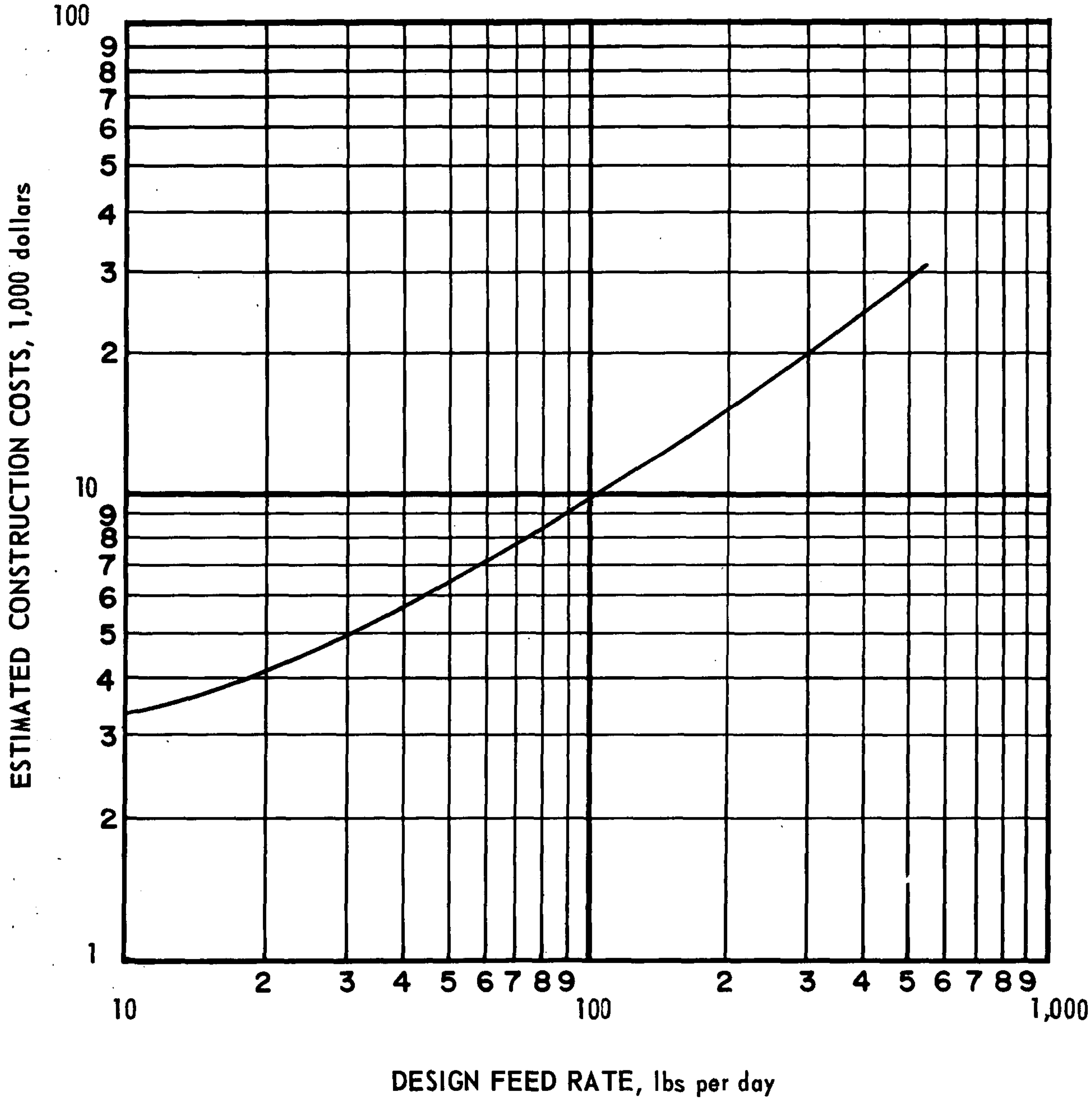
FIGURE A–13



ESTIMATED CONSTRUCTION COSTS

SLUDGE DRYING BEDS

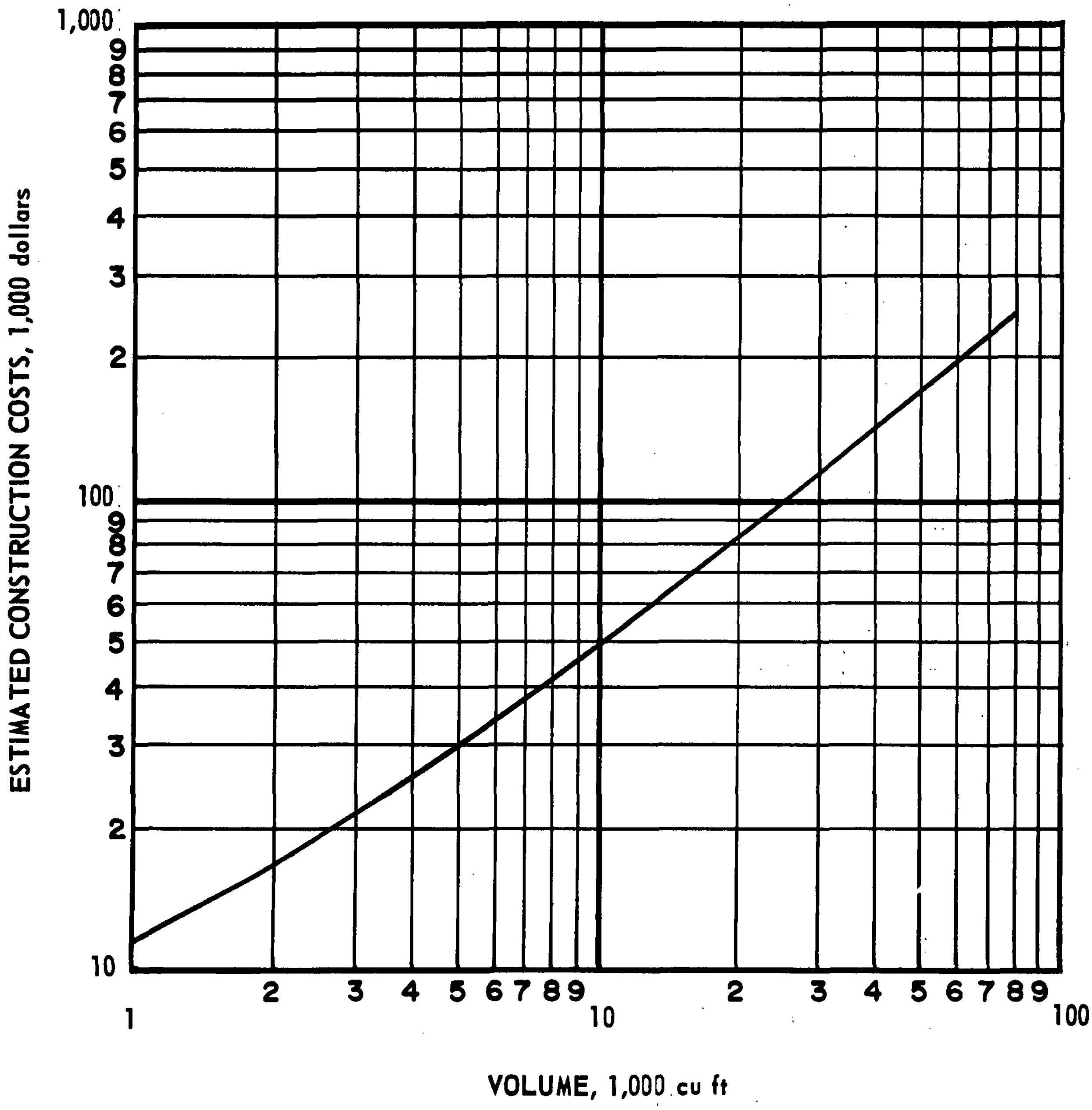
FIGURE A–14



ESTIMATED CONSTRUCTION COSTS

CHLORINE FEED EQUIPMENT

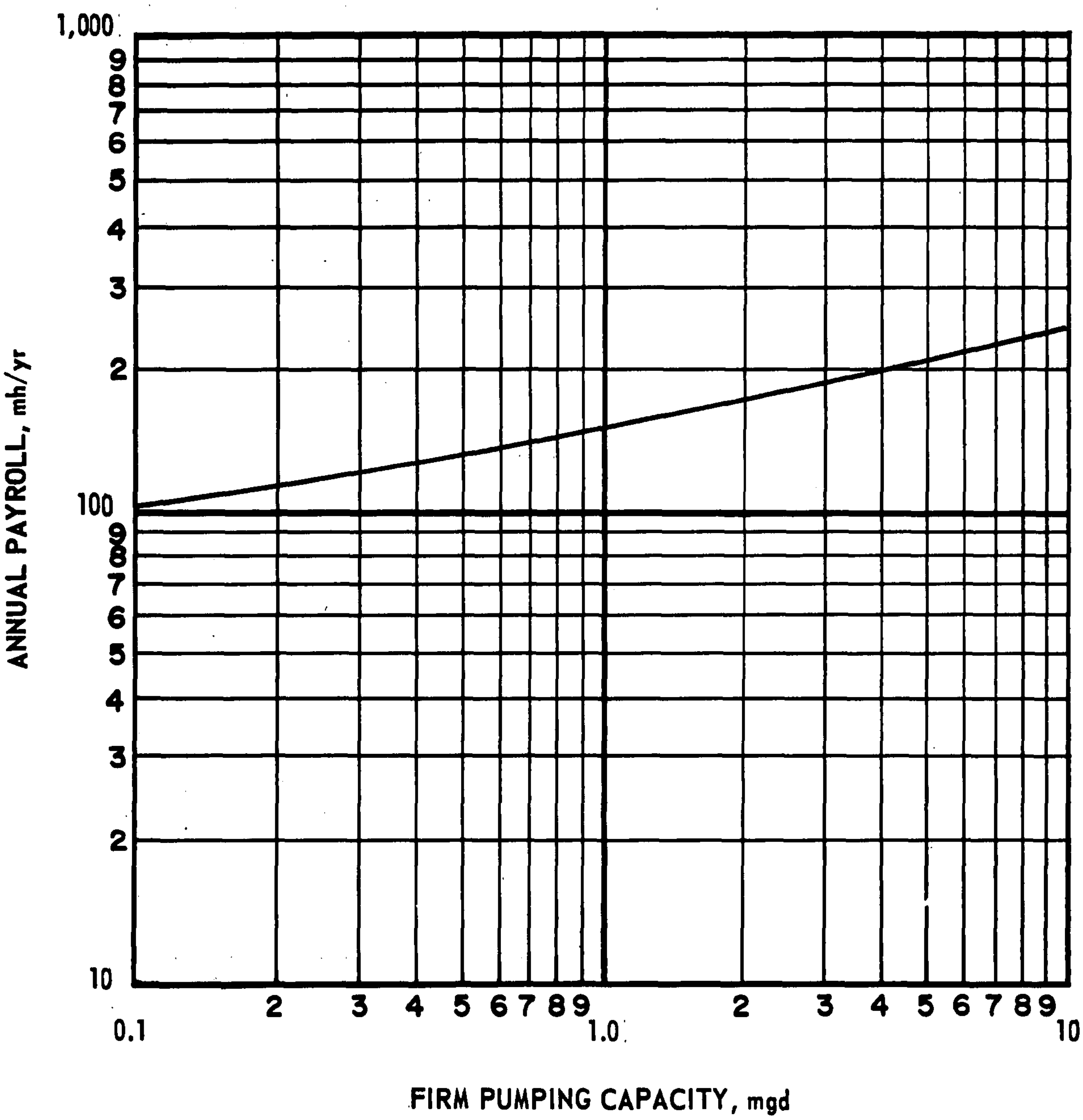
FIGURE A–15



ESTIMATED CONSTRUCTION COSTS

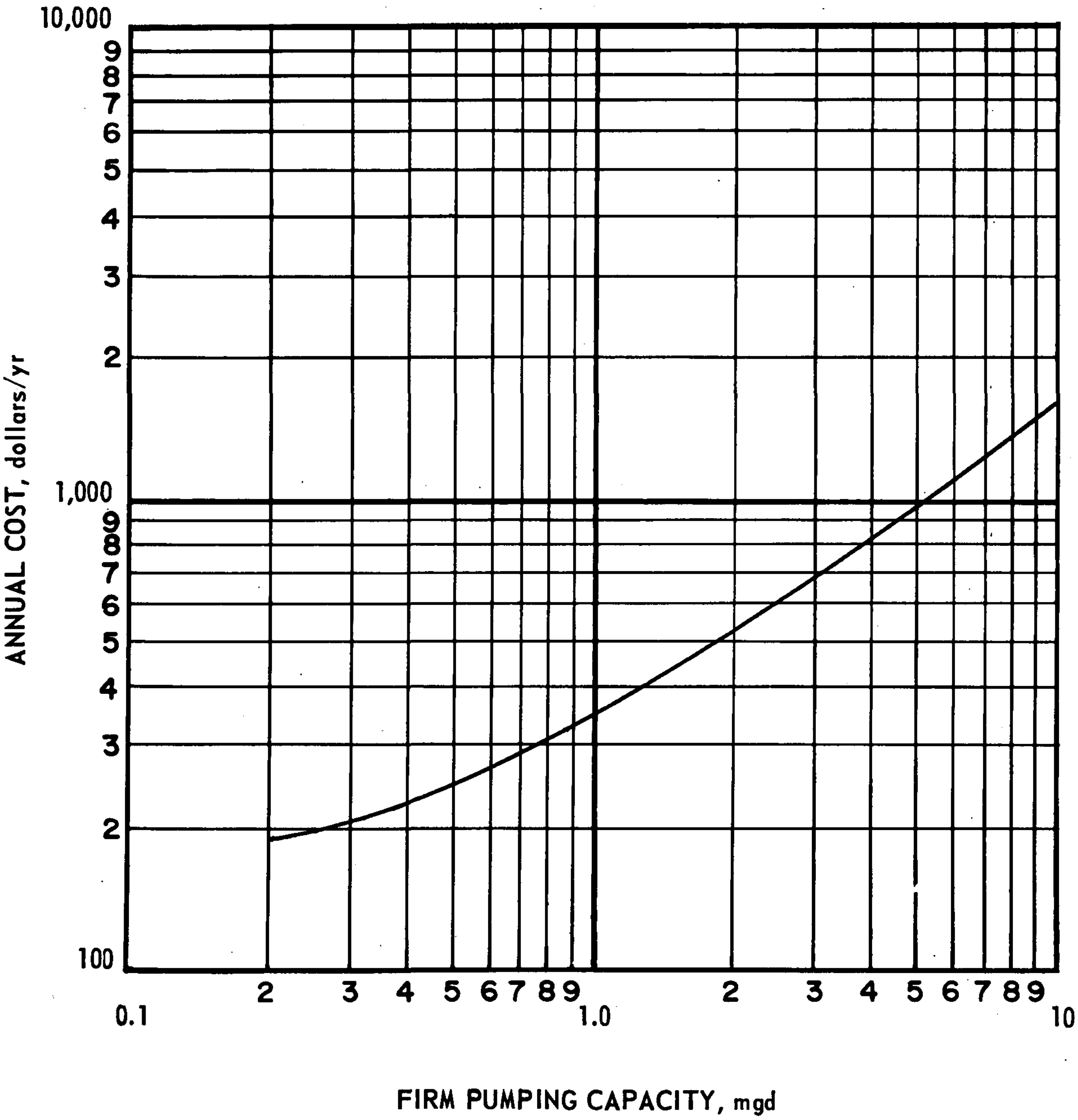
CHLORINE CONTACT BASINS

FIGURE A-16



LABOR
OPERATION & MAINTENANCE REQUIREMENTS
WASTEWATER PUMPING

FIGURE A-17

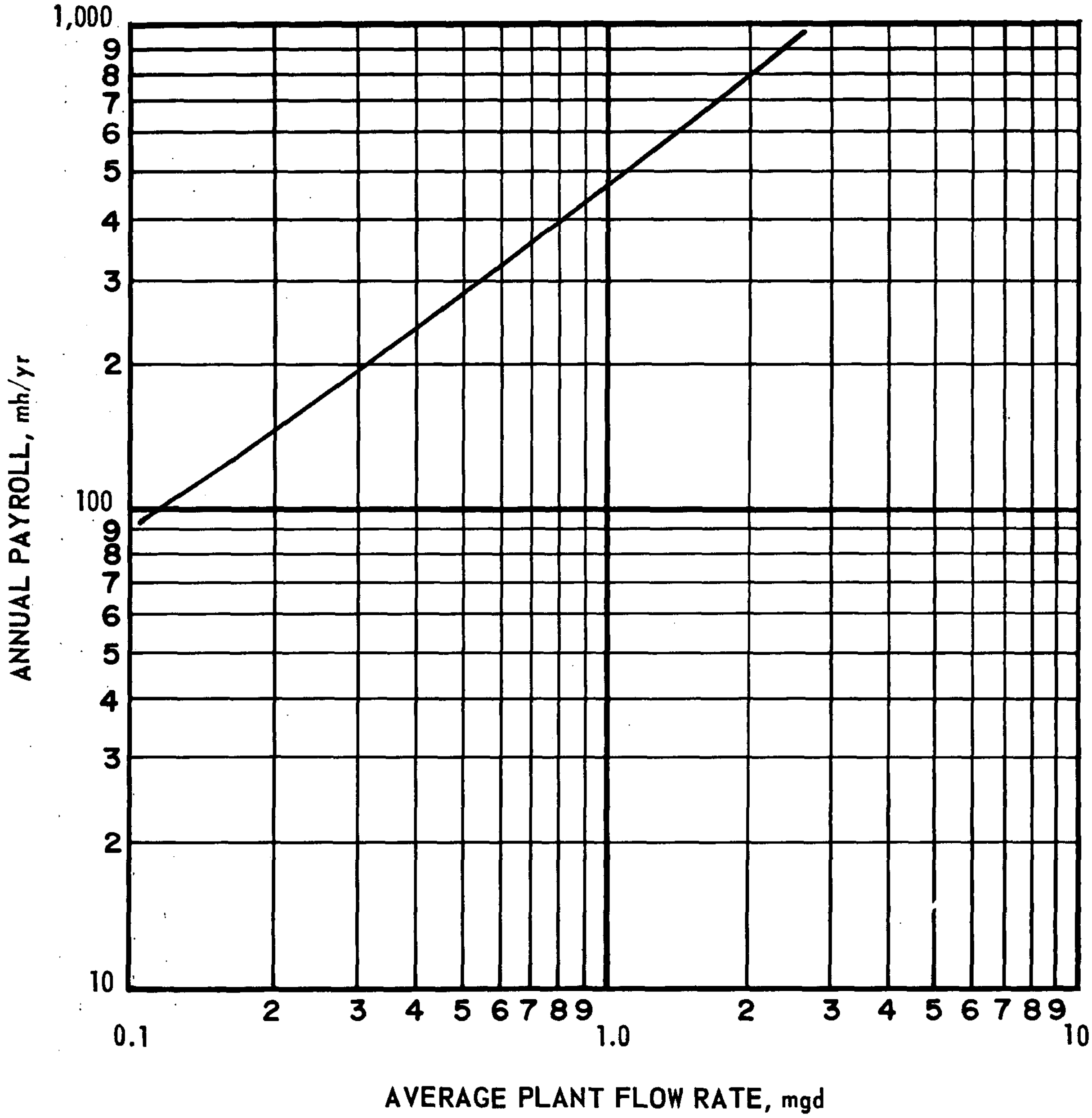


MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

WASTEWATER PUMPING

FIGURE A-18

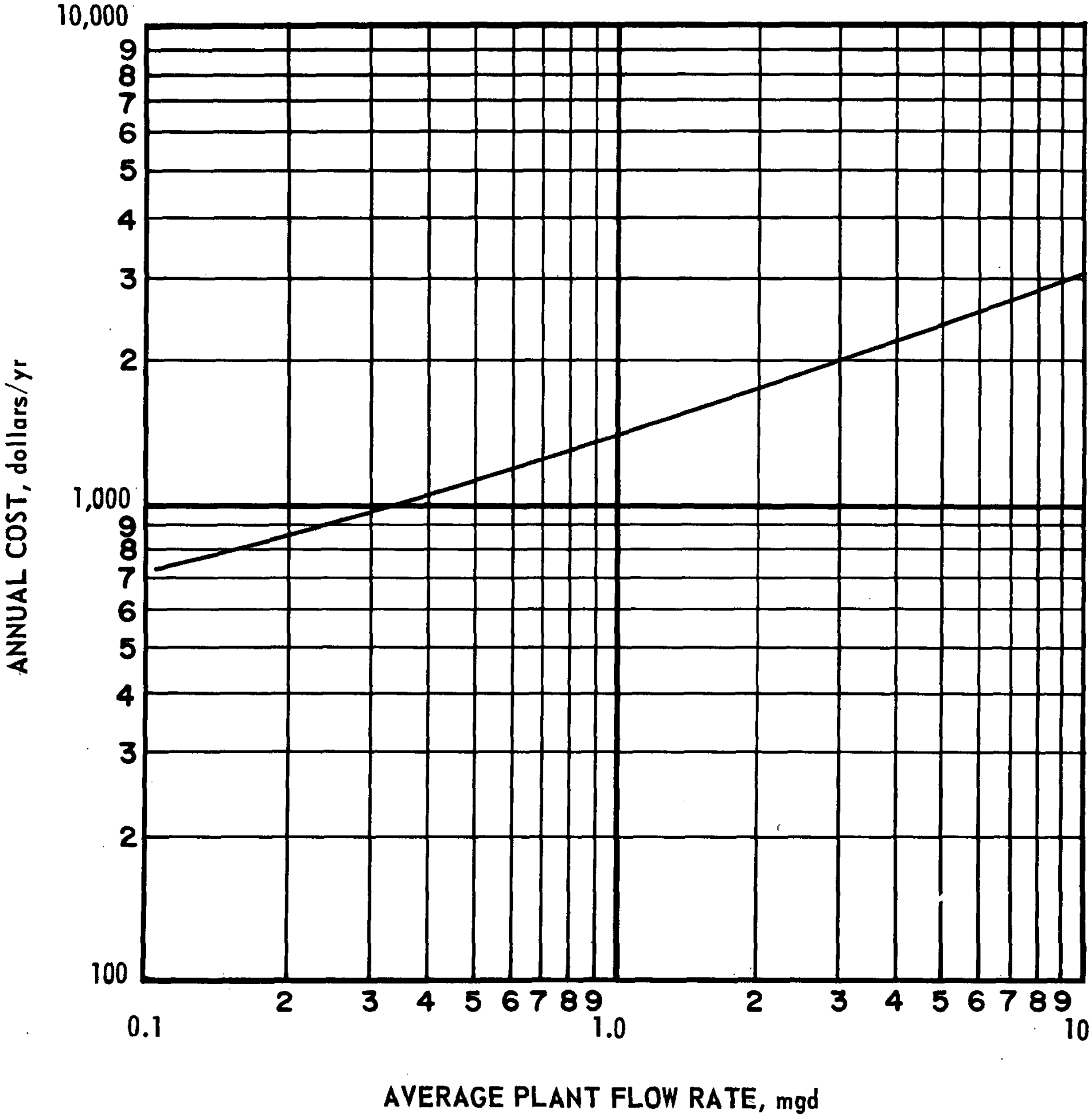


LABOR

OPERATION & MAINTENANCE REQUIREMENTS

PRELIMINARY TREATMENT

FIGURE A-19

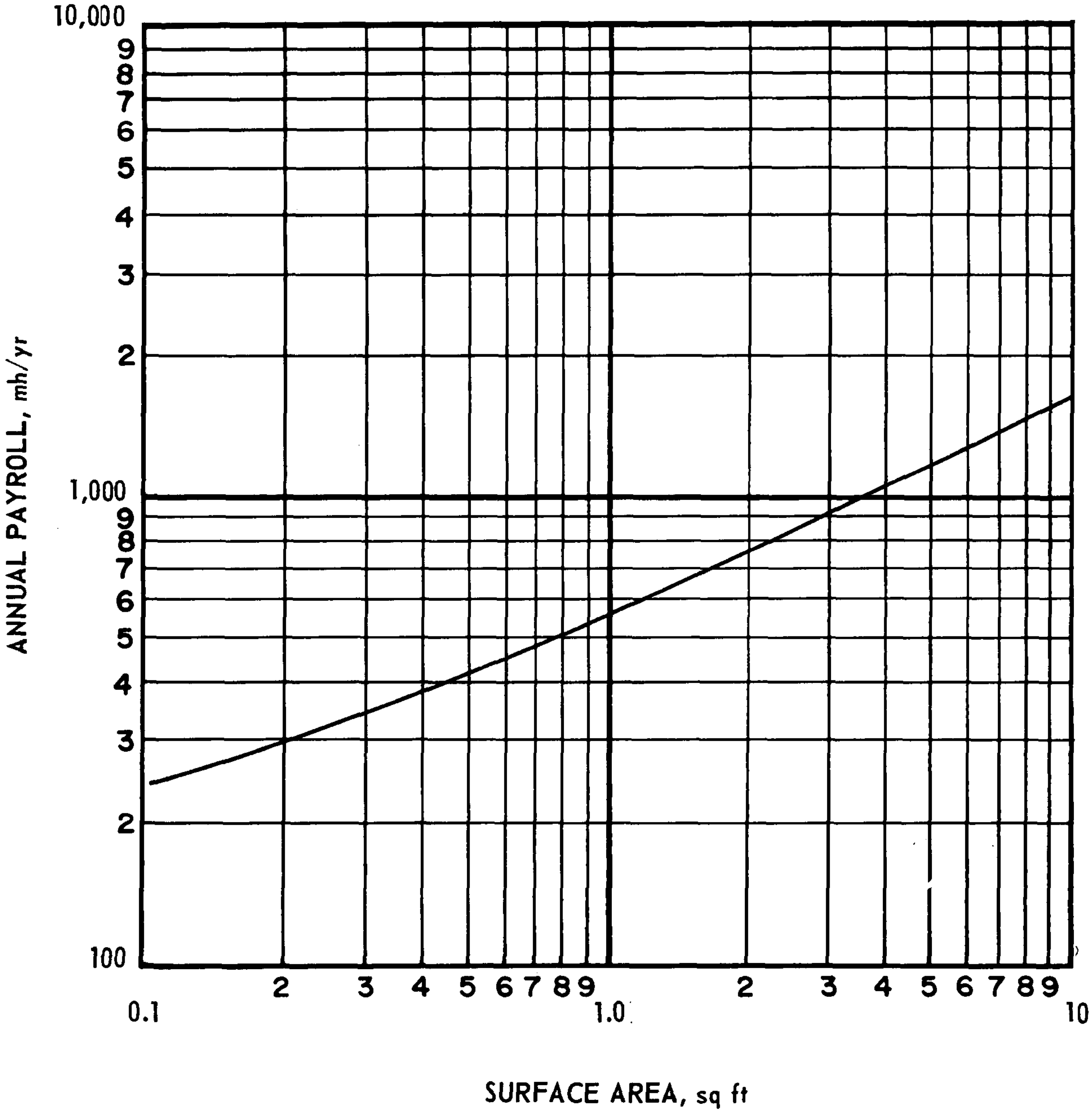


MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

PRELIMINARY TREATMENT

FIGURE A-20

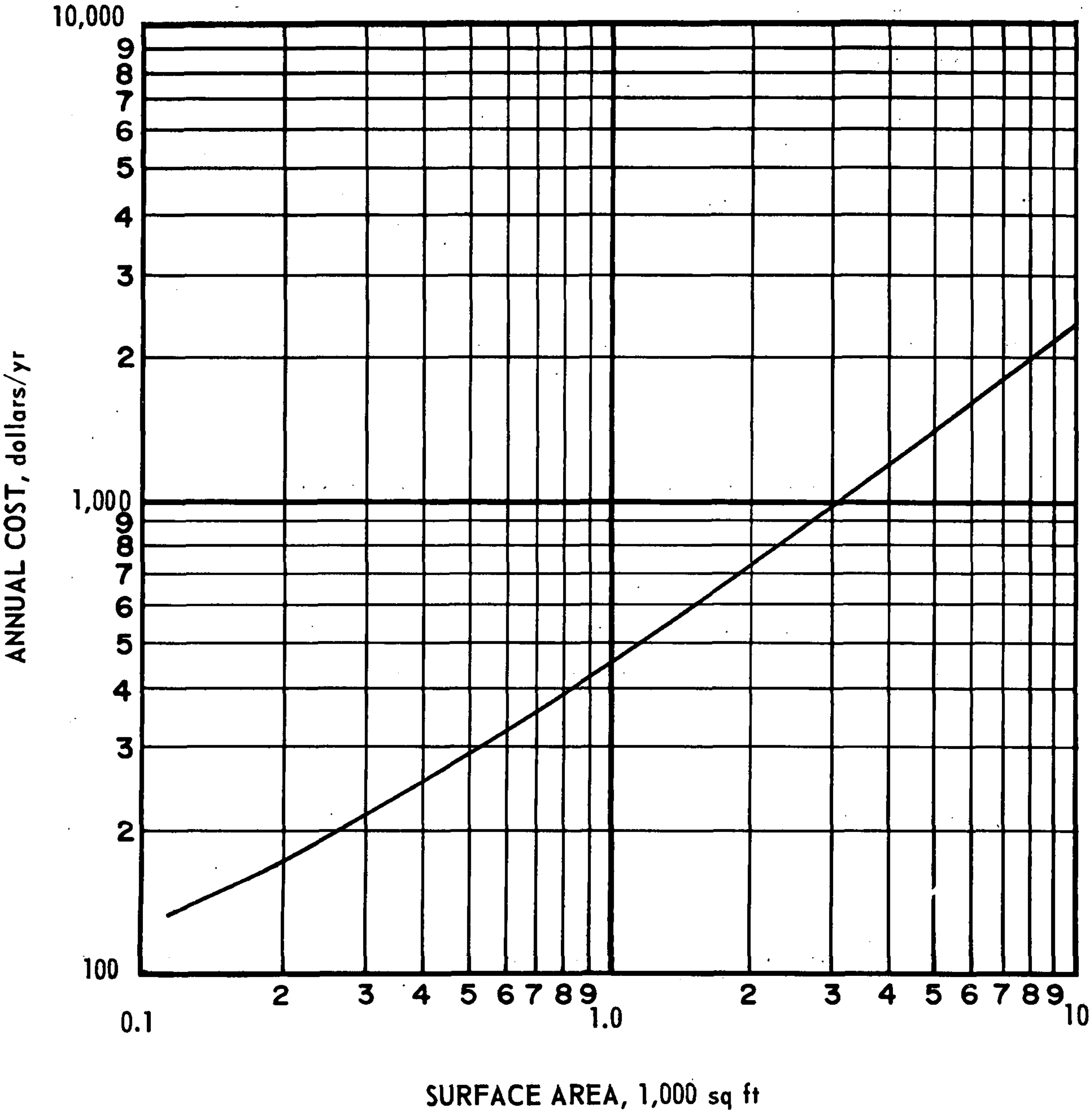


LABOR

OPERATION & MAINTENANCE REQUIREMENTS

SEDIMENTATION

FIGURE A-21

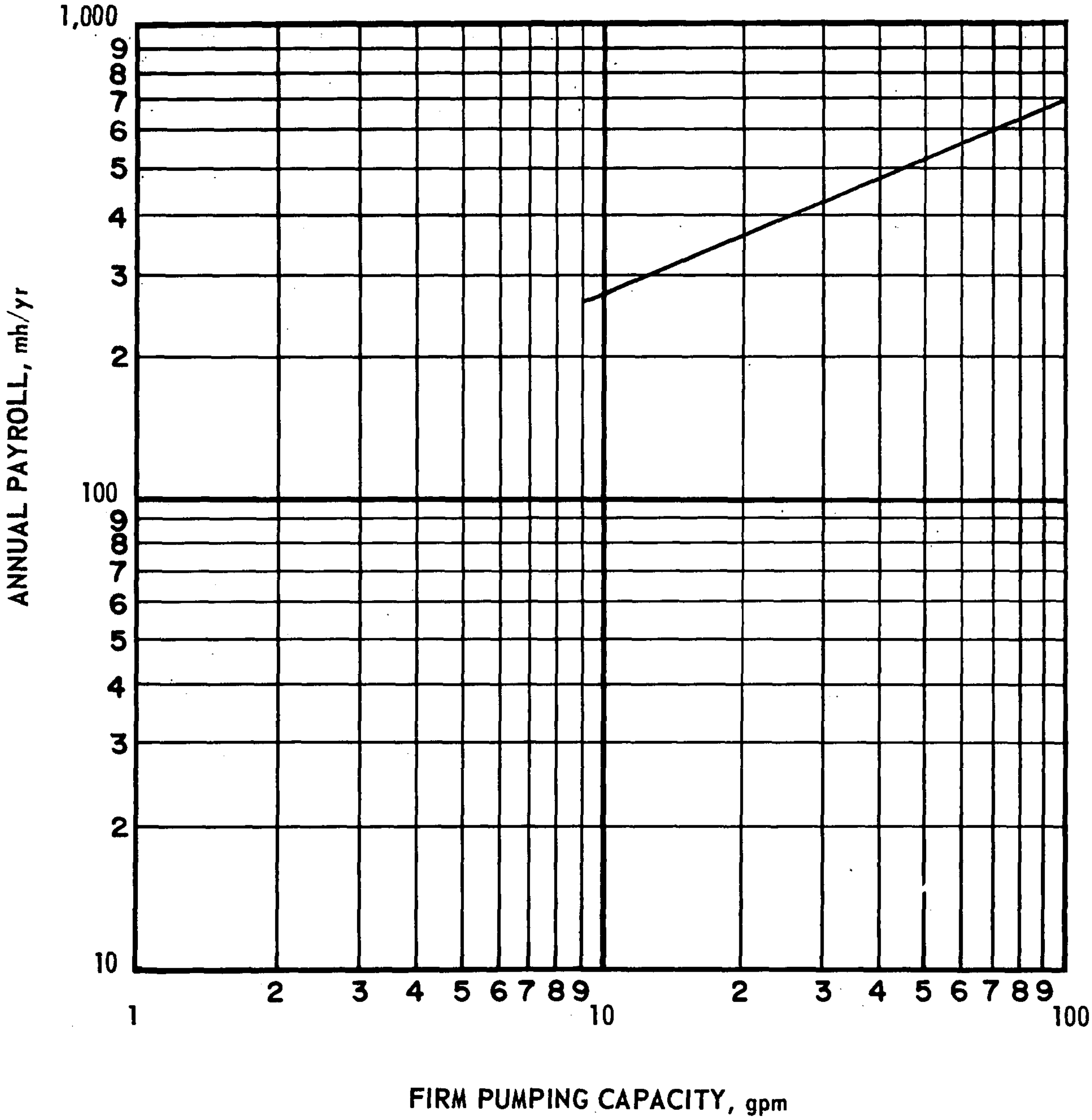


MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

SEDIMENTATION

FIGURE A–22

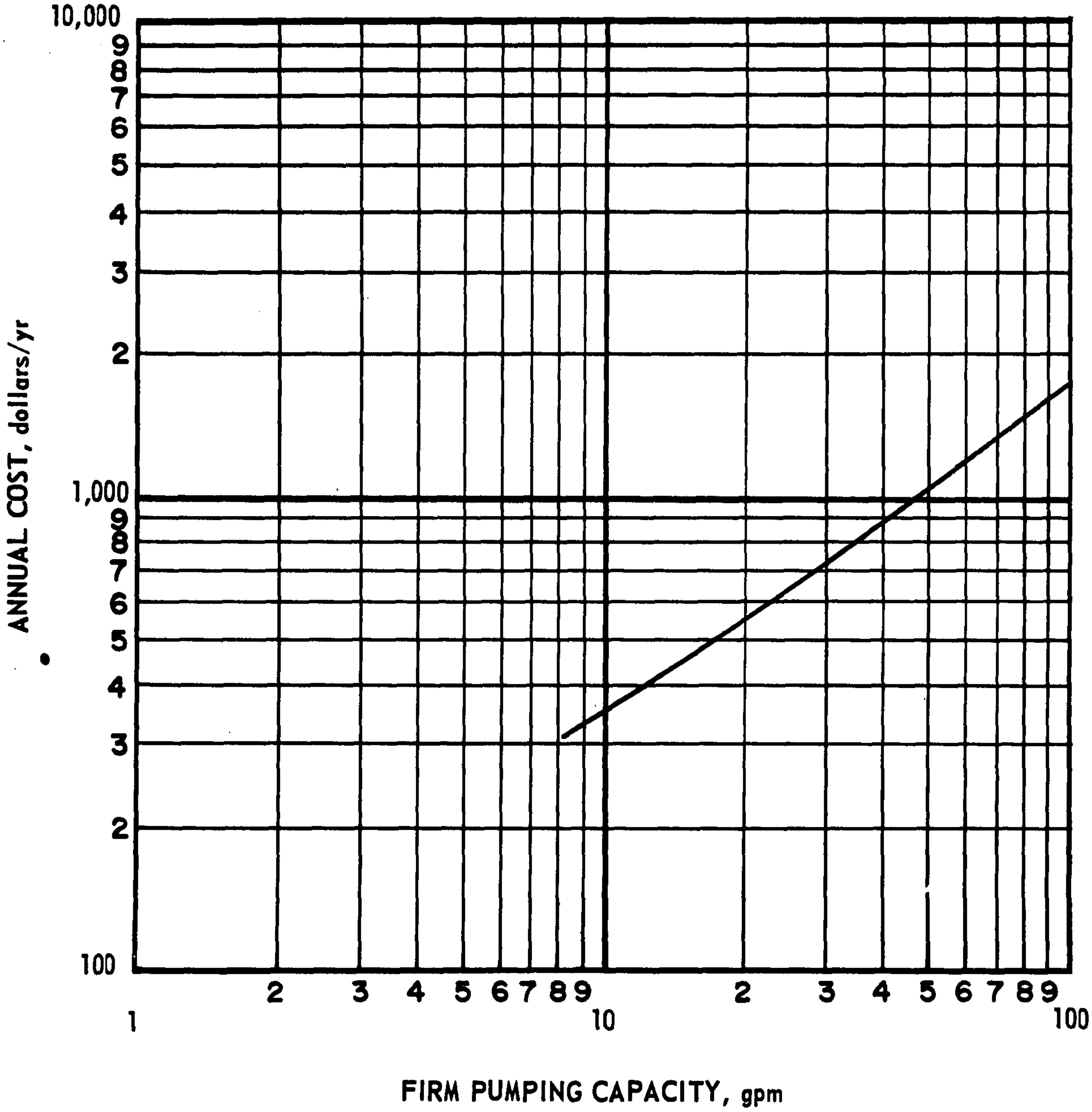


LABOR

OPERATION & MAINTENANCE REQUIREMENTS

SLUDGE PUMPING

FIGURE A–23

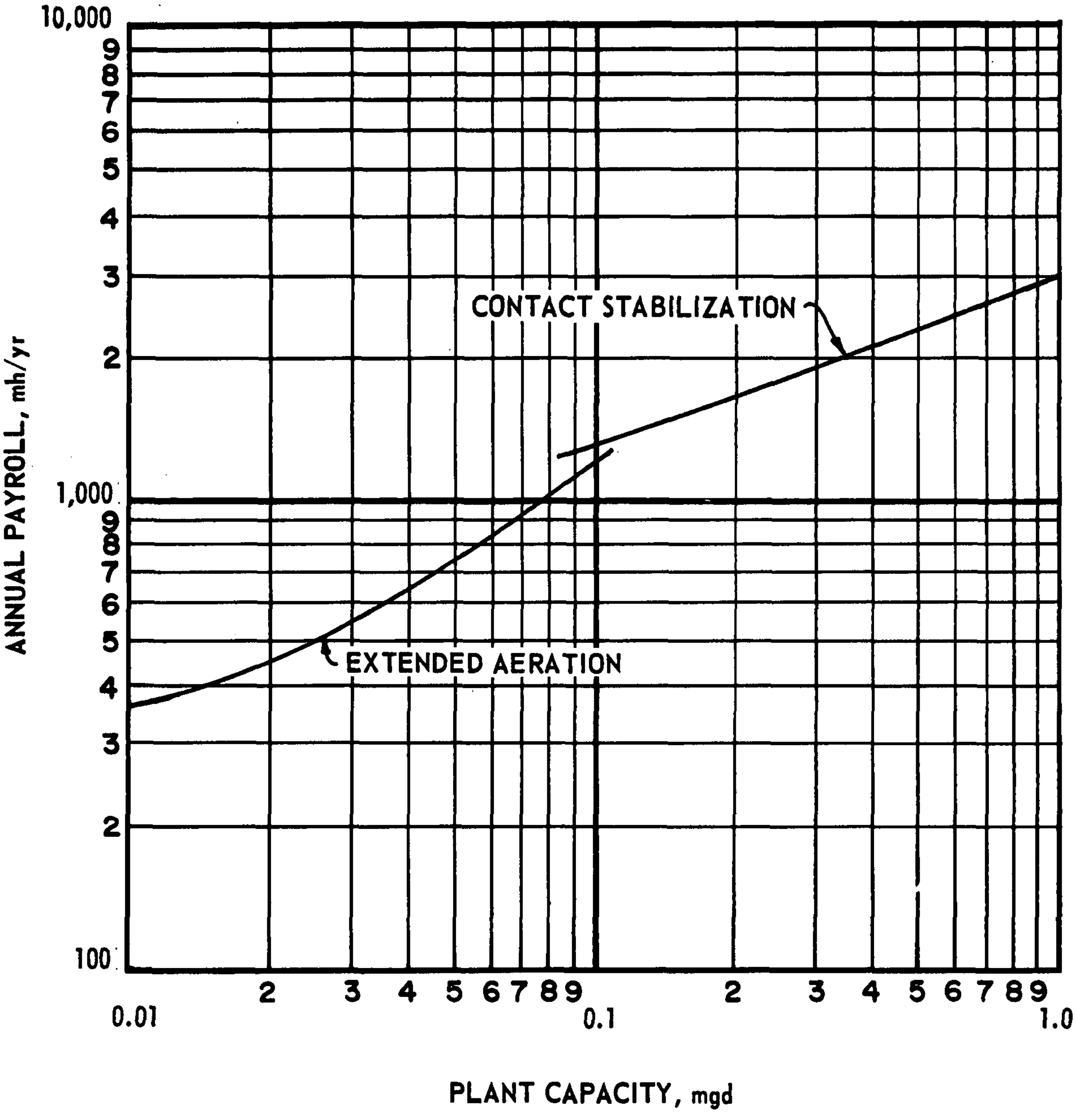


MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

SLUDGE PUMPING

FIGURE A-24



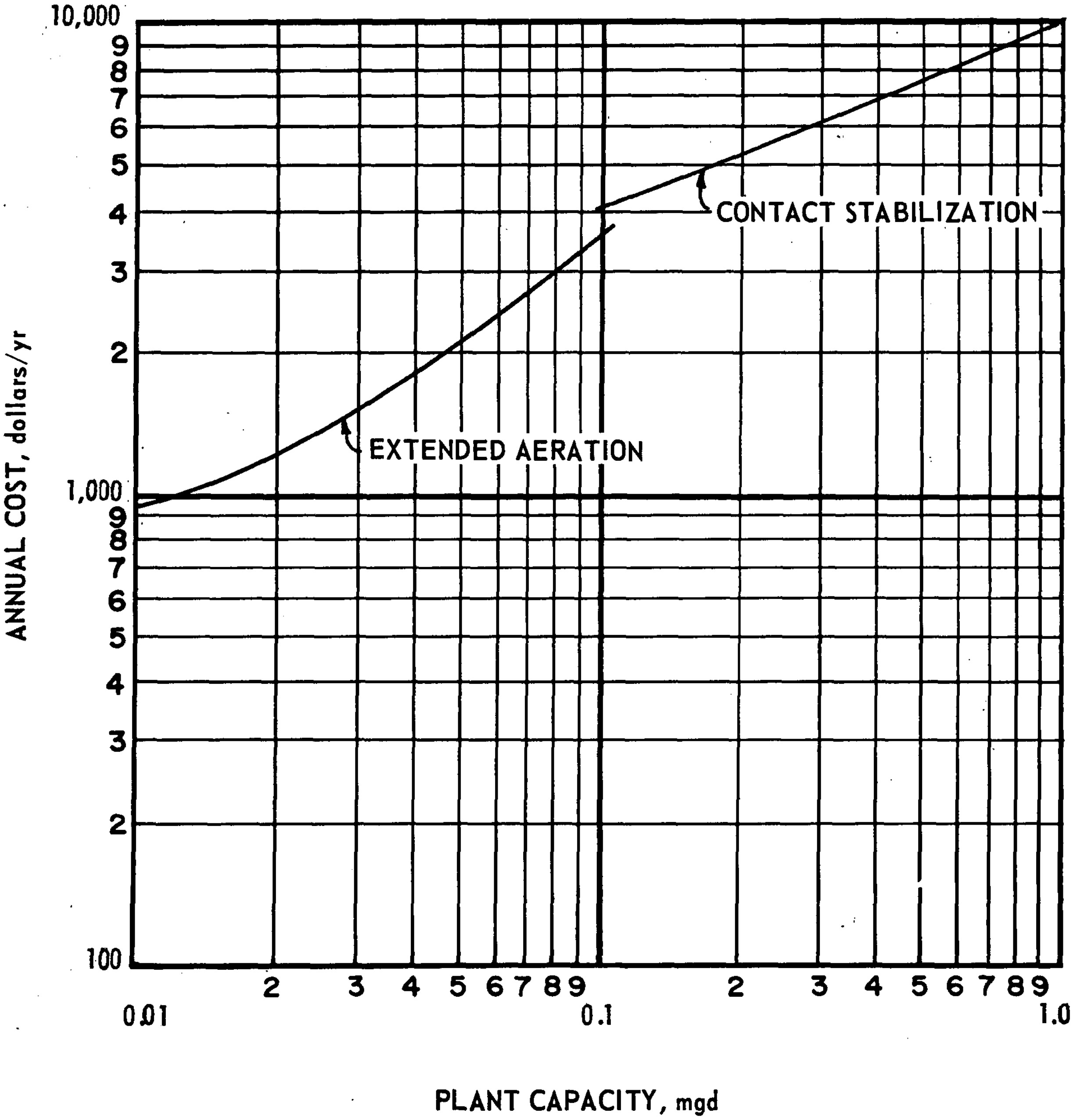
Note: Extended Aeration
Includes labor for sedimentation zone and aeration equipment.
Contact Stabilization
Includes labor for sedimentation basin, aeration equipment, aerobic digester

LABOR

OPERATION & MAINTENANCE REQUIREMENTS

PREFABRICATED ACTIVATED SLUDGE PLANTS

FIGURE A–25

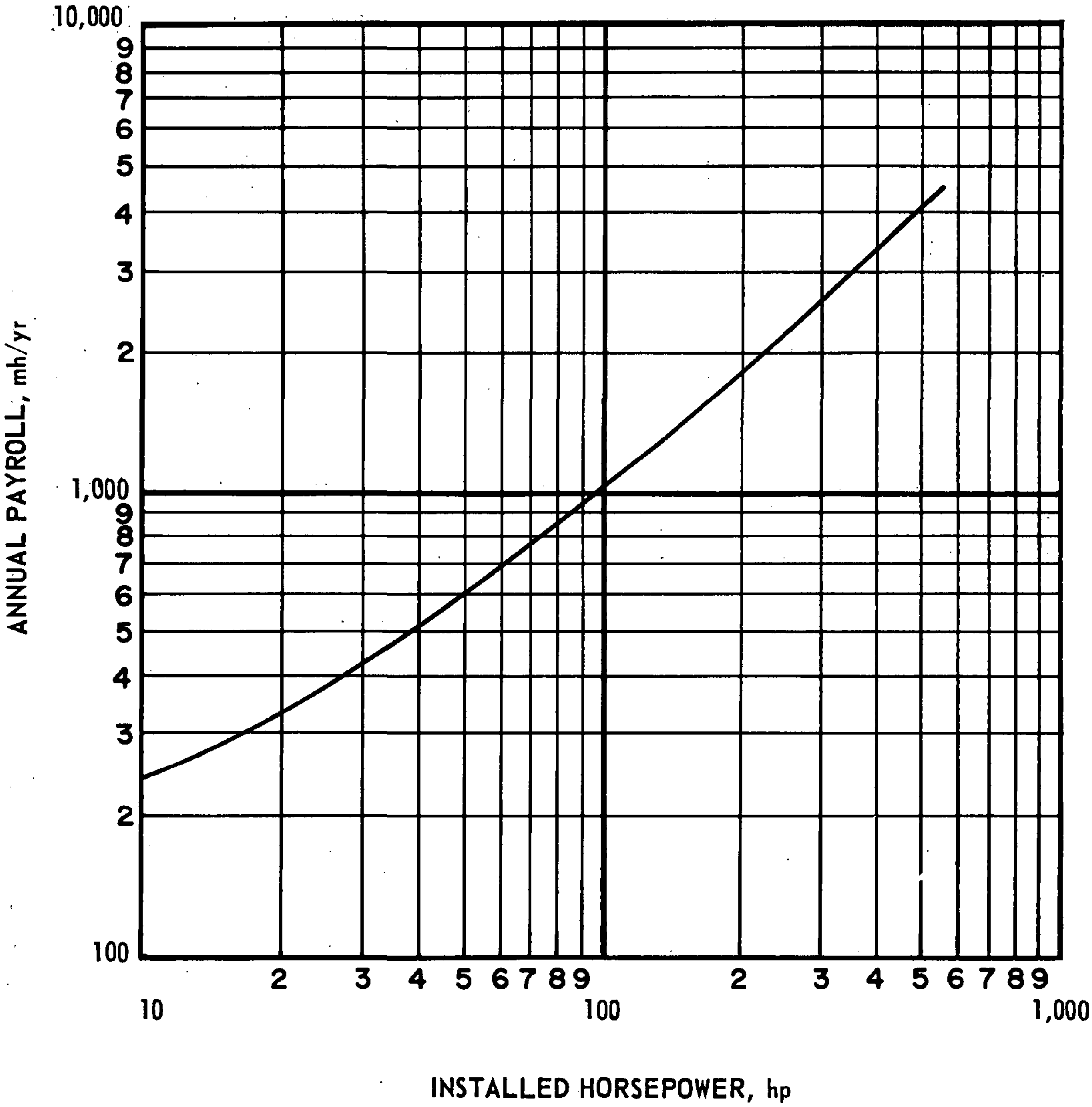


MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

PREFABRICATED ACTIVATED SLUDGE PLANTS

FIGURE A-26



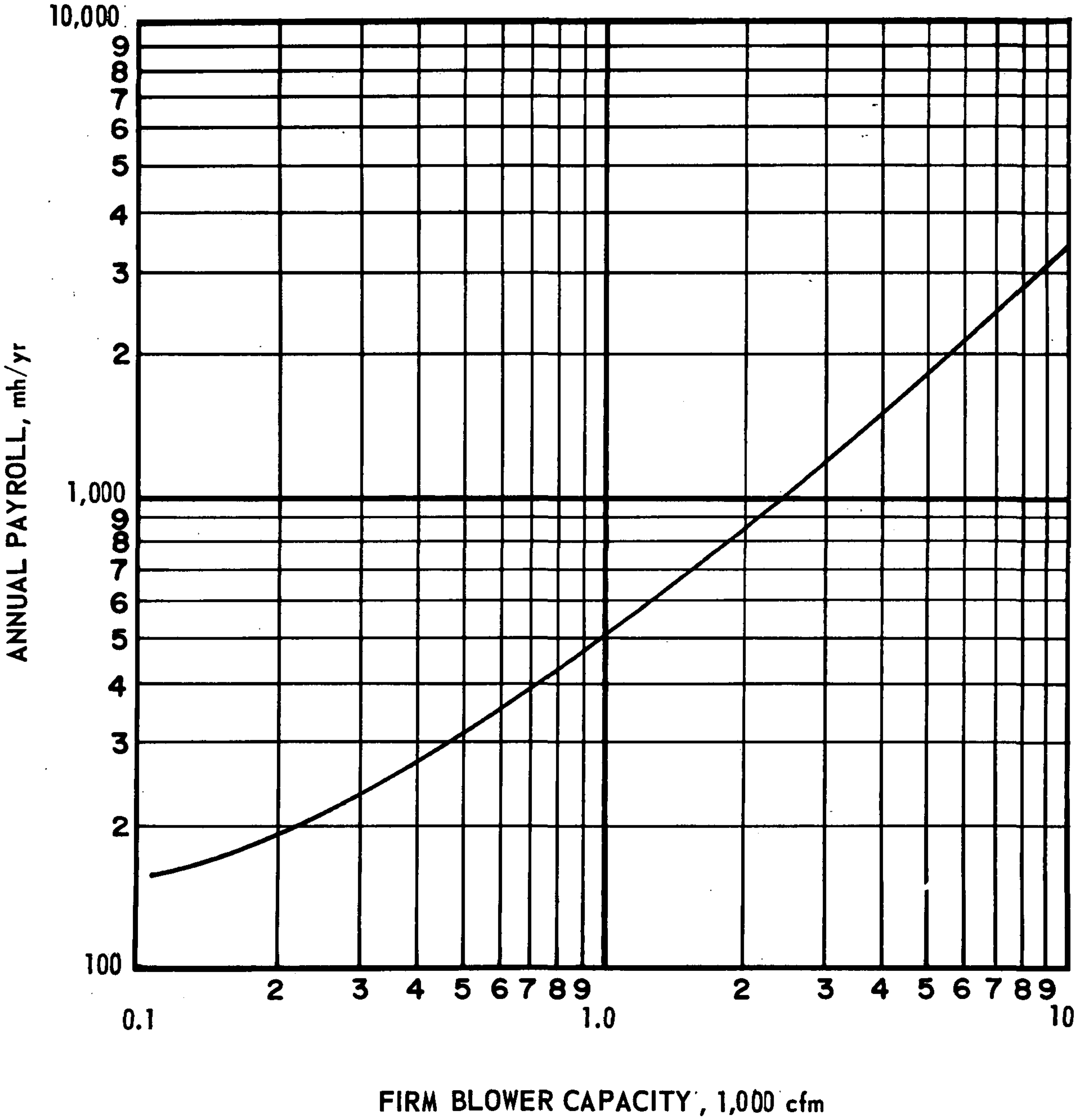
LABOR

OPERATION & MAINTENANCE REQUIREMENTS

CUSTOM BUILT AERATION BASINS USING

MECHANICAL AERATION

FIGURE A–27



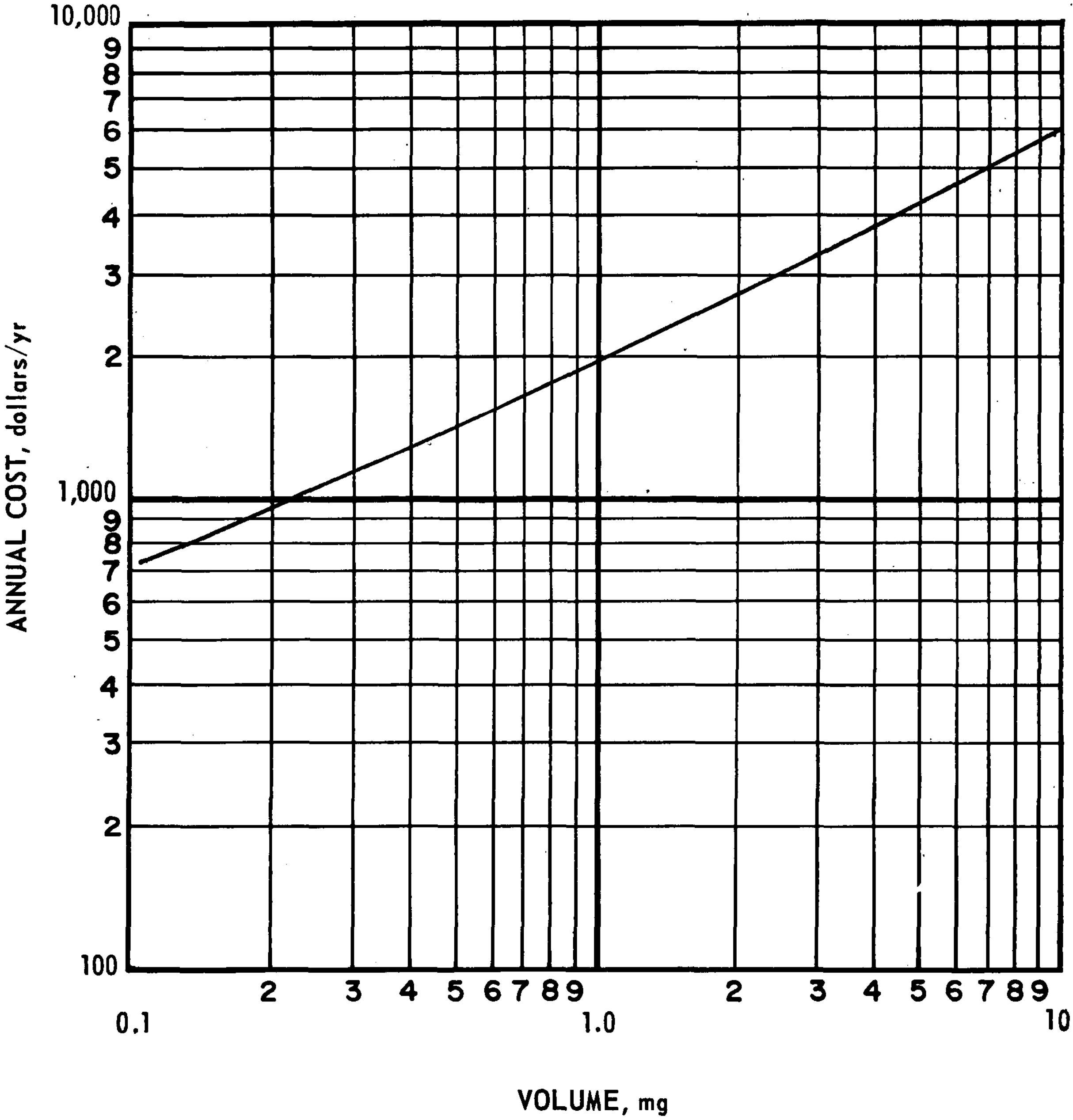
LABOR

OPERATION & MAINTENANCE REQUIREMENTS

CUSTOM BUILT AERATION BASINS USING

DIFFUSED AERATION

FIGURE A–28

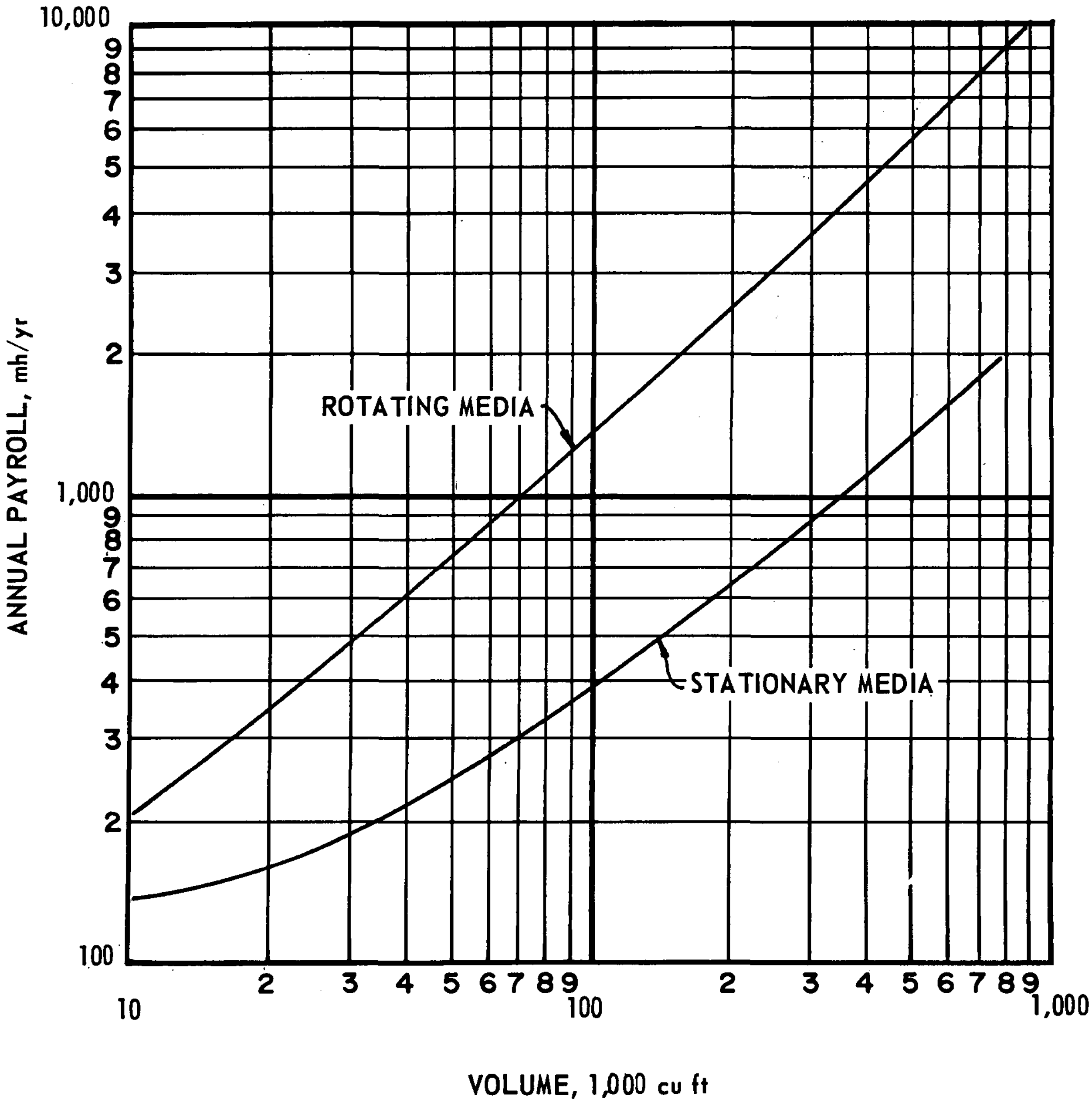


MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

CUSTOM BUILT AERATION BASINS

FIGURE A–29

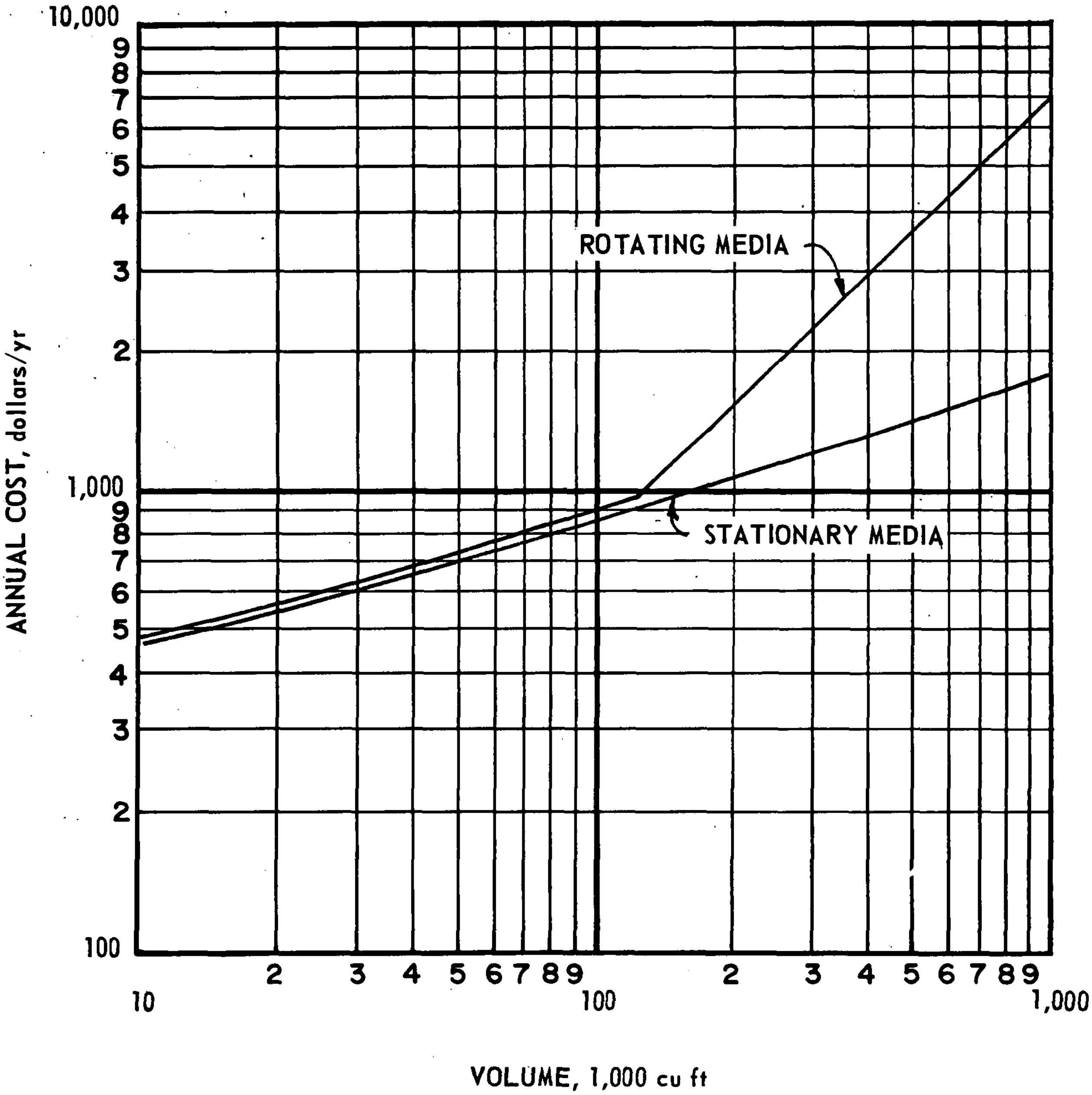


LABOR

OPERATION & MAINTENANCE REQUIREMENTS

TRICKLING FILTERS

FIGURE A–30

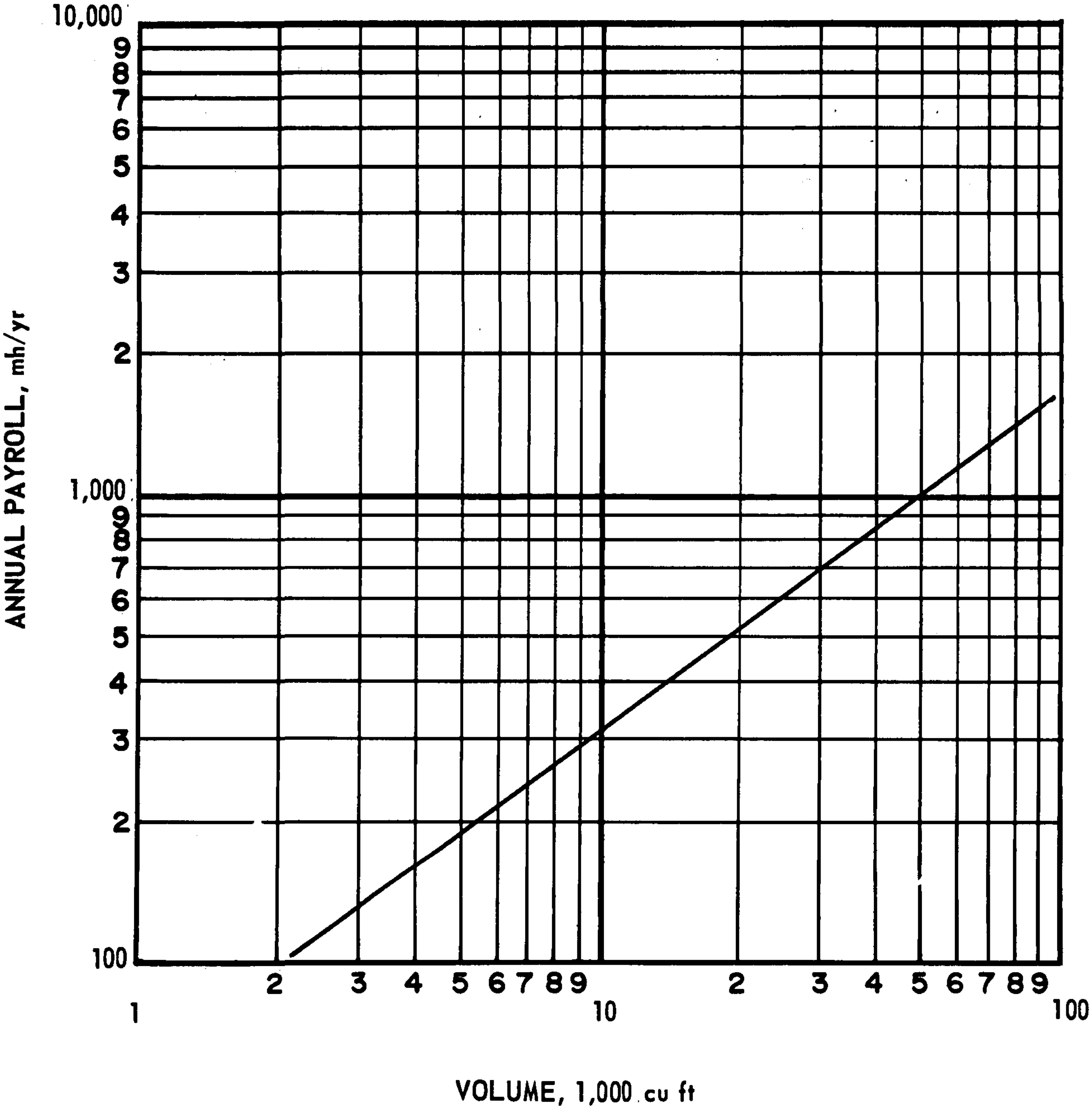


MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

TRICKLING FILTERS

FIGURE A-31

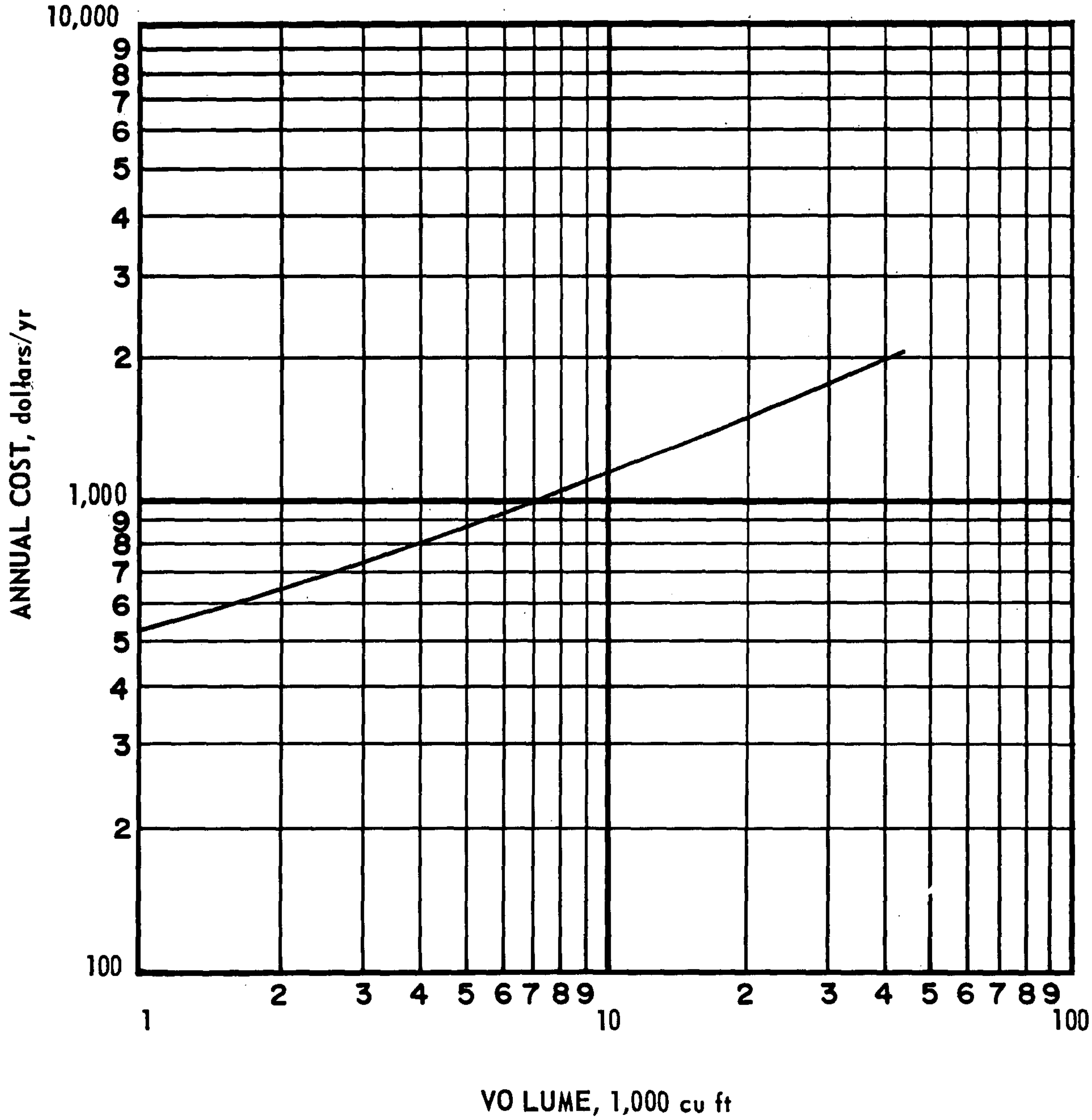


LABOR

OPERATION & MAINTENANCE REQUIREMENTS

ANAEROBIC DIGESTION

FIGURE A-32

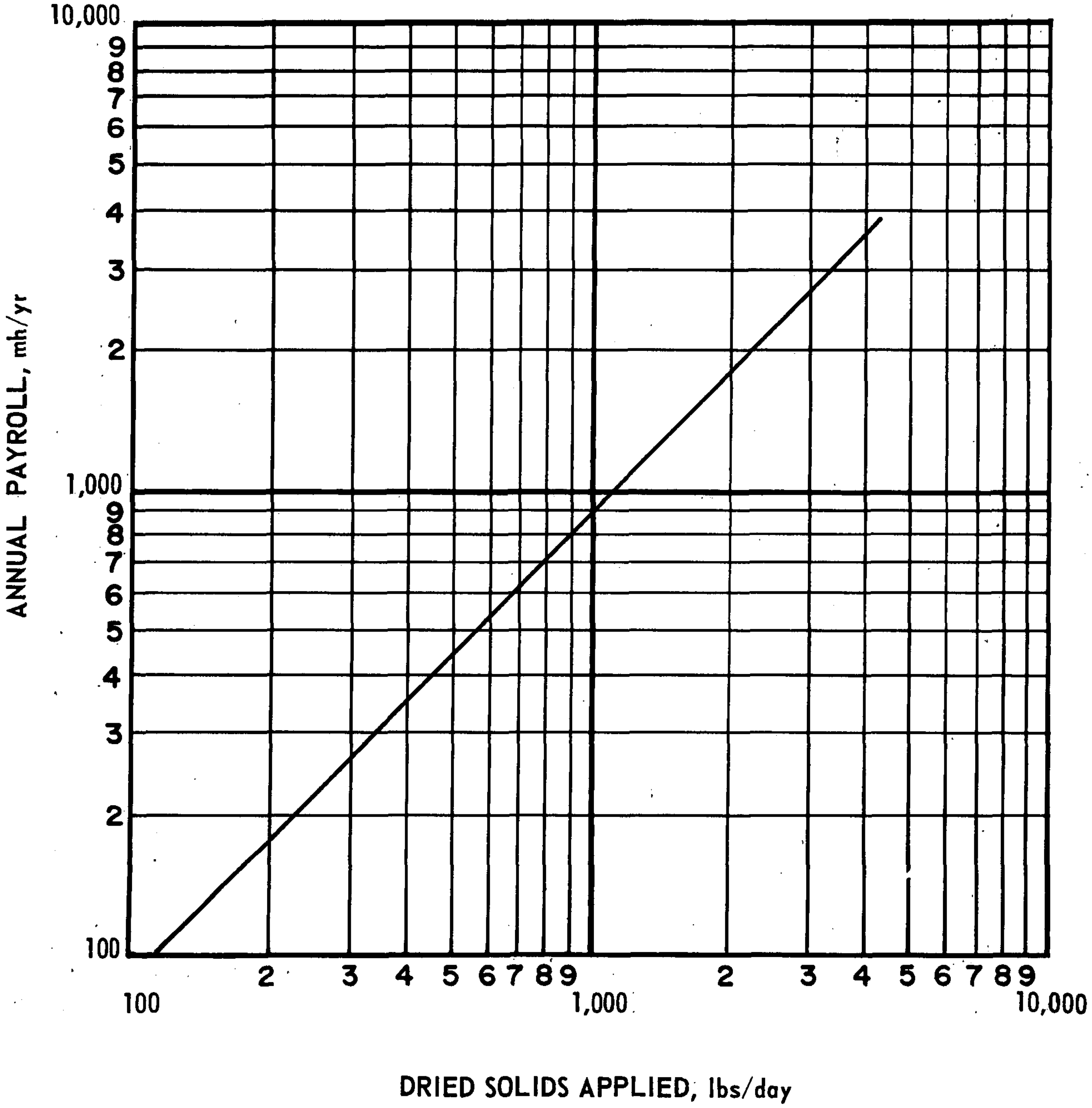


MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

ANAEROBIC DIGESTION

FIGURE A-33

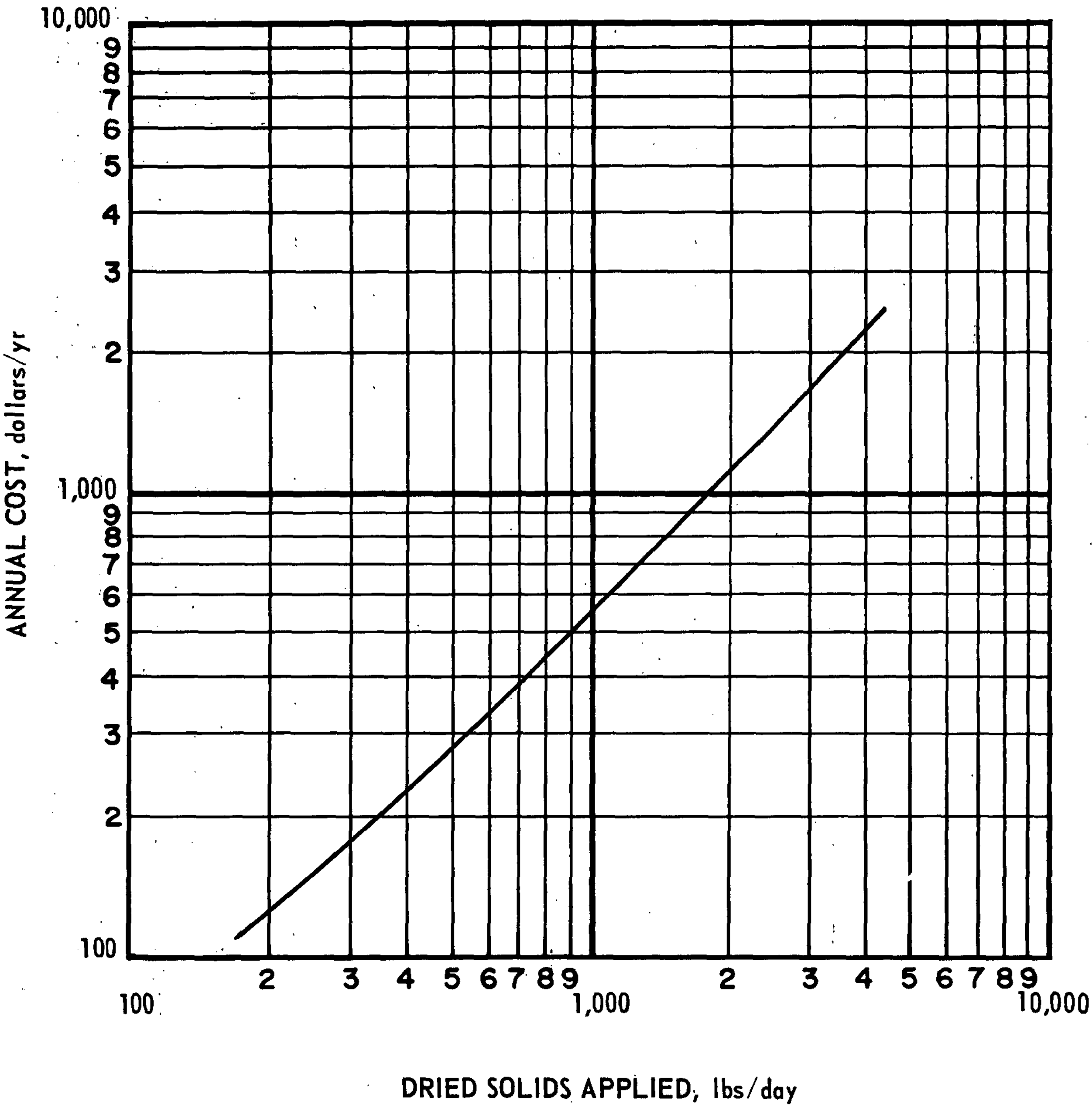


LABOR

OPERATION & MAINTENANCE REQUIREMENTS

SLUDGE DRYING BEDS

FIGURE A–34

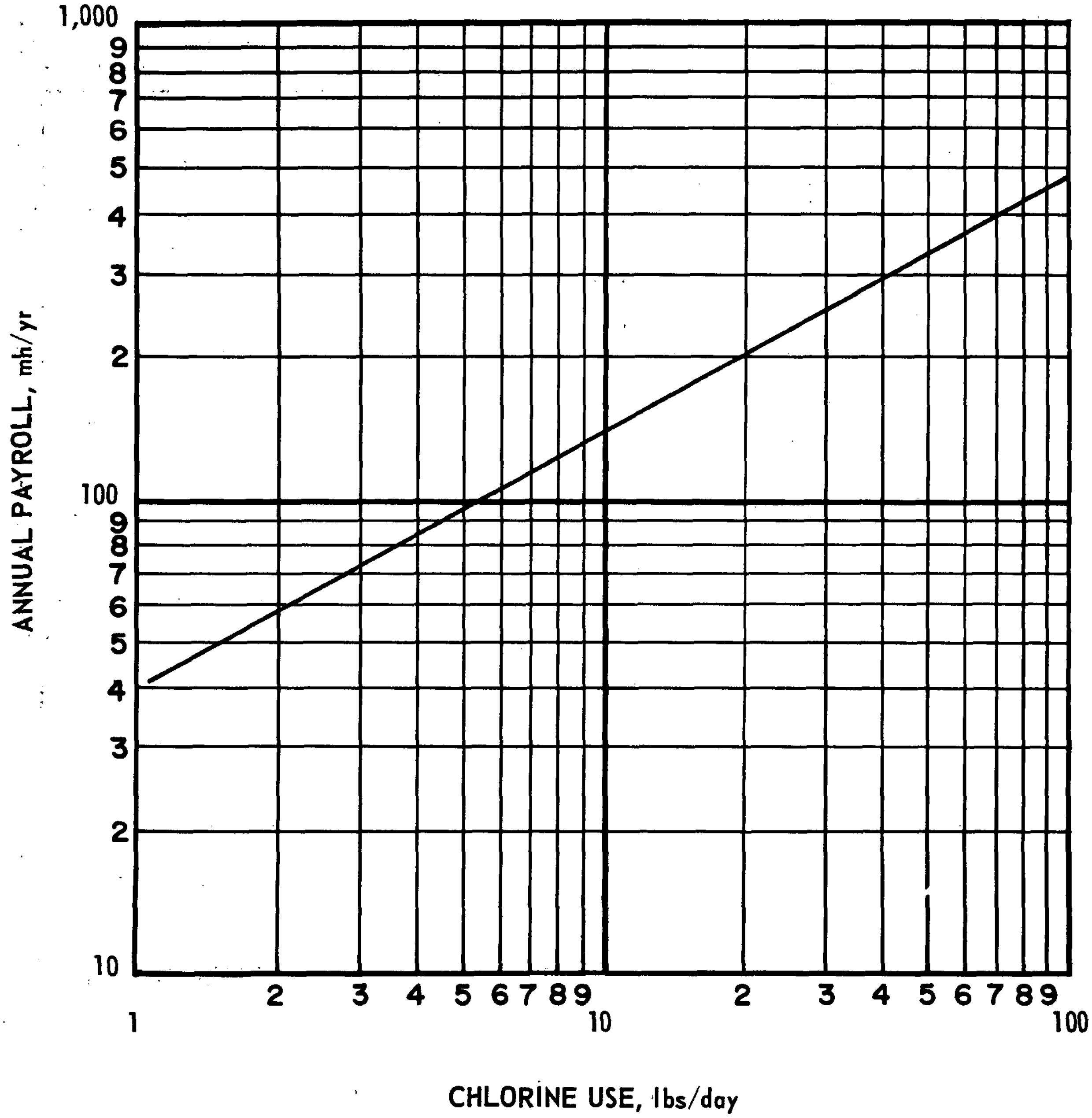


MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

SLUDGE DRYING BEDS

FIGURE A-35

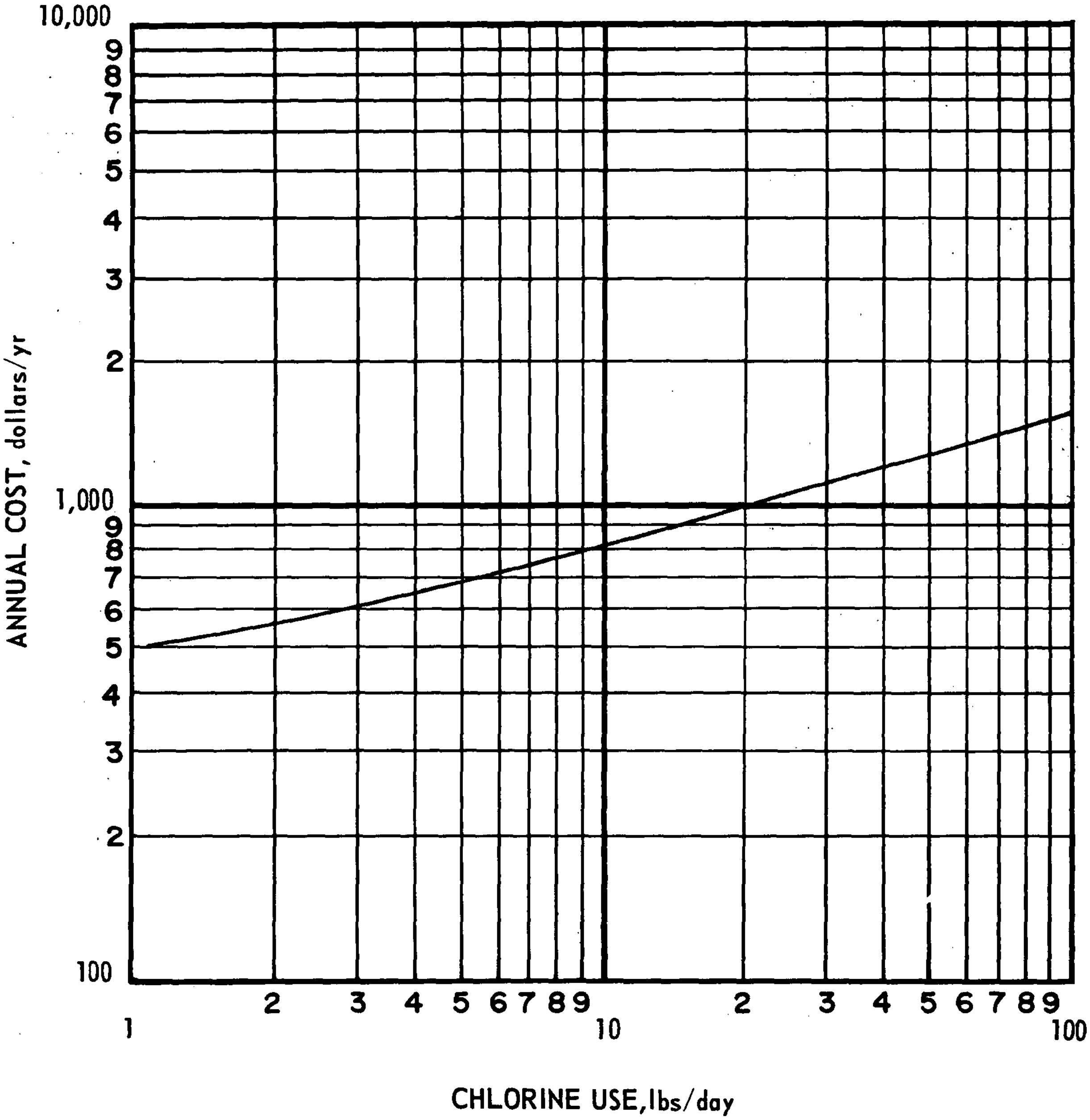


LABOR

OPERATION & MAINTENANCE REQUIREMENTS

CHLORINATION

FIGURE A-36

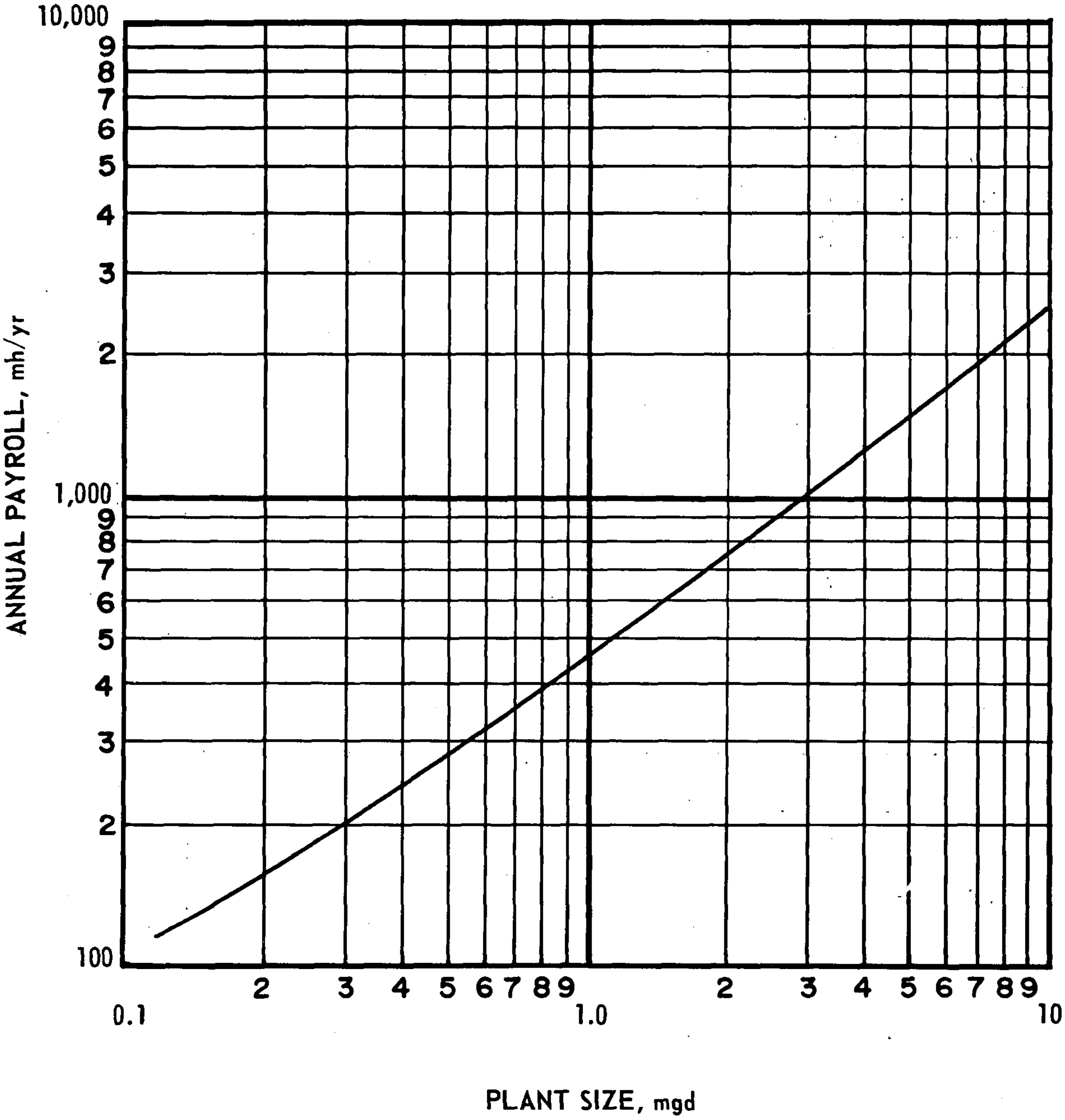


MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

CHLORINATION

FIGURE A–37

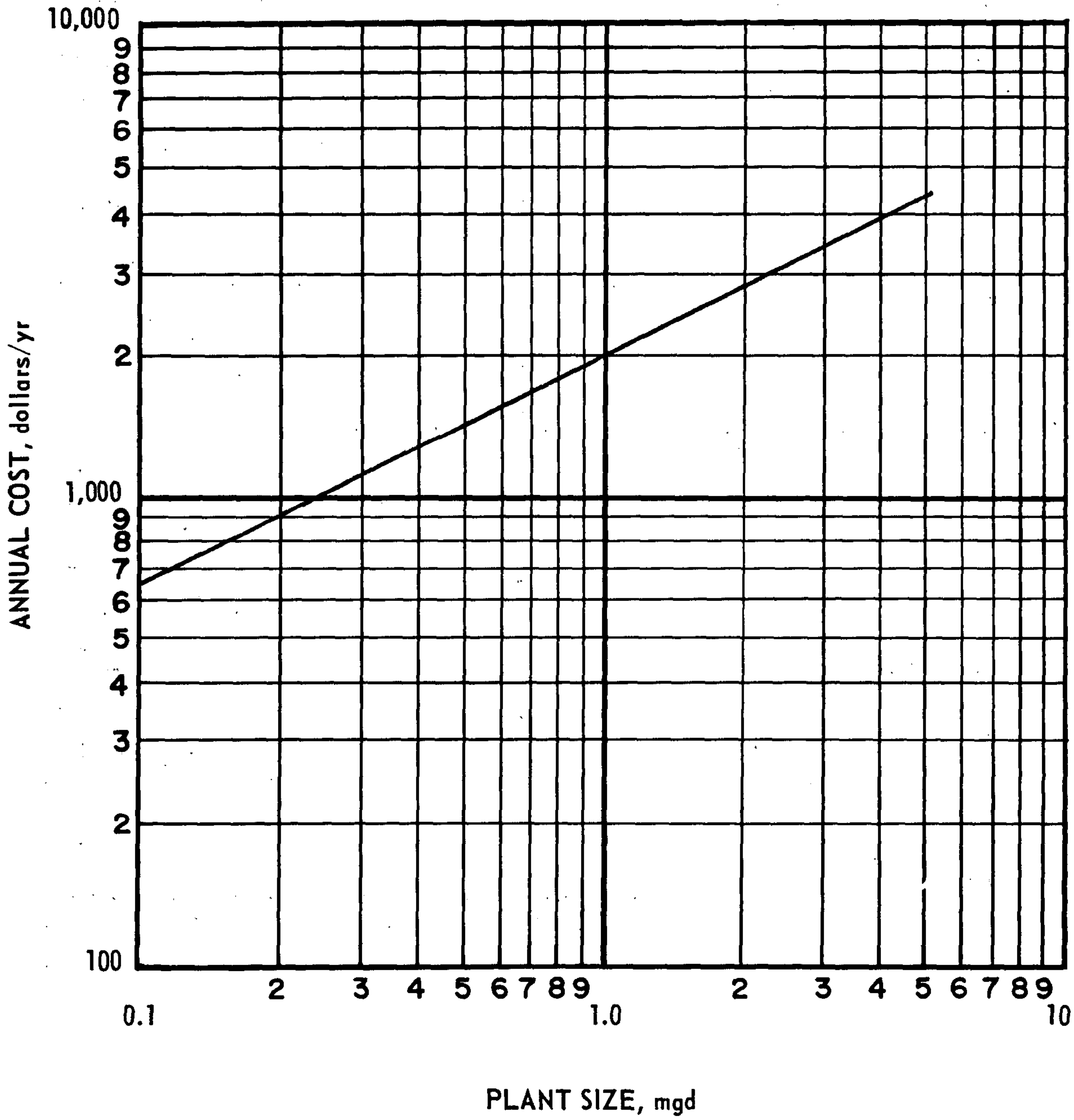


LABOR

OPERATION & MAINTENANCE REQUIREMENTS

ADMINISTRATION

FIGURE A-38

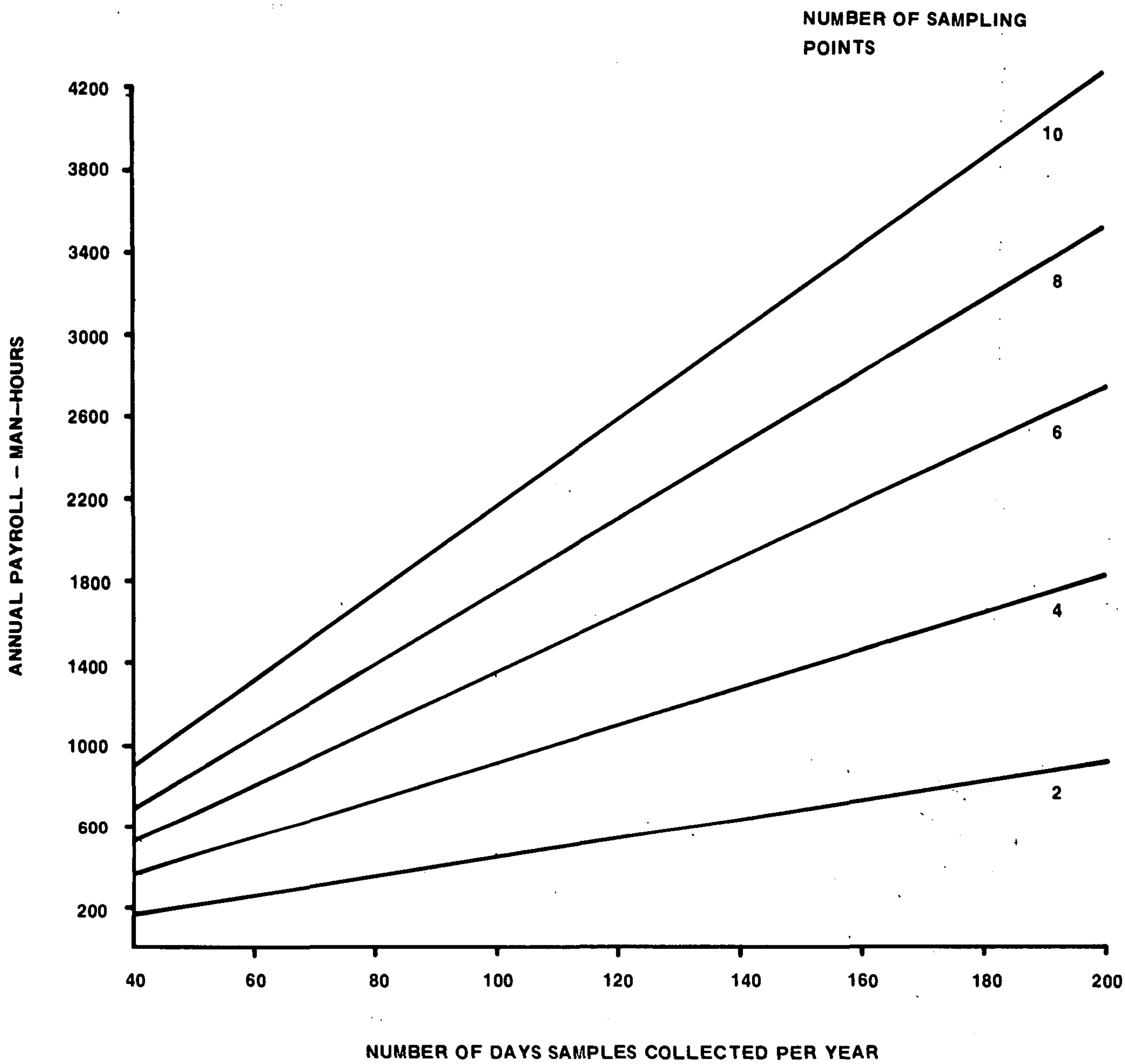


MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

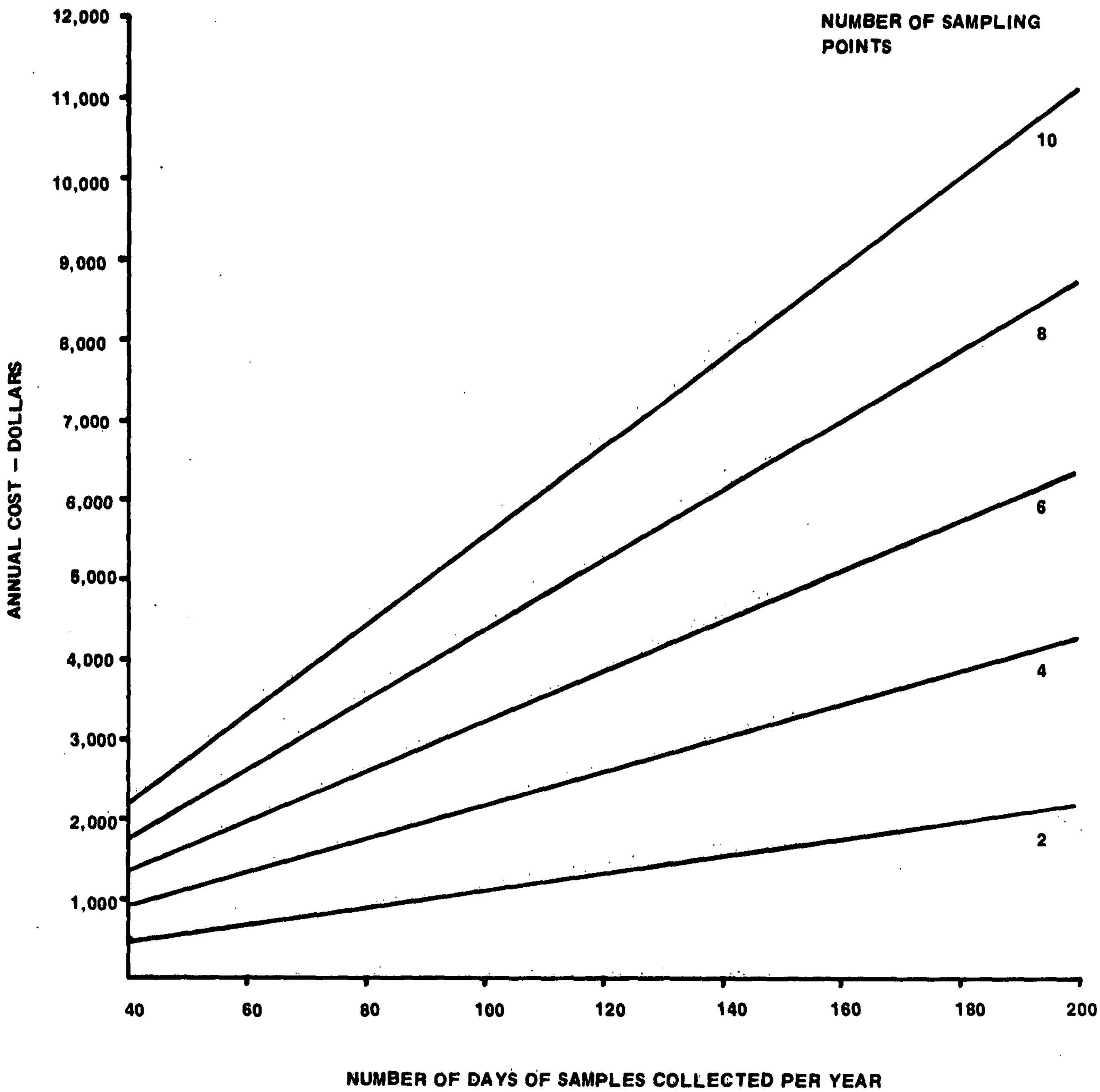
ADMINISTRATION

FIGURE A–39



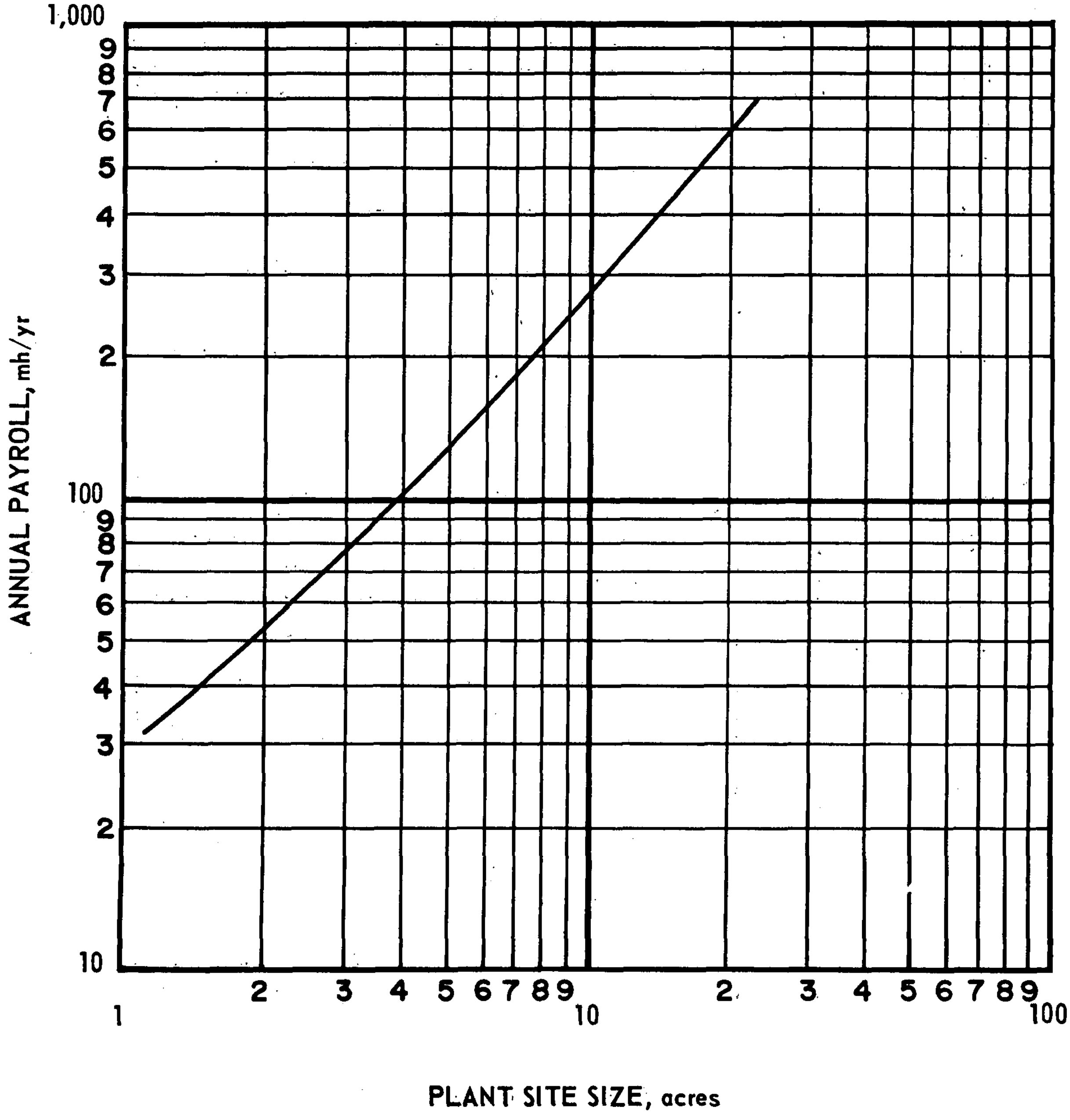
LABORATORY MAN-HOUR REQUIREMENTS

FIGURE A-40



LABORATORY, MISCELLANEOUS SUPPLY COSTS

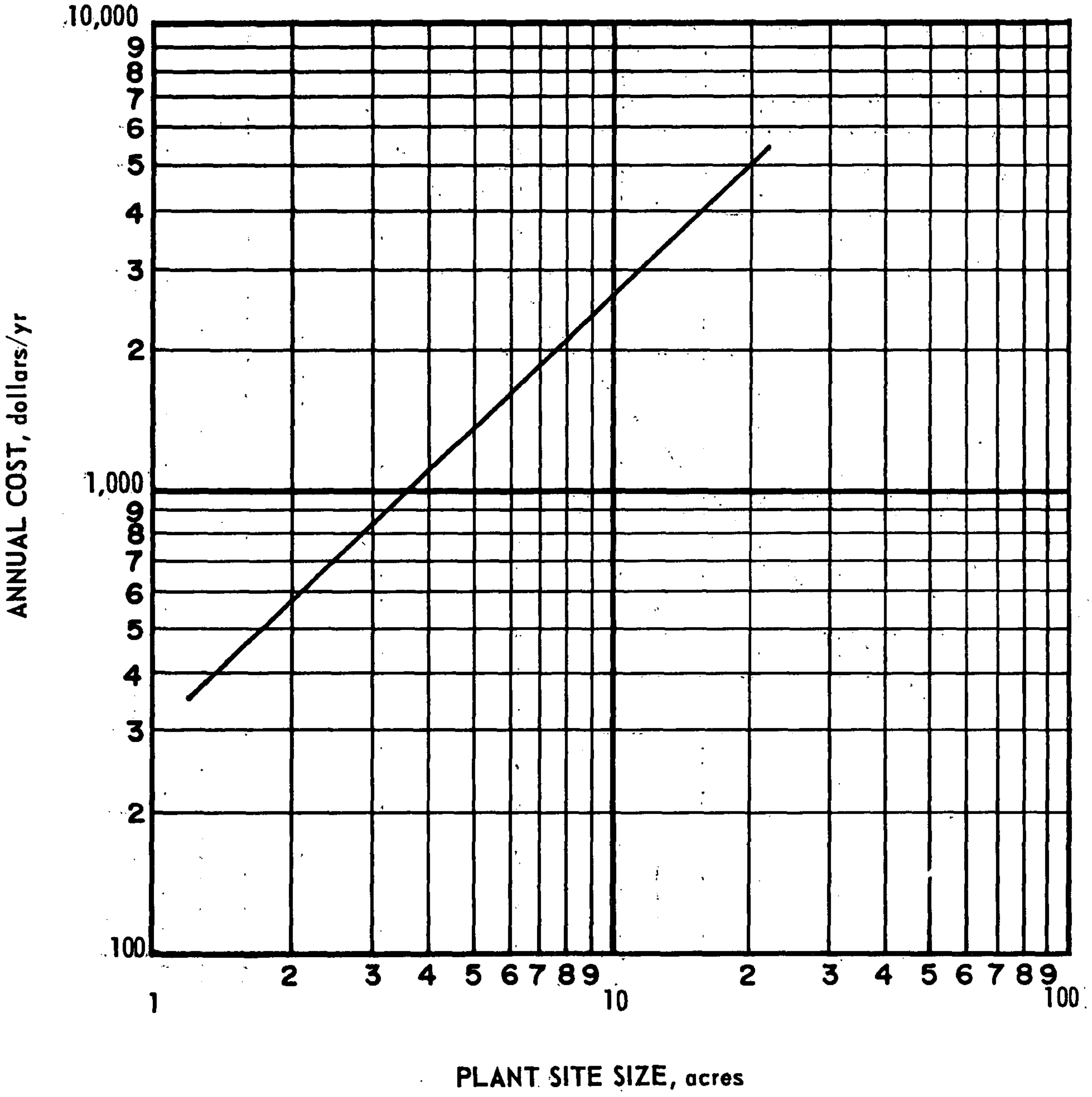
FIGURE A–41



OPERATION & MAINTENANCE LABOR REQUIREMENTS

YARDWORK

FIGURE A-42



MISCELLANEOUS SUPPLY COSTS

OPERATION & MAINTENANCE REQUIREMENTS

YARDWORK

FIGURE A-43

APPENDIX B

COST COMPARISON SUMMARY

PREFABRICATED EXTENDED AERATION - CASES I & II

	Plant Capacity - mgd		
	0.01	0.05	0.10
Capital Cost $	144,400	277,400	422,500
Operating Cost - $/Year			
Labor @ $9/MH	7,281	10,845	17,550
Power @ 3¢/KWH	531	1,005	4,170
Chlorine @ $250/TON	47	47	143
Misc. Supply	2,400	3,880	6,170
Annual O & M $/YEAR	10,259	15,777	28,033
Amortised Capital (6½% - 25 YRS)	11,838	22,741	34,637
Equivalent Annual Cost	22,097	38,518	62,670
Unit Cost ($/1000 GAL)	6.05	2.11	1.72

PREFABRICATED CONTACT STABILIZATION - CASE I

	Plant Capacity - mgd		
	0.10	0.50	1.0
Capital Cost $	463,700	988,500	1,578,300
Operating Cost - $/Year			
Labor @ $9/MH	20,160	36,360	53,280
Power @ 3¢/KWH	2,850	13,410	26,580
Chlorine @ $250/TON	143	712	1,186
Misc. Supply	7,720	14,720	18,970
Annual O & M $/YEAR	30,873	65,202	100,016
Amortised Capital (6½% - 25 YRS)	38,015	81,039	129,391
Equivalent Annual Cost	68,887	146,511	229,407
Unit Cost ($/1000 GAL)	1.89	0.80	0.63

PREFABRICATED CONTACT STABILIZATION - CASE II

	Plant Capacity - mgd		
	0.10	0.50	1.0
Capital Cost $	596,450	1,343,100	2,107,500
Operating Cost - $/Year			
Labor @ $9/MH	23,310	42,300	62,550
Power @ 3¢/KWH	5,910	25,710	57,030
Chlorine @ $250/TON	143	712	1,186
Misc. Supply	8,440	16,040	20,770
Annual O & M $/YEAR	37,803	84,672	141,536
Amortised Capital (6% - 25 YRS)	48,900	110,110	172,780
Equivalent Annual Cost	86,703	194,872	314,316
Unit Cost ($/1000 GAL)	2.38	1.07	0.86

CONVENTIONAL ACTIVATED SLUDGE - CASE I

	Plant Capacity - mgd			
	0.10	0.50	1.0	2.0
Capital Cost $	695,500	1,472,900	2,355,000	3,835,100
Operating Cost - $/Year				
Labor @ $9/MH	23,400	37,935	55,305	93,870
Power @ 3¢/KWH	3,570	11,610	22,920	45,750
Chlorine @ $250/TON	143	712	1,186	2,373
Misc. Supply	6,500	10,690	15,150	25,720
Annual O & M $/YEAR	33,613	60,947	94,561	167,713
Amortised Capital (6½% - 25 YRS)	57,018	120,750	193,066	314,407
Equivalent Annual Cost	90,631	181,697	287,627	482,120
Unit Cost ($/1000 GAL)	2.48	1.00	0.79	0.66

CONVENTIONAL ACTIVATED SLUDGE - CASE II

	Plant Capacity - mgd			
	0.10	0.50	1.0	2.0
Capital Cost $	753,740	1,611,200	2,537,000	4,162,700
Operating Cost - $/Year				
Labor @ $9/MH	23,760	40,365	70,305	101,970
Power @ 3¢/KWH	7,530	19,500	27,654	77,280
Chlorine @ $250/TON	143	712	1.186	2,373
Misc. Supply	6,550	10,890	15,350	26,120
Annual O & M $/YEAR	37,983	71,467	114,495	207,743
Amortised Capital (6% - 25 YRS)	61,800	132,100	208,000	341,300
Equivalent Annual Cost	99,783	203,567	322,495	549,043
Unit Cost ($/1000 GAL)	2.73	1.12	0.89	0.75

CUSTOM BUILT EXTENDED AERATION - CASES I & II

	Plant Capacity - mgd			
	0.10	0.50	1.0	2.0
Capital Cost $	424,300	1,008,800	1,696,800	2,898,600
Operating Cost - $/Year				
Labor @ $9/MH	15,660	32,445	52,110	92,160
Power @ 3¢/KWH	4,020	19,410	38,640	77,130
Chlorine @ $250/TON	143	712	1,186	2,373
Misc. Supply	5,290	9,770	13,480	20,900
Annual O & M $/YEAR	25,113	62,337	105,416	192,563
Amortised Capital (6½% - 25 YRS)	34,784	82,702	139,106	237,632
Equivalent Annual Cost	59,897	145,039	244,522	430,195
Unit Cost ($/1000 GAL)	1.64	0.79	0.67	0.59

OXIDATION DITCH - CASES I & II

	Plant Capacity - mgd			
	0.10	0.50	1.0	2.0
Capital Cost $	432,400	1,029,900	1,732,500	2,816,100
Operating Cost - $/Year				
Labor @ $9/MH	15,660	32,445	52,110	92,160
Power @ 3¢/KWH	4,020	19,410	38,640	77,130
Chlorine @ $250/TON	143	712	1,186	2,373
Misc. Supply	5,290	9,770	13,480	20,900
Annual O & M $/YEAR	25,113	62,337	105,416	192,563
Amortised Capital (6½% - 25 YRS)	34,784	82,702	139,106	237,632
Equivalent Annual Cost	59,897	145,039	244,522	430,195
Unit Cost ($/1000 GAL)	1.64	0.79	0.67	0.59

ROCK MEDIA TRICKLING FILTERS - CASE I

	Plant Capacity - mgd			
	0.10	0.50	1.0	2.0
Capital Cost $	768,600	1,583,600	2,570,900	4,375,500
Operating Cost - $/Year				
Labor @ $9/MH	18,450	26,955	45,855	79,560
Power @ 3¢/KWH	930	3,780	7,380	14,550
Chlorine @ $250/TON	143	712	1,186	2,373
Misc. Supply	6,120	10,300	14,300	22,270
Annual O & M $/YEAR	25,643	41,747	68,721	118,753
Amortised Capital (6½% - 25 YRS)	63,011	129,826	210,766	358,710
Equivalent Annual Cost	88,654	171,573	279,487	477,463
Unit Cost ($/1000 GAL)	2.43	0.94	0.77	0.65

ROCK MEDIA TRICKLING FILTER - CASE II

	Plant Capacity - mgd			
	0.10	0.50	1.0	2.0
Capital Cost $	790,440	1,747,400	2,989,500	5,194,500
Operating Cost - $/Year				
Labor @ $9/MH	18,540	27,585	47,025	81,360
Power @ 3¢/KWH	930	3,780	7,380	14,550
Chlorine @ $250/TON	143	712	1,186	2,373
Misc. Supply	6,200	10,390	14,300	30,190
Annual O & M $/YEAR	25,813	42,467	69,891	128,473
Amortised Capital (6% - 25 YRS)	64,800	143,250	245,100	425,850
Equivalent Annual Cost	90,613	185,717	314,991	554,323
Unit Cost ($/1000 GAL)	2.48	1.02	0.86	0.76

ROTATING BIOLOGICAL MEDIA - CASE I

	Plant Capacity - mgd			
	0.10	0.50	1.0	2.0
Capital Cost $	596,900	1,501,900	2,531,000	4,325,000
Operating Cost - $/Year				
Labor @ $9/MH	17,280	26,505	46,755	83,700
Power @ 3¢/KWH	1,950	8,280	16,980	33,150
Chlorine @ $250/TON	143	712	1,186	2,373
Misc. Supply	5,870	9,740	13,410	20,770
Annual O & M $/YEAR	25,243	45,237	78,331	139,993
Amortised Capital (6½% - 25 YRS)	48,900	123,100	207,500	354,600
Equivalent Annual Cost	74,143	168,337	285,831	454,593
Unit Cost ($/1000 GAL)	2.03	0.92	0.78	0.62

ROTATING BIOLOGICAL MEDIA - CASE II

	Plant Capacity - mgd			
	0.10	0.50	1.0	2.0
Capital Cost $	678,900	1,941,900	3,265,000	5,629,000
Operating Cost - $/Year				
Labor @ $9/MH	18,000	29,115	52,245	90,000
Power @ 3¢/KWH	3,390	14,880	30,780	60,150
Chlorine @ $250/TON	143	712	1,186	2,373
Misc. Supply	6,080	9,840	13,580	21,020
Annual O & M $/YEAR	27,613	54,547	97,791	173,543
Amortised Capital (6% - 25 YRS)	55,700	159,200	267,700	461,500
Equivalent Annual Cost	83,313	213,747	365,491	635,043
Unit Cost ($/1000 GAL)	2.28	1.17	1.00	0.87

APPENDIX C

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