EPA-600/9-76-014
July 1976

AREAWIDE ASSESSMENT

PROCEDURES MANUAL

VOLUME III

MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

APPENDIX G

PART I

URBAN STORMWATER MANAGEMENT TECHNIQUES: PERFORMANCE AND COST

G.1 Introduction

G.1.1 Purpose of Appendix G and Its Relationship to Chapter 6

The Chapter 2 preliminary assessment presented a broad identification of major pollutant sources within an urban area, together with a perspective on the relative magnitude of pollutant loads generated in an urban setting by intermittant sources from combined sewer overflows and storm drainage systems. In Chapter 3, alternative technical approaches for the estimation of stormwater-related waste flow and quality were discussed. Various methods of analyses including steady-state and time-varying water quality modeling were presented in Chapter 5 to determine the impact of storm related pollutants on the quality of receiving waters. Chapter 6 presents a methodology for the evaluation of stormwater runoff control alternatives. The Manual user is guided through a worksheet approach to the development of a least cost mix of structural and nonstructural pollution control alternatives to meet previously identified water quality goals.

In support of Chapter 6, Appendixes G and H have been prepared. Appendix G provides the estimated performance capability and pollutant removal unit cost input to the alternatives methodology of Chapter 6 from the standpoint of nonstructural control of urban stormwater runoff. This is in contrast to Appendix H which presents performance and cost data for structural solutions to point source and nonpoint source pollution control. In order to make Appendix G responsive to the data input needs of the alternatives evaluation methodology of Chapter 6, the discussions of nonstructural management techniques have been formated into Land Management Control Alternatives and Collection Systems Control Alternatives.

G.1.2 Description of Appendix G, Parts I and II

Appendix G is presented in two parts. Part I has been specifically researched and written to be in concert with other portions of the AAPM. The approach taken in the development of nonstructural performance and cost data is at the same level of detail and sophistication as has been used elsewhere in the manual's development of the assessment methodology. For discussions and example presentation pertinent literature data are used in conjunction with assumed or site-specific constraints to arrive at reasonable estimates of costs and generalized effectiveness of stormwater management alternatives. When site-specific data are available in a planning area, they are to be used to provide a more accurate analysis for the decision matrix. Those site specific data which should be used include total annual precipitation, runoff coefficient, land-use patterns and population density, miles of street, miles of curb, sewer miles, number of combined sewer overflows, number and volume of combined sewer overflows per year, and the like. Although the performance and cost estimates developed in Part I are based on reliable literature sources, the estimates should not be mechanistically applied to specific planning areas without considering whether the expressed or implied assumptions are representative of existing conditions.

Part II of this appendix consists of EPA Report No. 600/2-77-083, "Storm Water Management Model: Level I-Comparative Evaluation of Storage-Treatment and other Management Practices." The report describes simplified procedures for obtaining a preliminary estimate of the magnitude of stormwater discharges and the associated costs of control. The approach is similar to that used in Part I in that it is designed as a "desktop" procedure to compare selected alternative control technologies. It has been included in its entirety as Part II in order to provide an alternate approach to the evaluation of stormwater management practices. Unlike Part I which discusses a great number of individually-treated control techniques, Part II discusses only street sweeping, sewer flushing, catch/basin cleaning, and storage-treatment. However, it does present a graphical method for combining processes in series or parallel, arriving at a least-cost optimal combination of storage-treatment and other management practices using marginal cost analysis.

Part I of this appendix considers (1) Sources and Gross Pollution Loads of Urban Runoff, (2) Land Management Control Alternatives, and (3) Collection Systems Control Alternatives. The gross pollution loading relationships given in Chapter 3 are compared with quantitative measurements of individual sources such as street pollution and the buildup of municipal wastewater solids in combined sewers. The determination of the pollutional load establishes the baseline from which the performance capability and effectiveness of specific management techniques are calculated. The cost of various control measures are then estimated based on available literature references.

As previously mentioned, the techniques discussed in Part I are classified into two categories. The first category is Section G.3, Land Management Control Alternatives. The measures discussed are street cleaning, erosion control, storage in residential ponds, porous pavement, chemical use control, and air pollution control. The second category is Section G.4, Collection Systems Control Alternatives. The measures discussed are storage in existing sewer system, reduction or elimination of excessive infiltration and inflow, periodic sewer flushing and scraping, and catch basin cleaning.

Two additional management techniques, considered to be structural or semi-structural control methods, are also discussed in the second category. The first is sewer separations for which the conclusion is drawn that due to cost considerations this technique is feasible only in a developing area. The second semistructural control technique entails the installation of swirl concentrators at combined sewer overflow points. The discussion in this appendix considers swirl devices as flow regulators in combined sewer systems which will maximize in-line storage while providing a BOD removal capability not found in other flow regulating devices. On the other hand, the discussion of swirl devices in Chapter 6, covered in 6.4.2.7, Storage/ Treatment Methodology, considers these overflow regulators (which <u>may</u> be designed for solids removal) antecedent to off-line stormwater storage/ treatment systems.

For each stormwater management practice discussed, the same general approach is followed. A brief review of previous studies is given to describe the range of potential applicability and to point out the agreement or disparity of the literature data and results. The potential applicability, performance

capability and costs are presented in example format. In those instances were assumptions are made; for example, population density, land use activities in relation to street miles or sewer miles, precipitation and runoff coefficients; they are for illustrative use in the example. (For individual planning areas, site specific data are to be used where available, or a reasonable estimate in lieu thereof.) The costs which are given, therefore, are reflective of the example assumptions made and must be viewed in that light. Site-specific data applied to the example problem might substantially change the load reduction potential and cost projections in any individual planning area.

It is important to note that basic assumptions made or data used in Parts I and II are not identical and therefore the costs developed through a Part I approach may not be exactly comparable to the costs resulting from a Part II approach. For example, the cost of street sweeping given in Part I is $10.40 per curb-mile including disposal of sweepings versus $7.00 per curb-mile as given in Part II. The latter unit cost does not include disposal of the sweepings, nor does the optimization analysis provide for its inclusion.

From the standpoint of the context in which Parts I and II have been prepared and their relationship to the Chapter 6 methodology, the following comment may be obvious to users of the Manual but it bears additional attention. The Part I discussions of management alternatives have been prepared to furnish performance and cost inputs to Chapter 6. It is then the function of Chapter 6 to use the stormwater management practices performance and cost alternatives data from Appendix G, Part I, and the structural systems performance and cost alternatives data from Appendix H, to arrive at an optimal mix of structural and nonstructural pollution reduction alternatives which satisfy the previously developed load-reduction imperatives.

Conversely, Part II is a self-contained nonstructural/semi-structural alternatives evaluation methodology which provides a least-cost solution based on a limited number of technical alternatives, and which does not optimize among structural and nonstructural techniques. Again, it is being included in this appendix to demonstrate that more than one evaluation approach is available.

G.1.3 Literature Sources and Data Reliability

The text which follows is designed to fill a need for specific and illustrative information on management alternatives to input a first-level decision matrix. The process performance and cost data are not intended to be a complete listing or compilation of all the available literature concerning the sources, loadings, and alternative management practices for the control of urban nonpoint source pollution. There are other assessment and state-of-the-art documents available for an overview, and many of these have been used as a reference source for the material presented in this appendix.

The first level source of information is the library of EPA reports generated by the grant and contract activities of the Storm and Combined Sewer Section, Municipal Environmental Research Laboratory, Cincinnati. As a result of these and other EPA Research and Development projects, much information has been generated in defining problems and in the application of management techniques to the abatement of urban stormwater pollution. From some reports, information has been extracted directly. However, other reports were useful for their bibliographies which referred to original studies. In this case the original source of data could be examined and analyzed more critically for specific inclusion herein.

For much of the work presented in this appendix, data from the original, recent, and not so recent studies were used. As input to the development of the sources and gross pollution loads of urban stormwater runoff, data from those studies were analyzed and, in the oft occurring instances of irreconcilable data due to differences in experimental design or unrealistic experimental conditions, those data most closely approximating real-world experience were used. As a result of the evaluation of background data, the AAPM user can utilize the analytical results and cost performance estimates with confidence in a first-level planning exercise.

At this point, it should be restated, as it was stated originally in Chapter 6, that from the universe of research, demonstration, and on-site studies which have been carried out for the assessment of nonstructural technology for control of urban stormwater runoff pollution, a welter of data for performance capabilities and costs is available. Since the

information presented in this appendix has been collected from available literature and because it will be used in a preliminary way for the evaluation of alternative pollution control options with respect to performance and costs, it has been necessary to establish a quality measurement to relate the data from all sources.

The concept of "relative reliability" represents a reasonable indication to the user of this information regarding the nature and extent of the experience/data base upon which the cost or performance information is based. Although the judgement of the relative reliability of the information is subjective, it establishes for the user a relative confidence level for his alternative evaluation, indicating in the appropriate area of the alternative spectrum where data are deficient or where data are sufficient for reasonable confidence in definitive decision making. Using the relative reliability of the data, the user is directed away from making a major decision among a number of potentially usable techniques based on insufficient or unreliable information.

The relative reliability concept is stressed in order to convey to the user community that the information presented here, upon which decisions are based, results from studies conducted with differing levels of sophistication. Although the best available information has been used, further research and experience in nonstructural management techniques will result in establishing a greater confidence in the precision and accuracy of currently available data and a greater confidence in the application of the data in a decision-making process.

G.2 Sources of Gross Urban Runoff Pollutant Loads

G.2.1 Sources

Water pollutants from urban areas resulting from rainfall originate from a number of sources. For example, separate storm sewer pollution is generally recognized as pollution washed off the urban area by rainfall. Combined sewers used for both stormwater and municipal sewage have an additional pollutional component due to solids accumulation and escaping wastewater. During dry weather periods municipal sewage solids settle out in the sewer and are flushed into the receiving stream by the next rainfall of sufficient

intensity to cause resuspension and overflows. Also, since a portion of the combined sewage overflows upstream of the treatment plant during wet weather, a portion of the municipal sewage generated by the population of the urban area during a storm event goes directly to the receiving stream.

Stormwater loads may be reduced by employing various combinations of structural end-of-pipe control techniques and urban runoff management practices. The objective of Part I of this appendix is to examine the potential effectiveness and costs of the more prominent management practices available. Structural alternatives; such as deliberate interception, storage and treatment; will not be considered except for purposes of comparison. These are analyzed in detail in Chapter 6 and in Part II of this appendix.

G.2.2 Determination of Source Loads

In order to assess the effectiveness and costs of each of the various urban stormwater management practices the relative magnitude of pollutant loads from combined sewer overflows and from separate stormwater runoff must be estimated. Each of these loads is affected differently by the application of the various management options. For example, street sweeping can reduce the amount of urban dust-and-dirt which will be washed off the urban area into the receiving stream during wet weather. Similarly, periodic combined sewer flushing can reduce the amount of settled solids which are washed into the receiving streem during rain storms.

A relationship between combined sewer and separate sewer BOD pollutant loadings which is often used is that of Heaney et al. (1). Using actual sewer effluent measurements from seven urban areas it was reported that for equivalent area and precipitation, combined sewer BOD loadings averaged 4.12 times higher than separate sewer loadings. An independent survey of available data conducted by Lager and Smith (2) found that average BOD concentrations in combined sewage are 3.83 times those found in separate sewers (115 mg/1 vs. 30 mg/1).

Rather than merely employing these average values (which are calculated independent of community size), it is necessary for the purpose of this appendix to attempt to confirm these ratios through a quantitative analysis of measurements of individual sources of urban stormwater pollutants, such

as street pollutants and the accumulation of municipal wastewater solids in sewers, and to compare these values with the gross loading relationships developed in Chapter 3.

G.2.2.1 Separate Stormwater Loads

Many attempts have been made to relate stormwater loads to land use. One general procedure for predicting runoff quality that was outlined in Chapter 3 is based upon research done by the University of Florida for the Environmental Protection Agency. This procedure, which relates runoff quality to land use and population density, estimates loads via the following relationships:

$$M_s = \alpha(i,j) \cdot R \cdot p_1 (PD_d) \cdot \gamma \quad \text{(G-1)}$$

$$M_c = \beta(i,j) \cdot R \cdot p_1 (PD_d) \cdot \gamma \quad \text{(G-2)}$$

where:

M_s = pound of pollutant (j) from land use (i) with separate and unsewered conveyance (lbs/acre-yr)

M_c = pound of pollutant (j) from land use (i) with combined sewer conveyance (lbs/acre-yr)

$\alpha(i,j)$ = constant for pollutant (j) and land use (i) with separate and unsewered conveyance (lbs/acre-in)

$\beta(i,j)$ = constant for pollutant (j) and land use (i) with combined sewer conveyance (lbs/acre-in)

R = annual precipitation (in/yr)

$p_1 (PD_d)$ = population function

PD_d = population density (persons/acre)

γ = street sweeping effectiveness factor

Applying these relationships to a hypothetical urban area possessing the average developed land use characteristics of 248 urbanized areas in the United States (58.4% residential, 8.6% commercial, 14.8% industrial, and 18.2% other developed) Heaney et al. developed the following expression for the average annual BOD load from separate storm sewers as a function of population density (1):

$$\bar{M}_s = 0.467 \cdot R \cdot (0.142 + 0.218PD^{0.54}) + 0.457R \quad \text{(G-3)}$$

where:

$\bar{M}_s$ = average annual BOD loading from separate storm sewers (lb BOD/acre-yr)

R = annual precipitation (in/yr)

PD = population density of urbanized drainage area (persons/acre)

It is important to emphasize that similar relationships may be developed for other pollutant parameters of interest, e.g., suspended solids or nutrients. Most evaluations of urban management practices, however, have been conducted with BOD removal as the primary concern. For example purposes the estimations and analyses in this appendix will also be developed on a BOD basis.

The average annual precipitation for the seven cities from which runoff data was used to develop Equation (G-3) was 36 inches/year. Therefore, substituting this value for R, Equation (G-3) becomes:

$$\bar{M}_s = 18.8 + 3.67\ PD^{0.54} \quad \text{(G-4)}$$

Population density, in turn, is known to increase with community size in a fairly regular way, as described by the following relationship derived by Smith (3):

$$PD = 0.3\ POP^{0.304} \quad \text{(G-5)}$$

where:

POP = community population

Equation (G-5) is then substituted into Equation (G-4) to obtain the following relationship for estimating BOD loading from separate storm sewers as a function of community population.

$$M = 18.8 + 1.9\ POP^{0.1642} \quad \text{(G-6)}$$

This relationship is shown in Figure G-1 (curve A). It can be seen that separate sewer BOD loading (lb BOD/acre-year) is relatively insensitive to community size, ranging from about 25-35 lb BOD/acre-year.

The validity of this loading relationship may be tested using published values of BOD accumulation upon urban surfaces. A portion of the pollutional

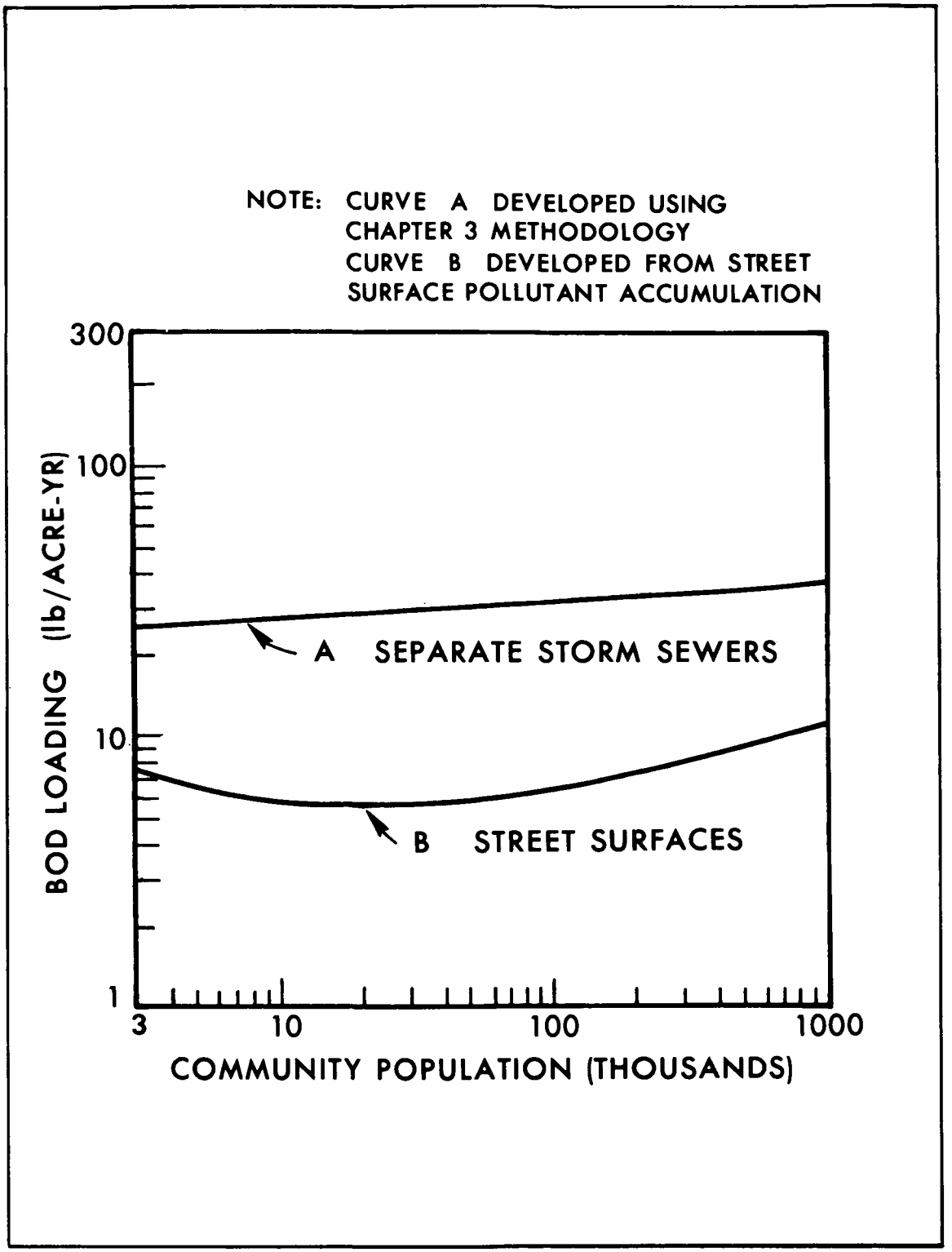


FIGURE G-1
ESTIMATED BOD LOADING IN URBAN RUNOFF
AS A FUNCTION OF COMMUNITY SIZE

load attributed to separate storm sewers is contributed by the dust-and-dirt which accumulates on the streets of the urban area. This load can then be reduced by means of street sweeping. During the summer of 1967, the American Public Works Association (4) conducted street sweeping measurements over a four-week period in the City of Chicago. The results of the study showed that the amount of litter which accumulates on the streets ranged from 26-422 lb/day/mile of curb. The dust-and-dirt fraction of the litter was defined as that fraction passing a screen with 1/8-inch openings. The amount of dust-and-dirt accumulated was measured as 21-275 lb/day/mile of curb. It was found that most of the BOD in street litter was contained in the dust-and-dirt fraction. The dust-and-dirt fraction was pulverized in a blender and dissolved in water. Without filtering, the BOD and COD of the water and dust-and-dirt mixture was found using standard techniques. The concentration of BOD in the dust-and-dirt fraction ranged from 3-7.7 mg/g. The amount of BOD accumulated varied with the type of land use and tended to be greater in high density residential areas. The weighted estimate for BOD accumulation on the streets of Chicago was 0.4 lb BOD/day/mile of curb or 0.8 lb BOD/day/mile of street. An interesting and, perhaps, significant result of the study was that the amount of BOD collected from the street was not related to the time since the last rainfall.

Using U.S. Department of Transportation data provided in the APWA report (4), miles of streets can be related to community size. This relationship is plotted as curve A in Figure G-2. Combining this with Equation (G-5) yields a relationship between community size and miles of streets/acre (curve B in Figure G-2), which when further multiplied by the APWA BOD accumulation rate for streets (0.8 lb BOD/day/mile x 365 days/year = 292 lb BOD/year/mile of street) yields curve B in Figure G-1. This provides a relationship for the potential annual BOD available from street surfaces as a function of community size.

Further examination of Figure G-1 reveals that the BOD loading values derived from the street sweeping experiments (curve B) are about one-fourth the loading when estimated from the BOD concentration found in separate storm sewers (curve A). A possible explanation for this factor difference is that the lower values of curve A are based upon dirt-and-dust available from street

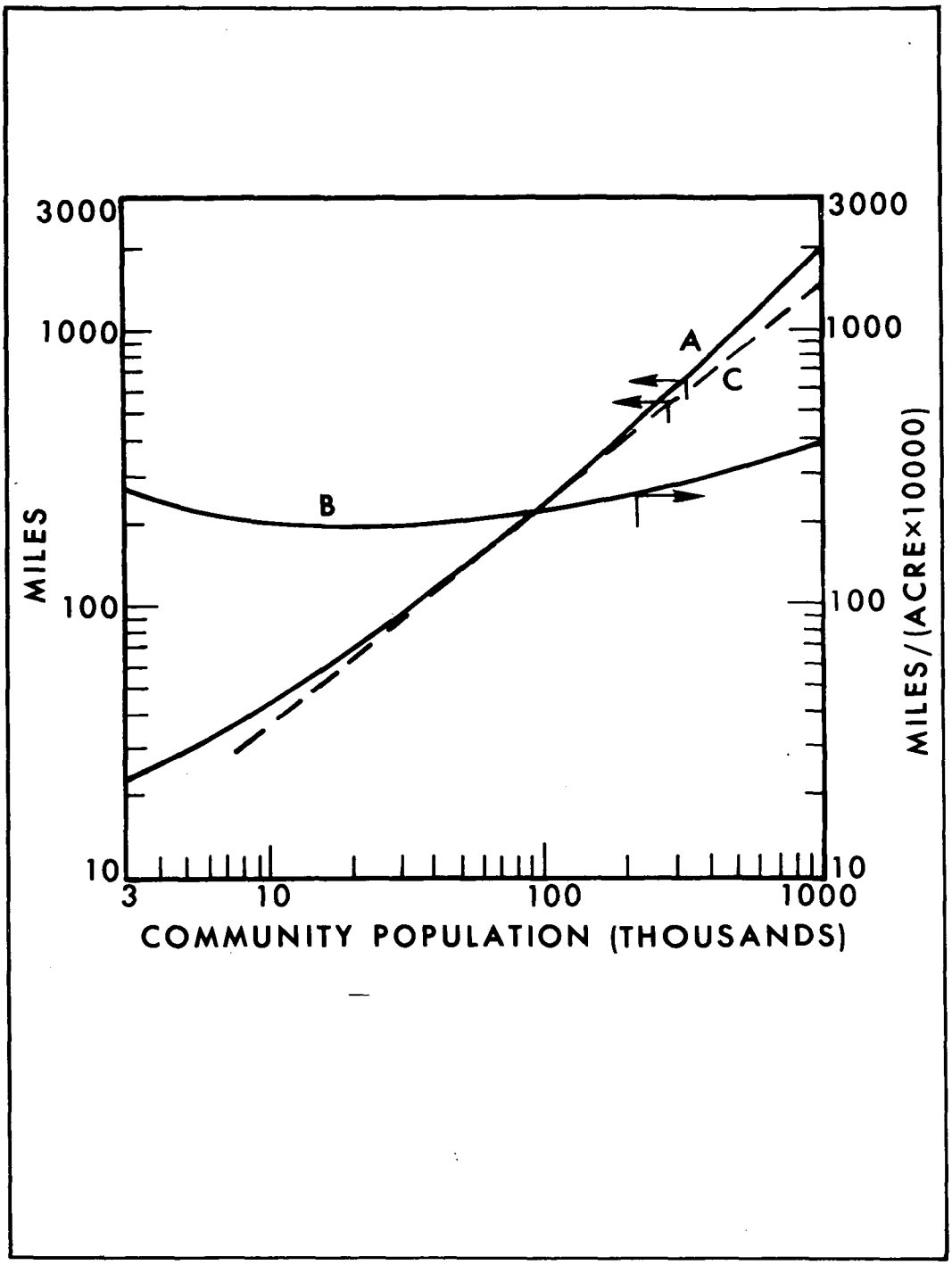


FIGURE G-2
RELATIONSHIP BETWEEN COMMUNITY SIZE AND (A) TOTAL MILES OF STREET; (B) STREET (SEWER) MILES PER 10,000 ACRES; AND (C) TOTAL MILES OF SEWER

surfaces only. These materials, however, accumulate at varying rates on all urban surfaces, including roof tops, parking lots, sidewalks, and pervious areas and are available to be washed from all these surfaces into separate storm sewers during rainfall events. This fact is reinforced by recalling that one of the conclusions of the Chicago study was that the amount of dust-and-dirt swept from the streets seemed to be independent of the time since the previous rainfall, probably due to the fact that rainfall tended to wash dust-and-dirt from the non-street areas onto the street.

As a rough approximation it might be assumed that the dust-and-dirt cover is, on the average, the same for all urbanized surfaces. In a careful study of urban land use for 53 central cities and 33 satellite cities Bartholomew (5) found that roughly 56% of the land was urbanized and that streets occupied an average of 28.1% of the urbanized area. Thus, it can be seen that the ratio of street BOD loading as recorded by APWA (Figure G-1, curve A) to the gross loading from the total urbanized area as estimated by Chapter 3 procedures (Figure G-1, curve B) is roughly the same as the ratio of street area to total urbanized area measured by Bartholomew, i.e., 25% vs. 28.1%. This tends to lend credence to both sets of estimates. More importantly, it leads to the further conclusion that the amount of BOD (in the form of dust-and-dirt) which can be removed from the streets (using current street sweeping practices) is not likely to exceed the amount actually on the street surfaces which accounts for only about 25% of the total loading.

G.2.2.2 Combined Sewer Loads

The experiments on buildup of sanitary sewage solids in a pilot sewer conducted by FMC Corporation (6) can be used in an attempt to account for the additional loading observed from combined sewers. In this work, FMC allowed raw sanitary sewage to flow through 12-inch diameter and 18-inch diameter pilot sewers both 790 feet long. The amount of solids accumulation was measures in terms of TOC (total organic carbon) as a function of time and distance along the sewer. As might be expected, the measurements were scattered but by averaging the accumulation per length of sewer per unit of time over the first 267 feet and over the entire length for the times used (1.75, 3.92, and 7.83 days) the average rate of accumulation was 2.09 lb TOC/mile/day. It was also found that the BOD of the accumulated solids

equaled about 1.8 times the TOC measurement. The suspended solids concentration of the raw sewage used by FMC was 133 mg/l which is low compared to a more normal value of 200 mg/l. If the measured accumulation is multiplied by the ratio 200/133 the value for normal sanitary sewage is 5.66 lb BOD/mile/day or 2067 lb BOD/mile/yr.

Thus, if the length of sewer can be stimated, the loading due to accumulation of sanitary sewage solids can also be estimated. One relationship developed by Smith (3) is shown below:

$$\text{Feet of installed sewer/capita} = 54\ (PD)^{-0.65} \qquad \text{(G-7)}$$

Combining this expression with Equation (G-5) the following relationship for miles of sewer versus community size results:

$$\text{Miles of sewer} = 0.02237\ POP^{0.8024} \qquad \text{(G-8)}$$

This relationship is shown plotted by the dashed line in Figure G-2 (curve C). It can be seen that the miles of sewer is very similar to the miles of street. The relationship for miles of street is probably more reliable as an estimate of length of sewer. If one assumes that the miles of sewer is equal to the miles of street, the expression for miles of sewer per acre as a function of community size becomes the same as curve B in Figure G-2. Multiplying miles of sewer/acre times 2067 lb BOD/mile/yr. gives the loading in terms of lb BOD/acre-yr. caused by flushing of settled sanitary sewage solids from combined sewers during the wet weather. This relationship is plotted as a function of community size in Figure G-3 (curve A).

Some raw sanitary sewage will also escape into the receiving stream during overflow periods and contribute to the difference between separate storm sewer and combined sewer loadings. Assuming that the production of sanitary sewage can be estimated as 0.17 lb BOD per capita per day, the equivalent loading is 62.05 (365 x 0.17) times the population density in persons/acre. Combining this relationship with Equation (G-5) gives the following equation for loading due to raw sewage escape:

$$\text{lb BOD/}_{\text{acre-yr.}} = 18.62\ POP^{0.304} \qquad \text{(G-9)}$$

A survey of combined sewer problems conducted by APWA (7) in 1967 showed that because of wet weather, treatment plants (197 plants surveyed) bypass an

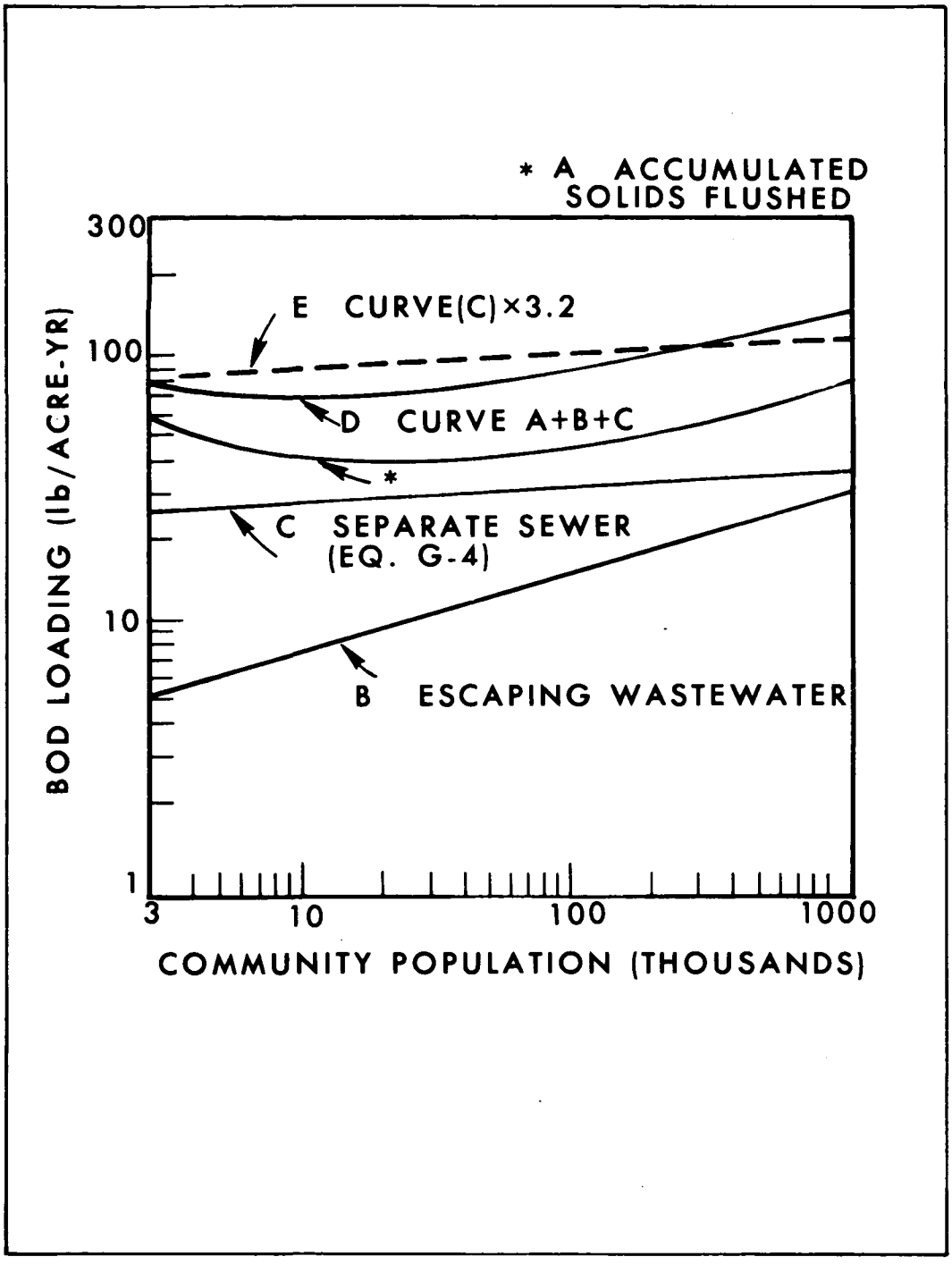


FIGURE G-3
ESTIMATED BOD LOADINGS AS A FUNCTION OF COMMUNITY SIZE FOR: (A) ACCUMULATED SOLIDS FLUSHED FROM SANITARY SEWERS; (B) ESCAPING WASTEWATER; AND (C) SEPARATE SEWERS.

average of 350 hr/yr or about 4% of the time. The fraction of raw municipal sewage which escapes during bypassing can be shown (8) to depend on the ratio of treatment plant capacity (MGD) to actual dry weather flow (MGD). The fraction escaping ranges from 48% when the ratio is ten, to 67% when the ratio is one. If a nominal value of 60% corresponding to a ratio of three is used, the loading caused by bypassed municipal sewage during wet weather can be estimated as 2.4% of that given by Equation (G-9), which is plotted as a function of community size in Figure G-3 (curve B).

Adding these two sources of BOD gives an incremental loading difference between storm and combined sewer discharges of 57 to 180 lb BOD/acre-yr. If this increment is now added to that estimated for pollution for from separate storm sewers using Equations (G-4), the overall estimate for combined sewer loading may be found. This is accomplished graphically in Figure G-3 by adding curves A, B, and C to derive curve D, the combined sewer loading relationship.

The seven measurements used for deriving the factor 4.12 are shown in Figure G-4. The dashed line (curve C) represents a factor of 4.12 times curve A. It can be seen that all but one of the measurements fall below the line. If the ratio of the loading for each point is divided by the equivalent estimate for separate storm sewers from curve A the average ratio becomes 3.2 instead of 4.12. The ratio of the loading for each point is divided by the equivalent estimate for separate storm sewers from curve A the average ratio becomes 3.2 instead of 4.12. The ratio of 3.2 is shown as curve B in Figure G-4. Using a factor of 3.2 instead of 4.12, the estimated loading relationship for combined sewers has been calculated and is shown as curve E in Figure G-3. Comparing curves D and E, it is seen that the two estimates agree well if the factor of 3.2 between loading of separate storm sewers and combined sewers is used. Table G-1 summarizes the calculations employed in defining the curve relationships in Figures G-1, G-2, and G-3.

From the analysis above, the relative loading functions for separate and combined sewer discharges have been derived from reported measurements of pollutant accumulation on streets and in sewers. Having arrived at this point it is now possible to discuss the relative efficiency and potential

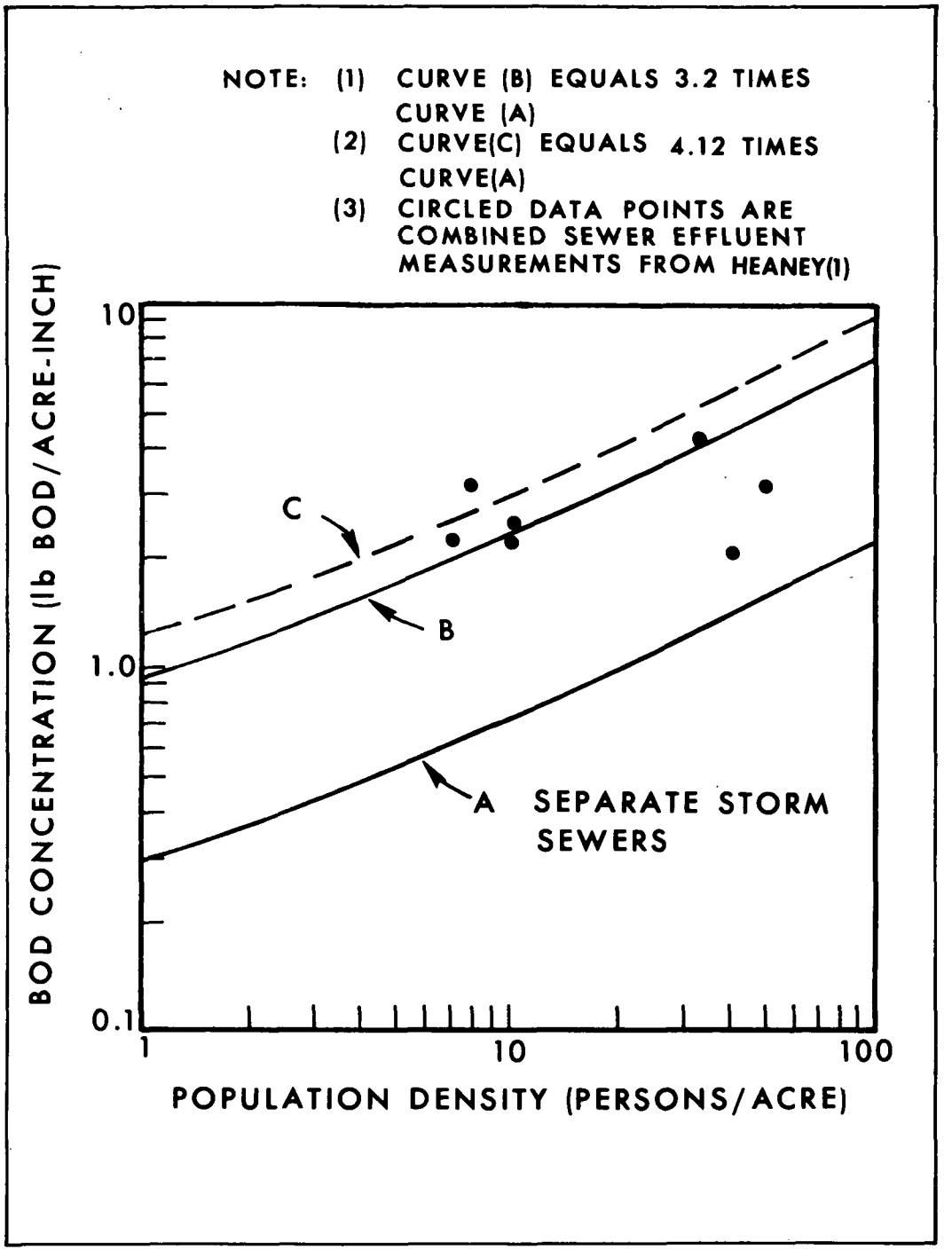


FIGURE G-4
BOD CONCENTRATION FOR SEPARATE STORM SEWERS AS A FUNCTION OF POPULATION DENSITY

TABLE G-1

URBAN AREA BOD LOADS (lb BOD/ACRE-YR) COMPUTED FROM STREET SWEEPING AND COMBINED SEWER SOLIDS BUILDUP MEASUREMENTS AS COMPARED TO ESTIMATES DETERMINED USING THE CHAPTER 3 METHODOLOGY

Community Population	Miles of Street/Acre, Figure G-2, Curve(B)	BOD Loading(lb BOD/Acre-yr)					
		Storm Sewers		Solids Flushed, Figure G-3 Curve (A)	Escaping Wastewater, Figure G-3 Curve (B)	Combined Sewers	
		Figure G-1, Curve(B)(APWA)	Figure G-1, Curve(A)(Ch.3)			Figure G-3, Curve (D)	Eq. G-4 X 3.2
3,000	0.0252	26.2	25.9	52.1	5.1	83.1	82.9
10,000	0.0195	20.3	27.4	40.3	7.3	75.0	87.7
30,000	0.0198	20.6	29.1	40.9	10.3	80.3	93.1
100,000	0.023	23.9	31.4	47.5	14.8	93.7	100.5
300,000	0.0285	29.6	33.9	58.9	20.6	113.4	108.5
1,000,000	0.038	39.5	37.2	78.5	29.8	145.5	119.0

costs of management techniques for controlling these pollutants.

G.3 Land Management Control Alternatives

G.3.1 Street Cleaning

The primary objective of municipal street cleaning practices is to enhance the aesthetic appearance of streets by periodically removing accumulated litter, debris, dust, and dirt. Such practices, however, do have an impact upon the amount and type of roadway pollutants which may be washed into receiving waters during runoff events. Consequently street cleaning is widely considered as a candidate management practice for reducing stormwater pollution loadings.

Streets are conventionally cleaned by manual or mechanically sweeping or by periodic flushing with water. The cleanliness of streets is also affected by the type and condition of pavement, the effectiveness of anti-litter programs, and the degree of air pollution (particulate) control in the urban area. The magnitude of the effect of these latter factors upon stormwater is difficult to assess except in general terms, due to the paucity of quantitative information. Litter control, primarily affecting large street debris such as cans, paper, bottles, is programmed entirely for aesthetic purposes and has little effect upon important water pollutants such as BOD, pesticides, heavy metals and small particulates. Streets in good condition are generally cleaner than those in poor condition; and concrete pavements tend to be cleaner than asphalt.

The purpose of this section will be to develop cost and effectiveness information for street cleaning techniques only. Because of the lack of good information no attempt will be made to develop cost/effectiveness relationships for litter control programs or pavement condition. Section G.3.6, however, provides an estimate of the impact of particulate air pollution control upon stormwater loads.

G.3.1.1. Street Sweeping

The most common methods of street sweeping include:

(1) mechanical (broom-type) sweepers

(2) manual

(3) vacuum sweepers

(4) combined vacuum and mechanical broom-type sweepers

Mechanical sweepers are most commonly used and, understandably, the majority of the currently available performance literature addresses this type of sweeper. Mechanical sweepers basically consist of a gutter and main broom which rotate at high speeds forcing the debris from the gutter and street surface onto a conveyor belt and subsequently into a hopper. Water is usually sprayed on the surface for dust control.

The effectiveness of mechanical sweepers is recognized to be a function of a number of factors, including: (1) particle size distribution of accumulated surface contaminants, (2) sweeping frequency, (3) number of passes, (4) equipment speed, and (5) pavement conditions.

Table G-2 displays the distribution of street surface pollutant fractions associated with various particle sizes (9). The interrelationship between mechanical sweeper efficiency and particle size is shown in Table G-3. The overall efficiency can be seen to be roughly 50 percent.

The effectiveness of street sweeping is also affected by the frequency of sweeping and the number of sweeper passes during each sweeping interval. As the sweeping frequency and number of passes increase the effectiveness of contaminant removal increases. The effect of sweeping frequency is described more fully in Part 2 of this appendix. For the purposes of the analysis of cost/effectiveness of mechanical sweepers in this section, however, the following relationship, developed by Sartor & Boyd (9), will be used:

$$\text{Efficiency} = 100\ (1 - M^*/M_o)\ (1 - e^{-0.7472P}) \qquad \text{(G-10)}$$

where M^* = ultimate residual dust-and-dirt, gm/sq.ft.

M_o = initial dust-and-dirt loading, gm/sq.ft.

P = number of passes made by sweeper

This relationship was determined from actual mechanical sweeper efficiency tests on naturally accumulated dust-and-dirt as well as a simulation developed to represent natural dust-and-dirt. Efficiency tests performed by equipment manufacturers and other research organizations were also cited in the studies.

TABLE G-2

STREET SURFACE POLLUTANTS ASSOCIATED WITH VARIOUS PARTICLE SIZES (9)

Measured Pollutant	Particle Size <43μ	43μ-246μ (% by weight)	>246μ
TS	5.9	37.5	56.5
BOD_5	24.3	32.5	43.2
COD	22.7	57.4	19.9
VS	25.6	34.0	40.4
Phosphates	56.2	36.0	7.8
Nitrates	31.9	45.1	23.0
Kjeldahl nitrogen	18.7	39.8	41.5
All heavy metals	51.2		48.7
All pesticides	73		27
PCB	34		66

TABLE G-3

INTERRELATIONSHIP BETWEEN MECHANICAL SWEEPER EFFICIENCY AND PARTICLE SIZE (9)

Particle Size (microns)	Sweeper Efficiency (% removal)
2,000	79
840-2,000	66
246-840	60
104-246	48
43-104	20
<43	15
Overall	50

It can be seen by Equation (G-10) that if the ratio of the ultimate residual divided by the initial loading is 0.05 the sweeping efficiency for one pass is 50%. Similarly, the efficiency of two-pass sweeping is 74% and the efficiency of three-pass sweeping is 85%.

The analysis of storm and combined sewer pollutant sources presented earlier in this appendix showed that about 25% of the separate storm sewer BOD load could be attributed to dust-and-dirt which can be swept from streets. This conclusion was based on the APWA (4) street sweeping experiments made in Chicago, which gave a weighted average street accumulation rate of 0.4 lb BOD/curb-mile/day. It should also be noted that the URS study by Sartor and Boyd (9) which resulted in Equation (G-10) also performed similar accumulation studies which produced significantly different values. The URS study measured street pollution in each of twelve cities for more than five land-use categories per city. The pollutant cover on a 40 ft. by 25 ft. section of street was measured by hand sweeping and flushing. The amount of BOD removed from these small sections averaged 14 lb BOD/curb-mile. Inquiries were made to determine the number of days since the street section was last swept. Based on this information, it was calculated that the street pollutant load averaged 4.5 lb BOD/curb-mile/day. This estimate is roughly ten times the APWA estimate of 0.4 lb BOD/curb-mile/day. Thus, the results of the two measurement programs are incompatible and cannot be averaged.

Because some logical value for loading is needed, the following evaluation must be made. The length of street swept by APWA was about 38 times the length swept by URS. Because of the greater amount of street sampled and the fact that URS did not sweep streets repeatedly but relied on local information on the time between regular sweepings it would appear that the APWA results form a more solid basis for estimating street pollutant loads.

Therefore, using the separate storm sewer loading relationship developed in Section G.2.2, and assuming that 25% of this load is on street surface and is available for removal, the maximum amount of BOD available for removal by street sweepers is 6-9 lb BOD/acre-year (25-37 lb BOD/acre-year times 0.25). If single-pass sweeping with an efficiency of 50% is used the load removal rate through sweeping will be 3-5 lb BOD/acre-year, or 12-14% of the separate

storm sewer load.

The cost of street sweeping has been reported by many sources and, according to Morris et al.(11), can vary from $2 to $14/curb-mile. Sartor and Boyd (9) cite a survey made by American City magazine where it was found that street sweeping costs varied from $2.18 to $8.42/curb-mile in 1970-71 dollars. The APWA report of street sweeping experiments in Chicago estimated the cost of single-pass mechanical street sweeping at about $10/curb-mile in 1968 dollars. The average cost of street sweeping in Cincinnati (1975) (12) was $8.40/curb-mile for mechanical sweeping and $45.68/curb-mile for hand sweeping. The cost of disposal of the sweepings was $3.36/cu.yd. The volume of sweepings collected was 0.6 cu.yd./curb-mile in Cincinnati which compares to a range of 0.18-0.56 cu.yd./curb-mile for the four cities surveyed by Sartor and Boyd (9). In Chicago the average volume of sweepings was 1.5 cu.yd./curb-mile. Since the amount collected in Cincinnati is about the average amount, the cost of disposal can be estimated at about $2/curb-mile (0.6 cu.yd./curb-mile x $3.36/cu.yd.) which, when added to the $8.40/curb-mile for sweeping, gives a total of $10.40/curb-mile.

The survey made by Sartor and Boyd (9) showed that the frequency of street sweeping computed by dividing the miles of street swept per year by the miles of streets scheduled for sweeping ranged from a low of 36 times/year in Baltimore to a high of 121 times/year in Phoenix. The average was 70 times/year, or every 5.2 days. Since the miles of street per acre range from a low of 0.019 to a high of 0.038, miles of street/acre can be multiplied by $/mile of street (2 x $10.40/curb-mile) times the average number of sweepings per year to find the cost in terms of dollars per acre per year. The cost of street sweeping is $28-$55/acre-year and this divided by the average loading removed (3-5 lb BOD/acre-year) gives the cost in dollars per pound of BOD removed as $7-$14/lb of BOD removed, for a single pass.

Vacuum street-sweepers may also be used. Three types of vacuum sweepers are available on the market today. The first is the so-called conventional vacuum sweeper which operates by sweeping the material from the path into approximately a one-foot windrow. This windrow, and only this windrow, is picked up by the vacuum. Dust problems are less evident than those created by mechanical sweepers; however, removal of fine particles is still questionable.

The second type features vacuum action over the entire path, assisted by a gutter broom. Still, the particulates which are embedded in the road surface do not receive sufficient agitation to become air-suspended and, therefore, the vacuum is unable to draw them into the hopper.

A relatively new concept is the regenerative air system. Sweepers of this type force air down onto the pavement, suspending the particles, which are then picked up by the vacuum suction. These sweepers may be the most efficient in removing fines.

The potential for overall debris control and, particularly, removal of dust-and-dirt by vacuum sweepers has been rather superficially reported in information currently available. An on-going EPA research investigation in San Jose, California is conducting a field examination of a variety of sweeper types, including vacuum sweepers (13).

Based on data from a pilot sweeping study, Field et al. noted that it could be feasible to improve removal efficiencies for various street surface contaminants through the use of sweepers employing a combination of broom and vacuum action (13). Under favorable conditions such equipment might be capable of removing up to 90% of deposited solids and perhaps as much as 67% of street surface BOD due to a greater efficiency in the removal of smaller particle sizes.

Costs of vacuum sweeping are reported to range widely from $2-$14/curb-mile. Regenerative air sweepers may cost from $3 - $10/curb-mile.

G.3.1.2 Street Flushing

Street flushing is a practice which is employed in many cities as an alternative to or in addition to street sweeping. Street flushers consist of an 800 to 3500 gallon water supply tank mounted on a truck or trailer which is equipped to allow water to be sprayed through three or more directional nozzles onto the street surface. As currently practiced, street flushing does not pick up contaminants but primarily serves to transport them from the center of the street surface to the gutter. The volume of water used is generally insufficient to transport the accumulated material to the nearest drain.

The relative surface cleaning efficiency of street flushing was compared to that of mechanical brush sweeping in the previously mentioned URS study (9). A mobile flushing unit with special high pressure, highly-effective, spray nozzles was used. The results of the tests which were conducted for similar mass loading, particle size, and surface type are shown in Figure G-5.

A comparison of the curves indicates that a street flusher of the type used in that study was a superior method for moving street contaminants, especially those in the dust-and-dirt fraction. Conventional street flushers, however, are not likely to be as effective as that used in the study.

Overall, unless greater than conventional flush volumes are used, street flushing will not on its own be an effective contaminant control practice. Even if street contaminants can be effectively washed into sewers, the practice will have impact only in areas which are served by combined sewers and also employ a sewer flushing program. The mere relocation of pollutants from street surfaces into sewers does not insure that they will not be subsequently washed into receiving waters during the next runoff event either in combined sewer overflows or direct discharge from separate storm sewer.

G.3.2. Sediment & Erosion Control

Soil erosion in urban areas is associated with construction of buildings, dams, sewerage facilities, roads, and new residential areas. Many adverse effects are caused by uncontrolled soil erosion. For example, sediment washed off denuded land can fill the pores of downstream land and destroy grass and other forms of vegetation thus reducing the infiltration capacity of the land and increasing the rate of runoff. Sediment will also clog storm sewers causing downstream flooding which can result in property damage. High sediment loads in natural streams make the water less fit for most human needs and increase the cost of potable water treatment. High sediment loads also can have a deleterious effect on aquatic life. The high runoff velocities resulting from the effects of erosion can erode stream banks causing further damage to the streams.

Although the acreage under construction is generally small compared to other forms of erodable land, the rate of erosion can be severe. For example, Ports (14) estimates that the rate of erosion from construction activity can

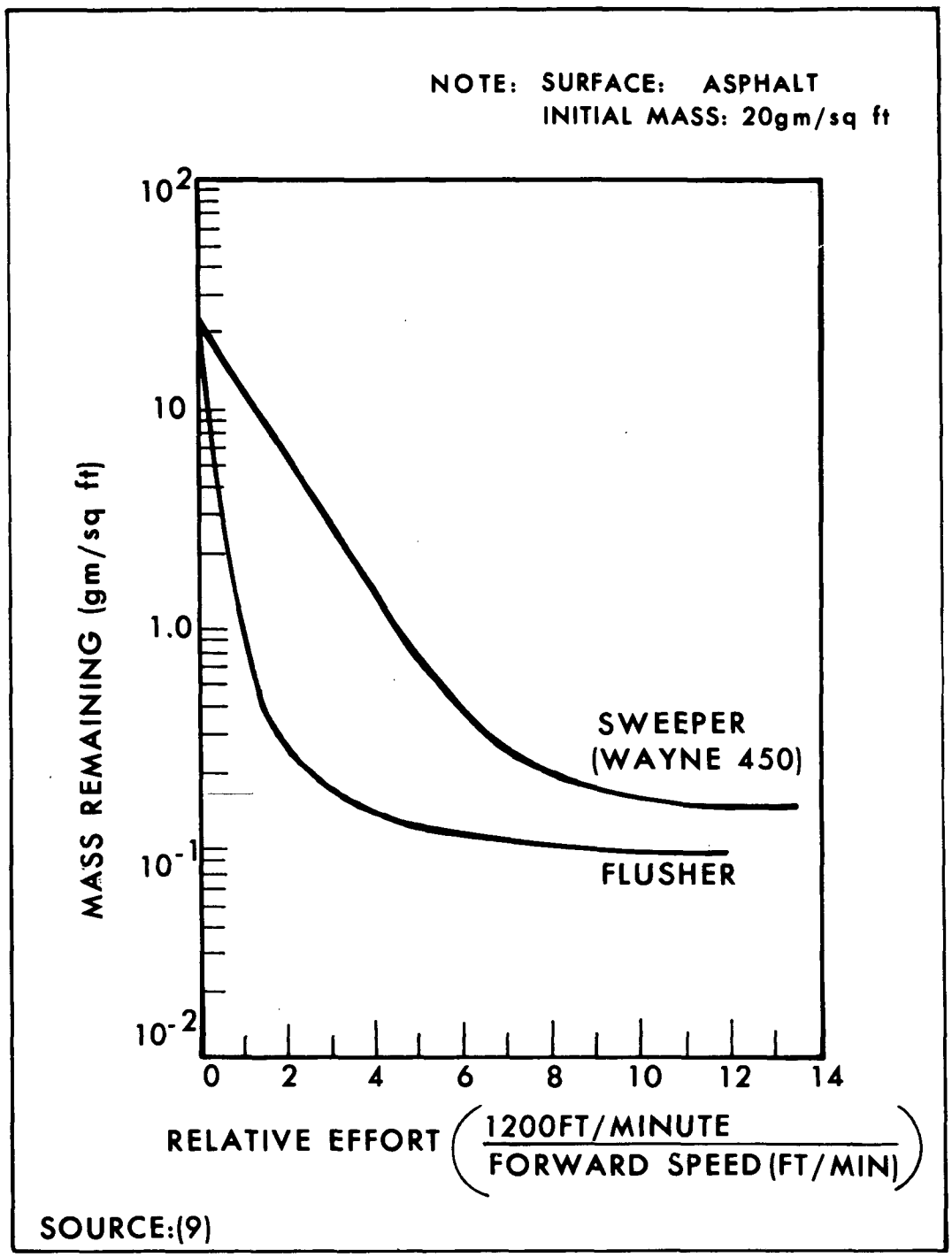


FIGURE G-5
COMPARISON OF CLEANING PERFORMANCES OF MOTORIZED STREET SWEEPING AND MOTORIZED STREET FLUSHING

be as high as 200 tons/acre-year as compared to about 0.2 tons/acre-year from forestland. Erosion from croplands is generally recognized as the single greatest source of sediment in natural streams. Ports estimates the rate of erosion from croplands to be in the range of 0.3 - 6 tons/acre-year.

In a study of nonpoint source pollution made by Midwest Research Institute in cooperation with Hittman Associates (15) the average erosion rate for construction was estimated as 75 tons/acre-year as compared to 0.4 tons per acre-year for forest and 7.5 tons/acre-year for cropland. This report also estimated the amount of land under construction at about 1.5 million acres as compared to 412 million acres of cropland. The U.S. Department of Agriculture (16) estimates that the total land area of the 50 states is about 2.3 billion acres and of this 1.1 billion acres is in farms and 333 million acres are in active croplands. Thus, if the erosion from croplands is assumed to be 6 tons per acre-year, the total sediment runoff for the U.S. is about two billion tons per year. Similarly, assuming the erosion from construction activities as 200 tons/acre-year, the total runoff for the U.S. is about 300 million tons per year. Thus, uncontrolled erosion from small land areas in urban construction activities might account for as much as 15% of the sediment runoff attributed to active croplands.

Other major sources of sediment are harvested forests and grassland. It has been estimated that both of these sources contribute about twice the amount estimated for construction activities (15). This report also estimated the sediment from urban construction activities at about three percent of the total runoff.

As much as 95 percent of the uncontrolled erosion which can occur at construction sites may be prevented using various combinations of relatively simple and inexpensive control techniques. These techniques were described in an Environmental Protection Agency report prepared by Hittman Associates (17), and range from the construction of sandbag and straw bale sediment barriers to use of diversion dikes, filter berms, flexible erosion control mats, gabions and chutes, flumes, and check dams. Table G-4 summarizes the calculated effectiveness of 10 erosion control systems (18).

TABLE G-4

PROMISING CONTROL SYSTEM AND EFFECTIVENESS

(After Reference 18)

System Numbers	Components	Percent Effectiveness
1	Seed, fertilizer, straw mulch. Erosion structures (normal). Sediment basins (0.04 ratio, and 70 percent of area)	91
2	Same as (1) except chemical (12 months protection) replaces straw	90
3	Same as (1) except chemical straw tack replaces asphalt	91
4	Seed, fertilizer, straw mulch. Diversion berms. Sediment basins (0.04 ratio and 100 percent area)	90
5	Seed, fertilizer, straw mulch. Downstream sediment basin (0.06 ratio)	93
6	Seed, fertilizer, chemical (12 months protection). Downstream sediment basin (0.06 ratio).	92
7	Seed, fertilizer, straw mulch. Downstream sediment basin using flocculants.	96
8	Same as (7) without straw mulch.	94
9	Chemical (12 months protection) sediment basin using flocculants.	94
10	Same as (9) with seed, fertilizer	96

Detailed costs for 25 erosion control methods were estimated in a report prepared for the Environmental Protection Agency by Engineering Science, Incorporated (18). Table G-5 summarizes the unit cost estimates developed, along with major assumptions used in preparing the estimates. The report also provides a method for estimating the overall cost of sediment control at specific sites as well as the damage costs which would result if no control were exercised.

It is difficult to generalize what a typical construction site erosion control system is and what its cost (e.g., $/acre) would be. The number, type, and cost of control measures are highly variable and very site and geographically specific because: (1) the erosion potential of sites vary widely with climate, slope, soil type, existing ground cover and other site conditions; (2) labor rates and material costs vary from region to region; (3) costs and feasible techniques are a function of project size, site terrain, and the type of construction activity (residential building, pipelines, highways, etc.); (4) it is sometimes difficult to distribute costs and benefits between sediment control objectives and aesthetic benefits (e.g., sodding and seeding practices, on-site ponds as sediment traps); and (5) techniques applied and their costs vary with the intended period of effectiveness, project life, and maintenance cost requirements.

Some attempts to evaluate average sediment control requirements for construction sites have been made. In 1972, for instance, Dow Chemical Co. performed an economic analysis for sediment control methods for watersheds undergoing urbanization (19). It utilized the Seneca watershed near Washington, D.C. as a model. The control practices included sedimentation basins, diversion berms, level spreaders, grade stabilization, sodded ditches, seeding and straw mulch. The total watershed system cost $1,125 per acre and controlled 91 percent of the erosion. It claimed that control systems with large sedimentation basins could boost control to 96 percent at a lesser total cost. Engineering Science (18) also estimated the cost of a "typical" erosion control program for a 2.95 acre commercial construction project. The control system included straw bale sewer inlet protectors, an earthen diversion dike, a straw bale diversion with a gravel weir, and a sediment retention basin. Using the unit cost values in Table G-5 the cost

TABLE G-5

COSTS FOR ON-SITE EROSION CONTROL MEASURES*

METHOD	UNIT COSTS California	Virginia	REMARKS
A) CONTROL STRUCTURES			
Gravel and Earth Check Dam	$1.84-0.83/cf	-	19 cf - 225 cf structures, resp.
Rock Riprap Check Dam	$7.00-8.17/cf	$5.99-$6.96/cf	56 cf - 300 cf structures, resp.
Concrete Check Dam	$22.15-8.04/cf	$20.03-$7.22/cf	51 cf - 891 cf structures, resp.
Concrete Chute	$32.45/ft 5.40/ft^2	$28.35/ft 4.72/ft^2	40 ft chute, 3 ft wide sides, 3 in thick
Diversion Dike (Interceptor Dikes)	$4.51/ft 0.48/cf	$3.70/ft 0.39/cf	405 cf dike,43 ft long
Erosion Check	$3.43/ft	$2.65/ft	150 ft jute mesh
Filter Berm (Filter Inlets)	$5.50/ft 0.39/cf	$5.11/ft 0.37/cf	810 cf gravel berm
Flexible Downdrain	$7.34/ft	$7.26/ft	300 ft unit with end connections
Flexible Erosion Control Mats	$1.18/ft^2	$1.11/ft^2	based on 33,000 ft^2 channel lining install.
Gabions	$3.34/ft^2 1.72/ft^2 1.41/ft^2	$2.76/ft^2 1.54/ft^2 1.26/ft^2	10 ft^{2}x1 ft install. 100 ft^{2}x1 ft " 1000 ft^{2}x 1 ft "
Level Spreader	$3.80/ft 1.91/ft 1.63/ft	$3.16/ft 1.57/ft 1.36/ft	15 ft long 44 ft long 78 ft long
Sandbag Barriers	$3.10/bag 12.40/ft	$2.44/bag 9.76/ft	filling & placing 180 bags/day~45'x4' barrier
Sectional Downdrain	$14.55/ft 10.91/ft	$11.85/ft 9.13/ft	44 ft unit 234 ft unit
Sediment Retention Basin	$0.51-0.39/cf	$0.42-0.33/cf	earth structures 30 to 40 ft long and 6-8ft h
Straw Bale Inlet Protection	$55/inlet	$46/inlet	
Straw Bale Barriers	$3.93/ft	$3.31/ft	2 ft bales x one high
B) GROUND COVERS			
Excelsior Mat	$12,000/ac	$10,200/ac	includes seed & fertilizer
Jute Mesh on One-Acre Plot	$7,700/ac	$6,700/ac	
Straw or Hay (blower applied)	$1,200/ac	$1,100/ac	
Woodchips	$8,000/ac	$7,200/ac	3" cover
4" Square Plugs of Sod	$11,300/ac	$10,300/ac	includes seed & fertilizer
Sodding	$14,800/ac	$14,300/ac	
Chemical Soil Stabilizer	$1,300/ac	$1,250/ac	
Hydromulch	$858-434/ac	$738-373/ac	1 acre - 30 acre job, resp.

*All costs in 1972 dollars

was estimated at $1,340/acre (California) and $1,110/acre (Virginia).

Engineering-Science also concluded that the technique which appears to be the most cost-effective is hydromulching. Hydromulching is a technique where wood chips are mixed with grass seed, fertilizer and water to form a slurry which is then sprayed on the denuded land. The wood chips tend to hold the soil and encourage the growth of the grass which then acts as an effective erosion deterrent. The cost of hydromulching in 1973 dollars ranges from about $850/acre when only one acre is treated to a low of about $344/acre when the land area is 30 acres or greater. For the example commercial construction site described above, the use of hydromulching would add $720/acre (53.7%) and $680/acre (6.13%) to the erosion control program costs in California and Virginia, respectively.

Cost estimates have also been made in various studies for determining the economic, aesthetic, and environmental damages which may result from not controlling erosion. The Dow Chemical Study estimated that miscellaneous damages of erosion from construction sites could reach $1,500 per acre (19). These damages included: a need for turbidity control in water treatment; dredging costs downstream; flooding; adverse effects to commercial and recreational fishing; boating and aesthetics. Engineering Science also made cost estimates of removing silt from homes, streets, stream channels, and in domestic water treatment (18). Estimated costs for sediment removal are shown in Table G-6. The study concluded that the cost of controlling erosion at construction sites is in most cases significantly less than the cost of correcting the damage caused by uncontrolled erosion.

G.3.3 Site Runoff Control

On-site or upstream runoff control refers to storage for detention (short term) or retention (long term) of runoff prior to its entry into a drainage system. In urban areas, roofs of structures represent a large part of the impervious surfaces which increase runoff. In many communities with combined sewers, the drainage from such roof areas is discharged into the sewer system via the single property sewer connector. The pollution potential of roof runoff consists of particulate air pollution dustfall, tree leaves, and bird droppings. Other impervious surfaces such as paved parking lots,

TABLE G-6

COSTS FOR SEDIMENT REMOVAL MEASURES

(Adapted from Reference 18)

Removal Measure	Unit Costs	
	California	Virginia
Sediment Removal from Streets	$8/cy	$6.60/cy
Sediment Removal from Basements	$77/cy	$65/cy
Sediment Removal from Storm Sewers Bucket Line Cleaning	$144/cy	$122/cy
Sediment Removed from Storm Sewers Rodding & Hydraulic Flushing of Storm Sewer	$68/cy	$62/cy

plazas, etc., having their own classes of pollutants, generally drain into the municipal sewerage system, causing sudden increases in pollutional load and wastewater volume during a storm event. In areas of lesser population densities, although the absolute and relative magnitudes of impervious surfaces decrease, similar effects of a storm event exist but to a lesser degree. In the latter situation in addition to runoff from impervious areas, surface runoff from pervious areas also contributes a pollutional load, generally erosion solids; and results in a sudden incremental volume to either the sewerage system or to the natural drainage network of the area.

Detention - In its simplest form, detention means capturing stormwater and controlling its release rate in order to decrease downstream peak flow rates. Oftentimes, on-site storage does or can be designed to provide for the dual or multi-benefits of aesthetics, recreation, recharge, irrigation, or other uses. For example, groundwater supplies are replenished by detention-recharge. However, the potential for groundwater pollution must also be considered. The design essentials include a contained area that allows the stormwater to pond and a release structure to control the rate at which the runoff is allowed into the drainage system. The release structure is usually a simple construction, such as a small-diameter pipe draining the basin or an orifice plate placed at a sewer inlet. The capacity of the pipe or orifice limits the flowrate to a level acceptable to the downstream system. Where the depth of ponding has to be limited, the release structure will have an automatic overflow to prevent excessive ponding.

Successful variations of detention that take advantage of facilities primarily used for other purposes are ponding on parking lots, plazas, and recreation and park areas. The fundamental approach is the same as for other forms of detention but low cost is implied.

Surface ponding is the most common form of detention being used by developers. In most cases the facilities are carefully planned so that the ponding area is a dual-use facility for recreation and athletics that enhances the value of the site when dry. Variable level ponds have a permanent water level during dry weather and increased holding capacity during storm conditions. The permanent lakes have aesthetic and recreational appeal which increases lot values. Basins that are dry between storms are often designed to be

used as baseball fields, tennis courts, and general open space. Parking lots can be made to serve as low-depth storage ponds by sloping the side and constructing drain outlets. Side slopes are restricted to about 4 percent for traction in the winter, and the pond depth is limited by the need for people to reach their vehicles. Obviously, a truck terminal lot can be allowed to pond to a greater depth than a supermarket lot.

Other variations of detention that have proven successful for metropolitan application are ponding on plaza areas and ponding on roof tops. The basic approach is the same as for other forms of detention. The outlet from the ponding area must be constructed to allow runoff to accumulate during peak storm conditions. The depth that can accumulate on plazas must be limited to approximately 3/4 inch because of pedestrians, but it is possible to design plazas so that portions can be flooded without inconvenience. Metropolitan roof tops provide an excellent opportunity for stormwater detention. Most are flat, watertight, and structurally designed to take loads greater than that of ponded stormwater. It adds very little to the cost of a new building to ensure structural conditions for ponding. The detention is controlled by a simple drain ring set around the roof drains. As the roof begins to pond, flow is controlled by orifices in the ring; extreme flows overflow the ring to prevent structural damage to the roof.

Retention - The precipitation/infiltration process is the most important method of replenishing the groundwater reservoirs that serve as potable water supplies for many areas of the county. The decreased infiltration and increased water demand caused by urbanization will stress groundwater supplies unless recharge areas are set aside. Although large-scale urban stormwater recharge programs have not been implemented because of potential groundwater pollution, onsite retention and recharge has been developed for small watersheds. Retention basins are usually variable-depth ponds designed with no outlet or only a bypass for exceptional flow conditions. They fill during storms and help maintain the groundwater during dry intervals. If groundwater protection is important, heavily polluted runoff is not used for recharge. The pollution control value is in the decreased volume of runoff entering downstream systems, as was discussed in the previous section.

Retention is also practiced as controlled onsite storage where groundwater recharge is not important. In a typical example, the California Division of Highways has built retention basins to dispose of highway runoff in the San Joaquin Valley. These basins were developed from 1- to 6-acre depressions that had originally been excavated for embankment material. Infiltration capacity is sometimes improved by excavating 6- to 10-foot deep trenches or vertical drains and backfilling with porous material. Maintenance is minimized by providing low-velocity channels ahead of the basins to help settle suspended particles. The areas are scarified once a year to decrease the surface clogging effects of organic solids. Advantages of the ponds include total containment of the highway runoff pollutants and the recreational asset to local cities that can result from an overall plan to landscape the basins as additional park land. The alternative to retention is to construct sewers to carry the runoff to receiving waters. Therefore, the economic advantages depend on the size and length of sewer that would be required.

Design Considerations - The acceptability of onsite storage as a pollution control alternative depends on the mitigation of apparent adverse factors, including the safety hazard to children, maintenance difficulties, mosquito breeding, algae growth, the land area required, possible poor appearance of dry ponds, and the responsibilities of ownership.

Safety - The safety features depend on the secondary use of the facility. Obviously, a dual-purpose recreational lake cannot be fenced to prevent access. Typical safety features include shallow bank slopes, fences, and outlet guards.

Maintenance - Debris removal, care for the landscaping, and maintenance of the outlet structure are all part of the routine operation of a retention facility.

Mosquito breeding and algae growth - Both mosquito and algae problems can be eliminated from dry basins by ensuring that the areas dry out completely between uses. For permanent ponds, these problems are more difficult to control. Mosquito breeding can be upset by controlling grass at the shoreline, varying the water depth every few days, or stocking the ponds with larvae-eating fish.

Land area required - The best way to overcome objections to land set aside as a detention pond is to recognize that the area can be an asset as open space. Housing near greenbelts and pond areas usually has a higher market value if the open space is aesthetically designed.

Poor appearance of dry ponds - Detention ponds are most presentable when a grass cover is kept on the basin slopes and floor. Grasses can be grown that will withstand periodic flooding. If retention basins contain water for long periods of time or need to be vegetation-free for better infiltration, appearance objections may be overcome by sight barriers such as trees.

Responsibility of ownership - In most cases the responsibilities of operation and ownership should be assumed by a public agency. The equipment, manpower, and expertise required for operation and maintenance is beyond the abilities of homeowner associations and developers.

Effectiveness and cost - The effectiveness of storage ponds in reducing pollutional loads can be estimated from the following exercise. In Chapter 16 of the Soil Conservation Service Hydrology Handbook (20) the following relationship was developed based on the use of a triangular hydrograph and a number of heuristic constraints:

$$q_p = 484\ A\ Q/(D/2 + 0.6\ T_c) \qquad \text{(G-11)}$$

q_p = peak rate of runoff, CFS

A = catchment area, sq. miles

Q = amount of rainfall, inches

D = duration of rainfall, hrs

T_c = time of concentration, hrs

In the context of runoff storage it is seen in the equation that the peak rate of runoff (q_p) is directly proportional to the amount of rainfall (Q) and the size of the catchment area (A). If, for example, the runoff from 25% of the urban drainage area flows into storage ponds, the peak runoff rate will be reduced by 25%.

In order to estimate the effect that residential storage ponds have on reducing combined sewer overflows the following computations must be performed. The fraction of the drainage area from which runoff flows into storage ponds

is called Fa, and K is the ratio of treatment plant capacity (TPC) to dry weather flow (DWF). When storage ponds are used, a corrected value for K (called K*) can be computed as follows:

$$K^* = (K - 1) / (1 - Fa) + 1 \qquad \text{(G-12)}$$

This procedure has been followed for the situation in which 25% and 40% of the land area drains into ponds, and the results are shown in Figure G-6. This figure shows that while the use of on-site storage does have a potential for reducing the annual hours of overflow, the reduction is strongly dependent on the K ratio. If, for example, the K ratio is 10 and 40% of the land area drains into ponds, the number of hours that overflows occur per year are reduced from 65 hours/year to 46 hours/year. This is a 29.2% reduction in annual overflow hours and a 40% reduction in runoff volume. However, a more representative raio (K) of TPC:DWF ratio is 4. At K = 4, there is a 12% reduction in the number of hours overflow per year when 25% of the drainage area runoff is impounded, and a 21.4% reduction in hours overflowing when 40% of the land area drains into ponds.

The feasibility of using neighborhood ponds for storage of runoff depends on the topography of the community and on the availability and cost of land. Although no general appraisal of the cost/effectiveness of ponds is applicable to all situations, one economic aspect of surface ponding is derived from the savings realized by eliminating the construction of a conventional sewer project. As examples of the cost magnitude involved several surface ponding sites are listed in Table G-7 showing the cost comparison between estimates for drainage systems using ponds to decrease peak flows and estimates for conventional strom sewer construction.

In the absence of technical feasibility and economic estimates to the contrary, the following type of reasoning can be used when the characteristics of the specific community are known. If the ponds are sized to contain the runoff from a total of eight inches of rain, the range of water level for the ponds is two feet, and the volumetric runoff coefficient is 0.25, the ratio of drainage area to ponds area will be 12. Thus, if 25% of the surface drains into the ponds, the pond area will be 2.1% of the drainage area. The cost of ponds was estimated by Black and Veatch (22) as roughly

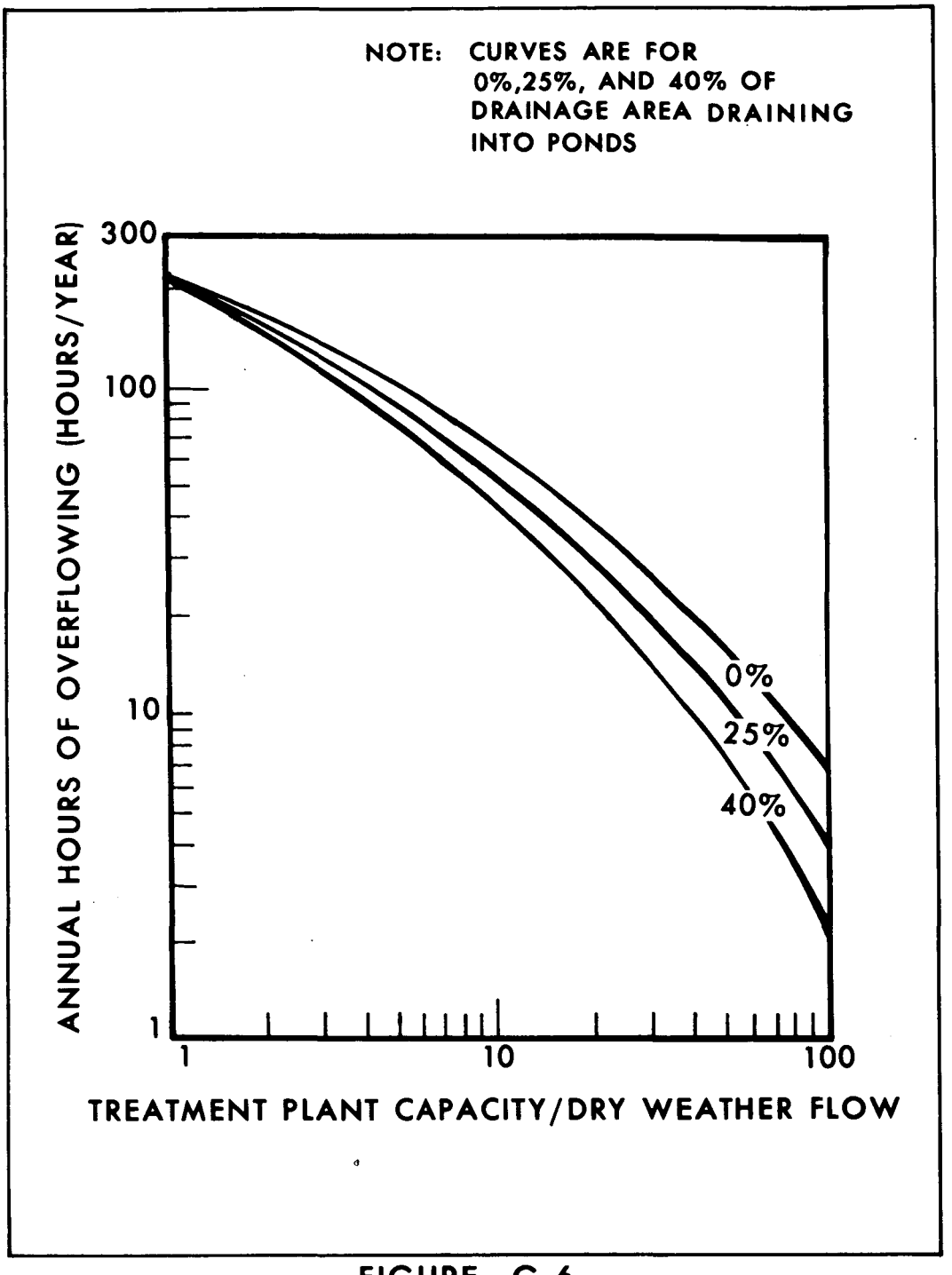


FIGURE G-6
EFFECT OF UPSTREAM STORAGE ON ESTIMATED ANNUAL HOURS OF OVERFLOW VERSUS TREATMENT PLANT HYDRAULIC CAPACITY

TABLE G-7

COST COMPARISON BETWEEN SURFACE PONDING TECHNIQUES AND CONVENTIONAL SEWER INSTALLATION (21)

Site	Description	Cost estimate, $	
		With Surface Ponding	With Conventional Sewers
Earth City, Missouri	A planned community including permanent recreational lakes with additional capacity for storm flow	2,000,000	5,000,000
Consolidated Freightways, St. Louis, Missouri	A trucking terminal using its parking lots to detain storm flows	115,000	150,000
Ft. Campbell, Kentucky	A military installation using ponds to decrease the required drainage pipe sizes	2,000,000	3,370,000
Indian Lakes Estates, Bloomington, Illinois	A residential development using ponds and an existing small diameter drain	200,000	600,000

$15,000/acre in current dollars. If the land cost is taken as $5000/acre the total cost of ponds will be $20,000/acre. In terms of dollars per acre of total drainage area this is $500/acre. Amortizing the ponds at 6% over a life of 50 years the cost of owning the ponds will be $31.72/acre-year. If the pollutional load of the runoff is 30 lb BOD/acre-year and the ponds reduce the load by 12.5% to 25% the cost of BOD reduction will be in the range $4.23 to $8.46 per lb BOD removed. Thus, it appears that the use of neighborhood ponds for storage of runoff may be a feasible pollution control alternative.

As mentioned in the beginning of this section on on-site control of runoff, another alternative for reducing the amount of combined sewer overflowing is removal of roof downspout drainage from the combined sewer. If downspout drainage is directed onto the lawn area a major fraction of the rainfall runoff from roofs is likely to infiltrate into the ground. In the ten acre area studied by APWA (4) and Tholin and Keifer (23) about 31% of the area of the individual building lots was roof area. About 17.2 percent of the ten acre area was covered by paved streets and alleys. The pervious graction including roofs, streets, and alleys was 0.379. If the runoff coefficient for lawn area is taken as 0.1 and the runoff coefficient for streets and roofs is taken as 1.0 the composite runoff coefficient with downspouts emptying into the combined sewer will be 0.44 as compared to 0.25 when the downspouts are directed onto the lawn area. This represents a 43 percent reduction in the runoff coefficient. Since some of the roof drainage will fail to infiltrate and perhaps flow ultimately into the combined sewer this estimate is an upper limit. It might be expected that removing downspouts might reduce the annual hours of overflow by about 15-25 hours/year and effect a reduction in total runoff volume between 25-40%. This simple and inexpensive method appears to have significant potential for preventing combined sewer overflowing and thus reducing the amount of pollution which reaches the stream. The cost will be primarily measured in terms of inconvenience to homeowners.

G.3.4 Porous Pavement

The use of porous asphalt paving to improve infiltration of precipitation at the source prior to concentration is a technique which is still in the developmental stages. Its purpose is to increase infiltration and to reduce flood peaks. It is also used to reduce the need for separation of storm sewers and sanitary sewers, especially in cases where the system is already overloaded. The advantages of using this management technique are given as follows: (24)

1. Reduces the total volume of runoff from paved areas.
2. Can reduce the peaking effect of local floods.
3. Enhances groundwater supply.
4. Its use can often result in savings resulting from the elimination of the need for storm sewers and curbing.
5. An improvement in the quality of roadside vegetation may be seen due to improved water availability in the soil.
6. As contrasted with conventional paving materials, there would be some preservation of natural drainage patterns in the urban setting.

A number of potential disadvantages are given as follows: (24)

1. It has not been clearly established that the filtering effect of the sub-base results in a significant improvement in the quality of the runoff if that runoff is polluted. In those instances where the runoff can be severely polluted by substances which could reduce the quality of the groundwater, it is not recommended that porous pavement be used.
2. Under certain circumstances the surface may become clogged and its permeability reduced or eliminated. Examples of such conditions are inadequate maintenance, rain on a frozen surface, and certain conditions during snowmelt. These circumstances can all result in runoff.
3. When the usual accessories of conventional paving are required (curbs, etc.) the higher construction costs may affect other economic considerations.

4. A special maintenance program is required.

The feasibility of using proous pavement in urban areas to reduce the rate and volume of runoff was examined by Thelen et al. (25) of the Franklin Institute Research Laboratories, Philadelphia, Pennsylvania. They concluded that an open-graded asphalt concrete would be the most suitable material for porous pavement because of its superior physical characteristics, its low cost, and its ability to be laid by conventional paving methods. Other types of porous pavement such as bricks assembled on rods with spacers, rubber sheeting over honeycomb, or bricks with lugs to control spacing have been suggested and might be appropriate for parking lots or aircraft runways.

The potential advantage of using porous pavement appears to be good but many problems have yet to be fully resolved. For example, water which infiltrates into the surface and the base course must be able to slowly infiltrate into the subgrade. Thelen et al. estimate that the coefficient of permeability must be at least 0.01 ft/day; characteristic of a sand-clay mixture. Freeze-thaw laboratory experiments were performed by Thelen et al. and it was concluded that the asphaltic porous surface would be able to hold its strength after 265 freeze-thaw cycles. However, a more realistic way of testing the porous pavement would be to lay a test length of highway in a northern climate. The hope is that aerobic bacteria which should be able to live in the surface and base courses of the pavement will help to rid the open spaces of organic material which will tend to clog the interstices. However, much of the dust-and-dirt which accumulates on streets is inorganic and the possibility of gradual clogging has not been adequate investigated.

As compiled by Delaware University, Water Resources Center, the design criteria and specifications are as follows: (24)

Porosity of Asphalt: Lab studies at the Franklin Institute (25) revealed that a porous asphaltic concrete containing 55% asphalt by weight and aggregate graded to allow water flow of 76 inches per hour to be optimal porous road material.

Aggregates: The aggregates used for porous pavement construction should satisfy ASTM-C-693-71 except in requiring the gradation to be the "open-graded type", but also requiring a soundness test ASTM-694-62.

As observed in Table G-8, the surface thickness and the thickness of the aggregate base course depend on the load bearing capacity of the subgrade soil and the traffic volume carried. The base thickness ranges from 6 inches for light traffic to 22 inches for highway traffic. The surface thickness ranges from 4-7 inches. The reservoir capacity of the pavement ranges from 2.4 inches of rain at a 6-inch base thickness to 7.65 inches of rain at a base thickness of 22 inches. Thus, it can be seen that even with 6 inches of base the reservoir capacity is sufficient to store a very large fraction of the storms. Since streets and highways cover about 28% of the developed urban area it is not unreasonable to visualize porous pavement as having an effect similar to ponds holding the runoff from 28% of the drainage area.

Capacity of Subsurface Reservoir: For the Philadelphia area a subsurface reservoir depth of 16 inches is recommended, capable of providing temporaty storage for a design storm of 5.4 inches per hour. Maximum daily precipitation is the critical factor and base courses of 7 inches are recommended for areas with 3 inches maximum precipitation per rainy day, 14 inches thickness for 6 inches maximum daily precipitation.

Soil Permeability: A soil capable of absorbing 5 inches of rainfall in a 10-day period, having a permeability of over 0.042 ft/day is adequate.

Maintenance: The surface of porous paving must be cleaned regularly and after each storm using a vacuum-type road sweeper to keep surface porosity as great as possible.

Additional design criteria and specifications are:

1. On slopes in excess of 5% porous paving should not be used. On shallower slopes a gravel-filled swale takes runoff and base flow.

2. Where soil is poorly drained, trench drains may be used at edges of porous paving areas or in a lattice work to lead excess water off-site.

TABLE G-8

REQUIREMENTS FOR SURFACE AND BASE COURSE AND SUBSURFACE CAPACITY

Traffic Load	CBR[1]	DTN[1]	Surface Thickness (inches)	Base Thickness (inches)	Reservoir Capacity (in. of rainfall)		
					Surface	Base	Total
Light	2	1	4	6	.60	1.80	2.40
	2	10	4	12	.60	3.60	4.20
	2	20	4-1/2	13	.66	3.90	4.56
Medium	2	50	5	14	.75	4.20	4.95
	2	100	5	16	.75	4.80	5.55
Heavy	2	1000	6	20	.90	6.00	6.90
	2	5000	7	22	1.05	6.60	7.65

Notes: 1. DTN = Design Traffic Number; CBR = California Bearing Ratio

2. A minimum surface thickness of 4 inches is used regardless of DTN

3. The estimated volume of voids in the base aggregate is 40%.

4. Frost bearing: If the combined surface and base thickness is less than anticipated frost penetration, additional base is required.

3. A single "V" drain may be used to intercept longitudinal base flow during exceptional storms or to divert base flow from fill.

4. In areas where grading puts exceptional pressure on areas of subsurface reservoirs, excess water can be removed via a tile drain system.

5. Additional informtion on specifications is found in the EPA report "Investigations of Porous Pavements for Urban Runoff Control" (25).

Cost Guidelines: Under contract, the University of Delaware built a 3100 sq. yd. porous parking lot in 1973. The cost of two inches of asphalt topping was $5.50 per sq. yd. as compared to $3.33 per sq. yd. of conventional asphalt.

An estimate of costs of conventional and porous pavements prepared by the Franklin Institute in 1972 and updated to January, 1975 is shown in Table G-9.

TABLE G-9

COSTS OF CONVENTIONAL AND POROUS PAVEMENTS
($ per sq. yd.) (26)

Type	Porous Pavement		Conventional Pavement	
	Range	Average	Range	Average
Parking Lot	8.43-8.53	8.48	4.87-7.75	6.31
Residential Street				
Low Capacity	8.44-8.49	8.47	6.40-12.80	9.60
High Capacity	8.25-8.75	8.51	11.13-16.13	13.64
Business Street				
Suburban	8.87-9.29	9.04	9.20-13.20	11.20
City	9.32-11.09	10.21	18.93-29.60	24.27
Rural Road	8.01-11.09	8.59	9.65-17.60	12.27
Highway, 2-lane	9.07-10.56	9.81	11.39-27.27	19.33

To summarize, the use of porous pavement has many potential advantages such as reducing the rate of runoff, replenishing the ground water supplies, eliminating the need for curbs and storm sewers for highway drainage, improving safety by eliminating the formation of puddles during heavy rains,

and relief of flash flooding. On the other hand, questions such as the susceptability of the pavement to clogging, the durability of the pavement during winter weather under heavy traffic loads, and the problem of adequate drainage into the subgrade remain to be satisfactorily resolved.

Although there may be cost savings due to elimination of curbings and stormwater drainage systems when porous pavement is used, the placement and sizing of storm sewerage systems is generally determined by ordinance in most municipalities. Porous paving is a relatively new development, and few local regulations permit its use. Where it can be used, however, regulations may not permit any reduction in the size of storm drains.

The cost of porous pavement is roughly equivalent to the cost of conventional pavement but significantly less than conventional pavement equipped with storm drains. The potential advantage of using porous pavement appears to be good but many problems have yet to be fully resolved.

G.3.5 Chemical Use Control

Most surfaces in an urban setting are treated, altered, or otherwise exposed to chemicals either applied by man or resulting directly from man's activities. The urban environment accumulates the refuse from man's activities and the process of urbanization increases the accumulation. Contaminants of many kinds accumulate on urban surfaces from dustfall, littering, leakage from street vehicles, and many other sources. Particles discharged into the air from industrial, commercial, or construction activities will either settle to the ground if they are too heavy to remain airborne or will be washed out in the next rainfall. On occasions these washouts can occur many hundreds of miles distant from the original source of the air contamination. Streets in industrial areas have been found to be particularly contaminated, containing soluble compounds of heavy metals and organics. Concentrations of metals in runoff have been found to be ten to one hundred times greater than in sanitary sewage, and contain copper, cadmium, zinc, and lead in high enough levels in street runoff to kill certain aquatic organisms (27).

Traffic residuals are another major source of chemicals in stormwater runoff. Some pollutants such as lead compounds are released to the atmosphere and then settle to the ground. Indicative of this is the high correlation

between traffic density and lead fallout measured in dustfall. Quantitatively, less than 5 percent by weight of the traffic-related deposits originate directly from motor vehicles, but these pollutants are among the most important by virtue of their potential toxicity. Spills or leaks or motor vehicles lubricants, antifreeze, and hydraulic fluids result in grease, petroleum and n-paraffins in runoff. Lead and lead oxide stem from fuels and the wear of tires. Zinc loadings may, in part, result from motor oils. Wear of engine parts allows the deposition of copper, nickel, and chromium. Finally, asbestos fibers are left from the wear of clutch and brake linings.

In addition to highly active industrial and commercial centers, urban surfaces remote from these activity centers are also sinks and sources of chemicals. Suburban sidewalks, planter strips, lawns and backyard garden plots, driveways, and roadside green belts, etc. are contributory to the total pollutional load of runoff. Deicing salts used on streets and highways contribute not only salt contamination in acute and chronic doses, but also are the source of ferric ferrocyanide, sodium ferrocyanide, chromate, and phosphate which are used as special additives in much of the highway salt in common use. High nutrient levels from fertilizer application on lawns and garden plots, and specific pesticides resulting from tree spraying, weed control, and insect control on public and private lands have been noted.

Nutrients - Data on loading intensities of nutrients found on street surfaces have been presented by Sartor and Boyd (9), and shown in Table G-10.

As noted in the report, the strength values vary somewhat from one land-use category to another, but only over a moderate range. These data, which are based on the analysis of samples from numerous cities, imply that all street surface contaminants are similar in nutrient composition from site to site.

Pesticides - The widespread and often indiscriminate use of pesticides is a cause for concern due to their toxicity and persistence in the environment. Although it is difficult to identify specific sources of pesticides, particularly at a sublethal chronic level, reports of fish kills and subsequent chemical analyses do substantiate the transport of this class of toxic compounds to receiving waters. Sartor and Boyd (9) reported the loading

TABLE G-10
NUTRIENTS IN STREET SURFACE CONTAMINANTS
VARIATION WITH LAND-USE CATEGORY (9)

	STRENGTH (% by weight)	LOADING INTENSITY (lb/curb-mi)	LOADING INTENSITY (lb/1,000 sq ft)
Phosphates			
Residential	0.113	1.07	12.3
Industrial	0.142	3.43	39.4
Commercial	0.103	0.29	3.41
Kjeldahl Nitrogen			
Residential	0.218	2.04	23.8
Industrial	0.163	3.94	67.1
Commercial	0.157	0.45	5.17
Nitrates			
Residential	0.0064	0.063	0.70
Industrial	0.0072	0.178	2.00
Commercial	0.0600	0.172	1.96

Note: The term "strength," as used here, refers to the amount of contaminant contained in the dry solids collected from the street surface (on a weight basis), i.e., a phosphate value of 0.1 percent would be equivalent to 1 lb of phosphate per 1,000 lb of sample.

intensities of pesticides found in street surface samples which they tested (Table G-11), and the pesticide concentration measured in the total solids on the streets tested, distinguishing among residential, industrial, and commercial land-use activities (Table G-12).

Additional clarification of the data indicated that DDD, DDT, and dieldrin tended to be associated with the finer particles (DDD:60 wt %, < 246 μ; DDT & dieldrin:80 wt %, < 246 μ) found on the streets.

Heavy Metals - As was the case in the discussion of pesticides, interest in the occurrence of heavy metals in urban runoff is not so much in the BOD implications of their presence but rather because of their potential toxicity in receiving streams. Data from Sartor and Boyd (9), presented in Table G-13

TABLE G-11

PESTICIDE LOADING INTENSITIES

(10^{-6} lb/curb-mi)

	p,p'-DDD	p,p'-DDT	DIELDRIN	ENDRIN	LINDANE	METHOXY-CHLOR	METHYL PARATHION	PCB's	TOTAL OF ALL PESTICIDES AND PCB's
San Jose I	67	110	11	2	17	0	20	1,200	1,427
San Jose II and Seattle	120	170	27	0	0	0	0	1,100	1,417
Phoenix II, Atlanta and Tulsa	34	13	24	0	0	0	0	65	136
Milwaukee	0.5	1.0	10	0	3.1	8500	0	3,400	12,000
Bucyrus	83	60	17	0	0	1600	0	650	2,451
Baltimore	100	30	3.0	0	0	170	0	1,000	1,300

TABLE G-12

PESTICIDE CONCENTRATIONS IN TOTAL SOLIDS (ppm) (9)

	p,p-DDD	p,p-DDT	DIELDRIN	ENDRIN	LINDANE	METHOXY-CHLOR	METHYL PARATHION	PCB's
Residential								
San Jose I	0.082	0.15	0	0	0	0	0	0.81
Milwaukee	0	0	0.009	0	0	2.5	0	2.0
Baltimore	0.11	0.030	0	0	0	0.19	0	0.99
Industrial								
San Jose I	0.060	0.091	0.031	0	0.031	0	0.037	1.5
Milwaukee	0	0	0	0	0.001	3.6	0	2.0
Baltimore	0.020	0.020	0.018	0	0	0	0	1.0
Commercial								
San Jose I	0.040	0.030	0	0.058	0	0	0	0.60
Milwaukee	0.020	0.031	0	0	0	1.8	0	0.99
Baltimore	0.020	0.031	0	0	0	0	0	0.51

shows the heavy metals loading intensities (in pounds per curb mile) as distributed among the major land-use categories.

TABLE G-13

HEAVY-METALS LOADING INTENSITIES
(lbs/curb-mile)

Land Use	Chromium	Nickel	Lead	Copper	Zinc	Mercury
Residential	2.0	0.5	15.7	4.8	16.8	4.8
Industrial	4.7	2.2	14.8	7.7	29.2	0.8
Commercial	1.0	0.3	3.5	1.8	3.0	1.5

The authors noted in their presentation that the samples collected from street surfaces in the cities studied were composited in various ways to reflect a particular trend of interest. This applies to the data shown in Table G-13 and, therefore, should be treated accordingly when used in further analysis.

For samples collected from five cities there was little trend for the individual metals to be distributed in relation to particle size. An exception to that was lead. There seemed to be a distinct tendency for lead to be associated with fine particles (<246μ). The assumption has been made that the principal source of lead is antiknock gasoline additives, in which case the results are as expected from particulate automobile exhaust emissions.

Deicing Compounds - The salting of streets and highways to control ice and snow is beneficial in cutting down on highway deaths, injuries, and property damage. Because of the large quantities of salt used annually in salting streets, adverse environmental effects have been reported (28,29,30). Among the many are high chloride levels in snow melt runoff, sewage, and surface streams; high chloride levels and density stratification in lakes and impoundments; ground and surface water supply contamination; widespread damage of roadside soils, vegetation, and trees; high levels of chromium and ferrocyanide compounds used as road ice additives; and roadway damage and vehicle corrosion.

Extensive use of salt for deicing of streets and highways started in the early 1950's when the total annual usage in the U.S. was about 500,000 tons and has grown since then until the current usage is about 10 million tons/year. Salt usage for deicing in the State of Massachusetts has risen from about 25,000 tons/year in 1955 to about 225,000 tons/year in 1972. This represents an annual increase of about 13.6%. Since the miles of streets and highways also increased over that 18-year period the average annual application rate increased from 4.3 tons/lane-mile in 1955 to 23.7 tons/lane-mile in 1972, a rate of increase of about 10% per year. The application rate needed for deicing is very dependent on the climate. For example, Cincinnati, Ohio and Ottawa, Canada both have a population of about 500,000 with about 900 miles of streets but the salt usage in Cincinnati averages about 11,000 tons/year or 6 tons per lane-mile per year while the salt usage in Ottawa is 72,000 tons/year or 39 tons/lane-mile per year.

Cost Considerations - In recent years many have questioned the advisability of heavy use of salt for deicing, particularly, the use of the "bare pavement" policy. In a study by Abt Associates (28), it was estimated that the annual cost of adverse effects associated with the use of salt for deicing was 2.9 billion dollars. The greatest single damaging effect was attributed to corrosion of automotive vehicles at a cost of 2 billion dollars per year. Other damage estimates made in this report were 500 million dollars per year in damage to highway structures, 150 million dollars per year in damage to health and water supplies, 50 million dollars per year in damage to vegetation, and finally 10 million dollars per year in damage to utilities.

On the other hand, the Abt report did not attempt to quantify the beneficial effects of highway deicing through salt application. In order to establish an order of magnitude, this aspect of the question was addressed in a report (31) prepared by the Institute for Safety Analysis (TSIA) for the Salt Institute of Alexandria, Virginia. The total annual benefits associated with highway deicing was estimated at 18.4 billion dollars in the TSIA report. About 80% of the total was associated with lost wages and lost productive capacity caused by late arrival at work when no salt is used for deicing of streets and highways. TSIA estimated that about 42 million persons are employed in the snow belt at an average wage of $4.54/hour. Thus,

if each worker is delayed by two hours on each of twenty inclement weather days per year the total wages lost will be about 7.6 billion dollars per year. Workers engaged in manufacturing were estimated as 13.9 million within the snow belt. The value of lost productive capacity was estimated as $9 per hour. Thus, if each manufacturing worker is late for work by two hours on each of 20 inclement days per year the total loss in productive capacity is computed as 7 billion dollars annually. These two estimates account for 14.6 billion dollars annually or about 80% of the grand total for benefits.

Other categories of benefits estimated in the TSIA report were loss of wages because of absenteeism, loss of revenue earned in shipment of goods by truck, and increased use of automotive fuels. Wages lost as a result of absenteeism was estimated by assuming that 10 percent of the 42 million workers in the snow belt will be absent on each of 20 inclement days per year. Assuming an 8-hour day and $4.54 per hour in wages this benefit totals about 3 billion dollars annually. Potential losses to intercity truck revenue was based on an estimate of 93 million vehicle-miles traveled daily in the snow belt and an estimate that 70% of the vehicles-miles were load-carrying or revenue-producing. The revenue produced per vehicle-mile was estimated as $1.15 and a two-hour delay was assumed on each of 20 inclement days per year. Pick up and delivery trucks within the snow belt travel an estimated 189 million vehicle-miles per day or 13.5 million vehicle-miles per hour assuming a 14-hour work day. Of these vehicle-miles, it was estimated that about 48% were loaded and generating revenue at the rate of $2.20 per vehicle-miles. Using these assumptions the estimate for revenue lost due to a two-hour delay on each of 20 inclement days per year was computed as 600 million dollars annually.

The increased cost of automotive fuel was estimated from the additional fuel consumption known to be required for travel on snow- and ice-covered streets and highways. About 2,287 million vehicle-miles are traveled by all automotive vehicles in the snow belt daily. At an average gasoline mileage of 20.8 miles per gallon the total fuel consumption in the snow belt is about 110 million gallons/day. Estimating the cost of fuel at $0.60 per gallon the daily fuel cost is about 66 million dollars per

day. The increase in fuel consumption caused by hard-packed snow and ice cover on the streets was estimated at 15%. Thus, the daily cost for additional fuel was about 10 million dollars. Multiplying this by the estimated 20 inclement days per year gives a total annual additional fuel cost of 200 million dollars.

Although TSIA presented convincing evidence that time lost in responding to traffic accidents and other forms of medical emergencies can result in loss of life no attempt was made to place a monetary value on this aspect of the problem.

TSIA also reevaluated the categories of salt damage presented by the Abt Associates report. The TSIA estimate for total annual damage was one billion dollars as compred to the Abt Associates estimate of 2.9 billion dollars.

Although additional work might profitably be done to more precisely estimate the costs and benefits associated with the use of salt for deicing highways and streets it is clear from these two reports that neither of the two extremes of heavy salt use to implement the "bare pavement" policy or a prohibition on the use of salt are likely to represent the best solution.

As presented by Field et al., (29) aside from chemical melting, various methods for deicing are available or have been conceived which may become more prominent in the future especially when communities realize that a price must be paid to alleviate the pollutional effects of wintertime salting. Some of these methods are:

1. External and in-slab thermal melting systems.
2. Stationary (or pit) and mobile thermal "snow melters".
3. Substitute deicing compounds.
4. Compressed air or high-speed fluid streams in conjunction with snowplow blade or sweepers to loosen pavement bond and lift snow.
5. Snow adhesion reducing substances in pavement.
6. Pavement substances that store and release solar energy for melting.
7. Electromagnetic energy to shatter ice.

8. Road and drainage design modifications to enhance runoff, reduce wintertime accidents, and capture snowmelt for treatment or control.

9. Salt retrieval or treatment possibly enhanced by the addition of chelating agents.

10. Improved tire or vehicular design to reduce deicer requirements. Although it is recognized that power, maintenance, and chemical costs for the above systems are high when compared to rock salt, municipalities, such as Burlington, Massachusetts have expressed a deep willingness to explore and demonstrate new methods regardless of cost. Burlington has recently suspended roadway salting practices when a study indicated that their well water chloride concentrations could exceed the recommended limit of 250 mg/l if salting was continued. It should also be pointed out that experience, operational data, and knowledge of environmental effects are lacking for the substitute chemical deicers.

G.3.6 Control of Particulate Fallout

The provisions of Section 208 of the Federal Water Pollution Control Act Amendments, as well as a variety of 208 planning guidance documents, mandate coordination between air quality maintenance planning and areawide waste treatment planning. Particulate material released into the atmosphere precipitates by gravity or through entrainment with precipitation and may contribute significantly to the dust-and-dirt cover on urban surfaces. This material is then available for incorporation as pollutant load in urban runoff.

Major sources of such materials are stacks and vents, construction and excavation projects, agricultural operations, and exposed vacant land areas. Automobile traffic and commercial air traffic are also sources of fine airborne particulates. Materials may be generated from activities entirely within the 208 area or through transport from other areas by prevailing winds.

Many such forms of fallout are largely inert and would contribute only to turbidity and suspended solids levels in runoff. Others, however, may be

reactive and possibly impose BOD, nutrient, and toxics loadings upon receiving waters. The extent to which each source contributes to the accumulation of specific pollutants on urban surfaces is a complex issue, and one that has not been adequately defined. Nonetheless, some coarse estimates can be made, particularly with regard to nontransportation related particulates, to provide a judgement regarding the potential impact of point source air quality control upon runoff.

In the 1950's and 1960's contaminants falling to the surface by gravity or as a result of being entrained in rainfall were routinely measured by means of dustfall jars. Dustfall jars were partially filled with water and placed in an elevated position, such as a rooftop, for a period of one month. The contents of the jars were then analyzed for mass of particulate material, composition, particle size distribution, etc. The use of dustfall jars has been discontinued in most communities in recent years in favor of the high-volume sampler. Johnson et al. (32) gives average values for dustfall in seven U.S. cities which range from a low of 15.5 tons/sq mi/mo in Tacoma, Washington, to a high of 63.8 tons/sq mi/mo in New York City. APWA (4) reported that the average dustfall in Chicago was 36.9 tons/sq mi/mo in 1966 and 41 tons/sq mi/mo in 1965. APWA also showed that the amount of dustfall depended on the month of the year, being least in August and maximum in February.

The amount of dust-and-dirt swept from streets as reported by APWA (4) was 79.2 lb/day/curb-mile. This estimate can be converted as follows to tons/sq mi/mo when the miles of street per acre is known and it is assumed that the area of the streets is about 28% of the total urban land area.

$$\text{tons/sq mi/mo} = 5431 \text{ (miles of street/acre)}$$

The miles of street per acre in the 10-acre test area used by APWA (4) was 0.037. Therefore, the amount of dust-and-dirt was equivalent to 201 tons/sq mi/mo. Thus, it can be concluded that the measured dust-and-dirt originating from dustfall was approximately 30% of the total surface accumulation estimated from street sweeping experiments. Johnson (32) gave percentages ranging from 16-33% of the total dust-and-dirt which might originate from dustfall. These might well be over-estimates because there

is evidence that conversion from coal to oil and gas for heating and the imposition of air pollution regulations has reduced the amount of pollutants in urban air.

Thus, it may be seen that control of particulate air pollution will benefit pollution control. For illustrative purposes, the cost of particulate control will be compared with that of street cleaning.

One of the major sources of atmospheric particulates is coal-fired power generating stations. For this example a plant having the following characteristics will be considered: capacity - 1000 megowatts; conversion rate - 0.85 lb coal/kw-hr.; coal content - 15% ash; 85% of ash entrained in flue gas (no control); volume of gas - 4 million actual cubic feet per minute (acfm); electrostatic precipiator efficiency - 99%.

Using these assumed data the potential amount of flyash emitted from an uncontrolled stack can be estimated at approximately 900 million pounds per year. The cost of a 4-million acfm electrostatic precipitator installation is about $21 million (33). This amount amortized at 17% per year, typical of industrial depreciation rates, results in an annual cost of $3,570,000. Operating and maintenance cost will be about $178,000 and the cost of power at three cents per kw-hr is about $850,000 per year. The total cost of removing over 99% of the particulate is about $4.6 million dollars/year. Thus, the cost of removing particulate material at the source is about a half cent per pound.

On the other hand, the cost of mechanical street sweeping to remove dust-and-dirt from street surfaces is more expensive. Using the APWA accumulation rate for Chicago (79.2 lb/day/curb-mile), a seven day sweeping frequency, 50% sweeper efficiency, and the average sweeping cost developed in Section G.3.1 ($10.40/curb-mile), the cost of removing dust-and-dirt may be estimated at roughly four cents per pound of dust-and-dirt removed.

Thus, it appears that removing the particulate material at the source is less expensive by a factor of nearly 8. In smaller sized power plants the cost will be somewhat higher per pound of particulate removed but not likely to equal the cost of removing dust-and-dirt from the street by sweeping. It should also be remembered that particulate control would effect all urban land area, including streets. However, on a BOD basis it is difficult to estimate the impact of particulate source control. For the example above

the effect upon potential BOD loadings from urban runoff would be negligible. For other industrial or municipal sources which could contain oxygen demanding materials the impact could be greater.

G.4 Collection Systems Control Alternatives

G.4.1 Introduction

The goal of management practices applied to collection systems is either hydraulic control to minimize the volume of storm runoff and municipal sewage which overflow into the receiving stream without treatment, or loading controls to minimize the pollutants which are contained in urban stormwater runoff and combined sewer overflows. During combined sewer flows in excess of treatment plant capacities are a storm event diverted around the treatment plant and are generally discharged directly into a receiving stream without treatment. Since the overflowed excess is comprised of untreated sanitary wastes and a wide variety of environmentally disruptive substances, a potential public health hazard and an adverse stream quality impact results.

It is the purpose of this section of this appendix to review the quantification of combined sewer overflow events and volumes, and to review the management alternatives for controlling the volume and/or pollutant loadings attributable to combined sewer overflows.

G.4.2 Volume Control

G.4.2.1 Review of Available Analytical Techniques

A number of reports have attempted to estimate the annual number of hours during which combined sewer overflows are likely to occur as a function of interceptor capacity expressed as a multiple of DWF. One of the first was by McKee (34) who estimated the number of times overflow occurred and the fraction of sanitary sewage it contained based on the intensity distribution of the hourly rainfall.

Assuming the population density and the rate of inflow both increase as the runoff coefficient increases, the calculated that the rainfall intensity producing a runoff flow equal to dry weather flow approximates 0.01 inch/hour for all communities. He also estimated that the first 0.03 inches of each rainfall event would be used in wetting the land surface and would result

in no runoff. While this correction made little difference in the rainfall distribution pattern, rainfall time was essentially reduced from 6.64% to 5.56%. Implicit in this analysis was the assumption that the runoff coefficient was the same for all intensities. The result of McKee's analysis is shown in Figure G-7 (curve A) as the number of hours per year combined sewer overflows will occur as a function of interceptor capacity expressed as multiples of the dry weather flow rate.

The results of a study of combined sewer overflows in Detroit were presented by Palmer (35). His analysis was for a part of Detroit having a population density of 40 persons/acre and an estimated dry weather flow of 162 gal/capita/day. Following McKee's lead, Palmer estimated the equivalent dry weather flow at 0.01 inch/hour, and also assumed that rainfall equal to or less than 0.03 inch/hour would produce no runoff. The number of hours per year in which the rainfall exceeded 0.03 inch per hour was 263 hours, or 3% of the time. Palmer's estimate of annual hours of combined sewer overflow as a function of interceptor capacity is shown as curve B in Figure G-7.

St. Louis was the site of a similar analysis of combined sewer overflows made by Shifrin and Horner (36). The area studied had a population density of 27 persons/acre and the estimated dry weather flow of 166 gal/capita/day. They estimated the rainfall equivalent of dry weather flow at 0.007 inch/hour and assumed that 0.4 inch of depth is required to wet the land surface. In this analysis only a very limited range of interceptor capacity was considered. The results are shown as curve D in Figure G-7.

Johnson (37) made a study of the number and duration of overflow events at eight overflow points in the Washington, D.C. combined sewer system. His analysis was similar to those of McKee and Palmer. The results, shown as curve C in Figure G-7, demonstrated that the measured annual hours of overflow exceeded the calculated estimates for six out of the eight overflow sites studied.

Benjes et al (38) made a study of expected combined sewer overflow hours/year in Kansas City. Benjes estimated the dry weather sanitary flow as equivalent to 0.0035 inch/hour, and assumed that 0.04 inch of rain was required to wet the land surface. The results of this study are shown as curve E in Figure G-7.

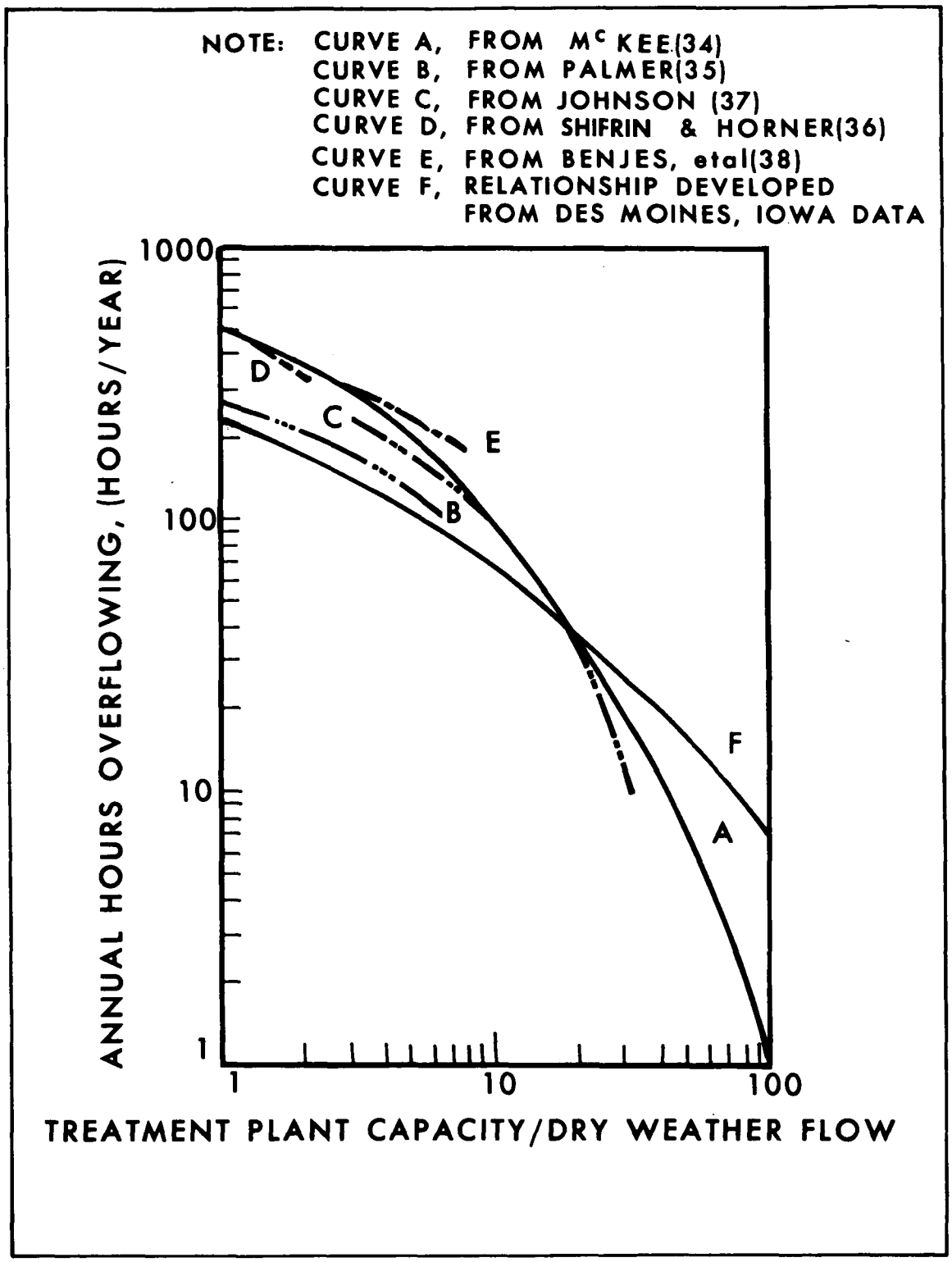


FIGURE G-7

ESTIMATED ANNUAL HOURS OF OVERFLOW VERSUS TREATMENT PLANT HYDRAULIC CAPACITY

Each of the foregoing studies of combined sewer overflow frequency recognized that a threshold rainfall exists, below which no measurable runoff occurs. Palmer believed that the intensity must exceed 0.03 inch/hour before runoff will occur and the other studies all eliminated some quantity (0.02-0.04 inches) from the initial hours of each event. Also, all five studies incorporated an estimated runoff coefficient into the calculation of the amount of rainfall which is equivalent to dry weather flow. The same runoff coefficient was then implicitly assumed for all intensities above the threshold value. All five investigators concluded that the capacity of the interceptors must be very large in order to effect a significant reduction in the annual hours of combined sewer overflows. As a result of the study made by Johnson, a ratio of 30 between interceptor design flow and dry weather flow was adopted for use in portions of the Washington, D.C. area. According to Johnson at a WWF:DWF ratio of 3 the combined sewers would overflow 240 hours per year. When the WWF:DWF ratio was increased to 30 the overflow hours per year would be reduced to 13 hours per year. This represents a reduction in the time of overflow of about 95%. The principal weakness of the analyses described above is the assumption that the runoff coefficient is independent of the rainfall intensity. In order to avoid having to make this assumption, Davis and Borchardt (39) made measurements of the runoff coefficient in Des Moines based on the rational method. Measurements were made for two catchments: catchment 0-11 having a population density of 10.7 persons/acre and catchment S-3 having a population density of 5.3 persons/acre. Calculated runoff coefficients for each catchment as a function of rainfall intensity are shown in Figure G-8. A regression line for catchment 0-11 data has been plotted in Figure G-8, and the equation relating the runoff coefficient to rainfall intensity is shown below:

$$C = 0.3836 + \ln(I)/9.48248 \qquad \text{(G-13)}$$

I = rainfall intensity, in/hr

I_r = runoff, in/hr

$C = I_r/I$

Extrapolating the regression line in Figure G-8 it is seen that the rainfall intensity which produces no runoff would be about 0.026 inch/hour. This

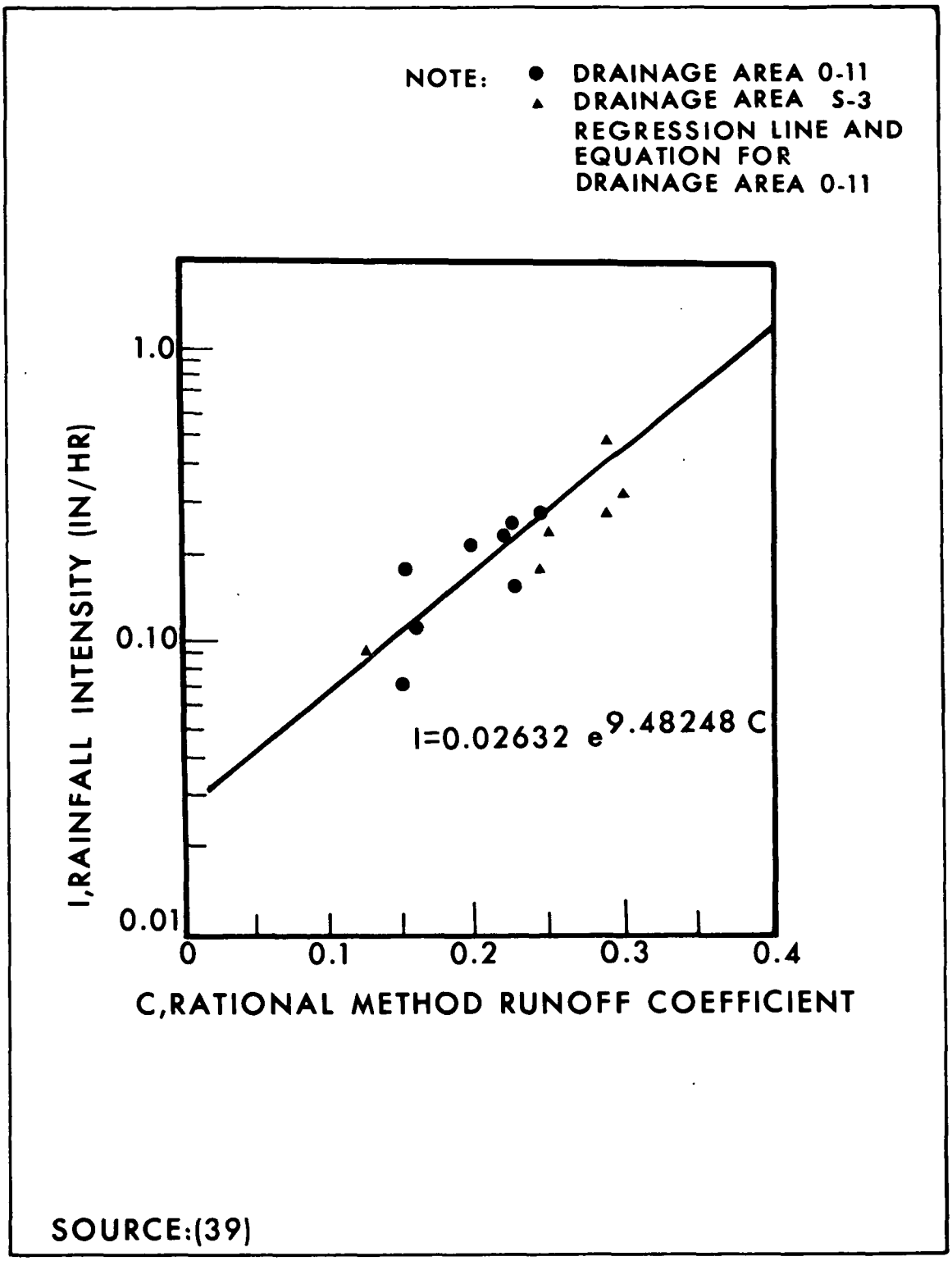


FIGURE G-8
RATIONAL METHOD RUNOFF COEFFICIENT
VERSUS RAINFALL INTENSITY

figure agrees well with the estimates of 0.03-0.04 inch per hour which had been assumed by previous investigators.

The relationship between runoff coefficient and rainfall intensity is known to depend on percent imperviousness which, in turn, is believed to be related to population density. In their Des Moines study, Davis and Borchardt (39) found that for an urban area with a population density of 10.7 persons/acre the rate of runoff could be related to rainfall intensity as follows:

$$CFS/Ac = I(0.380 + \ln(I)/9.526) \qquad \text{(G-14)}$$

I = rainfall intensity, in/hr

Using the relationships of Chapter 3, the value for C_v at a population density of 10.7 persons/acre is about 0.35 as compared to a value of 0.38 at a population density of 13.1 persons/acre. Thus, for a rough estimate, the runoff coefficient shown in Figure G-8 for catchment 0-11 would be increased by 10% to find the expected runoff coefficient for a population density of 13.1 persons/acre.

To convert a distribution of hourly rainfall intensities to a distribution of runoff rates, Equation (G-13) can be used. Using the data of Davis and Borchardt (39), a distribution of hourly equivalent runoff is plotted in Figure G-9.

However, runoff rates are more meaningful when expressed as multiples of the dry weather flow of municipal sewage. If the production of municipal sewage is estimated as 100 gal/capita/day, the flow will be 0.00131 mgd/acre at a population density of 13.1 which is equivalent to a runoff rate of 0.00201 inches/hour.

If the rainfall intensity is less than 0.026 inch/hour no runoff will occur. The distribution of rainfall intensities shown in Figure G-9 indicates that about 46% of the rainfall hours have intensities greater than 0.026 inch/hour. Therefore, since the total rainfall hours was 489 hours/year in Des Moines over the 10-year period between 1950 and 1960, the combined sewer would overflow 225 hours/year if the interceptor capacity is just equivalent to the dry weather flow. If the runoff rate is equal to the dry weather flow, the rainfall intensity will be 0.0403 inch/hour and this intensity of rainfall

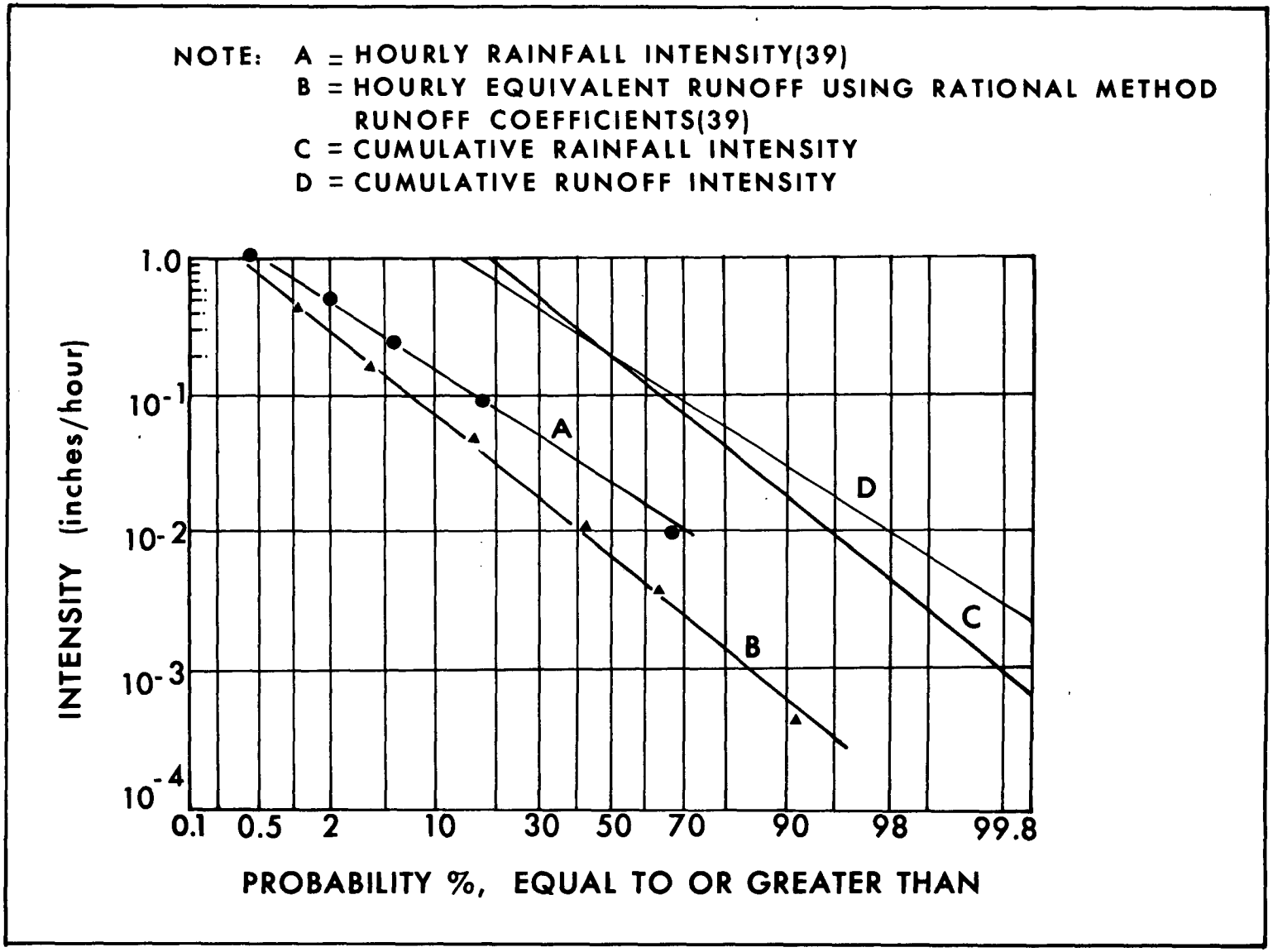


FIGURE G-9
PROBABILITY DISTRIBUTION OF RAINFALL AND RUNOFF INTENSITIES

will be exceeded 196 hours per year. Using this reasoning, the relationship between WWF:DWF ratio and annual hours of overflowing has been developed for the Des Moins data, and is shown as curve F in Figure G-7. It can be seen that the potential for limiting the number of hours during which overflowing occurs is significant. If the capacity of the treatment plant is 3.5 times the dry weather flow the total annual overflow will be reduced from 225 to 125 hours, which is a reduction of 44%. A 90% reduction in the number of hours of overflow could be achieved but only if the plant capacity is 35 times the dry weather flow.

It is generally accepted that a large fraction of the total overflow volume is contributed by a few high-intensity storms. As a result, the potential for making a major reduction in the total amount of overflow by increasing the capacity of the plant and sewerage system is considerably reduced. To quantify this facet of the situation the first moment of the distribution of runoff rates must be found.

Since both the distribution of rainfall intensities and the distribution of runoff rates appear to be log-normally distributed the first moment of the distribution functions can be computed as given in Smith (8). The mean rainfall intensity for Des Moines was computed as 0.0672 inch/hour and this value when multiplied by the total number of rainfall hours (489) gives a total accumulation of 32.86 inches per year. Similarly, the mean value for runoff is 0.03546 inch/hour and this value times 225 hours/year of runoff gives an accumulated runoff amount of 7.97 inches per year. Thus, the average runoff coefficient for Des Moines at a population density of 13.1 is 7.97/32.86 or 0.243.

By using the first moment of the distribution of runoff rates, the fraction of the total amount of runoff (7.97 inch/year) which will escape when the capacity of the plant is some multiple of the dry weather flow can be found. The results of this calculation are shown by the zero percent curve in Figure G-10. It can be seen that to reduce the runoff volume which overflows by 50% the capacity of the treatment plant must be 100 times the dry weather flow rate.

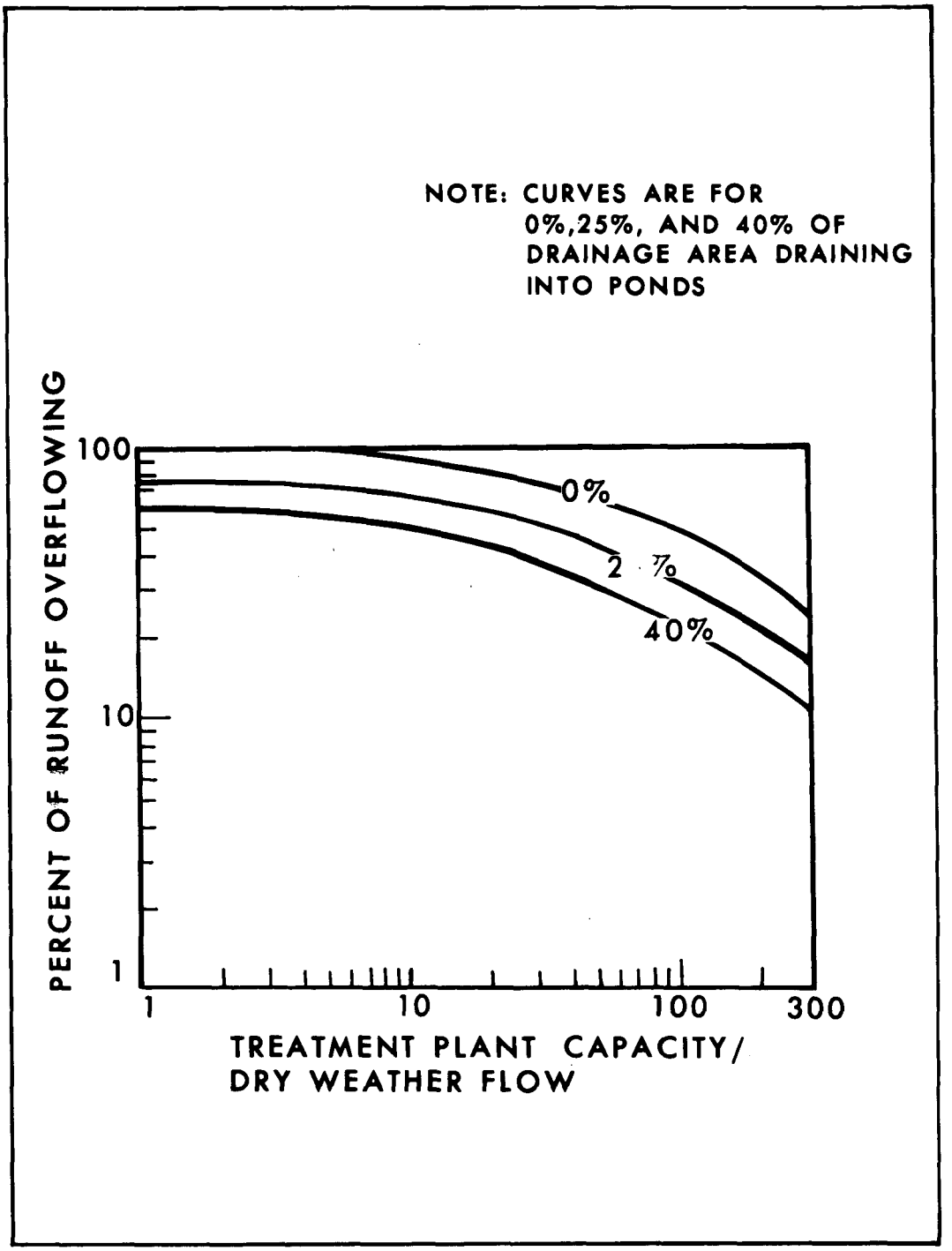


FIGURE G-10
EFFECT OF UPSTREAM STORAGE ON PERCENTAGE OF TOTAL RUNOFF VOLUME OVERFLOWING VERSUS TREATMENT PLANT HYDRAULIC CAPACITY

If the concentration of pollutants in the runoff were the same in each volume of runoff this estimate of the fraction of runoff escaping would lead to a pessimistic view of the potential for limiting pollution by increasing the capacity of the plant and sewerage system. However, since there is some evidence that a larger fraction of the pollutants are present in runoff from smaller storms, and especially in the first flush, the analysis based on number of overflow hours may have more validity. If the concentration of BOD is assumed to be the same for all runoff the concentration can be computed as follows:

$$\text{BOD, mg/l} = 4.422\ (\text{BOD loading, lb BOD/acre-yr})/(\text{annual runoff, in/yr}) \qquad \text{(G-15)}$$

Thus it can be seen that if the annual runoff is 7.97 in/yr and the storm sewer loading is 30 lb BOD/acre-yr the average BOD concentration will be 16.1 mg/l. Similarly, if the combined sewer loading is 100 lb BOD/acre-yr the average BOD concentration will be 55.5 mg/l.

G.4.2.2 Storage in the Sewerage System

The municipality of Metropolitan Seattle has installed and demonstrated (40) a computer augmented treatment and disposal system (CATAD) which monitors and controls storage of runoff in a combined sewer system serving an area of 13,120 acres. It was shown that storage of storm runoff in trunks and interceptors can reduce the volume of runoff escaping by about 74% when the regulator gates are manually controlled from a central control panel, and by about 90% when the gates are controlled automatically by computer algorithms.

Although performance measurements made on the CATAD system were very scattered there is little question that, in sufficiently large communities, storage of runoff in the sewerage system is a viable strategy for reducing or eliminating the volume of runoff which escapes into the receiving stream. For example, by means of the following simple analysis the storage volume available in the interceptors can be roughly estimated. If the interceptors are designed to transmit K times the dry weather flow with a Manning roughness coefficient of 0.013, the required interceptor diameter can be found from the following relationship.

$$D = 0.308 \ (K \ DWF/S^{1/2})^{0.375} \qquad \text{(G-16)}$$

D = interceptor diameter, ft

K = multiple of DWF

DWF = dry weather flow, mgd

S = interceptor slope, ft/ft

In the 1967 APWA survey (7) of combined sewer problems it was found that of the 109,000 miles of sewer installed about 7,600 miles or 7% were interceptors. Thus, if the length of interceptor is taken as 7% of the total sewer length shown in Figure G-2, and the interceptor diameter is found from Equation (G-16), the volume of the interceptor can be calculated as follows:

$$V = 0.002171 \ D^2 \ L \qquad \text{(G-17)}$$

V = interceptor volume, mg

L = interceptor length, miles

If dry weather flow is taken as 100 gpd/capita, the interceptor is sized to transmit four times the dry weather flow, and the slope of the interceptor is assumed to be 0.0003 ft/ft, the diameter of the interceptor will vary from about 2 ft at a population of 10,000 to a diameter of 7.5 ft at a population of about 220,000. Up to a population of 45,000 the slope must be increased to provide the minimum velocity of 2 ft/sec necessary to provide adequate scouring. Since 7.5 ft diameter is near the upper limit of current practice it has been assumed that for populations above 220,000 multiple 7.5 ft diameter interceptors will be provided.

Based on the assumptions itemized above, the interceptor volume can be expressed by the following expressions as a function of the drainage area in acres.

$$V = 0.078 \ (Acres/1000)^{2.3} \qquad \text{(G-18)}$$

$$V = 0.0217 \ (Acres/1000)^{2.76} \qquad \text{(G-19)}$$

Equation (G-18) applies from 10,000 to 17,500 acres or from 100,000 to 220,000 population and Equation (G-19) applies from 17,500 to 50,000 acres or for a population greater than one million.

The CATAD system served an area of 13,120 acres and had a design storage of 29.8 mg, however, it was found that the actual storage was only 17.83 mg or about 60% of the design storage. Substituting an area of 13,120 acres into Equation (G-18) gives an estimate of 29.1 mg which shows that the estimates made agree reasonably well with the CATAD system.

The important conclusion to be drawn from Equations (G-18) and (G-19) is that the available interceptor storage volume increases very rapidly with the area drained. The volumetric runoff coefficient measured in the CATAD testing program was 0.152. Using this value for C_v and the ratio of design storage to actual storage of 0.6, Equations (G-18) and (G-19) can be used to express the volume of rain (inches) which can be stored in the sewerage system for any size drainage area.

$$\text{Stored rainfall, inches} = 0.01134\ (\text{Acres}/1000)^{1.3} \qquad \text{(G-20)}$$

$$\text{Stored rainfall, inches} = 0.003155\ (\text{Acres}/1000)^{1.76} \qquad \text{(G-21)}$$

Equation (G-20) is used with Equation (G-18), and Equation (G-21) is used with Equation (G-19). Since the area of the metropolitan area is a function of the population the inches of rainfall which might be stored can also be expressed as a function of the population as follows.

$$\text{Stored rainfall, inches} = 3537 \times 10^{-6}\ (\text{POP}/1000)^{0.905} \qquad \text{(G-22)}$$

$$\text{Stored rainfall, inches} = 637.8 \times 10^{-6}\ (\text{POP}/1000)^{1.225} \qquad \text{(G-23)}$$

From Equation (G-20) it can be seen that for the Metro Seattle case where the drainage area was 13,120 acres, a maximum of 0.322 inches of rain can be stored in the collection system. Rainfall records for the Seattle area showed that rainfall exceeded 0.1 in/day for 88 days/year, 0.25 in/day for 46.6 days/year, and 0.5 in/day for 17.6 days/year. These three points appear to fit an exponential distribution expressed as follows:

$$F(I) = 1 - e^{-4I} \qquad \text{(G-24)}$$

I = rainfall intensity, in/day

$F(I)$ = fraction of rain days with intensities less then I

The total number of days/year in which some rain fell was 130 days. Therefore, it can be seen that rain intensity will be less than 0.322 in/day for

72.4% of the 130 days/year or 94 days/year. This is clearly a significant reduction in the number of days overflowing occurs.

It is also important to examine the volume of runoff which is contained. The first derivative of Equation (G-24) is the probability density function. Multiplying the probability density function by I, and integrating from zero to infinity gives a total annual rainfall of 32.5 inches/year. The amount of rain falling in days with intensities less then I can be expressed as follows.

$$C = 32.5(1 - e^{-4I}(1+4I)) \qquad \text{(G-25)}$$

C = cumulative rainfall, in/yr

From Equation (G-25) it can be seen that the total amount of rainfall contained during days when the intensity was less than 0.322 in/day is 11.98 in/year or about 37% of the total annual amount. However, during days when the intensity exceeds 0.322 in/day the first 0.322 in/day can also be stored.

Since it has already been shown that 94 days/year had intensities less than 0.322 in/day, it follows that 130 minus 94 or 36 days/year will have intensities greater than 0.322 in/day. Thus, of 0.322 inches stored during each of the 36 days, an additional 11.59 inches of rain will be contained. Adding 11.59 to 11.98 gives a total of 23.6 in/year contained. Therefore, a total of about 73% of the runoff can be contained. This estimate agrees well with the findings of the CATAD system using manual control of the gates. This analysis, however, implicitly assumed that the interval for storing and releasing the runoff was one day. If the adjustment period is less than one day greater fractions might be contained.

It can be seen from this simplistic analysis that storage in the collection system offers a significant potential for reducing the amount of overflow. In addition to overflow volume the potential for reducing pollution loading is also impressive because of the potential for containing small storms and the first flush of larger storms where a large fraction of the pollution is believed to originate.

The total construction cost for the CATAD system was about 3.9 million dollars in mid-1968 which would correspond to about 8.5 million current dollars. This

corresponds to about $650/acre drained. Amortizing at 6% over 20 years gives $56.67/acre/year. Operating cost was estimated at $5/acre in 1974 or about $6/acre in current dollars. If 75% of the pollutional load or about 75 lb BOD/acre-year is contained the cost in terms of pollution control is about $0.84 per pound of BOD removed.

G.4.2.3 Use of Combined Sewer Regulators

Combined sewer regulators are designed to intercept all dry weather flows and automatically bypass wet weather flows which are greater than the interceptor and/or treatment plant capacity. Originally, a low dam or weir was constructed across the combined sewer downstream from a vertical or horizontal orifice. Flows dropping through the orifice were collected by the interceptor and conveyed to the treatment plant. Although more sophisticated mechanical regulators were developed to improve control over the diverted volumes, no specific consideration was given to quality control. More recently, several regulators have been developed which are capable of providing both quality and volume control through induced hydraulic flow patterns that tend to separate and concentrate the solids from the main stream (41,42,43).

Conventionally designed regulators can be subdivided into three major groups: (1) static, (2) semiautomatic dynamic, and (3) automatic dynamic. The grouping reflects the general trend toward the increasing degree of control and sophistication and, of course, the capital and operation and maintenance costs. Conventional regulator design, use, advantages, and disadvantages are well covered in the literature (41,44).

Recent emphasis has resulted in the development of several new and innovative regulators both in the United States and in Europe (45). Those showing the greatest promise are undergoing prototype testing. Regulators included in this group are fluidic devices, swirl concentrators, broad-crested inflatable fabric dams, and automatic slide gates and tide gates. Improved regulators developed in England include the vortex regulator, high side-spill weir, stilling pond regulator, and the spiral flow regulator. The choice of a regulator must be based on several factors including: (1) quantity control, (2) quality control, (3) economics, (4) reliability, (5) ease of maintenance, and (6) the desired mode of operation (automatic or semiautomatic).

G.4.2.4 Use of Swirl Concentrators as Combined Sewer Regulators

The swirl regulator/concentrator has shown outstanding potential for providing both quality and quantity control, and is believed to be practical for installation at overflow points along an interceptor. In the following illustrative example the average number of combined sewer overflow points per 1000 acres of catchment and per 1000 population was given in the 1967 APWA survey (7) of combined sewer problems. These are shown in Figures (G-11) and (G-12). Construction cost for swirl concentrators was estimated by Benjes (46) and the cost is expressed by the following relationship in December 1976 dollars.

$$\text{Construction cost, dollars} = 772.72\ A^{0.7727} \qquad \text{(G-26)}$$

A = swirl concentrator surface area, sq. ft.

The following sizing relationship was developed by APWA (45) for 90% removal of suspended solids.

$$A = 30(CFS)^{0.7588} \qquad \text{(G-27)}$$

CFS = peak flow rate, cfs

Assuming that the total overflow rate (cfs/acre) is equally divided among the various overflow points, the following relationship for construction cost in terms of dollars/acre can be derived by substituting Equation (G-27) into Equation (G-26).

$$\text{Construction cost, \$/acre} = 10{,}700(CFS/Ac)^{0.586}(N/Ac)^{0.414} \qquad \text{(G-28)}$$

N/Ac = number of overflow points per acre

CFS/Ac = overflow rate, cfs/acre

Operating and maintenance cost given by Benjes (46) can be expressed as follows in terms of annual man-hours. No power or chemicals are required. The manpower consists of one inspection of each swirl concentrator every two weeks (2 hr/inspection) and washing of the concentrator after every overflow event. It is assumed that 30 overflows will occur each year.

$$\text{Annual man-hours per concentrator} = 52 + 78.16\ A^{0.306} \qquad \text{(G-29)}$$

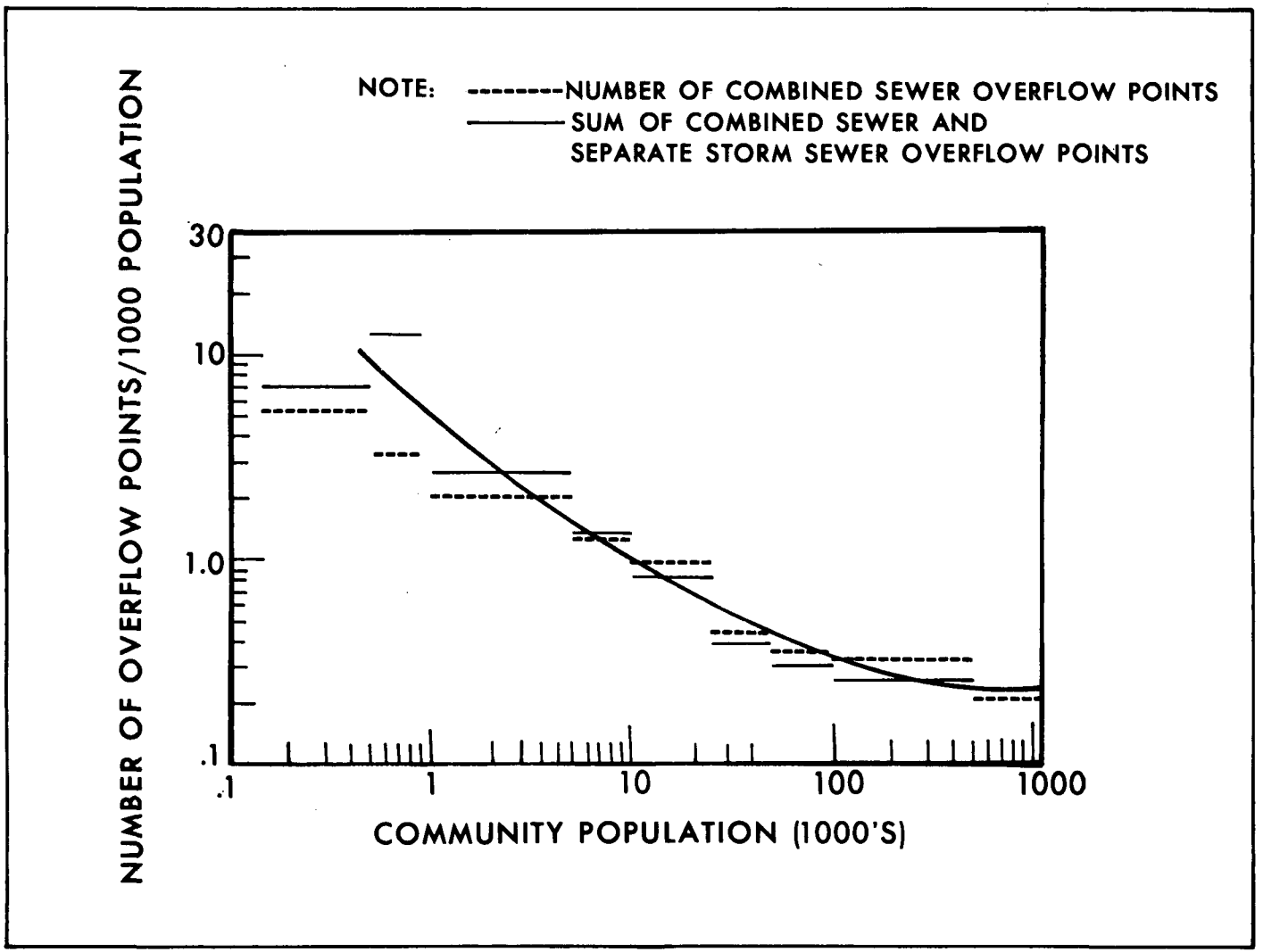


FIGURE G-11
NUMBER OF SEWER OVERFLOW POINTS PER 1000 POPULATION VERSUS COMMUNITY POPULATION

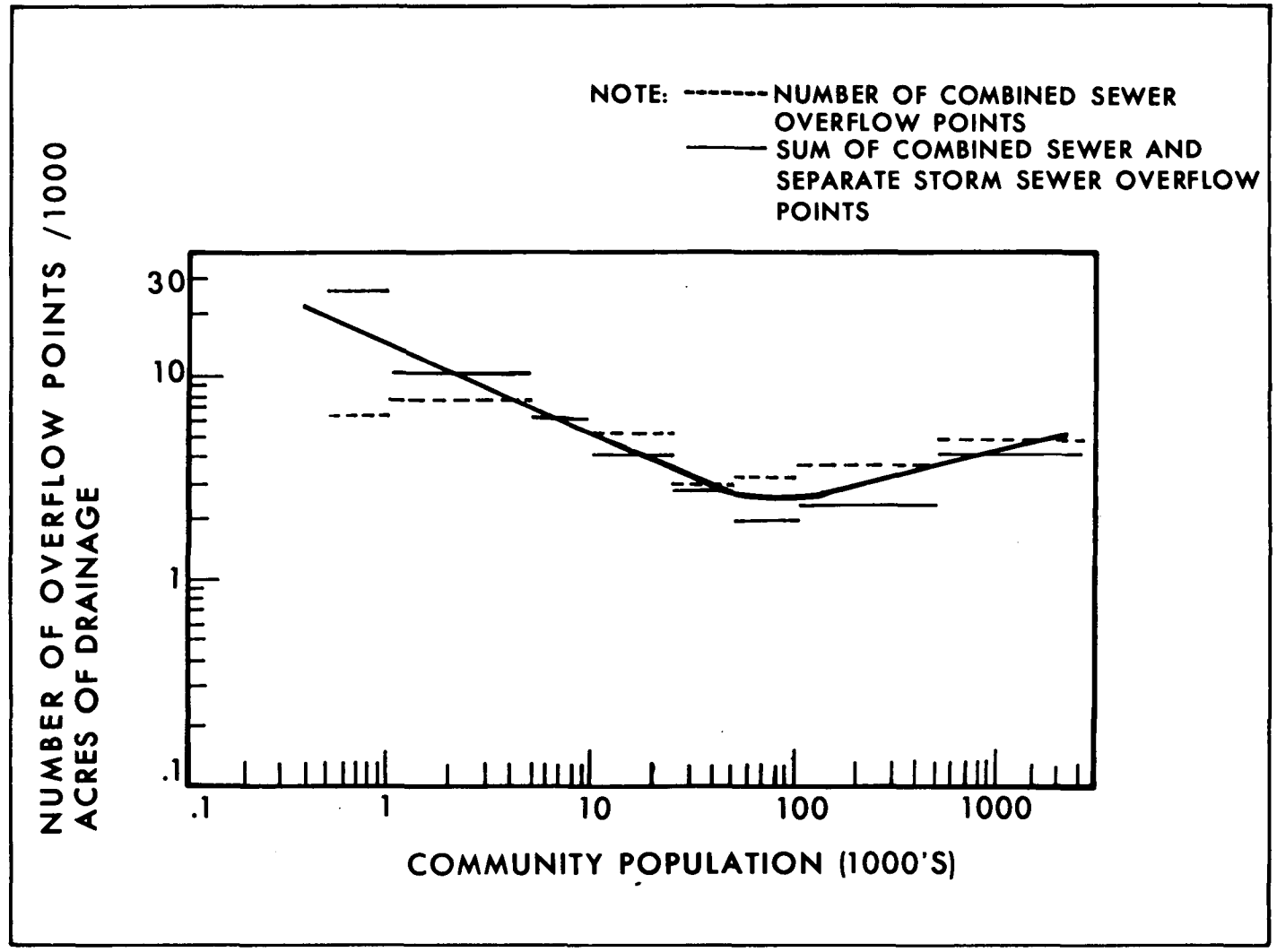


FIGURE G-12
NUMBER OF OVERFLOW POINTS PER 1000 ACRES OF DRAINAGE AREA VERSUS COMMUNITY POPULATION

Combining Equations (G-28) and (G-29) and assuming a manpower cost of $10/hour the following relationship is derived for operating and maintenance cost expressed as dollars/acre-year.

$$\text{O\&M cost, \$/acre-yr} = \$520\ (\text{N/Ac}) + \$2213\ (\text{N/Ac})^{0.768}\ (\text{CFS/Ac})^{0.232} \qquad \text{(G-30)}$$

It can be seen that both the construction and the operating and maintenance costs depend on the number of overflow points per acre (N/Ac) and the runoff rate in CFS per acre.

If the production of municipal sewerage is assumed to be 100 gal/day/capita the rate of flow for municipal sewerage can be expressed as 0.000155 CFS/acre times the population density. If the population density is 10.7 and the capacity of the interceptor is assumed to be 3.5 times the rate for municipal sewage the interceptor capacity will be 0.0058 CFS/acre. Rates of flow exceeding this amount will be assumed to overflow.

From Figure (G-9) it can be seen that in Des Moines (39), the intensity of rainfall will exceed one inch/hour in only about 0.5% of the rainfall hours or 2.5 hours/year. Using one inch/hour as the design point for the sizing of swirl concentrators, and using Equation (G-14), it is calculated that the runoff rate at 1 inch/hour will be 0.380 CFS/acre. By subtracting the capacity of the interceptor (0.0058 CFS/acre) from the runoff rate the overflow rate is calculated to be 0.074 CFS/acre. In Figure G-12 it is seen that the number of overflow points per acre of urban land varies with the population. A representative value is about 0.0035 overflow points per acre.

An average construction cost per acre for swirl concentrators can now be found by substituting a value of 0.374 CFS/acre and a value of 0.0035 for number of overflow points per acre into Equation (G-28). For the 50,000 population community the construction cost is $586/acre, but in very small communities the cost could be as much as $1000/acre. Taking the useful life of the swirl concentrators as 20 years, the interest rate as 6%, and the ratio of capital to construction cost as 1.35 the amortization cost will be about $70/acre-year but this amount might be as high as $120/acre-year for small communities. Operating and maintenance cost can be estimated by substituting values for N/Ac and CFS/Ac into Equation (G-30). If 0.374 CFS/ac

and 0.0035 overflow points per acre are used, the annual operating and maintenance cost will be about $25/acre-year. This could be as high as $78/acre-year for very small communities where the value for N/Ac is 0.015. Thus, the total cost of owning and operating swirl concentrators in most communities is likely to be about $95/acre-year, but could be as high as $200/acre-year for very small communities.

The swirl concentrator is capable of removing about 50% of the BOD from the overflow. Thus, if the annual load from combined sewers is 196 lb BOD/acre-year, 48 lb BOD/acre-year might be removed by the swirl concentrators. Dividing the previously calculated $95/acre-year by 48 lb BOD/acre-year gives an estimate of $1.98 per pound of BOD removed when swirl concentrators are installed at all overflow points of a combined sewerage system. This is two-thirds the cost of $2.94/lb BOD removed which is estimated later in Section G.4.2.6 for installing separate sewers in undeveloped land.

G.4.2.5 Reduction in Infiltration/Inflow (I/I)

A serious problem results from both excessive infiltration and high inflow rates into sewer systems: both limit the capacity of the sewerage system and the treatment plant to handle runoff resulting in increased amounts of overflows and treatment plant bypasses. Infiltration usually occurs from groundwater sources seeping into sewer pipes from leaky joints, crushed or collapsed pipe segments, leaky lateral connections, or other pipe failures. Inflow usually occurs from surface runoff, from roof leaders, cellar and yard drains, foundation drains, commercial and industrial "clean water" discharges, drains from springs and swampy areas, depressed manhole covers, cross-connections, and the like. The subject of I/I control strategies and costs has been thoroughly reported by Lager and Smith (2). The information presented in this section has been adopted from their report.

G.4.2.5.1 Inflow Control

Correction of inflow conditions is dependent on regulatory action on the part of city officials, rather than on public construction measures. If elimination of existing inflows is deemed necessary because of adverse effects of these flows on sewer systems, puming stations, treatment plants,

or combined sewer regulator-overflow installations, new or more restrictive sewer-use regulations may have to be invoked.

The effects of inflows into sewers can be greatly reduced by a variety of methods. Many authorities advocate the discharge of roof water into street gutter areas--or onto on-lot areas in the hope that it will percolate into the soil (47). Discharging roof or areaway drainage onto the land or into street gutters reduces the immediate impact on the sewer system be allowing reduction of the volume and attenuation of the flow. The use of pervious drainage swales and surface storage basins within urban areas allows the stormwater to percolate into the ground.

Depressed manholes (those with vented covers in street areas where runoff can pond over the cover) can be repaired or the covers replaced with unvented covers.

The elimination of inflow sources is generally found to be cost effective. Some of the most common sources of inflow are summarized in Table G-14. Estimates for the flow contribution by each source and the cost of eliminating that source are summarized in the following table.

TABLE G-14

COST ESTIMATES FOR INFLOW ELIMINATION

Source elimination	Flow rate, in gallons per minute	Rehabilitation cost in 1974 dollars,$
Leakage around manhole covers	10-20	50-75
Holes in manhole covers	50-100	100-125
Foundation drain disconnection	10	300-1,200
Roof leaders disconnection	10	50-75
Cross-connection plugging	250-450	100-500
Catch basin	300	3,000-5,000
Ditch or storm sewer, infiltration sanitary sewer (per manhole reach)	60-80	500-2,500
Area drain disconnection	50-200	50-350

G.4.2.5.2 Control of Infiltration

The average rate of infiltration represents a minor fraction of the hydraulic capacity but the peak infiltration rate can be several times the dry weather flow. For example, the most commonly used allowance standard for infiltration is 500 gallons per inch of pipe diameter per mile per day. Santry (48) itemized the length and diameter of sewer pipe in three communities and the average pipe diameter ranged from 9.82 inches at a total sewer length of 2822 miles to a low of 6.51 inches at a length of 6.3 miles. The three data points are well represented by the following relationship.

$$D_a = 5.75\ L^{0.67348} \qquad \text{(G-31)}$$

D_a = average pipe diameter, inches

L = total length of sewer, miles

At a population of 50,000 the length of sewer will be about 128 miles, the average pipe diameter will be about 8 inches, and the population density will be about 8 persons/acre. Assuming the dry weather flow as 100 gal/capita/day the flow of municipal sewage will be 800 gal/acre/day. The infiltration rate allowed is 4000 gal/mile/day. Since the miles of sewer per acre is about 0.021, the infiltration rate allowed is 84 gal/acre/day or about 10.5% of the dry weather flow. In terms of dry weather flow the infiltration allowances ranged from 10% of dry weather flow to as much as four times dry weather flow. It can be seen from Figure G-7 that if the infiltration equals twice dry weather flow, the capacity of the treatment plant, in multiples of dry weather flow, must be reduced by two. For example, if the capacity of the treatment plant is four times dry weather flow with no infiltration the annual hours of overflowing will be reduced from 225 to 117. However, with infiltration at twice the dry weather flow the reduction will be only to 165 hours/year representing an additional 48 hours/year of overflowing. In terms of the additional volume of runoff escaping as a result of excessive infiltration the effect is much less.

The problem of infiltration involves two basic areas of concern: (1) prevention in new sewers by adequate design, construction, inspection, and testing practices, and (2) the elimination or cure of existing infiltration by proper survey, investigation, and corrective measures. Control of

infiltration in new sewer systems involves engineering decisions and specification of the methods and materials of sewer construction, pipe, joints, and laying procedures and techniques. Correction of existing sewer infiltration can be accomplished by three basic approaches: (1) replacing the defective component, (2) sealing the existing openings, and (3) building within the existing component.

In order to conduct a sewer system evaluation or a system analysis, the following approach is taken:

(1) Identify the scope and nature of the infiltration/inflow problem.

(2) Establish an end objective. (The amount of infiltration/inflow that can be economically eliminated).

(3) Isolate those general sections of the sewer system where infiltration/inflow is occurring.

(4) Formulate a plan which can be economically justified for investigating and locating specific areas from which the major infiltration/inflow emanates.

(5) Formulate a plan which will assure correction of the infiltration/inflow problem along with alternatives for rehabilitation and a prediction of the end results that can be expected.

The costs for an evaluation survey for the determination of excessive infiltration and inflow is summarized in the following table.

TABLE G-15

EVALUATION SURVEY COST ESTIMATES (49)

Phase	Estimated costs per foot for specific areas of sewer system, in 1974 dollars, $
Physical survey	0.15-0.25
Rainfall simulation	0.25-0.50
Preparatory cleaning	0.30-0.90
Internal inspection	0.40-0.70

Based on the findings of survey and the engineers' reports, a decision is made as to which sewers will not need any work, which sewers can be successfully grouted, relined, or replaced to eliminate the sources of infiltration/inflow, and the amount of street repair that will be required.

The correction alternatives include (1) replacement of broken sections, (2) insertion of various types of sleeves or liners, (3) internal sealing of joints and cracks with gels or slurries, and (4) external sealing by soil injection grouting. Additional detailed information is available in recent EPA reports on jointing materials (47,50) and sealants (47,50,51,52).

G.4.2.5.3 Rehabilitation Costs

Rehabilitation costs for any one area is site specific. Generalized estimates, however, may be calculated based on some unit costs for certain classes of repair. The following cost information has been adopted from EPA Report No. 430/9-75-021 (53).

G.4.2.5.3.1 Sewer Replacement Costs

The cost curve for replacing the existing gravity sewer with a new pipe of the same size is shown in Figure G-13. Included in the costs are the costs for site preparation, excavation, backfill, pavement, pipe materials, removal of existing pipe, pipe installation, reconnection of one house service connection for every 20 feet of pipe and field inspection. In deriving the cost curve, it was assumed that the depth of cover over the crown of the pipe is 9 feet, the pipe is laid in moderately wet soil conditions, the excavations are limited to earth excavations and the cost required to remove the existing pipe is 50% of that required to install the new pipe. The cost required for sewage bypassing during construction is not included.

For preliminary estimations, this curve should be sufficient in most applications. For more detailed cost estimations, individual costs should be developed based on the actual field conditions. Factors which may affect the cost for sewer replacement are shown in Table G-16.

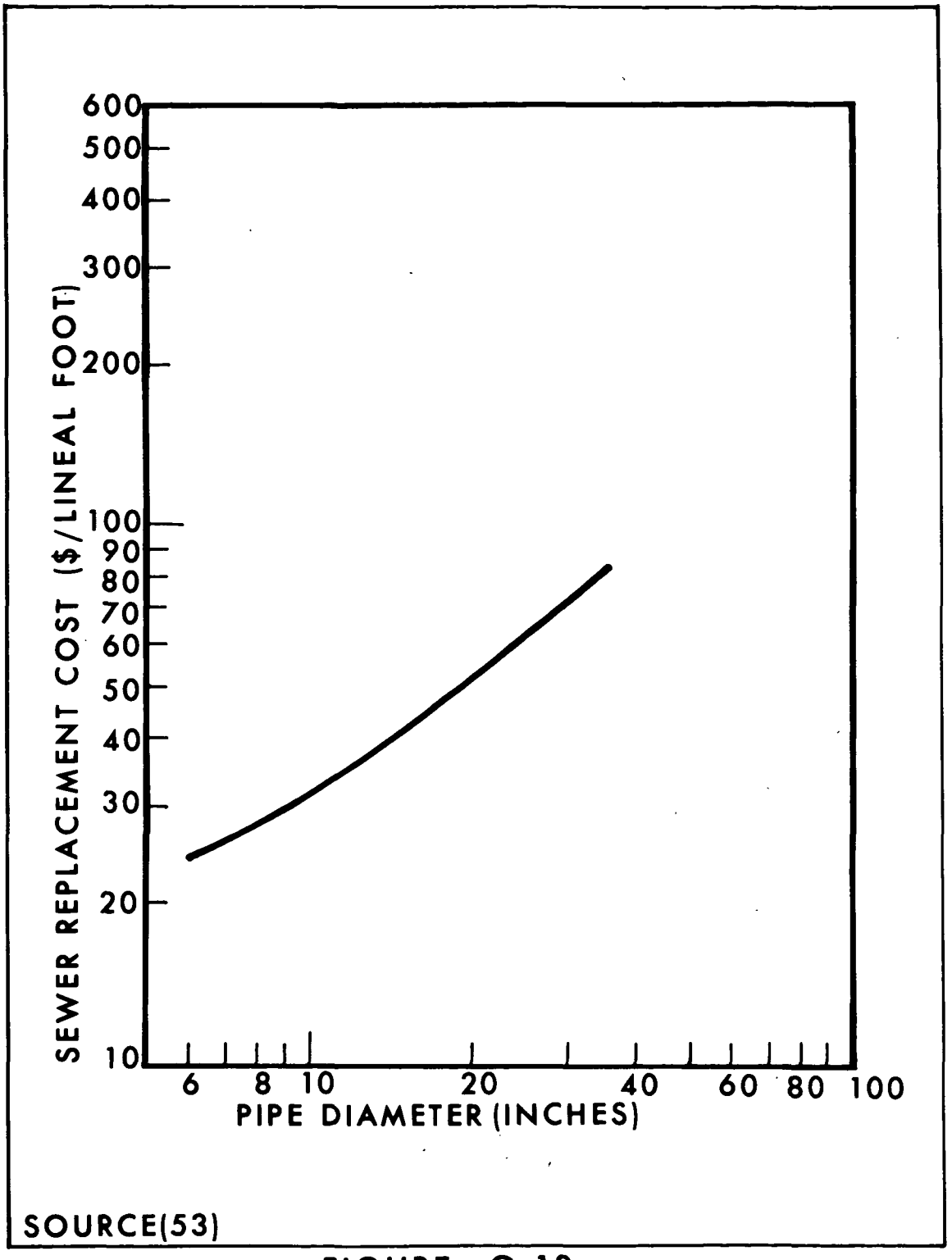


FIGURE G-13
SEWER REPLACEMENT COST VERSUS PIPE DIAMETER

TABLE G-16

SEWER REPLACEMENT COST CRITERIA (53)

Size of pipe
Depth of pipe
Type of service
Type of pipe
Removal of existing pipe
Number of service connections to be made
Groundwater elevation
Proximity to other utilities
Pipe transportation requirements
Infiltration allowance requirements
Access to site work
Availability of storage area for pipe materiald and equipment
Availability of storage area for excavated materials
Weather conditions
Availability and cost of labor

G.4.2.5.3.2 Pipe Lining (Polyethylene) Costs

The cost curve for pipe lining with polyethylene pipe is shown in Figure G-14. Included in the costs are the costs for site preparation, insertion pit, pipe materials, pipe welding, pipe installation, connection of one house service connection for every 20 feet of pipe, pipe sealing off in manholes and mobilization. It was assumed that the depth of cover over the crown of the sewer is 9 feet.

Factors which may affect the pipe lining cost are shown in Table G-17. They should be considered in developing more refined cost data.

G.4.2.5.3.3 Grouting Costs

The costs for chemical grouting of sewer pipes are shown in Figure G-15. The costs are developed based on the following assumptions (54):

(1) Length of manhole section: 300 feet

(2) Type of pipe: Vitrified clay

(3) Depth of flow: Less than 20% of pipe diameter

(4) Type of joint: Factory made

(5) Joint spacing: 4 feet

(6) Access to manholes: Readily accessible

(7) Manhole opening: 21 inches

(8) Manhole diameter: 4 feet

(9) Manhole condition: Structurally sound with steps for access

(10) Manhole depths: 6-8 feet

(11) Hazardous gas: None present

(12) Random vs. successive manhole sections: All sections requiring grouting are successive

(13) Mobilization distance: Within 100 miles

(14) Weather conditions: Mild temperature and no storm

(15) Traffic control: None required

(16) Chemical grout used: Acrylamide gel or urethane foam

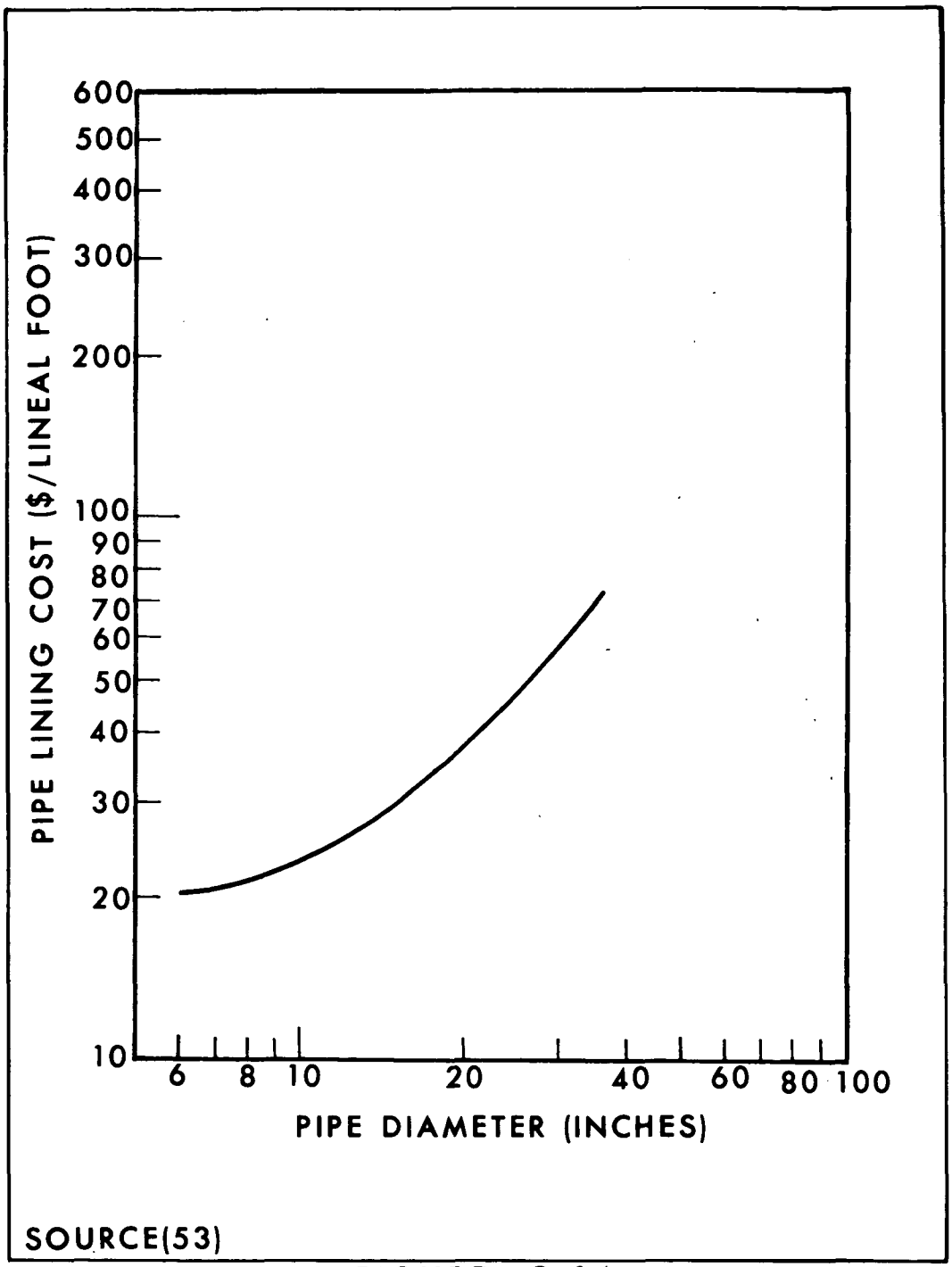


FIGURE G-14
PIPE LINING (POLYETHYLENE) COST
VERSUS PIPE DIAMETER

TABLE G-17

PIPE LINING COST CRITERIA (53)

Size of sewer
Length of sewer
Depth of sewer
Grade and direction change of sewer
Depth of flow in sewer
Size of liner pipe
Liner pipe wall thickness required
Annulus grouting requirements
Number of service connections to be made
Type of surface restoration required
Pipe transportation requirements
Type of manhole "seals" required
Extent of sewer cleaning required
Technique to "prove" or preinspect sewer lines
Excavation requirements
Groundwater elevation
Access to site of work
Availability of electrical power for fusing
Availability of storage area for pipe materials and equipment
Availability of storage area for excavated materials
Mobilization distance
Availability and cost of labor

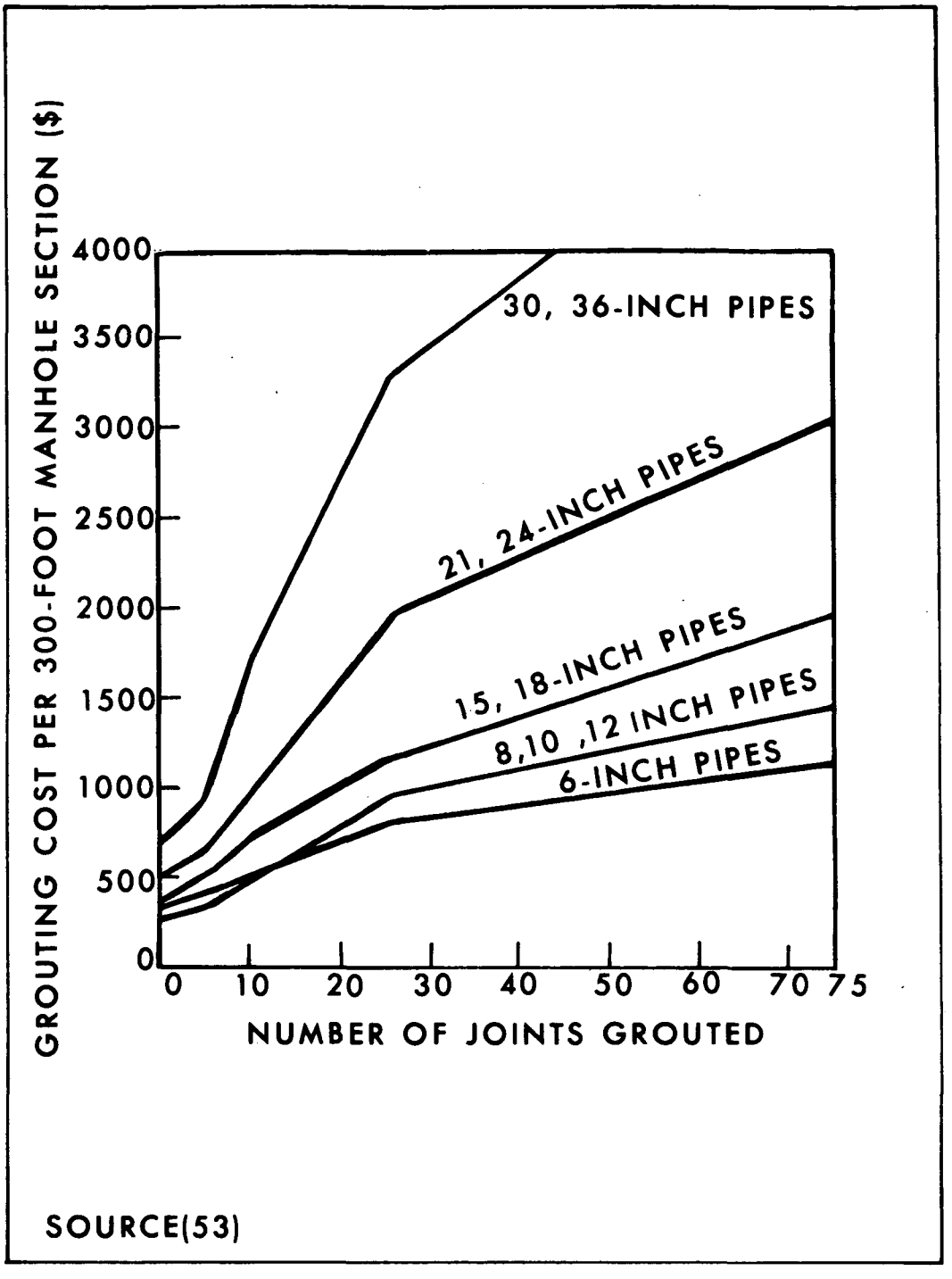


FIGURE G-15
GROUTING COST VERSUS NUMBER OF
PIPE JOINTS GROUTED

TABLE G-18

SEWER LINE GROUTING COST CRITERIA (53)

Mobilization distance
Weather condition
Terrain
Type of soil
Access to manholes
Manhole opening
Manhole size
Manhole cleanliness
Manhole depth
Hazardous gases in manhole
Type of pipe
Pipe size
Pipe alignment
Pipe grade
Pipe cleanliness
Depth of flow
Flow rate
Ability to plug
Type of joint
Joint spacing
Offset joints
Intruding service connections
Structurally damaged pipe
Random vs. successive manhole sections
Availability and cost of labor

G.4.2.6 Sewer Separation

In the 1967 APWA (7) report the cost of sewer separation was shown to vary enormously depending on whether partial or complete separation was being considered. The cost given for partial sewer separation averaged about $25-30 per foot of sewer in December 1976 dollars. Thus, the cost of installing sewers initially based on sewer lengths given in Figure G-2 ranges from about $2800-$5500/acre. Assuming the sewers have a life of 50 years and the interest reat is 6% installed, they will cost from $177 to $394/year. The cost to the homeowner for the house connection will be about one-third of the sewer cost in terms of dollars/capita. Thus, the cost of owning the sewerage system will range from $235/acre-year to $465/acre-year.

The cost of sewer separation in developed areas is much greater. For example, the cost of sewer separation in developed areas was given for several communities by Lager and Smith (2). The cost ranged from a low of about $4800 per acre in Bucyrus, Ohio, to a high of $150,400 per acre in Boston. Nationwide costs for sewer separation ranged from $9,250 to $35,580 per acre. Thus, it appears that sewer separation in highly developed urban areas is essentially impractical but installation of separate sewers at the time the land is developed is a feasible alternative for reducing pollution. At a population of 50,000 the annual cost of owning separate storm sewers costing $27.5 per linear foot is about $194/acre-year.

The pollution control potential for separate sewers can be estimated from the analysis of BOD load sources. About 30% of the annual load is associated with pollutants washed from urban areas and 70% of the load is caused by solids which settle in combined sewers to be washed into the receiving stream later by stormwater. If the load from separate storm sewers is 30 lb BOD/acre-year the corresponding load from combined sewers might be estimated at about 96 lb BOD/acre-year. Thus, by providing separate sewers the reduction in pollution load will be about 66 lb BOD/acre-year. Dividing $194/acre-year by the pollution load removed from the receiving stream (66 lb BOD/acre-year) the cost of BOD removal is about $2.94 per lb of BOD. This analysis applies only if the sewers are constructed in undeveloped land. In the developed area the cost could be greater by a factor of ten.

G.4.3 Loading Control

The potential impact of wet weather pollution discharges from combined sewers can also be limited through collection system management techniques aimed at controlling the quantity of pollutant available for discharge from the sewers.

The analyses of stormwater pollution sources in Section G-2 estimated that the annual BOD load from combined sewers ranges from 80-120 lb BOD/acre-year. Of this amount, 65% or 52-74 lb BOD/acre-year is attributable to municipal sewage solids which settle in sewers and are later flushed into receiving waters by storms. Thus, it is apparent that reasonably effective measures to limit the availability of this material for discharge with overflow will have a significant impact upon the magnitude of stormwater loads.

Three methods of load control will be discussed in this section; sewer flushing, sewer scraping, and catch basin cleaning. The cost effectiveness of structural sewer separation will also be discussed here to provide a cost contrast with the semistructural management approaches. Part II of this appendix also considers sewer flushing in the level I SWMM analysis of best combinations of storage/treatment, street sweeping and catch basin cleaning.

G.4.3.1 Sewer Flushing

Periodic sewer flushing has been proposed as one way of limiting the pollution from combined sewers. The goal of sewer flushing is to maintain the inventory of BOD exerting solids in the combined sewer system at a low level, so that, when storms of sufficient intensity to flush the solids from the sewer occur, the pollutional impact will be minimized. A number of flushing approaches are available, including flushing stations, in-line storage, and portable tanker units.

A flushing station technique developed by FMC (6) involves installation of a watertight bag in existing manholes with pump, valves, and controls to draw screened raw sewage from the sewer for storage in the inflatable bag. The screened sewage in the bag is released periodically with sufficient energy to flush settled solids from the lateral sewer downstream of the flushing station. In-line storage systems have been described by Pisano (55). They involve a system of internal dams which are employed to block the flow of sewage at critical upstream points for instantaneous release of sewage to

scour downstream segments without creating an artificial overflow. The tanker approach merely involves gravity or pressurized dumping of flush water into strategic manholes to provide scouring action.

Because each flushing wave becomes attenuated by wall friction and other internal pipe configuration flushing has limited usefulness beyond certain distances. It has been estimated that up to 1200 feet of small- and medium-diameter sewer can be flushed by a single flush station (56). FMC, in their study, concluded that from two to four flush stations are needed for every nine acres of urban land served by combined sewers. If the miles of sewer per acre is 0.025 (corresponding to an average community size of 170,000, Figure G-2) this would mean flush stations would have to be placed from 297 to 594 feet apart.

Recent studies in Boston, however, have developed a method for estimating pollutant loadings associated with dry weather sewage solids deposition in combined sewer systems (57). The predictive equations relate the total daily mass of pollutant deposition accumulations within a collection system to physical characteristics of collection systems such as per capita waste rate, service area, total pipe length, average pipe slope, average diameter and other more complicated parameters that derive from analysis of pipe slope characteristics. A comparative error analysis of the model for a test case collection system indicated a relative error in predicting solids deposition ranging from 8 to 18%. Suspended solids were also found through regression analyses to be a useful surrogate for BOD, COD, and nutrient forms. An important outcome of the work was the determination of the relationship between the distribution of solids deposition and cumulative pipe length in the system which is reproduced as Figure G-16. The curves reveal that between 80 and 90% of the deposition of suspended solids (and by association, BOD) occurs in only 50% of the system. These curves can be used in a preliminary fashion to estimate the total length of sewer which may have to be flushed.

The location of sewers which will likely benefit from flushing may then be approximated knowing the location of lines with average slopes less than those indicated. Obviously, other factors affect the location in severity

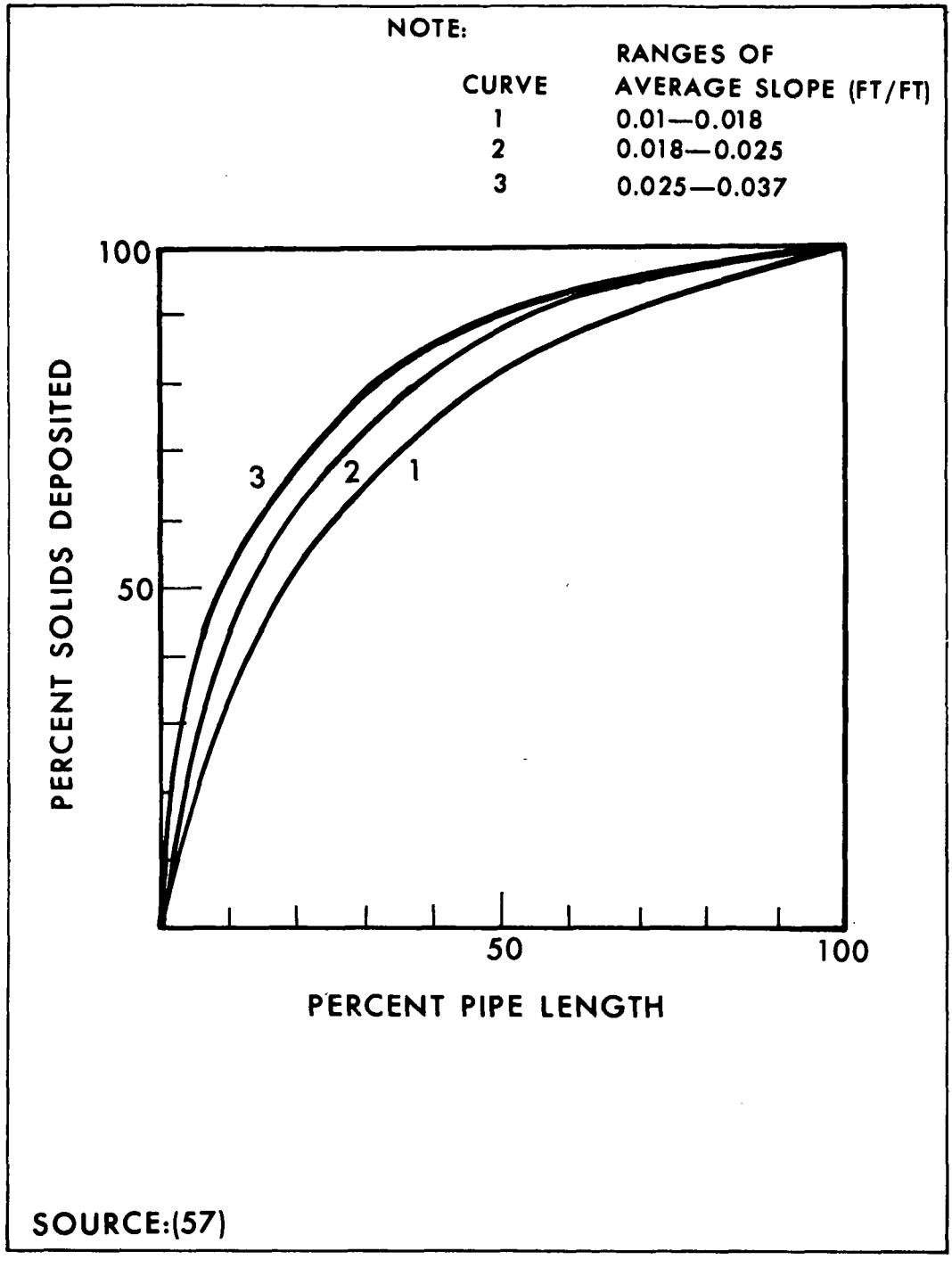


FIGURE G-16
CUMULATIVE DISTRIBUTION OF SOLIDS DEPOSITED VERSUS PIPE LENGTH

of deposits. The reader is referred to the study report to evaluate the specifics of flushing system design (57).

By calculation of daily mass balance in a sewer system the inventory of deposited solids after (n) days of daily flushing may be found to be represented by the following:

$$M_n = L(F_r + F_r^2 + F_r^3 + ...F^n) + M_o F_r \qquad \text{(G-32)}$$

where:

n = number of days

M_n = lbs of solids remaining

M_o = initial lbs of solids

F_r = fraction of solids remaining after each flush

L = daily solids accumulation (lbs/day)

For large values of (n) the last term in Equation (G-32) will approach zero. If the ratio M_n/L is then multiplied by F_r and subtracted from the expression for M_n/L it can be seen that for large values of n Equation (G-32) will approximate the following expression:

$$M_a = LF_r/(1 - F_r) \qquad \text{(G-33)}$$

M_a = mass of BOD stored in the sewers, lb

Flushing experiments conducted by FMC showed that the flushing efficiency ($1-F_r$) ranged from a minimum of 35% to a maximum of about 75% (6). The volumes of water stored in the flushing station ranged from 300 gallons to 900 gallons. The recommended frequency for sewer flushing was once-per-day.

If one applies a cleaning efficiency of 35% (F_r = 0.65) to Equation (G-33) the sewer solids inventory will stabilize (within 1%) at 1.86 times the daily accumulation. When the cleaning efficiency is 50% (F_r = 0.5) the inventory will stabilize within seven days at the daily accumulation and when cleaning efficiency is 75% the inventory will stabilize within four days at one-third the daily accumulation. Therefore, Equation (G-33) can be used in most cases to estimate solids inventory, but for short time intervals between storm events Equation (G-32) can be used to estimate the inventory washed out by

the stormwater. A key factor in estimating the efficacy of sewer flushing is the average time interval between rainfall events of sufficient intensity to cause overflow of the combined sewer. If the design capacity of the treatment plant is based on a production rate of 100 gpd/capita at a population density of 13.1 the size of the plant will be 0.00131 mgd/acre of urban area served. In the interceptor is sized at 3.5 times the treatment plant capacity the capacity of the interceptor 0.00459 mgd/acre. If the runoff coefficient for rate of runoff, as used in the Rational Method, had a value of 1.0 this threshold value of 0.00459 mgd per acre would be equivalent to a rainfall intensity of 0.007 inches per hour. However, it is known that at these low values of rainfall intensity the runoff coefficient is small and the relationship (Equation (G-14)) developed by Davis and Borchardt (39) between rainfall intensity and runoff coefficient shows that the rainfall intensity which will result in a runoff of 0.007 in/hr is near 0.007 in/hr. According to the distribution of hourly rainfall intensities given by Davis and Borchardt (39) rainfall exceeding 0.07 in/hr will occur during about 22% of the rain hours or about 108 hours/year. Thus, the average interval between rainfall intensities which are of sufficient intensity to cause overflow of the combined sewer is about 3.4 days.

In the FMC report (6) on sewer flushing the expected flushing efficiency was estimated at 60-75%. However, to be conservative the once-a-day flushing efficiency will be taken as 50% here. From Equation (G-32) the solids inventory will be 88% of the daily accumulation if the interval between storms is three days and 94% if the interval is four days. Thus, the inventory at an interval of 3.4 days can be estimated at 90.4% of the daily accumulation. The reduction in BOD reaching the stream can then be computed as (3.4 - 0.904)/3.4 or 73.4%. Since the load computed from settled solids was 52-79 lb BOD/acre-year, the BOD reduction is 38-58 lb BOD/acre-year.

A generalized cost of sewer flushing is difficult to estimate, because only limited cost data are available and the number of flushing points per acre is not known with a great degree of assurance. Based on the findings of the Boston study (57) it is likely that the number of flushing points in the system may be less than previously estimated, perhaps closer to the two stations per nine acres reported by FMC (6) rather than four per nine acres.

For the mechanized daily flushing system developed by FMC (6) the cost per unit (or cost per manhole) was estimated at $5500 in 1972 dollars. Thus, the capital cost per acre is about $1250 or approximately $2000 per acre in current dollars. Amortization over 20 years at 6% gives a debt service of $174/acre/year. FMC recommended an operation and maintenance allowance of $102 per month for the two unit per nine acre installation which converted to current dollars becomes roughly $217/acre/year. The total cost under these conditions is then $391/acre/year (debt service plus operation and maintenance). If the previously estimated BOD reduction with daily sewer flushing is used (35-58 lb BOD/acre-year) the cost of sewer flushing will range from $5.75-$10.30/lb BOD/removed.

Sewer flushing can also be done with manual labor. APWA (4) reports that in 1960 all of the sewers in Chicago were flushed approximately 3.2 times/year. The cost of flushing was estimated in 1960 dollars as $42 per mile of sewer flushed. Assuming 0.025 miles of sewer per acre, the cost of manually flushing all sewers once would be $1.05/acre in 1960 dollars or $2.65/acre in current dollars. The annual cost of daily flushing in current dollars would be $967/acre-year. This estimate is higher than the estimate for automatic sewer flushing but if the flushing was done every two days, the cost of manual flushing might be competitive with automatic flushing.

G.4.3.2 Catch Basin Cleaning

Catch basin cleaning is one of the most discussed nonstructural pollution control measures. The potential for pollution abatement by this method, however, is likely to be small.

Historically, the role of catch basins was to minimize sewer-clogging by trapping coarse debris and to reduce odor emanations from the sewers by providing a water seal. In early sewer systems, catch basins were important because of the number of impaired streets, the use of flat grades, inefficient means of sewer-cleaning, and low flows in the sewer systems. With improvements in street surfacing, attention to design for self-cleaning velocity in sewers, and the advent of street sweeping and improved sewer-cleaning techniques, their benefits have been reported as being marginal.

Despite the purported reduced need, catch basins remain very prevalent in many sewer systems. The APWA report (4) on street accumulation reported that Chicago had 3815 miles of sewer in 1960 with 200,000 catch basins. The AVCO report on stormwater pollution in Tulsa reported 330 miles of installed sewers with 22,000 catch basins (58). Thus, the average number of catch basins in Chicago was 52.4/mile of sewer in Chicago and 66.7/mile of sewer in Tulsa. Averaging these gives an estimate of 60 catch basins/mile.

The Metcalf and Eddy report on catch basin technology (59) quoted a 1928 survey of catch basin dimensions conducted by American City Magazine. Ninety-six American cities, the District of Columbia, and four Canadian cities participated in the survey. The average catch basin volume available for storage of solids and water was 1.45 cu. yds. The APWA report of street accumulation quoted catch basins cleaning statistics for the City of Chicago in the period 1946-1960. The average amount of material removed from each catch basin was 0.7 cu yd/yr. The cleaning frequency was once or twice per year. Thus, it appears that when catch basins are cleaned about once a year, the available volume is about 50% water and 50% solids. The solid material is wet with a density of about 110 lb per cu. ft.

The best data on the amount of pollution contained in uncleaned catch basins is given by Sartor and Boyd (9). Twelve uncleaned catch basins in San Francisco were completely stirred and the BOD concentration of the slurry was measured. The BOD concentrations ranged from 5 mg/l to 1500 mg/l with an average of 241 mg/l. The amount of water in a single catch basin just before annual cleaning is about 1 cu. yd. Using the average BOD concentration of 241 mg/l, the uncleaned catch basin will contain about 0.4 lb of BOD. Assuming 60 catch basins per mile of sewer and 0.025 miles of sewer per acre, this is equivalent to 0.61 lb BOD per acre. Assuming that the catch basins are cleaned annually, the amount of pollution removed in the cleaning process will be 0.61 lb BOD/acre-year. This constitutes only about 2% of the previously estimated 25-35 lb BOD/acre/year for separate storm sewers. Part II of this appendix also contains an analysis of catch basin effectiveness based on an assessment by Lager and Smith (2). The analysis reveals that for twice-per-year catch basin cleaning the expected

level of BOD removal from stormwater discharges would be slightly greater than 6%.

Sartor and Boyd (9) also flushed uncleaned catch basins with clean water at rates corresponding to storms with intensities ranging from 0.07-0.7 in/hr. The catch basins contained from 2000-3000 lb of solids. It was found that only about 1% of the solids were removed by the flushing effect. Thus, it appears that solids which accumulate in catch basins which are primarily inorganic are not readily flushed out by rainfall events.

The potential for reducing the amount of pollution which reaches the stream from separate storm sewers by means of catch basin cleaning is apparently small. The amount of inorganic solids which can be intercepted in catch basins, however, is a much larger percentage of the total solids loading. For example, the APWA report on street accumulation (4) estimated the amount of dust-and-dirt on the streets as 1.5 lb/day/100 ft of curb. Assuming the sewer length equals the street length and applying the factor of four mentioned previously, the amount of dust-and-dirt believed to be washed into the storm sewer is about 231,000 l/yr per mile of sewer. Annual cleaning catch basins in Chicago yielded about 0.7 cu. yd. of material which was 60% solids. Assuming 60 catch basins per mile, this is equivalent to a removal of about 75,000 lb. of solids per mile of sewer annually. This represents about 32% of the estimated annual load of dust-and-dirt.

Some cost information is available for cleaning of catch basins. The cost of catch basin cleaning in Chicago was given by APWA as $3.50 per catch basin cleaned in 1960 dollars. Catch basin cleaning costs in Buffalo, New York ranged from $4.59 in 1966-67 to $7.61 per catch basin in 1970-71 (60). The Chicago costs adjusted to December, 1976 become $8.80 per catch basin, and the Buffalo equivalent cost is $11.23. Sartor and Boyd (9) surveyed catch basin cleaning practice in seven cities and found that the cost of catch basin cleaning (1971) was $15.00 for hand-cleaning and about $7 per catch basin for mechanical cleaning. An average cost of $10/catch basin will be used here.

Lager and Smith (59) reported the results of an APWA survey of 299 cities which showed an average cleaning frequency of 2.3 times/year. Of the seven

cities surveyed by Sartor and Boyd (9) two reported a frequency of 2.3 times/year, two at a frequency of once a year, one at six times/year, and the remainder "as required".

Assuming a cleaning frequency of 2.3 times/year, 60 catch basins per mile, 0.025 miles of street/acre, and a cost of $10 per cleaning gives an overall cost for catch basin cleaning of $34.50/care-year. Divising the cost per acre-year by the estimated 0.61 lb BOD/acre-year gives the cost of BOD removal at $56.50 per lb of BOD removed. Therefore, it would appear that not only is the potential effectiveness of catch basin cleaning small but the cost is high.

REFERENCES

1. Heaney, Huber, Medina, Murphy, Nix, (University of Florida), "Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Volume II: Cost Assessment and Impacts", U.S. Environmentap Protection Agency, EPA-600/2-77-064, March 1977.

2. Lager, J.A., and Smith, W.G., "Urban Stormwater Management and Technology: An Assessment," U.S. EPA Report EPA-670/2-74-040, NTIS-PB 240 687, December 1974.

3. Smith, Robert, "Cost to the Consumer for Collection and Treatment of Wastewater:, U.S. Environmental Protection Agency, EPA 17090-07/70, July 1970.

4. American Public Works Association, Federal Water Pollution Control Administration, "Water Pollution Aspects of Urban Runoff", WP-20-15, January 1969.

5. Bartholomew, Harland, "Land Uses in American Cities", Harvard University Press, 1955.

6. Central Engineering Laboratories, FMC Corporation, U.S. Environmental Protection Agency, "A Flushing System for Combined Sewer Cleaning", EPA 11020 DNO 03/72, March 1972.

7. American Public Works Association, Federal Water Pollution Control Administration, "Problems of Combined Sewer Facilities and Overflows 1967", WP-20-11, December 1967.

8. Smith, Robert, "The Use of the Log-Normal Distribution for Rainfall Data, Internal EPA Memorandum to Record, February 10, 1977.

9. Sartor, J.D., and Boyd, G.B., "Water Pollution Aspects of Street Surface Contaminants", U.S. Environmental Protection Agency, EPA-R2-72-081, November 1972.

10. Sartor, J.D., Boyd, G.B., and Agardy, F.J., "Water Pollution Aspects of Street Surface Contaminants", Presented at the 45th Annual Conference of the Water Pollution Control Federation, Atlanta, Georgia, October 1972.

11. Morris, R.H., et al., "Water Pollution Abatement Technology-Capabilities and Costs: Urban Runoff", National Commission on Water Quality, NTIS Report PB-247-391, December 1975.

12. 1975 Annual Report, City of Cincinnati, Department of Public Works, Highway Maintenance Division.

13. Field, R., Tafuri, A.N., and Masters, H.E., "Urban Runoff Pollution Control Technology Overview", U.S. EPA Report 600/2-77-047, March 1977.

14. Ports, M.A., Sediment and Erosion Control Design Criteria", APWA Reporter, May 1975.

15. Midwest Research Institute and Hittman Associates, Inc., "Methods for Identifying and Evaluating the Nature and Extent of Nonpoint Sources of Pollutants", U.S. Environmental Protection Agency, EPA-430/9-73-014.

16. Agricultural Statistics 1973, U.S. Department of Agriculture.

17. Hittmann Associates, Inc., "Processes, Procedures, and Methods to Control Pollution Resulting from all Construction Activity", U.S. Environmental Protection Agency, EPA 430/9-73-007.

18. Engineering Science, Inc., "Comparative Costs of Erosion and Sediment Control, Construction Activities", U.S. Environmental Protection Agency, EPA-490/9-73-016.

19. Dow Chemical Company, "An Economic Analysis of Erosion and Sediment Control Methods for Watersheds Undergoing Urbanization", U.S. Department of the Interior (C-1677), February 1972.

20. Soil Conservation Service National Engineering Handbook; Section 4, Hydrology, August 1972.

21. Poertner, H.G., "Practices in Detention of Urban Stormwater Runoff, An Investigation of Concepts, Techniques, Applications, Costs, Problems, Legislation, Legal Aspects, and Opinions", APWA, Special Report No. 43, 1974.

22. Patterson, W.L., and Banker, R.F., "Estimating Cost and Manpower Requirements for Conventional Wastewater Treatment Facilities", U.S. Environmental Protection Agency, 17090 DAN 10/71, October 1971.

23. Tholin, A.L., and Keifer, C.J., "Hydrology of Urban Runoff", Trans. American Society of Civil Engineering Paper No. 3061.

24. Toubler, J., and Westmacott, R., "Water Resources Protection Measures in Land Development - A Handbook", Delaware University, Water Resources Center, Newark, Delaware, NTIS #PB-236-049, April 1974.

25. Thelen, Edmund, et al., "Investigation of Porous Pavements for Urban Runoff Control", Franklin Institute Research Labs., U.S. Environmental Protection Agency, 11034 DUY, March 1972.

26. "208 Areawide Wastewater Management Plan - Fifth Planning District Commission, Roanoke, Virginia, Moore, Gardner, and Associates, Inc., July 1976.

27. U.S. EPA, "Water Quality Management Planning for Urban Runoff", Report No. EPA 440/9-75-004, December 1974.

28. Murray, D.M., and Ernst, U.F.W., "An Economic Analysis of the Environmental Impact of Highway Deicing", U.S. Environmental Protection Agency, EPA-600/2-76-105, May 1976.

29. Field, R., Struzeski, E.J., Jr., Masters, H.E., and Tafuri, A.N., "Water Pollution and Associated Effects from Street Salting", U.S. Environmental Protection Agency, EPA-R2-73-257, May 1973.

30. Struzeski, E., "Environmental Impact of Highway Deicing", U.S. Environmental Protection Agency, 11040 GKK, June 1971.

31. Brenner, R., and Moshman, J., "Benefits and Costs in the Use of Salt to Deice Highways", The Institute for Safety Analysis, The Salt Institute, November 1976.

32. Johnson, R.E., et al., "Dustfall as a Source of Water Quality Impairment", Hour. Sanitary Engineering, Div. ASCE Vol 92 Sal p 245, February 1966.

33. Draft Report, EPA Contract No. 6802-1473, Industrial Gas Cleaning Institute, Stamford, Connecticut, 1977.

34. McKee, J.E., "Loss of Sanitary Sewage through Storm Water Overflows", Jour. Boston Soc. Civil Engineers, 34:2, April 1947.

35. Palmer, C.L., "The Pollutional Effects of Storm-Water Overflows from Combined Sewers", Sewage and Industrial Wastes, 22:2, 154, 1950.

36. Shifrin, W.G., and Horner, W.W., "Effectiveness of the Interception of Sewage-Storm Water Mixtures", JWPCF, 33:6, 650, 1961.

37. Johnson, F.C., "Equipment, Methods, and Results from Washington, D.C. Combined Sewer Overflow Studies", JWPCE, 33:7, 721, 1961.

38. Benjes, H.H., et al., "Storm-Water Overflows from Combined Sewers", JWPCF, 33:13, 1252, 1961.

39. Davis, P., and Borchardt, J., "Combined Sewer Overflow Abatement Plan, Des Moines, Iowa, "U.S. EPA Report No. EPA-R2-73-170, April 1974.

40. Leiser, C.P., "Computer Management of a Combined Sewer System", U.S. EPA, Report No. EPA-670/2-74-022, July 1974.

41. "Combined Sewer Regulation and Management: A Manual of Practice", American Public Works Association, 11022 DMU, Environmental Protection Agency, July 1970.

42. "The Swirl Concentrator as a Combined Sewer Overflow Regulator Facility", American Public Works Association, EPA-R2-72-008, Environmental Protection Agency, September 1972.

43. Field, R., "The Dual Functioning Swirl Combined Sewer Overflow Regulator/ Concentrator", EPA-670/2-73-059, Edison Water Quality Research Laboratory, Environmental Protection Agency, September 1973.

44. Metcalf & Eddy, Inc., "Wastewater Engineering : Collection, Treatment, Disposal, New Yrok, McGraw-Hill Book Company, 1972.

45. American Public Works Association, U.S. Environmental Protection Agency, "Relationship Between Diameter and Height for the Design of a Swirl Concentrator as a Combined Sewer Overflow Regulator", EPA-670/2-74-039, July 1974.

46. Benjes, H.H., "Cost Estimating Manual - Combined Sewer Overflow Storage and Treatment", U.S. Environmental Protection Agency, Report No. EPA-600/2-76-386, December 1976.

47. "Prevention and Correction of Excessive Infiltration and Inflow into Sewer Systems: A Manual of Practice", American Public Works Association, 11022 EFF, Environmental Protection Agency, January 1971.

48. Santry, I.W., "Sewer Maintenance Costs", J. Water Pollution Control Federation, 44:7, 1425, 1972.

49. Cesareo, D.J., and Field, R., "Infiltration-Inflow Analysis", Jour. Env. Engineering Division, ASCE, 101:5, 775, October 1975.

50. "Heat Shrinkable Tubing as Sewer Pipe Joints", The Western Company, 11024 FLY, Environmental Protection Agency, June 1971.

51. "Improved Sealants for Infiltration Control", The Western Company, 11020 DIH, Environmental Protection Agency, June 1969.

52. "Ground Water Infiltration and Internal Sealing of Sanitary Sewers", Montgomery County Sanitary Department, Montgomery, Ohio, 11020 DHQ, Environmental Protection Agency, EPA-430/9-75-021, December 1975.

53. "Handbook for Sewer System Evaluation and Rehabilitation", U.S. Environmental Protection Agency, EPA-430/9-75-021, December 1975.

54. "Preliminary Report for a Manual of Practice," The Sewer Rehabilitation Subcommittee, Technical Advisory Committee, American Public Works Association, 1975.

55. Pisano, W.C., in "Proceedings, Urban Stormwater Management Seminars", EPA Report WPD 03-76-04, January 1976.

56. Metcalf and Eddy, "Best Management Practices for Nonstructural Solutions to Stormwater Pollution Control", draft report EPA Contract No. 68-03-2437, December 1976.

57. Pisano, W.C., and Queiroz, C.S., "Procedures for Estimating Dry Weather Pollutant Deposition in Sewerage Systems", draft report, EPA Grant No. R804579, May 1977.

58. AVCO Economic Systems Corporation, Federal Water Quality Administration, "Storm Water Pollution From Urban Land Activity", 11034 FKL 07/70, July 1970.

59. Lager, J.A., and Smith, W.F., Metcalf and Eddy, Inc., "Catchbasin Technology Overview and Assessment", Unpublished report on EPA Contract 68-03-0274.

60. Buffalo Sewer Authority Annual Report 1970-71.

APPENDIX G, PART II

EPA-600/2-77-083
April 1977

STORM WATER MANAGEMENT MODEL: LEVEL I--COMPARATIVE EVALUATION OF STORAGE-TREATMENT AND OTHER MANAGEMENT PRACTICES

by

James P. Heaney
Stephan J. Nix
Department of Environmental Engineering Sciences
University of Florida
Gainesville, Florida 32611

Grant No. R-802411

Project Officers

Richard Field
Storm and Combined Sewer Section
Wastewater Research Division
Municipal Environmental Research Laboratory(Cinti.)
Edison, New Jersey 08817

Dennis Athayde
Urban Runoff Program
Non-Point Source Branch
Water Planning Division
Washington, D.C. 20460

WATER PLANNING DIVISION
OFFICE OF WATER AND HAZARDOUS MATERIALS
US ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, D.C. 20460

and

MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
US ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

DISCLAIMER

This report has been reviewed by the Municipal Environmental Research Laboratory and the Water Planning Division, Office of Water and Hazardous Materials, US Environmental Protection Agency, and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the US Environmental Protection Agency, nor does any mention of trade names or commercial products constitute endorsement or recommendation for use.

FOREWORD

The US Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solving and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communication link between the researcher and the user community.

Combined sewer overflows and urban stormwater discharges are a significant pollution source. This report describes simplified procedures to enable decision makers to obtain a preliminary estimate of the magnitude of this pollution source and the associated costs of control.

Francis T. Mayo, Director
Municipal Environmental Research Laboratory

Ned Notzen, Acting Director
Water Planning Division

PREFACE

This report is part of a series of documents on urban stormwater management which provides analysts with a wide variety of tools for evaluating alternatives ranging from simple desktop procedures as outlined in this report (Level I analysis) to sophisticated computer-based simulation using the original Storm Water Management Model (Level IV analysis). The companion document to this simplified procedure for comparing other management practices with storage-treatment options would be very useful in supplementing this report. The other report is titled:

> Heaney, J.P., W.C. Huber, and S.J. Nix, Storm Water Management Model: Level I--Preliminary Screening Procedures, EPA-600/2-76-275, Environmental Protection Technology Series, USEPA, 1976.

ABSTRACT

The original USEPA Storm Water Management Model (SWMM) provides a detailed simulation of the quantity and quality of stormwater during a specified precipitation event lasting a few hours. This model is widely used. However, it is too detailed for many purposes. Indeed, a wide range of evaluation techniques ranging from simple to complex procedures are needed. In particular, the 208 planning effort needs simplified procedures to permit preliminary screening of alternatives. In response to this need, four levels of stormwater management models are being prepared. This volume presents a "desktop" procedure to compare selected alternative control technologies.

A graphical procedure is described which permits the analyst to examine a wide variety of control options operating in series with one another or in parallel. The final result is presented as a control cost function for the entire study area which is the optimal (least costly) way of attaining any desired level of control. Given a specification regarding the desired overall level of control the user can determine the appropriate amount of each control to apply.

This methodology is applied to Anytown, U.S.A., a hypothetical community of 1,000,000 people. The results indicate the mix of treatment, storage, street sweeping, and sewer flushing which attains the specified pollution control level at a minimum cost.

This report is submitted as part of Grant No. R-802411 by the University of Florida under sponsorship of the US Environmental Protection Agency. Work was completed in December 1976.

TABLE OF CONTENTS

		Page
FOREWORD		iii
PREFACE		iv
ABSTRACT		v
LIST OF FIGURES		ix
LIST OF TABLES		xi
LIST OF SYMBOLS		xii
ACKNOWLEDGMENTS		xv
SECTION		
I	SUMMARY	1
	General Theory and Methodology	1
	Control Technologies	2
	Application to Anytown, U.S.A.	2
II	RECOMMENDATIONS	6
III	INTRODUCTION	7
IV	208 PLANNING AREAS	9
V	GENERAL THEORY AND METHODOLOGY	12
	Theory	12
	Marginal Analysis	12
	Production Theory	13
	Methodology	13
VI	CONTROL TECHNOLOGIES	26
	Street Sweeping	26
	Combined Sewer Flushing	36
	Catch-basin Cleaning	41
	Storage-Treatment	41

TABLE OF CONTENTS (CONCLUDED)

SECTION

VII APPLICATION TO ANYTOWN, U.S.A. 46
Problem Statement 46
Application 48

REFERENCES 65

GLOSSARY 67

APPENDICES

A. Quantity and Quality Analysis 69

B. Working Curves for Application to Anytown, U.S.A. 73

LIST OF FIGURES

FIGURE		Page
1	PRODUCTION FUNCTIONS.	14
2	GENERALIZED STORMWATER POLLUTION CONTROL NETWORK.	16
3	GRAPHICAL PROCEDURE FOR DETERMINING OPTIMAL CONTROL STRATEGIES, STEPS 1 AND 2	17
4	GRAPHICAL PROCEDURE FOR DETERMINING OPTIMAL CONTROL STRATEGIES, STEP 3.	20
5	GRAPHICAL PROCEDURE FOR DETERMINING OPTIMAL CONTROL STRATEGIES, STEPS 4 AND 5	22
6	GRAPHICAL PROCEDURE FOR DETERMINING OPTIMAL CONTROL STRATEGIES, STEPS 5 (CONCLUDED), 6 AND 7.	24
7	STORMWATER POLLUTION CONTROL TECHNOLOGIES - AVAILABILITY FACTORS EQUAL 1.0	27
8	IMPERVIOUSNESS AS A FUNCTION OF DEVELOPED POPULATION DENSITY	33
9	SWEEPING AVAILABILITY FACTOR AS A FUNCTION OF DEVELOPED POPULATION DENSITY.	33
10	PRODUCTION FUNCTIONS FOR STREET SWEEPING.	34
11	PRODUCTION FUNCTION FOR COMBINED SEWER FLUSHING	38
12	PRODUCTION FUNCTION (ISOQUANTS) FOR STORAGE-TREATMENT, ATLANTA, GEORGIA.	44
13	TOTAL COST CURVE FOR STORAGE-TREATMENT, STORM SEWERED AREAS, ATLANTA, GEORGIA	45
14	STORMWATER POLLUTION CONTROL NETWORK FOR ANYTOWN, U.S.A.- AVAILABILITY FACTORS EQUAL 1.0	49
15	MARGINAL COST CURVES FOR THE PARALLEL OPTIONS, STORM AREAS - - MEDIUM AVAILABILITY FACTORS	51

LIST OF FIGURES (CONCLUDED)

FIGURE

16	TOTAL COST CURVES FOR THE PARALLEL OPTIONS, STORM AREAS . . .	52
17	TOTAL COST CURVE FOR STORAGE-TREATMENT, STORM AREAS	54
18	ISOQUANTS OF THE OVERALL FRACTION OF BOD REMOVED, STORM AREAS - - HIGH AVAILABILITY FACTORS	55
19	ISOQUANTS OF THE OVERALL FRACTION OF BOD REMOVED, STORM AREAS - - MEDIUM AVAILABILITY FACTORS	56
20	ISOQUANTS OF THE OVERALL FRACTION OF BOD REMOVED, STORM AREAS - - LOW AVAILABILITY FACTORS	57
21	TOTAL COST CURVE FOR ALL OPTIONS, STORM AREAS - - HIGH AVAILABILITY FACTORS	58
22	TOTAL COST CURVE FOR ALL OPTIONS, STORM AREAS - - MEDIUM AVAILABILITY FACTORS	59
23	TOTAL COST CURVE FOR ALL OPTIONS, STORM AREAS - - LOW AVAILABILITY FACTORS	60
24	TOTAL COST CURVES FOR ALL DRAINAGE SYSTEM SERVICE AREAS . .	64

LIST OF TABLES

TABLE		Page
1	ANNUAL COST OF OPTIMAL STRATEGY FOR ANYTOWN, U.S.A., PRESENTED BY TYPE OF SEWERAGE SYSTEM - MEDIUM AVAILABILITY FACTORS . .	4
2	ANNUAL COST OF OPTIMAL CONTROL STRATEGY FOR ANYTOWN, U.S.A., PRESENTED BY TYPE OF CONTROL TECHNOLOGY FOR DIFFERENT ASSUMED AVAILABILITY FACTORS	5
3	COMPARISON OF ANNUAL COST OF OPTIMAL CONTROL STRATEGY FOR ANYTOWN, U.S.A. USING STORAGE-TREATMENT ALONE AND IN COMBINATION WITH OTHER MANAGEMENT PRACTICES.	5
4	PERCENT OF STREET POLLUTANTS IN VARIOUS PARTICLE SIZE RANGES	29
5	BRUSH-TYPE SWEEPER EFFICIENCY FOR VARIOUS PARTICLE SIZE RANGES	30
6	AVERAGE VALUES OF GUTTER LENGTH.	37
7	UNIT COSTS OF STREET SWEEPING.	37
8	EXAMPLE PROBLEM EVALUATING CATCH-BASIN PERFORMANCE	42
9	LAND USE AND POPULATION CHARACTERISTICS OF ANYTOWN, U.S.A.	47
10	ANNUAL WET- AND DRY-WEATHER FLOWS AND BOD LOADS FOR ANYTOWN, U.S.A.	50
11	OPTIMAL STRATEGY FOR 25, 50, 75, AND 85 PERCENT OVERALL BOD REMOVAL, ANYTOWN, U.S.A. - HIGH AVAILABILITY FACTORS	61
12	OPTIMAL STRATEGY FOR 25, 50, 75, AND 85 PERCENT OVERALL BOD REMOVAL, ANYTOWN, U.S.A. - MEDIUM AVAILABILITY FACTORS . . .	62
13	OPTIMAL STRATEGY FOR 25, 50, 75, AND 85 PERCENT OVERALL BOD REMOVAL, ANYTOWN, U.S.A. - LOW AVAILABILITY FACTORS	63

LIST OF SYMBOLS

A_p	Area served by option p, ac
A_{SF}	Area served by combined sewers to be flushed, ac
A_{SW}	Area to be swept, ac
AR	Annual runoff, in/yr
$\alpha(i,j)$	Coefficient for storm and unsewered areas for pollutant j on land use i, lb/ac-yr-in
$\beta(i,j)$	Coefficient for combined areas for pollutant j on land use i, lb/ac-yr-in
C_p	Cost per unit of effort, $\$/X_p$-yr
C_{SF}	Cost per mile of sewer flushed, $/mile
C_{SW}	Cost per curb mile swept, $/curb-mile
CF_p	Total cost function for option p
DD	Daily dust and dirt accumulation rate, lb/day
DS	Annual depression storage, in/yr
DWF	Dry-weather flow, in/yr
ϵ	"Pick-up" efficiency of the street sweeping equipment
F	Pounds of pollutant per pound of dust and dirt
$f_i(PD_d)$	Population density function for land use i
G	Gutter density, curb-miles/ac
I	Total imperviousness, percent
I_S	Imperviousness due to streets only, percent
M_{DEP}	Combined sewer deposition pollutant (BOD) load, lb/yr

M_P	Pollutant load in area served by option p, lb/yr
M'_p	Pollutant load available to option p, lb/yr
M_{II}	Pollutant load available to all parallel options , lb/yr
M_W	Wet-weather pollutant (BOD) load, lb/yr
MC_p	Marginal cost per pound of pollutant removed by option p, $/lb
MC_{II}	Composite marginal cost per pound of pollutant removed by the parallel options , $/lb
MF_p	Marginal cost function for option p
MF_{II}	Composite marginal cost function for the parallel options
m_{Dw}	Annual dry-weather BOD load, lb/ac-yr
m_{DEP}	BOD load of combined sewer deposition, lb/ac-yr
m_p	Unit pollutant load in area served by option p, lb/acre-yr
m_W	Annual wet-weather pollutant (BOD) load, lb/ac-yr
N_D	Number of dry days since the last storm
N_S	Number of days between street sweepings
n	Number of times the streets were swept since the last storm
P	Annual precipitation, in
P_1	Total pollutant at the beginning of the storm, lb
P_o	Pollutant remaining at the end of the last storm, lb
PD_d	Population density in the developed area, persons/ac
PF_p	Production function for option p
ϕ_p	Fraction of pollutant load available to option p ($0 \leq \phi_p \leq 1.0$)
ϕ_{SF}	Fraction of wet-weather BOD load available for flushing ($0 \leq \phi_{SF} \leq 1.0$)
ϕ_{SW}	Fraction of wet-weather BOD load available for sweeping ($0 \leq \phi_{SW} \leq 1.0$)

W_E	Net pollutant (BOD) discharge, lb/yr
W_p	Pollutant removal by option p, lb/yr
W_{II}	Pollutant removal by the parallel options , lb/yr
W_{SF}	BOD load in deposition removed by daily sewer flushing, lb/yr
W_{ST}	Wet-weather BOD removed by sweeping, lb/yr
$\overline{X}$	Input vector
X_p	Level of effort for process p ($0 \leq X_p \leq 1.0$)
X_{SF}	Fraction of combined sewerage system components flushed daily ($0 \leq X_{SF} \leq 1.0$)
X_{ST}	Input vector to storage-treatment option ($0 \leq X_{ST} \leq 1.0$)
X_{SW}	Fraction of days per year an area is swept ($0 \leq X_{SW} \leq 1.0$)
$\overline{Y}$	Output vector
Y_p	Fraction of available pollutant load removed by option p ($0 \leq Y_p \leq 1.0$)
Y_{II}	Fraction of pollutant removed by the parallel options ($0 \leq Y_{II} \leq 1.0$)
Y_ψ	Fraction of pollutant removed by the serial operation ($0 \leq Y_\psi \leq 1.0$)
Y_{SF}	Fraction of available BOD removed by flushing ($0 \leq Y_{SF} \leq 1.0$)
Y_{ST}	Fraction of BOD removed by storage-treatment ($0 \leq Y_{ST} \leq 1.0$)
Y_{SW}	Fraction of available BOD removed by sweeping ($0 \leq Y_{SW} \leq 1.0$)
Z_p	Total cost for process p, $/yr
Z_{II}	Composite total cost of the parallel options , $/yr
Z_ψ	Composite total cost of the serial operation, $/yr
Z_{SF}	Total cost of combined sewer flushing, $/yr
Z_{ST}	Total cost of storage-treatment, $/yr
Z_{SW}	Total cost of sweeping, $/yr

ACKNOWLEDGMENTS

Numerous individuals were very helpful in formulating and conducting specific phases of this study. Dennis Athayde, Richard Field, and Pat Waldo of USEPA provided many valuable suggestions and overall review. Dr. William Pisano of Energy and Environmental Analysis, Inc. provided information regarding their sewer flushing studies in Boston. George Hinkle and Richard Sullivan of the American Public Works Association provided data on street sweeping. John Lager of Metcalf and Eddy, Inc. provided information regarding catch-basin cleaning. Dr. Wayne C. Huber, University of Florida, reviewed an earlier draft of this document.

SECTION I

SUMMARY

Analysis of wet-weather pollution control alternatives is much more complicated than the traditional dry-weather sewage problem due to the highly variable flow and the much broader range of options to be evaluated. The highly variable nature of the flows requires statistical characterization of the properties of the runoff hydrographs and pollutographs using averaging times ranging from a single storm event to an annual series. The range of control options has been extended from examining only storage and treatment devices to inclusion of other management practices, e.g., street sweeping, sewer flushing, catch-basin cleaning. These units operate in series and/or in parallel with one another.

This report provides a simplified methodology for evaluating these other management practices in conjunction with storage-treatment options. A graphical solution technique is used to evaluate wet-weather control alternatives for Anytown, U.S.A., a typical U.S. city of 1,000,000 people. The results demonstrate the technique and provide a preliminary indication regarding the relative competitiveness of the various control options.

GENERAL THEORY AND METHODOLOGY

The optimal combination of storage-treatment devices and other management practices for wet-weather pollution control can be determined using marginal analysis from economic theory, and a graphical solution procedure. Marginal analysis indicates that more intensive use should be made of control alternatives with lower marginal costs, measured in dollars per pound of pollutant removed. As these activities are expanded, marginal costs increase to the point where other options become competitive. The entire analysis can be viewed as determining, at any specified marginal cost, the quantity of pollution which the various control options, in parallel, would offer to control. These results, for all options in parallel, are combined to yield a composite control cost curve. Then this composite option is evaluated with the downstream option(s) in series with it to yield the final result. The solution is guaranteed to be optimal because every option produces a diminishing marginal value of pollution control as its level of effort is expanded. For example, if sewer flushing is to be used as a control alternative the initial monies will be spent where it is most effective, e.g., cleaning the pipes with the heaviest deposition rate. As more money is spent, controls would be used on progressively cleaner sections of pipe. Thus, the pollution control effectiveness, per dollar invested, would

decrease. Also, constant unit control costs are assumed. As a consequence, marginal costs increase thereby guaranteeing that the control cost functions are convex and the resulting graphical solution is the optimal one.

CONTROL TECHNOLOGIES

For the purposes of this study, four technologies were considered: street sweeping, combined sewer flushing, catch-basin cleaning, and storage-treatment. Only combined sewered areas utilize all four technologies. Storm sewered areas do not require sewer flushing. In addition to flushing, unsewered areas do not use catch-basin cleaning or street sweeping if it is assumed that there are no gutters.

For street sweeping, an overall BOD removal efficiency of 0.5 is assumed. The assumed unit cost is $7.00/curb-mile swept ($4.35/curb-km). The performance of sweepers was estimated, for varying sweeping intervals and removal efficiencies, using a continuous simulation of one year of data for Minneapolis. A modified version of the street sweeping procedure described in SWMM was used.

Data on combined sewer flushing were obtained from studies in Boston, Mass. The results of these efforts indicated that a relatively small percentage of the pipes retain a substantial amount of the total deposition. The assumed annual costs of flushing per unit length of sewer line is $11.78/ft ($38.64/m). Daily flushing is assumed to remove 100 percent of the BOD deposited in the affected pipes.

Catch basins were found to be relatively ineffective as a wet-weather pollution control device due to their relatively small size in relation to the contributing drainage area. Thus, they were not investigated further.

The procedure for evaluating storage-treatment technologies was presented in our earlier work. Thus, this control technology was not discussed in detail.

APPLICATION TO ANYTOWN, USA

The methodology was applied to a hypothetical urbanized area, called Anytown, which has characteristics typical of the 248 urbanized areas in the US as listed below:

(1) Population Density (urbanized area): 5.14 persons/ac (12.70 persons/ha)

(2) Mean Annual Precipitation: 33.4 in (84.8 cm)

(3) Land Use Percentage (urbanized area): residential, 31.4%; commercial, 4.6%; industrial, 8.0%; other developed, 9.8%; undeveloped, 46.2%.

(4) Land Use Percentage (developed areas only): residential, 58.4%; commercial, 14.8%; industrial, 8.6%; other developed, 18.2%.

(5) Percent of Developed Area Served by Type of Drainage System: combined sewers, 14.4%; storm sewers, 38.3%; unsewered, 47.3%.

(6) Population Density of the Developed Area by Type of Drainage System--person/ac (persons/ha): combined, 16.7 (41.3); storm, 13.0 (32.1); unsewered, 4.6 (11.4); all developed areas, 9.6 (23.7).

The results of this analysis, presented by type of sewerage system, are shown in Table 1. Although the combined sewered area comprises less than 15 percent of the land area, about 40 percent of the total costs are incurred for this area because the loadings are higher and it is more cost-effective to control this portion of the total load.

A breakdown of total control costs, by type of technology and assumed availablility factors, is presented in Table 2. For the medium availability factors, storage-treatment is used for about 80 percent of total control. As expected, sweeping and flushing gain in relative importance as the availability factors increase. This effect is most pronounced for street sweeping. This type of sensitivity analysis is quite helpful in providing an indication of the importance of reliable estimates of the availability factors.

Lastly, the significance of the savings resulting from using management practices other than storage-treatment are evaluated in Table 3. The results indicate savings (relative to using storage-treatment only) of 6 percent, 21 percent, or 37 percent for 50 percent control for low, medium, and high availability factors, respectively. These results definitely indicate the need to evaluate all available control options in area-wide wastewater management planning.

TABLE 1. ANNUAL COST OF OPTIMAL STRATEGY FOR ANYTOWN, U.S.A., PRESENTED BY TYPE OF SEWERAGE SYSTEM - MEDIUM AVAILABILITY FACTORS

Type of System	Acreage[a] ac(ha)	Total Annual Cost ($ x 10^6/yr) for Indicated % BOD Control			
		25%	50%	75%	85%
Combined	15,100 (6,110)	0.48	1.65	4.50	7.86
Storm	40,100 (16,230)	0.12	0.82	3.56	7.43
Unsewered	49,500 (20,030)	0.56	1.42	2.82	2.82
Total	104,700 (42,370)	1.16	3.89	10.88	18.11

[a]Anytown has a population = 1,000,000

TABLE 2. ANNUAL COST OF OPTIMAL CONTROL STRATEGY FOR ANYTOWN, U.S.A., PRESENTED BY TYPE OF CONTROL TECHNOLOGY FOR DIFFERENT ASSUMED AVAILABILITY FACTORS.

Type of Control Technology	Total Annual Cost ($ x 10^6/yr) for Indicated % BOD Control and Assumed Availability Factors											
	25%			50%			75%			85%		
	Low	Med	High	Low	Med	High	Low	Med	High	Low	Med	High
Sweeping	0	0.16	0.24	0.21	0.59	0.88	0.56	1.45	2.84	0.94	2.58	3.80
Flushing	0.14	0.14	0.18	0.17	0.21	0.28	0.26	0.57	0.60	1.01	0.69	0.82
Storage-Treatment	1.18	0.86	0.43	4.25	3.09	1.95	11.50	8.86	5.25	19.31	14.84	9.72
TOTAL	1.32	1.16	0.85	4.63	3.89	3.11	12.32	10.88	8.69	21.26	18.11	14.34

TABLE 3. COMPARISON OF ANNUAL COST OF OPTIMAL CONTROL STRATEGY FOR ANYTOWN, U.S.A., USING STORAGE-TREATMENT ALONE AND IN COMBINATION WITH OTHER MANAGEMENT PRACTICES.

%BOD Control	Annual Cost ($ x 10^6/yr)				% Savings Over Storage-Treatment Only		
	Storage-Treatment (S-T) Only	Storage-Treatment and Other Options					
		Low	Medium	High	Low	Medium	High
25	1.47	1.32	1.16	0.85	10	21	42
50	4.95	4.63	3.89	3.11	6	21	37
75	13.39	12.32	10.88	8.69	8	19	35
85	22.42	21.26	18.11	14.34	5	19	36

SECTION II

RECOMMENDATIONS

This simplified methodology for evaluating urban stormwater pollution control alternatives is intended to serve as a preliminary screening device. It requires neither a computer nor an understanding of more refined analytical solution procedures. After the user understands the concepts and procedures, he may wish to substitute the appropriate analytical procedures using derived functions.

The results indicate significant savings if other management options are combined with storage-treatment options. Further savings can be realized by recognizing that a significant portion of the control costs can be assigned to other purposes.

SECTION III

INTRODUCTION

In recent years there has been the realization that stormwater from urban areas is a serious water pollution source. Abatement of this source will require a monumental effort in research, development, and implementation. In this study, a simple methodology is developed which provides a "first-cut" evaluation of the problem and the optimal control strategy.

Several technologies are available to control stormwater pollution. At present, emphasis is placed on storage-treatment control techniques [1]. However, other techniques are available, e.g., street sweeping, sewer flushing. These methods, used in conjunction with storage-treatment, may provide a more cost-effective pollution management package [2, 3, 4, 5, 6]. The resultant optimal mix of all control options is often referred to as "Best Management Practice" or BMP's.

With the potential control effectiveness of options other than storage-treatment established, the need has arisen for a methodology capable of determining, on a "first-cut" basis, the most cost-effective usage of these other options in conjunction with (or exclusive of) storage-treatment in the urbanized area. "First-cut" or preliminary analyses establish the magnitude of the problem and rapidly evaluate alternatives. This study derives a relatively simple methodology to obtain this "first-cut."

Several analytical techniques which can provide an optimal "mix" of control alternatives are available. Many require the use of computerized algorithms which defeat the need for simplicity. Nearly all require an accurate knowledge of the functional form of empirically derived relationships. A simple methodology was developed by Heaney, Huber, and Nix [7] but was limited to storage-treatment as a control alternative.

A graphical technique is chosen to provide a preliminary estimate of an optimal stormwater pollution control strategy. Graphical solution techniques do have drawbacks. They are relatively time consuming and more susceptible to human error. Nevertheless, there are definite advantages. Computational aides are not necessary and complex analytical procedures are avoided.

The next section presents a generalized description of a typical 208 planning area. Section V describes the procedure used to obtain an optimal strategy along with the economic theories used to derive the methodology. This procedure is applicable to a wide variety of stormwater pollution

control networks. Section VI discusses the various control technologies and develops the production functions and cost equations necessary for the methodology of Section V. Section VII is an application of the methodology to a hypothetical urban area known as Anytown, USA. Anytown is given the characteristics found for urbanized areas around the nation [1]. The optimal integrated control package is determined for this hypothetical situation. Appendix A presents a simplified method for estimating wet-weather quantity and quality. Equations to estimate dry-weather quantity and quality are also given for comparative purposes. Lastly, working curves for the application to Anytown are placed in Appendix B.

SECTION IV

208 PLANNING AREAS

The Federal Water Pollution Control Act Amendments of 1972 (PL92-500) are a comprehensive piece of legislation designed to implement a procedure by which virtually all sources of pollution to the nation's waters are to be eliminated and the purity of these waters restored [8]. The pollutants are discharged from both urban and rural areas and from point and nonpoint sources. Several goals were set forth by the Act:

(1) that the discharge of pollutants into navigable waters be eliminated by 1985;

(2) that a level of water quality be attained by July 1, 1983, that provides for the protection of aquatic life, wildlife, and recreation; and

(3) that areawide water quality management planning processes be developed and utilized.

Other provisions include funding for the necessary research and to aid in the implementation of management plans.

Section 208 of the Act sets overall guidelines for the development of areawide planning processes. The US Environmental Protection Agency, designated to carry out the intent of the Act, has published specific guidelines to aid local authorities in attaining the overall goals [9]. These guidelines state that the 208 planning procedure should proceed along the following lines:

(1) Identify the problems in meeting the 1983 goals of the Act.

(2) Identify all constraints and priorities pertaining to the 208 planning area.

(3) Identify all possible solutions to the problems.

(4) Develop alternative plans to meet the statutory requirements.

(5) Analyze the alternative plans for technologic and economic feasibility.

(6) Select an areawide plan.

(7) Seek approval for the plan.

(8) Periodically update the plan.

The selection of a specific plan should be based on cost effectiveness, feasibility, and public acceptance.

An important portion of the selected plan should be involved with the control of stormwater pollutant discharges. EPA guidelines specifically state the need for "an analysis of the magnitude of existing and anticipated urban stormwater problems" [9]. Additionally, techniques to better manage the existing drainage systems, thus preventing discharges at the source, and/or improved methods for the storage and treatment of urban runoff, should be developed.

Areawide management is conducted at the local level. In general, areawide plans should be developed for a region relatively homogeneous in its wastewater problems and ultimate discharge locations. Such an area will include one or several urbanized areas which are of primary concern to this study. In order to conduct an analysis of stormwater discharges from these areas and potential control strategies, a comprehensive inventory of the characteristics of each should be available. For the preliminary or "first-cut" analysis conducted in this study, these characteristics should include land use, sewerage system service areas and population served by each, and the mean annual precipitation.

A definition of an urbanized area is needed to properly delineate the areas of potential urban stormwater discharge. The U.S. Census describes an urbanized area as follows [10]:

(1) A central city (or adjacent cities) of 50,000 or more inhabitants.

(2) Settled areas in close proximity to the central city, including the following:

   a. Incorporated areas of 2,500 or more inhabitants or less than 2,500 if the area includes 100 or more closely settled housing units.

   b. Small parcels of land with a population density greater than 1,000 (386) inhabitants per square mile (km).

   c. Other small parcels of unincorporated land with less than the required population density that eliminate enclaves.

With this definition, the local planner can divide the urbanized acreage among five land use categories: residential, commercial, industrial, other developed (parks, institutions, etc.), and undeveloped. The definition of

an urbanized area allows for the inclusion of large areas of undeveloped lands not likely to be developed in the planning future. These areas should not be included in the following analysis. Additionally, the planner should delineate the area and population served by combined and storm drainage systems and unsewered areas within the remaining developed (or developing) area. With these data, the following graphical procedures may be applied to provide a "first-cut" evaluation of the urbanized area's stormwater problem and optimal control strategy.

SECTION V

GENERAL THEORY AND METHODOLOGY

This section presents the economic theories and general methodology necessary to determine an optimal stormwater pollution control strategy. The discussion is based heavily on production theory and marginal analysis from economics.

THEORY

Marginal Analysis

In its simplest terms, marginal may be defined as "extra." In economic terms, for example, marginal cost is defined as the extra cost associated with an additional unit of some commodity. In economic decision making marginal analysis determines whether an action results in a sufficient additional benefit to justify the additional cost.

Two basic rules governing the concept of marginal analysis are [11]:

(1) The scale of an activity should, if possible, be expanded so long as its marginal net yield (taking into account both benefits and costs) is a positive value; and the activity should therefore be carried to a point where this marginal net yield is zero.

(2) For optimal results, activities should, whenever possible, be carried to levels where they all yield the same marginal returns per unit of effort (cost).

As an example of rule (2), assume that product A, at a specific production level, is yielding $1.50 per $1.00 spent and product B is returning $2.00 per $1.00 spent. In this situation the firm is missing the opportunity to gain $0.50 by not transferring the $1.00 spent to manufacture product A to product B. Therefore, to assure the maximum return, both products should be manufactured at levels of equivalent marginal return or yield.

In stormwater pollution control these same concepts apply. Analysts should seek, in such cases, to utilize various control procedures at levels yielding an equivalent marginal cost.

Production Theory

A production process seeks to increase the utility of a commodity or commodities. In any such process certain technological relationships restrict the decision maker's options on input and output levels [12]. Consider an input vector to a production process, $\overline{X}$, defined as

$$\overline{X} = (X_1, X_2, \ldots, X_i, \ldots, X_n). \quad (1)$$

Similarly, the output vector, $\overline{Y}$, is defined as

$$\overline{Y} = (Y_1, Y_2, \ldots, Y_j, \ldots, Y_m). \quad (2)$$

The technological relationship between the input and output vectors, known as the production function, is

$$\overline{Y} = PF\ (\overline{X}) \quad (3)$$

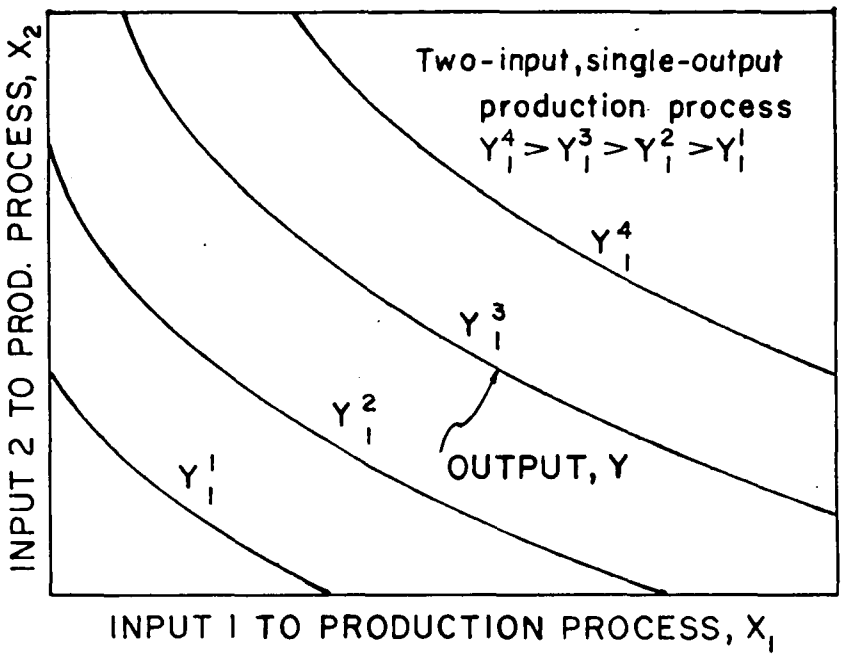
where $\overline{Y}$ is the maximum output attainable with input vector $\overline{X}$. In other words, any output Y_j may not be increased without a reduction in some other output Y_k or an increase in some input X_i. Examples of production functions are shown in Figure 1. The single-input, single-output production function shown may be viewed as a two-input, single-output function with one input held constant.

The shape of the production function is governed by the "law" of diminishing returns which states that, as an input to a production process is increased, with all other inputs held constant, a point will be reached beyond which any additional input will yield diminishing marginal output. For example, if a treatment plant experiences increases in raw sewage flow and no alterations are made to the facility, a flow will be reached where an increment in flow will result in a diminishing increase of pollutant removed.

METHODOLOGY

In this study, a stormwater pollution control option is defined as a unique set of conditions and control technologies. For example, although a particular control technology, such as street sweeping, may be used in several different subareas within an urbanized area, those subareas may have varying pollutant loading rates which affect the cost-effectiveness of the common technology. Also, within a particular subarea there may be several distinct pollutant sources requiring different control technologies.

Knowing the production function for each stormwater pollution control technology (production process) and with the control options defined, it is possible to graphically determine an optimal strategy. In the discussion that follows all production functions have been transformed into a single-input, single-output form and expressed in terms of the fraction of available pollutant removed, Y, as a function of the fraction of the level of effort, X. The definition of level of effort is dependent on the particular technology. For example, the level of effort for street sweeping is defined as the fraction of days during a year when sweeping occurs. All production functions and later functions or graphs are derived on an annual basis.



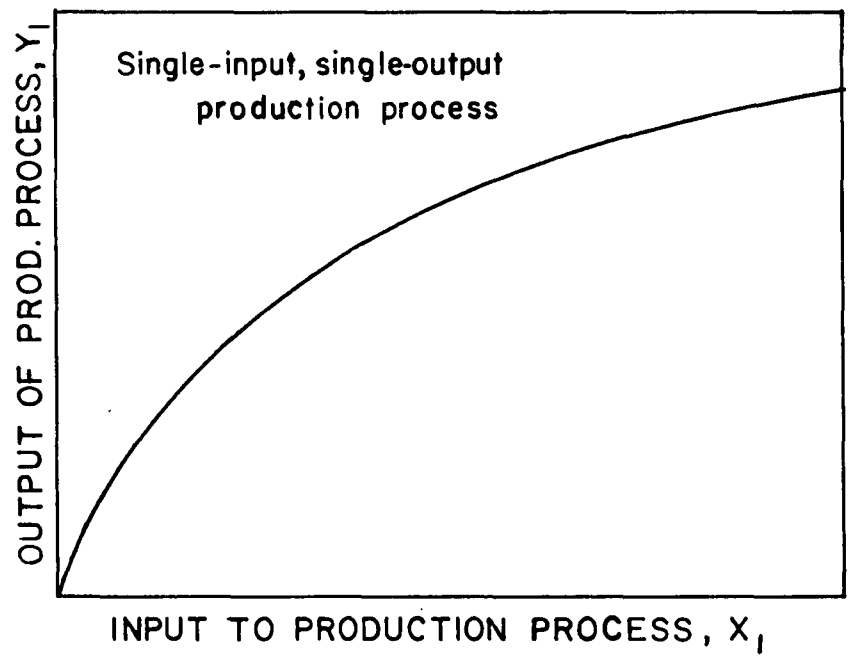


Figure 1. Production Functions

Before delving into the methodology a few more definitions are required.

In stormwater pollution control, options may operate in parallel, series, or a combination of both. A parallel operation is defined as one in which the effluent (untreated portion) of any one option does not act as the influent to any other parallel option. A serial operation is defined as one in which options are sequential with the effluent from one option acting as the influent to the next.

A network of series/parallel pollution control options is shown in Figure 2. In this example four options (p = 1, 2, 3, and 4) operate in parallel followed by one option (p = 5) operating in series with the parallel group. The pollutant flows through this network are shown in terms of the pollutant load in the area served by the parallel options, M_p, the fraction of the pollutant load available to option p, ϕ_p, and the pounds removed by each option p, W_p. The pollutant load available to each option, M_p', is the product of ϕ_p and M_p. The pollutant load M_4' is shown as the influent to an imaginary option (p = 4) that has zero pollutant removal capacity. This simply allows the residual pollutant loads to be routed to option 5 without passing through the other parallel options. For example, street sweeping does not reach the entire surface pollutant load of an area. Therefore, some portion may be washed off by runoff events and routed to a storage-treatment facility without having the opportunity to be removed by sweeping. The influent to option 5 is the pollutant load not removed by the parallel group. This network will serve as an example and reference throughout the remainder of this section.

Once the production functions are established, the first step is to construct the total cost curve for each option (see Step 1, Figure 3). Production functions for several specific pollution control technologies and methods to develop the total cost curves are discussed in Section VI. However, for the purposes of generalization, a total cost curve is defined as a function of the fraction of pollutant removed, i.e.,

$$Z_p = CF_p(Y_p) \qquad (4)$$

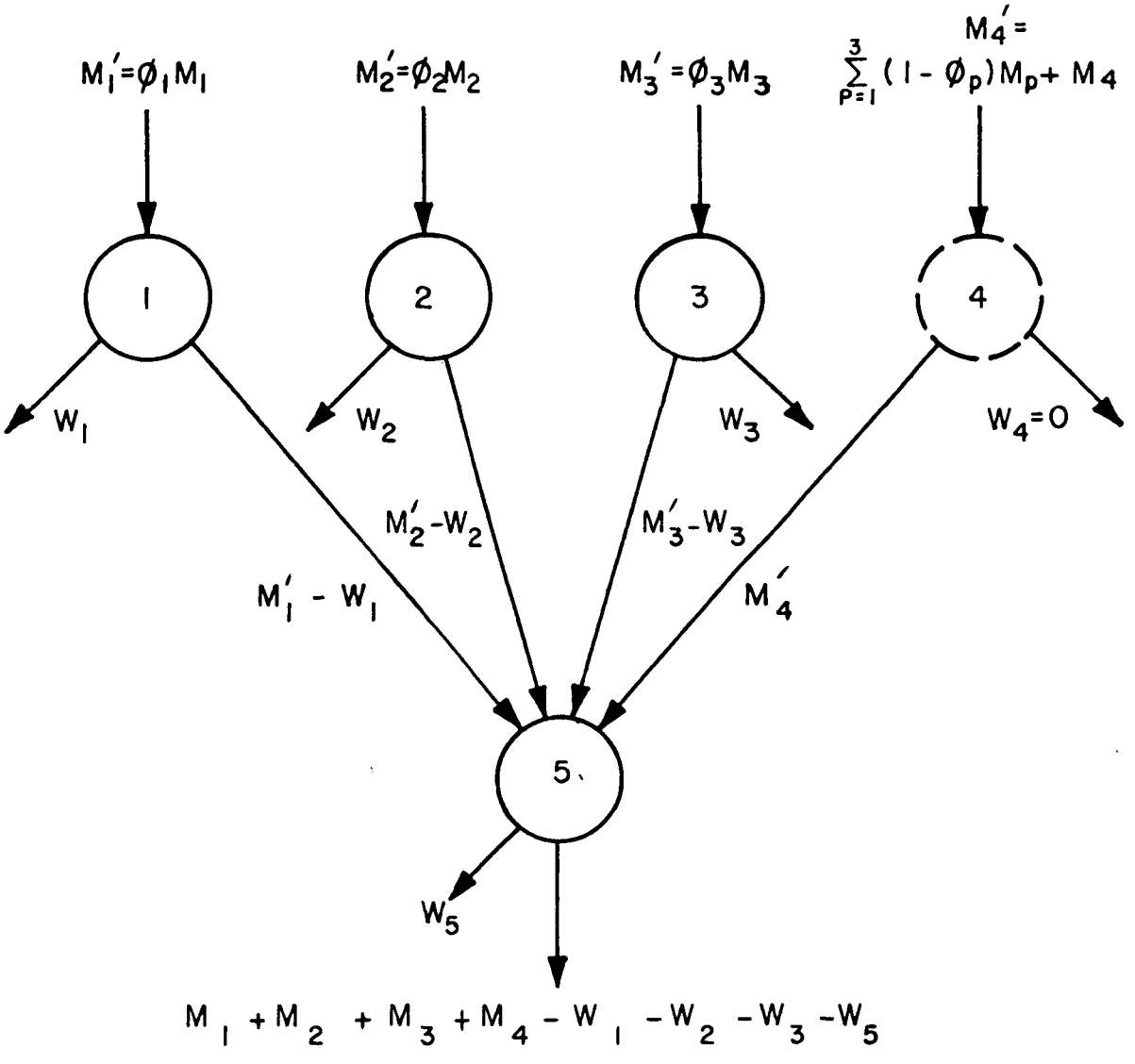
where Z_p = total cost for option p, \$/yr;

Y_p = fraction of available pollutant load removed by option p ($0 \leq Y_p \leq 1.0$); and

$CF_p(Y_p)$ = total cost function in terms of Y_p.

Recall that the fraction of available pollutant removed is the dependent variable of the production function. Thus, to derive the total cost curve, one only needs to reverse the axes of the production function and develop the relationship between the level of effort for option p, and the total cost. Mathematically, this may be stated as

$$X_p = PF_p^{-1}(Y_p) \qquad (5)$$



LEGEND

p = CONTROL PROCESS

M_p' = AVAILABLE POLLUTANT LOAD, lb/yr

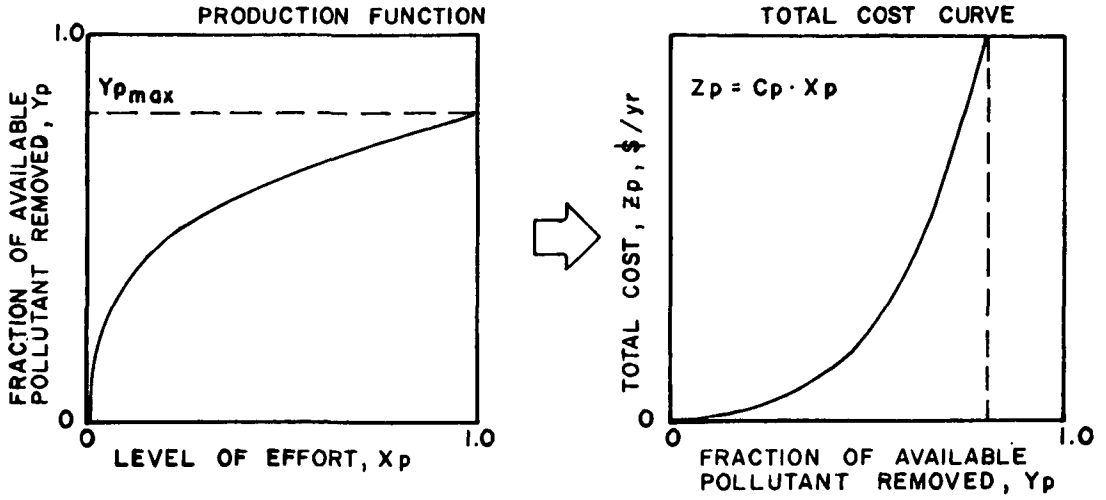
ϕ_p = AVAILABILITY FACTOR

M_p = POLLUTANT LOAD, lb/yr

W_p = POLLUTANT REMOVAL, lb/yr

Figure 2. Generalized Stormwater Pollution Control Network

STEP 1 : FIND TOTAL COST CURVE FOR EACH OPTION (p=1,2,3,4,and 5)



STEP 2 : FIND MARGINAL COST CURVE FOR EACH PARALLEL OPTION (p= 1,2,3, and 4)

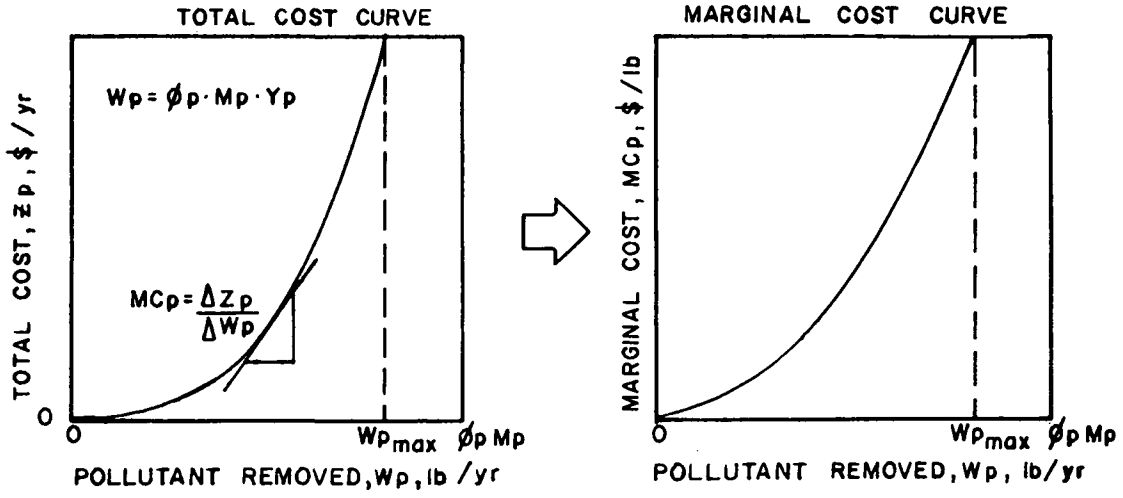


Figure 3. Graphical Procedure for Determining Optimal Control Strategies, Steps 1 and 2

where X_p = level of effort for option p ($0 \leq X_p \leq 1.0$);

Y_p = fraction of available pollutant load removed by option p ($0 \leq Y_p \leq 1.0$); and

$PF_p^{-1}(Y_p)$ = inverse of the production function for option p.

If total cost is assumed to be a linear function of the level of effort, then

$$Z_p = C_p \cdot X_p \tag{6}$$

where C_p = annual cost of option p per unit of effort, $\$/X_p$.

Substituting equation (5) into equation (6) yields

$$Z_p = C_p \cdot PF_p^{-1}(Y_p). \tag{7}$$

Equation (7) is the desired form of the total cost function (equation 4).

The next step is to generate the marginal cost curve for each parallel option (see Step 2, Figure 3). This curve gives the relationship between the marginal cost per pound of pollutant removed at any level of pollutant removed. The marginal cost curve is the first derivative of the total cost curve. However, the total cost curves for the parallel options must be converted from the fraction of pollutant removed to pounds removed. This is accomplished using the following equation,

$$W_p = M_p' \cdot Y_p \tag{8}$$

where W_p = pollutant removal by option p, lb/yr;

M_p' = pollutant load available to option p, lb/yr; and

Y_p = fraction of available pollutant load removed by option p ($0 \leq Y_p \leq 1.0$).

The maximum value of W_p, $W_{P_{max}}$, depends on the maximum removal efficiency of option p, $Y_{P_{max}}$. The equation used to find M_p' is

$$M_p' = \phi_p \cdot M_p \tag{9}$$

where ϕ_p = fraction of pollutant load (M_p) available to option p ($0 \leq \phi_p \leq 1.$); and

M_p = pollutant load in area served by option p, lb/yr, and $M_p = m_p \cdot A_p$;

where m_p = unit pollutant load in area served by option p, lb/ac-yr; and

A_p = area contributing pollutants to option p, ac.

The pollutant load per acre, m_p, can be found using methods described in Appendix A. The normalized version of the total cost curve (equation 4) may now be written as

$$Z_p = CF_p(W_p) \tag{10}$$

since only the units of the abscissa of the total cost curve are being changed. Utilizing equation (10), the marginal cost curve is described by the following equation,

$$MC_p = \frac{d(CF_p(W_p))}{dW_p} = MF_p(W_p) \tag{11}$$

where MC_p = marginal cost per pound of pollutant removed by parallel option p, \$/lb; and

$MF_p(W_p)$ = the marginal cost function in terms of W_p.

Graphically, the marginal cost curve is determined by finding the slope, $\Delta Z_p/\Delta W_p$, of the total cost curve at several values of W_p. These values, plotted against the values of W_p, give an approximation of the marginal cost curve. The marginal cost curves are increasing functions of pounds of pollutant removed. This result necessarily follows from the earlier assumptions of a concave production function and constant unit costs.

Once the marginal cost curves are developed for the parallel options, a composite marginal cost curve may be constructed (see Step 3, Figure 4). This single curve summarizes the effect of the entire parallel group (p = 1, 2, 3, and 4). This is accomplished by adding the marginal cost curves with respect to the ordinate of the marginal cost curves. In other words, at several equivalent values of MC_p for the parallel options, the corresponding W_p's are summed. The composite marginal cost curve is

$$MC_{II} = MF_{II}(W_{II}) \tag{12}$$

where MC_{II} = composite marginal cost per pound of pollutant removed by the parallel options, \$/lb;

W_{II} = pollutant removal by the parallel options, lb/yr, (= $W_1 + W_2 + W_3$, in the example network); and

$MF_{II}(W_{II})$ = the composite marginal cost function in terms of W_{II}.

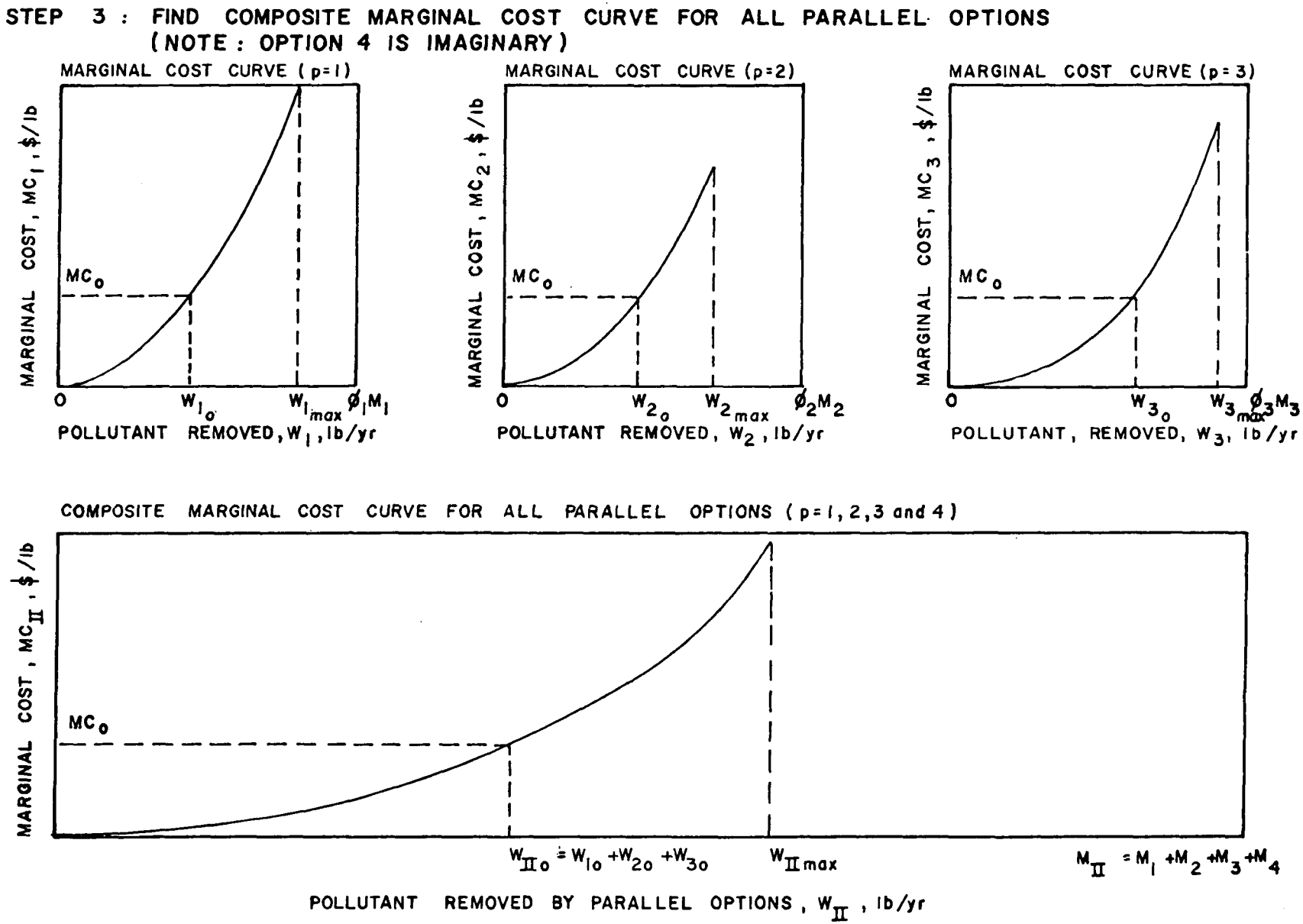


Figure 4. Graphical Procedure for Determining Optimal Control Strategies, Step 3

The composite total cost curve for the parallel options is constructed by integrating the composite marginal cost curve (see Step 4, Figure 5), i.e.,

$$Z_{II} = \int_{o}^{W_{II_{max}}} MF_{II}(W_{II})dW_{II} \qquad (13)$$

where Z_{II} = composite total cost of the parallel options, \$/yr; and

$W_{II_{max}}$ = maximum pollutant removal by the parallel options ($W_{1_{max}} + W_{2_{max}} + W_{3_{max}}$ in the example network), lb/yr.

At this point the economic behavior of the parallel group has been condensed into a single equivalent "option." Therefore, the problem has been reduced to one with two options in series. Next, the two-option serial operation is aggregated into a single equivalent "option" representing the entire example network. Although the procedure will be unique to a two-option serial operation, this will not limit the number of options in series that may be analyzed. Any number of options may be aggregated by simply working with pairs until condensed to one equivalent "option." The previous procedure for the parallel case may be applied to any number of options.

A two-option serial operation may be viewed as a production "process" with two inputs and one output. The production function can be described using isoquants (see Figure 1), i.e., lines of input combinations capable of producing a constant output and having the following characteristics [12]:

(1) Isoquants cannot intersect. Intersection would imply that the same input levels are capable of producing different output levels.

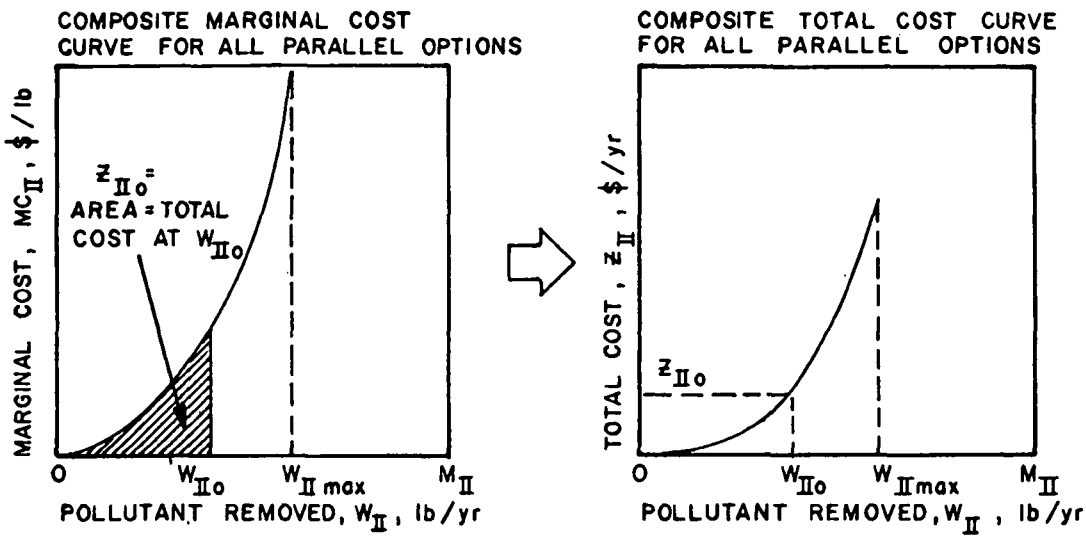
(2) Isoquants slope downward to the right because increased use of one input requires the lessened use of the other input.

(3) Isoquants are convex to the origin due to the inability of one input to be substituted for another at a specific level of output.

The inputs are the total costs of each option in series and the output is the fraction of pollutant removed by the serial operation. In this particular case, the inputs are the composite total costs for the parallel group, Z_{II}, and the total costs for the subsequent option, Z_5.

Before constructing the isoquants of the fraction removed by the serial operation, both total cost curves must be in terms of the fraction of pollutant removed. The curve for option 5 was constructed earlier (see

STEP 4 : INTEGRATE COMPOSITE MARGINAL COST CURVE TO OBTAIN COMPOSITE TOTAL COST CURVE FOR ALL PARALLEL OPTIONS



STEP 5 : FIND ISOQUANTS OF THE FRACTION OF POLLUTANT REMOVED BY OPTIONS IN SERIES, $Y_{\mathcal{E}}$

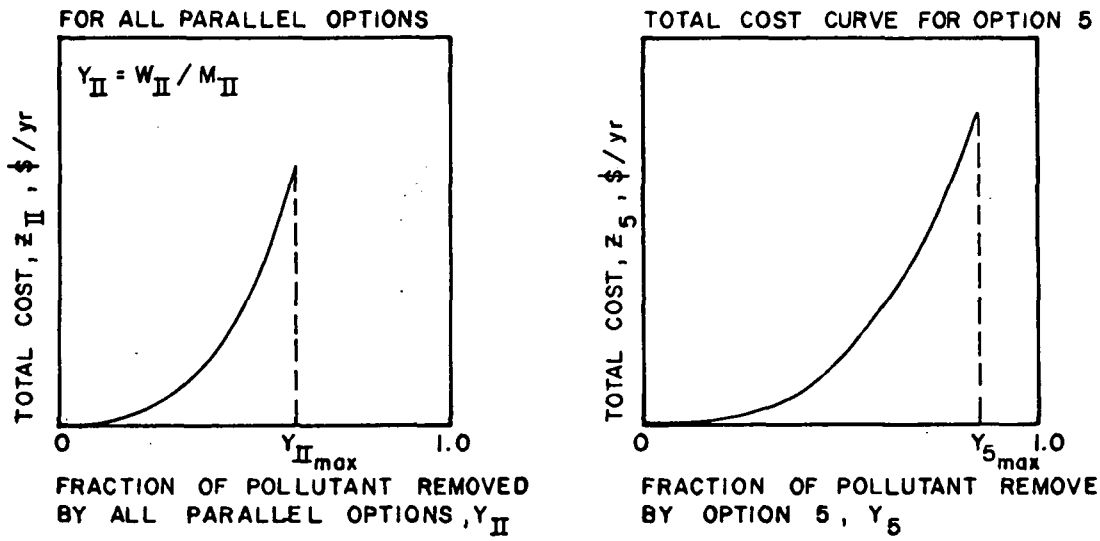


Figure 5. Graphical Procedure for Determining Optimal Control Strategies, Steps 4 and 5

Step 1, Figure 3) and is already in the proper form. The composite total cost curve for the parallel group was left in terms of the pounds of pollutant removed (equation 13). The following equation is used to convert to the fraction removed:

$$Y_{II} = W_{II}/M_{II} \tag{14}$$

where M_{II} = pollutant load available to all parallel processes ($= M_1 + M_2 + M_3 + M_4$), lb/yr.

These curves must be in terms of the fraction removed due to the nature of the serial operation. Essentially, one input is passing through two options, as opposed to a parallel operation where each input is independent. Therefore, the action of one affects the other and it becomes necessary to optimize the fraction removed by each option and then determine what quantity of pollutant was removed by each, rather than the reverse.

Constructing the isoquants of the overall fraction of pollutant removed requires several combinations of Y_{II} and Y_5 capable of providing the desired overall fraction. This is determined by the following equation,

$$Y_\psi = Y_{II} + Y_5(1-Y_{II}) \tag{15}$$

where Y_ψ = fraction of pollutant removed by the serial operation.

Equation (15) states that the fraction of pollutants removed by the serial operation is the sum of the removal from the first option, Y_{II}, and the incremental removal due to the second option, $Y_5(1-Y_{II})$. By noting the Z_{II} and Z_5 corresponding to the various combinations of Y_{II} and Y_5 from each of the total cost curves, the isoquants may be drawn (see Step 5, Figures 5 and 6).

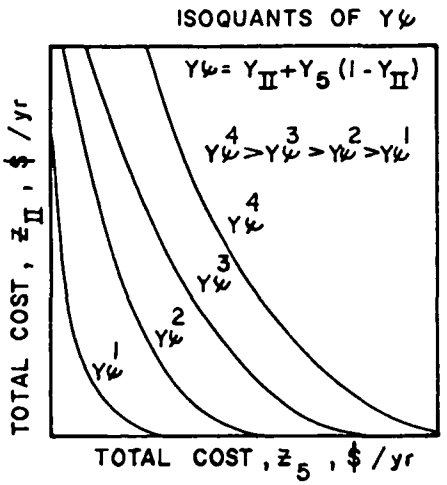
The next step is to develop the optimal expansion path from the isoquants by constructing points of tangency between the isoquants and isocost lines. As the name suggests, isocost lines are lines of equal cost. The isocost lines are given as

$$Z_\psi = Z_{II} + Z_5 \tag{16}$$

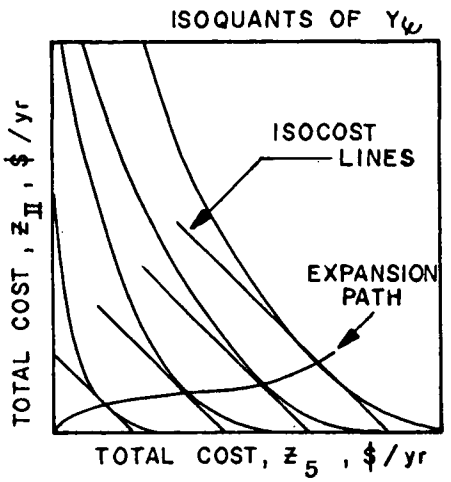
where Z_ψ = composite total cost of the serial operation, $/yr.

The slope of this linear equation is -1. Therefore, to find the point of tangency simply requires that the point on the isoquant tangent to a line of a negative unit slope be located. These points determine the optimal or least-cost combination of costs from each option. The optimal solution may fall at a corner point. Connecting these points gives the optimal expansion path (see Step 6, Figure 6). The final step is to construct a composite total cost curve for the serial operation. This may be done by plotting the values of Z_ψ against the corresponding values of Y_ψ found on the optimal expansion path (see Step 7, Figure 6). This curve, for the

STEP 5 : (CONTINUED)



STEP 6 : FIND OPTIMAL EXPANSION PATH



STEP 7 : FIND TOTAL COST CURVE FOR ALL OPTIONS (p= 1,2,3,4, and 5)

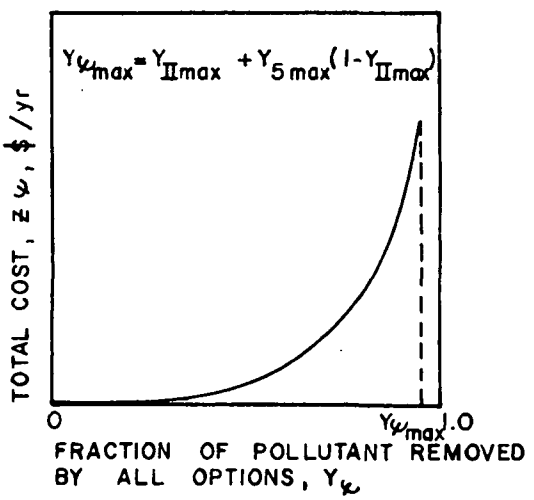


Figure 6. Graphical Procedure for Determining Optimal Control Strategies, Steps 5 (concluded), 6 and 7

example network shown in Figure 2, represents the final total cost for the entire network as a function of the overall fraction of pollutant removed. If there were subsequent options the curve would merely represent a composite of two options in series that could next be combined with the following process.

The methodology for establishing optimal strategies has been developed. At this point, a planner could select an overall removal fraction and proceed, in reverse, through the seven steps shown in Figures 3 through 6 and determine the optimal operating levels of each option. This is demonstrated by the example application found in Section VII.

SECTION VI

CONTROL TECHNOLOGIES

Section V set up a methodology by which engineers and planners could determine an optimal strategy with any number of control options. For the purposes of this study, four technologies will be considered: street sweeping, combined sewer flushing, catch-basin cleaning, and storage-treatment. Typically, these technologies operate as shown in Figure 7. Only combined sewered areas may utilize all four technologies. Storm sewered areas do not require sewer flushing to remove sanitary sewage deposition. In addition to flushing, unsewered areas do not have catch basins or street sweeping since materials are transported to adjacent pervious areas. (It is assumed that there are no gutters.) For combined sewered areas the network operates in the following manner. A portion of the street solids are removed by sweeping and a portion are washed off during runoff events and partially captured by catch basins which are subsequently cleaned by artificial means or flushed by a storm surge. The sanitary sewage deposited in the sewer lines is also flushed artificially or by storm surges. The pollutants flushed from the catch basins and sewer lines during storm surges are sent to the wet-weather storage-treatment facility. The material removed from the catch basins is normally sent to a sanitary landfill. The material flushed from the sewers is sent to the dry-weather treatment plant.

This section develops the production function for each technology. Additionally, the relationships to construct the total cost curves are given. With this information, the methodology of Section III may be applied to any urbanized area.

The term "pollutant" has been used almost exclusively up to this point. However, the pollutant BOD will be the parameter of concern in this and remaining sections. BOD is the most commonly used indicator of general water pollution levels. The same method could be used for any other single pollutant.

STREET SWEEPING

The sweeping of roadways is a long-established practice in American cities. However, the primary purpose of this activity is the removal of unsightly debris. Recent studies indicate that a portion of the material found on the streets (and therefore a potential pollution source during runoff events) may be removed by a conscientious sweeping program [4, 5]. The particle size and pollutant distribution of street contaminants are shown

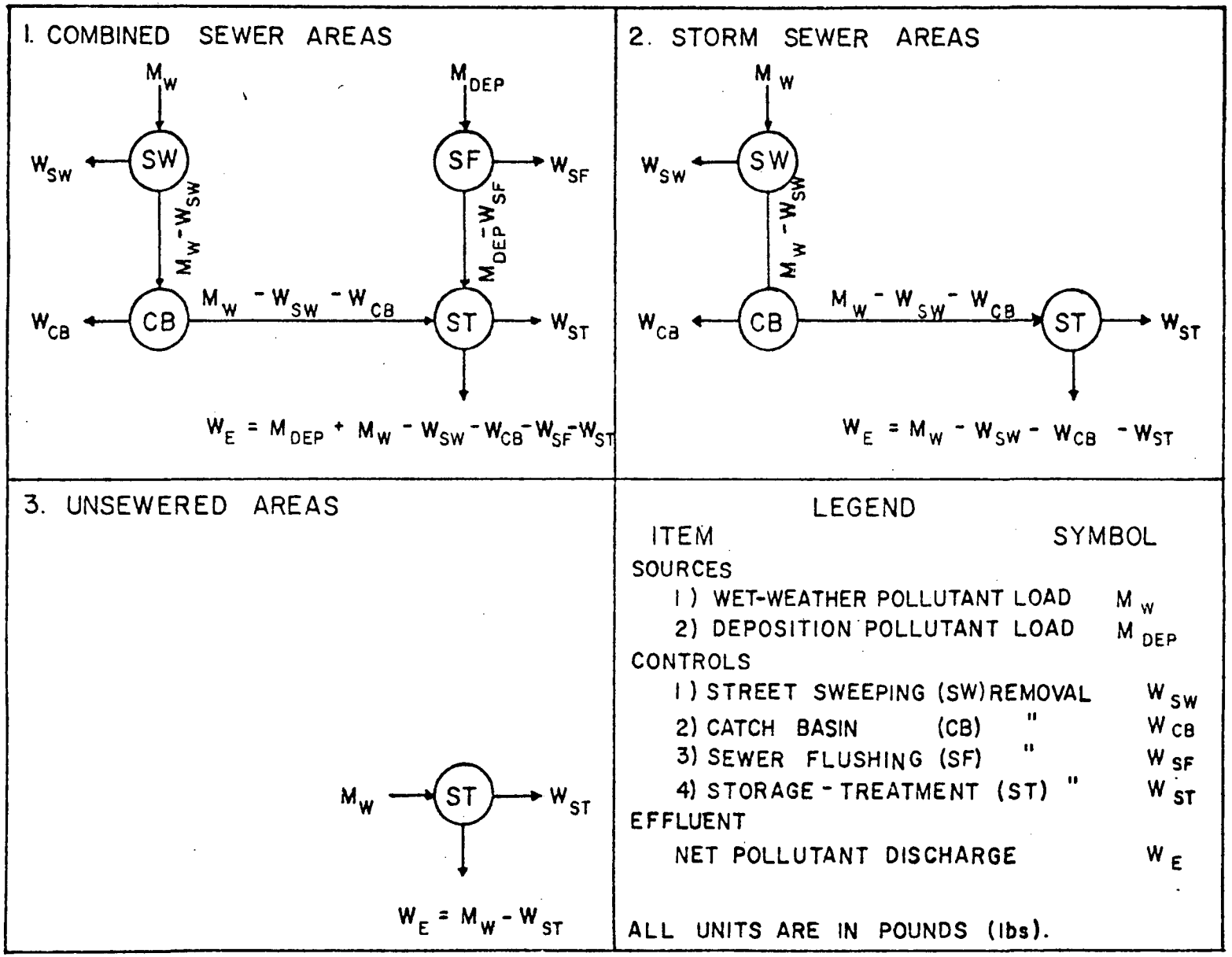


Figure 7. Stormwater Pollution Control Technologies - Availability Factors Equal 1.0

in Table 4.

Street sweeping may be performed manually or mechanically, with the latter enjoying more widespread usage. Mechanical sweepers are divided into two categories: brush-type and vacuum-type. Removal efficiencies with brush-type sweepers for various particle sizes are shown in Table 5. The overall efficiency is 0.50 with the coarser materials enjoying higher efficiencies than fine particles. APWA reports that vacuum type sweepers have achieved efficiencies of greater than 0.95 [5]. Of course, the increased efficiency of vacuum sweepers results in a substantially higher cost over brush-types.

Street sweeping has several advantages and disadvantages as a pollution control technique. Some favorable characteristics are the

(1) control of pollutants at the source; and

(2) dual purpose of sweeping for pollution control and esthetics.

Unfavorable traits include

(1) relatively low efficiency as a pollution control measure;

(2) sweeper's history as a traffic hazard;

(3) removal of only the portion of the load located near the gutter; and

(4) problems of vehicular parking along the streets.

Although sweeping has a relatively low removal efficiency, in a coordinated system of storage-treatment and other management practices, it may prove to be a viable alternative.

Street sweeping may be considered a production process (as described earlier). Indeed, all pollution control techniques may be described as such. Therefore, it is possible to describe the technological relationship between the input and output of street sweeping in terms of a production function. In this case, the input is the fraction of the days on which sweeping occurs during a year and the output is the fraction of BOD removed.

In order to generate the production function a model was developed to simulate the conditions within an urbanized area and the effect of street sweeping. Hourly rainfall is converted to runoff using a simple runoff coefficient, and subsequently accumulated BOD is removed by scheduled sweeping or a runoff event. The model makes use of the following assumptions:

(1) The average removal efficiency for BOD is equivalent to that of all particle sizes. This is assumed because of the apparent consistency of the portion of BOD

TABLE 4. PERCENT OF STREET POLLUTANTS IN VARIOUS PARTICLE SIZE RANGES

Pollutant	Percent of Pollutant Associated with Each Particle Size Range Particle Size (microns) >2,000	840 → 2,000	246 → 840	104 → 246	43 → 104	< 43
Total Solids	24.4	7.6	24.6	27.8	9.7	5.9
Volatile Solids	11.0	17.4	12.0	16.1	17.9	25.6
BOD_5	7.4	20.1	15.7	15.2	17.3	24.3
COD	2.4	4.5	13.0	12.4	45.0	22.7
Kjeldahl Nitrogen	9.9	11.6	20.0	20.2	19.6	18.7
Nitrates	8.6	6.5	7.9	16.7	28.4	31.9
Phosphates	0	0.9	6.9	6.4	29.6	56.2

Source: Sartor, J.D., and Boyd, G.B., "Water Pollution Aspects of Street Surface Contaminants," USEPA Report EPA-22-72-081, November 1972, p. 7.

TABLE 5. BRUSH-TYPE SWEEPER EFFICIENCY FOR VARIOUS PARTICLE SIZE RANGES

Particle Size (microns)	Sweeper Efficiency (%)
2000	79
840→2000	66
246→ 840	60
104→ 246	48
43→ 104	20
<43	15
Overall	50

Source: Sartor, J.D., and Boyd, G. B., "Water Pollution Aspects of Street Surface Contaminants," USEPA Report EPA-22-72-081, November 1972, p. 10.

contained within the particle size ranges (see Table 4).

(2) A runoff event encompasses consecutive hourly runoff occurrences and intermittent dry periods not to exceed twelve hours. For example, an intermittent dry period of twelve hours will start a new event; a period of eleven hours or less will not [1].

(3) No sweeping occurs during an event. If an event and a scheduled sweeping coincide the streets are simply not swept until the next scheduled time.

(4) Only one pass is made per sweep.

The pollutant washoff functions incorporated in the model were identical to the functions found in the SWMM and STORM models [13, 14]. However, the methods of pollutant build-up and removal by street sweeping are somewhat different. SWMM and STORM allow the linear build-up of pollutants as long as the elapsed time from the previous runoff event is less than the street sweeping interval. The relationship is

$$P_1 = F \cdot DD \cdot N_D + P_o \tag{17}$$

where P_1 = total pollutant load at the beginning of the storm, lb;

F = pounds of pollutant per pound of dust and dirt;

DD = daily dust and dirt accumulation rate, lb/day;

N_D = number of dry days since the last storm; and

P_o = pounds of pollutant remaining at the end of the last storm (event).

If the number of days since the last runoff event is greater than the sweeping interval, the following equation is employed by SWMM and STORM.

$$P_1 = P_o(1-\varepsilon)^n + N_S \cdot DD \cdot F \cdot [(1-\varepsilon)^n + (1-\varepsilon)^{n-1} + \ldots + (1-\varepsilon)] + DD \cdot F \cdot (N_D - nN_S) \tag{18}$$

where N_S = number of days between street sweepings;

n = number of times the streets were swept since the last storm; and

ε = "pick up" efficiency of the street sweeping equipment.

The major flaw in this procedure is that the street sweeping "counter" is set to zero at the end of every runoff event. In other words, after the end of an event, N_S days must pass before sweeping occurs. Therefore, it is conceivable that the streets will never be swept according to this procedure. For example, assume that N_S is 20 days. If the longest dry period during a year is 15 days, STORM and SWMM will fail to simulate any sweeping--even though an interval of 20 days was specified.

To correct this deficiency, the model developed for this study merely establishes a sweeping schedule from which no deviation is allowed except in the case of a coincident runoff event. This is not an entirely accurate assumption, for public works departments certainly have the flexibility to alter their sweeping schedule. However, this is not considered to be a serious error. When sweeping does occur, the amount of pollutant removed is taken as the product of the accumulated pollutants available and the "pick-up" efficiency.

Not all of the accumulated pollutants are available for removal by sweeping. There are considerable amounts on parking lots, driveways, and other impervious areas not subject to sweeping by municipal units. Total and street imperviousness as a function of developed population density is shown in Figure 8 [1]. If pollutants are assumed to be uniformly distributed over the impervious area and that only the pollutants in the street are sweepable, then

$$\phi_{SW} = \frac{I_s}{I} \approx 0.6\ PD_d^{-0.2} \quad \text{for } PD_d \geq 0.1 \tag{19}$$

where ϕ_{SW} = sweeping availability factor, i.e., proportion of pollutant load which is sweepable ($0 \leq \phi_{SW} \leq 1.0$);

I_s = imperviousness due to streets only, percent; and

I = total imperviousness, percent.

A plot of equation (19) (Figure 9) shows that ϕ_{SW} ranges from about 0.43 at PD_d = 5 persons/ac (12.4 persons/ha) to about .35 at PD_d = 15 persons/ac (37.1 persons/ha). In actuality, a disproportionate amount of the pollution is located on the streets or is delivered to the streets prior to entering the final drainage canals. To test the sensitivity of the result to ϕ_{SW}, evaluations will be made with ϕ_{SW} = 0.40 representing a lower bound ϕ_{SW} = 1.0 representing the upper bound, and ϕ_{SW}=0.70 representing an average value.

Running the model with several different efficiencies (ranging from 0.1 to 0.9) and sweeping intervals (ranging from 1 to 42 days) generated the family of production functions shown in Figure 10. The model was run using data for the developed areas of Minneapolis, Minnesota (including hourly precipitation for 1971). The production function for a "pick-up" efficiency of 0.50 is used to describe the typical mechanical sweeping operation. This corresponds to the efficiency shown in Table 5 for brush-type sweepers. The efficiency could be any value suitable to local conditions. For

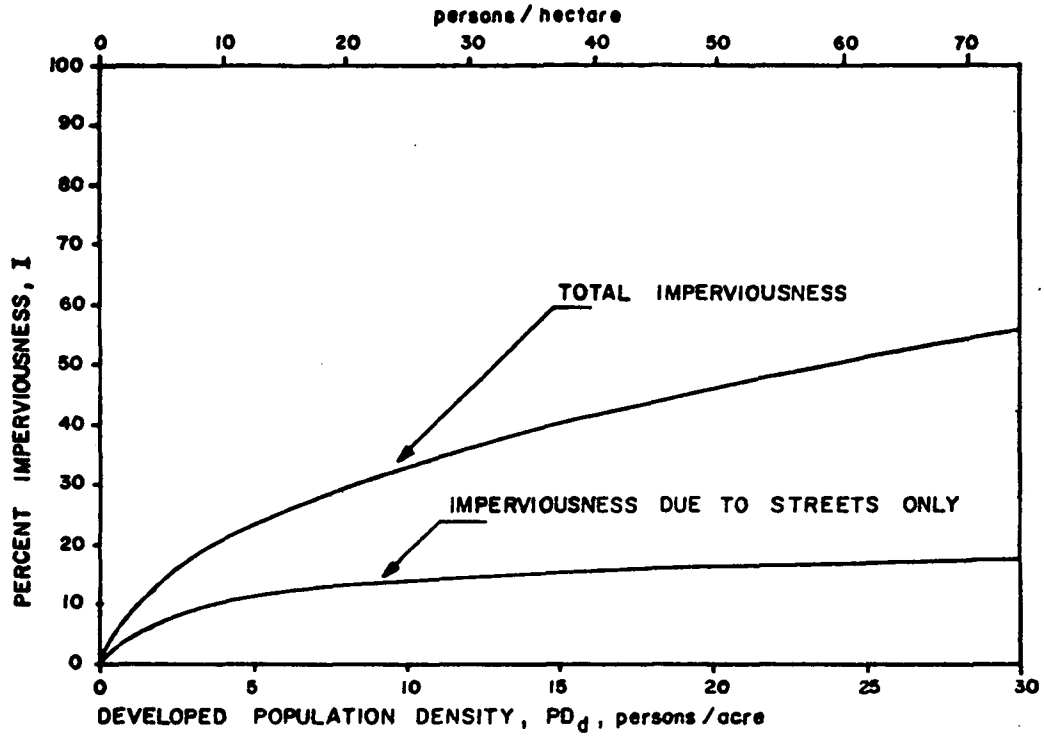


Figure 8. Imperviousness as a Function of Developed Population Density

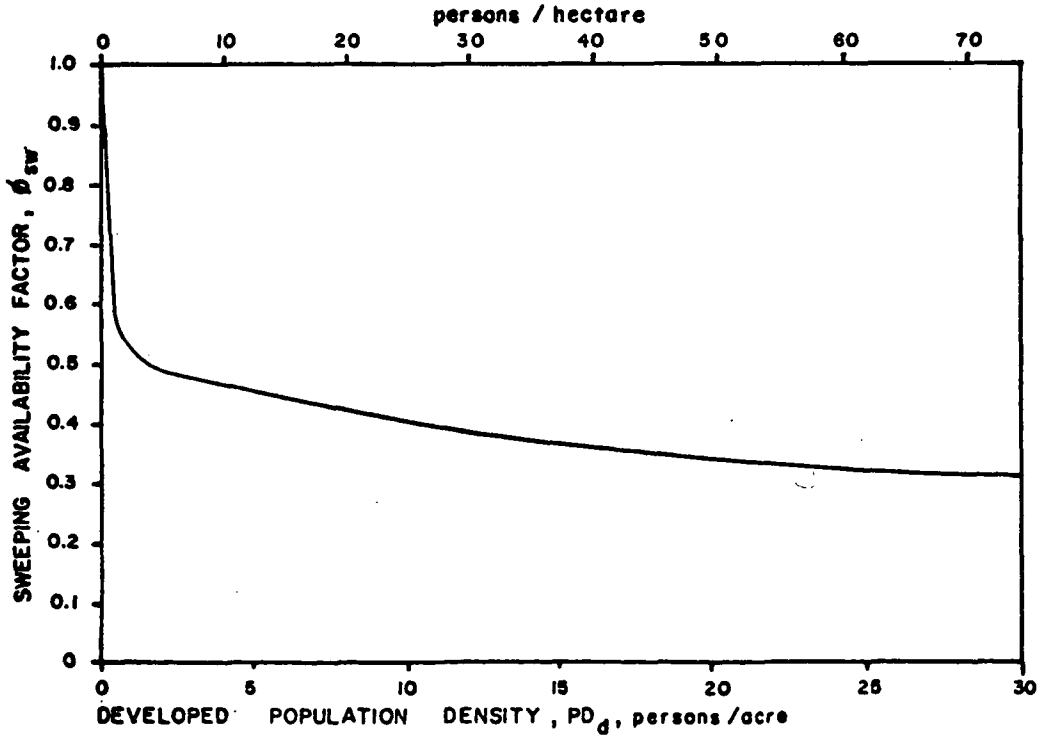


Figure 9. Sweeping Availabiility Factor as a Function of Developed Population Density

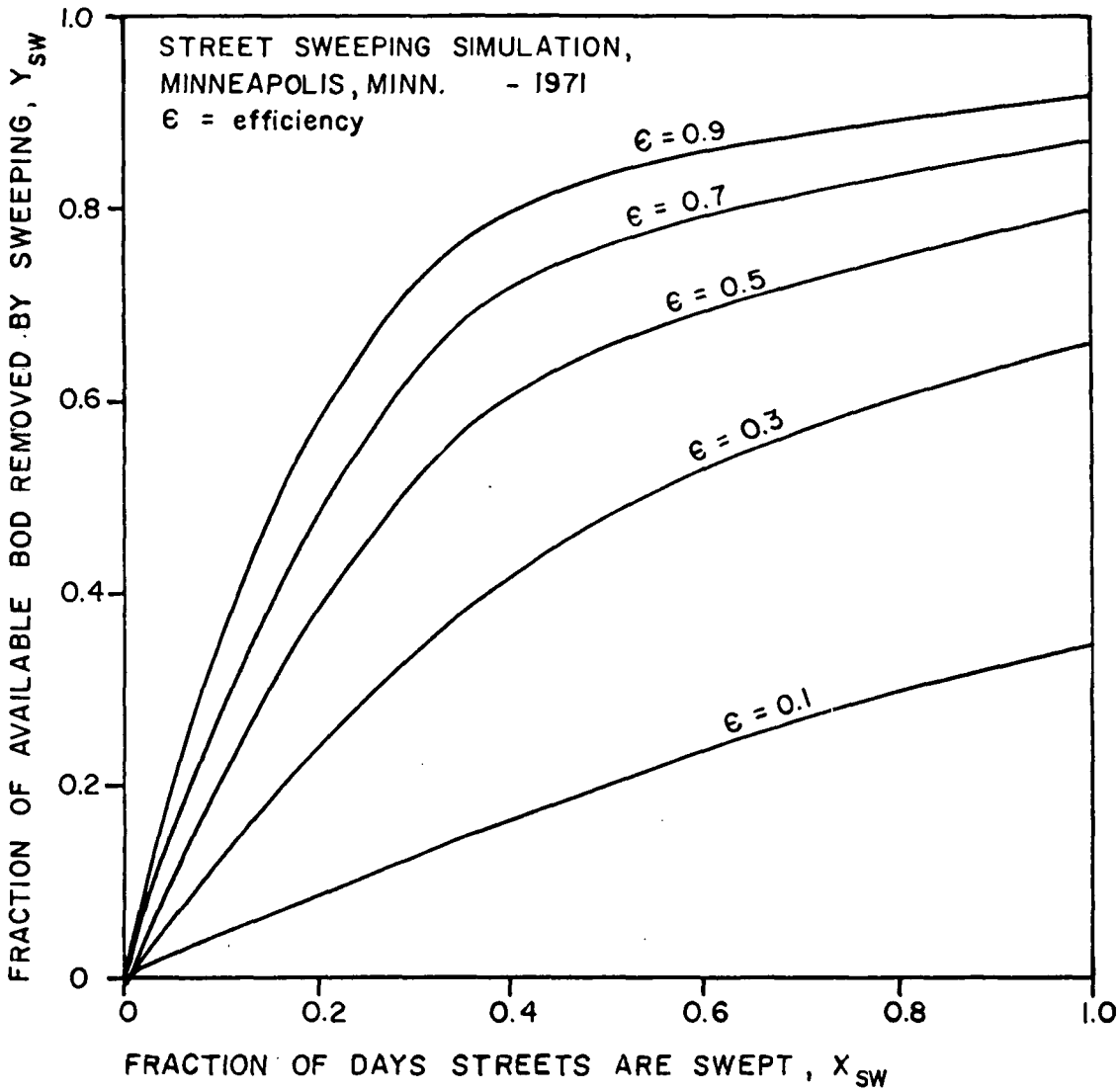


Figure 10. Production Functions for Street Sweeping

example, the use of vacuum sweepers would dictate the use of a higher efficiency.

The production function generated by the model is in terms of the fraction of BOD removed annually. In order to generate the total cost curve the fraction removed must be converted to the pounds of BOD removed (as described in Section V). Therefore,

$$W_{SW} = (m_W - m_{DEP}) \cdot A_{SW} \cdot \phi_{SW} \cdot Y_{SW} \qquad (20)$$

where W_{SW} = wet-weather BOD removed by sweeping, lb/yr;

m_W = annual wet-weather BOD load, lb/ac-yr;

m_{DEP} = annual wet-weather BOD load due to combined sewer deposition, lb/ac-yr;

A_{SW} = area to be swept, ac;

ϕ_{SW} = sweeping availability factor, i.e., proportion of BOD load which is sweepable ($0 \leq \phi_{SW} \leq 1.0$); and

Y_{SW} = fraction of available BOD removed by sweeping ($0 \leq Y_{SW} \leq 1.0$).

The unit pollutant loads, m_W and m_{DEP} may be found using methods described in Appendix A.

Assuming that the total annual cost of sweeping, Z_{SW}, is a linear function of the number of sweepings, the total cost curve for any urbanized area (or subarea) is given by the following equation,

$$Z_{SW} = 365 \cdot C_{SW} \cdot G \cdot A_{SW} \cdot X_{SW} \qquad (21)$$

where C_{SW} = cost per curb mile swept, $/curb-mile;

G = gutter length in A_{SW}, curb-miles/ac;

X_{SW} = fraction of days per year the area is swept ($0 \leq X_{SW} \leq 1.0$); and

$X_{SW} = PF_{SW}^{-1}(W_{SW})$, the inverse production function of street sweeping expressed in terms of W_{SW}.

The value for gutter length, G, may be estimated by an equation developed by Graham et al. [15],

$$G = 0.0782 - 0.0668(.839)^{PD_d} \qquad (22)$$

where PD_d = population density for developed areas, persons/ac.

Also, some average values for gutter length are shown in Table 6. The sweeping cost per curb mile, C_S, is more difficult to determine. APWA reports a wide range of values for several street sweeping cost parameters from over 160 municipalities [16]. The median and mean for each parameter are given in Table 7. The median value of $7.00/curb mile ($4.35/curb-km) swept will be used because of the large variance in the data.

COMBINED SEWER FLUSHING

Combined sewers often experience dry-weather sewage deposition. These solids accumulate in the sewers until removed by a storm surge or by artificial flushing. The deposition carried away by runoff discharges directly to the receiving water if the dry-weather treatment plant is bypassed. Controlled flushing allows the sewers to be cleaned without adding pollutants to the water body. Instead, these pollutants are routed to the dry-weather plant for treatment.

As with street sweeping, sewer flushing is not a new idea; its primary purpose has been to improve hydraulic capacity and self-scouring ability. However, several reports indicate the ability of a flushing program to remove a substantial portion of the pollutants associated with deposition [2, 3, 6]. Several systems are available for sewer flushing, e.g., flushing stations, in-line storage, and portable tankers. Flushing stations are tanks placed at strategic locations that release the required flushing volumes. Varying degrees of automation are utilized [3]. In-line storage involves a system of internal dams used to block the sewage flow at upstream points for rapid release to scour downstream elements [2]. The use of tankers merely requires that the trucks be dispatched to the system components requiring flushing.

Pisano has investigated the deposition problem in two Boston systems--Dorchester and South Boston [2]. The two systems have a total of 2666 sewer elements with an average length between manholes of 191 ft (58.2m) per segment. Through a simulation model it was estimated that 10.3 percent of the daily dry-weather solids in the Dorchester system was deposited in the lines. The South Boston system retained 6.6 percent. Also, the Dorchester system retains 75 percent of the total deposition in only 18 percent of the system components. South Boston retains 63 percent in 22 percent of the components. Therefore, a relatively small portion of the combined sewer elements retain a substantial amount of the total deposition. As expected, pipe slope was reported to be the major factor in determining potential deposition problems.

Pisano provided the data used to develop the sewer flushing production function [2]. The input to this process is the fraction of the combined sewer segments flushed daily. The output is the fraction of available BOD removed annually by flushing. The production function is based on information regarding the relative concentration of deposition within the two systems of Boston (e.g., 75 percent of the deposition is found in 18 percent of the Dorchester system elements). The function, shown in Figure 11,

TABLE 6. AVERAGE VALUES OF GUTTER LENGTH

Land Use	Curb or Gutter Length	
	miles/ac (km/ha)	ft/ac (m/ha)
Residential	0.059 (0.235)	312 (235)
Commercial	0.070 (0.279)	370 (279)
Industrial	0.034 (0.136)	180 (136)
Other	0.023 (0.091)	121 (91)

Source: Heaney, J.P., Huber, W.C., Medina, M.A., Murphy, M.P., Nix, S.J., and Hasan, S.M., "Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Volume II: Cost Assessment," USEPA Report EPA-600/2-77-064B, March 1977.

TABLE 7. UNIT COSTS OF STREET SWEEPING

	Street Sweeping Costs			
	$/curb-mile ($/curb-km)	$/yd^3 ($/m^3)	$/ton ($/metric ton)	$/capita/year
Mean	86.61 (53.82)	22.14 (28.96)	31.31 (34.51)	1.54
Median	7.00 (4.35)	13.79 (18.04)	14.28 (15.74)	1.23

Source: Unpublished data from American Public Works Association, 1976.

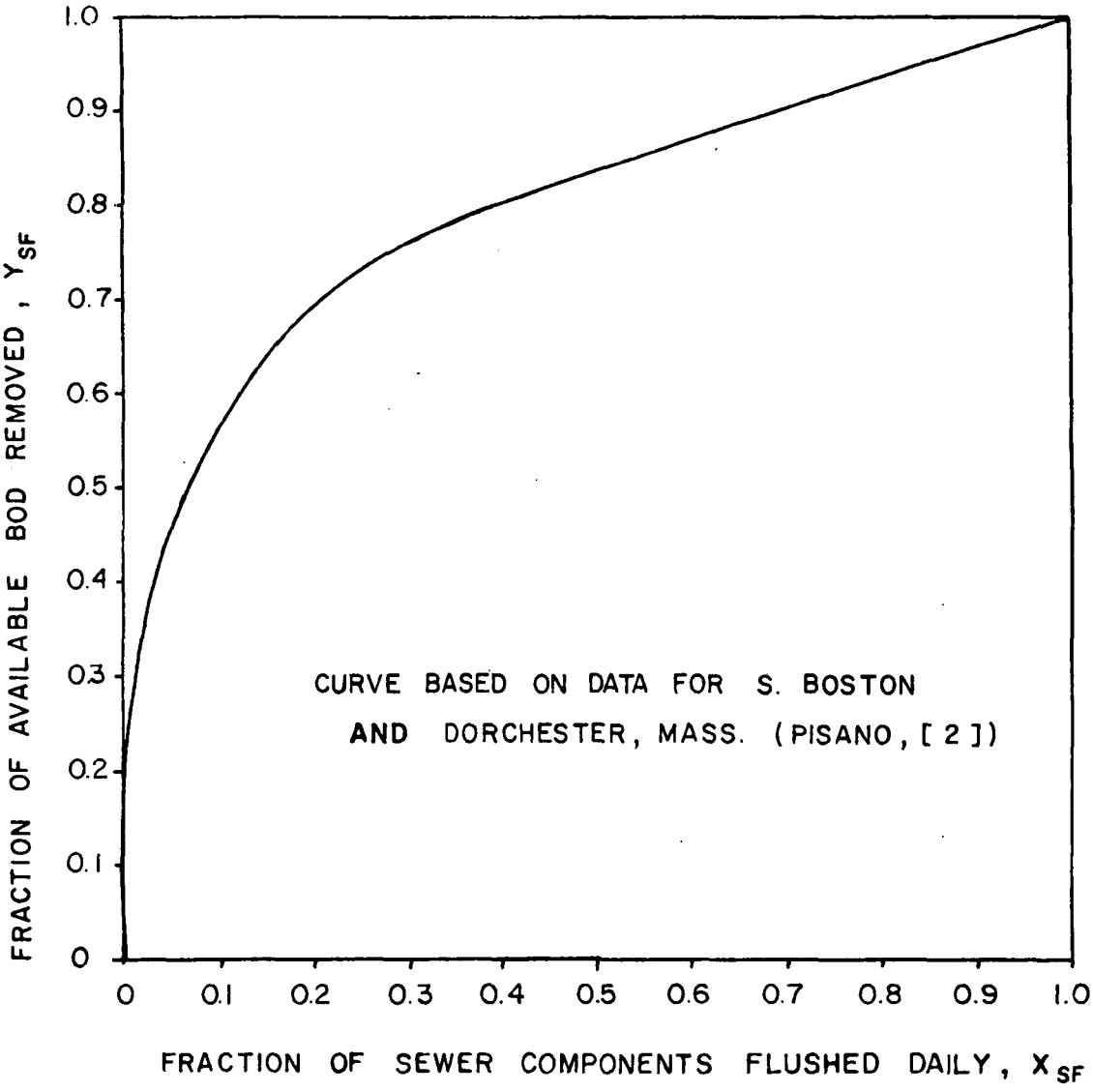


Figure 11. Production Function for Combined Sewer Flushing

is based on several assumptions:

(1) A flushing program cleanses a designated portion of the system elements daily.

(2) Daily flushing will remove virtually 100 percent of the available daily deposition. This assumes that the sewers have been properly maintained and do not contain beds of debris. Of course, not all of the annual deposition will be available for flushing, i.e., some portion will be removed by runoff events.

(3) Pisano reports deposition in terms of suspended solids. BOD, the pollutant of concern here, is assumed to be deposited in a similar manner [2].

(4) All components or pipe sections are ranked, according to the severity of the deposition problem, for flushing priority.

(5) The BOD flushed daily is routed to the dry-weather treatment facility. The flushing volumes are small enough to prevent an artificial combined sewer overflow.

Inspection of the production function reveals that sewer flushing also exhibits decreasing marginal output as the input is increased. In fact, the effect is dramatic after approximately 10 or 20 percent of the sewers are flushed daily.

To develop a total cost function for any urbanized area, it is necessary to convert the production function in terms of the fraction of BOD removed to the pounds of BOD removed annually. This is accomplished by the following equation,

$$W_{SF} = \phi_{SF} \cdot m_{DEP} \cdot A_{SF} \cdot Y_{SF} \qquad (23)$$

where W_{SF} = BOD in deposition removed by daily sewer flushing, lb/yr;

ϕ_{SF} = sewer flushing availability factor, i.e., proportion of pollutant load that is flushable ($0 \leq \phi_{SF} \leq 1.0$);

m_{DEP} = annual BOD load of combined sewer deposition, lb/ac-yr;

A_{SF} = area served by combined sewers to be flushed, ac; and

Y_{SF} = fraction of available BOD removed by flushing ($0 \leq Y_{SF} \leq 1.0$).

A method of estimating the combined sewer deposition BOD load, m_{DEP}, is found in Appendix A. A more refined procedure is being developed. Results should be available later this year [17].

Assuming that the total annual costs of sewer flushing, Z_{SF}, are a linear function of the system footage to be flushed, the total cost curve for any area is given by the following equation,

$$Z_{SF} = C_{SF} \cdot (0.40G) \cdot A_{SF} \cdot X_{SF} \qquad (24)$$

where C_{SF} = cost per mile of sewer flushed, \$/mile;

$0.40G$ = sewer length in A_{SF}, miles/ac = 40 percent of gutter length;

X_{SF} = fraction of combined sewerage system components flushed daily ($0 \leq X_{SF} \leq 1.0$); and

$X_{SF} = PF_{SF}^{-1}(W_{SF})$, the inverse sewer flushing production function in terms of pounds of BOD removed.

This equation and the construction of the total cost curve require several assumptions:

(1) Flushing will be performed by the in-line storage and sudden release of dry-weather flow at upstream locations [2].

(2) The redeposition of BOD at the trailing edge of the resulting flush wave is assumed to be negligible [2].

(3) The cost per unit length of flushed sewer line is constant; regardless of pipe size, type, slope, or Manning's n.

(4) The length of combined sewers per acre is assumed to be 40 percent of the gutter length. Equation 22 or Table 6 may be used to estimate the gutter length, G.

Pisano indicates that the total present worth of the cost (capital and operation/maintenance) per in-line flushing module (inflatable dam) and the initial cleaning is \$22,500. It is assumed that one module is needed per segment requiring flushing. The annual cost per module is \$2,250 (8% interest, 20 year service life). The Boston system segments have an average length of 191 feet. Therefore, the annual cost of flushing per unit length of sewer line is \$11.78/ft (\$38.64/m) or \$62,200/mile (\$38,600/km).

CATCH-BASIN CLEANING

In a study dealing with overall catch-basin performance, Metcalf and Eddy, Inc. define a catch basin as [18]:

> a chamber or well, usually built at the curb line of a street, for the admission of surface water to a sewer or subdrain, having at its base a sediment sump designed to retain grit and detritus below the point of overflow.

This definition implies that catch-basins are not intended to remove BOD or suspended solids, but act primarily as grit chambers designed to prevent the clogging of sewer lines. However, the catch basin does act as a sedimentation tank capable of removing some portion of the BOD. Unfortunately, the small portion of BOD removed may be flushed from the sump during runoff events. Catch basins act much as septic tanks, but are subject to highly variable and, often, overwhelming flows. Lager and Smith estimate the typical pollution control effectiveness in an example shown in Table 8 [18]. The efficiency of BOD removal, calculated below using data from Table 8, i.e.,

$$\text{Efficiency} = \frac{\text{removal - loss}}{\text{input}} = \frac{(345{,}800 - 262{,}500)\ 100}{25{,}000 \text{ basins } (50 \text{ storms})(1.04 \text{ lb})}$$

$$= 6.4\%$$

indicates that the expected removal level is not significant. For this reason, catch-basin cleaning will not be analyzed further.

STORAGE-TREATMENT

The remaining method of controlling stormwater pollution involves storage and/or treatment of the collected runoff. Storage-treatment facilities operate in series with the management practices.

A variety of storage and treatment technologies are available. Examples of storage include

(1) in-line storage,

(2) tanks,

(3) lagoons, and

(4) tunnels.

Treatment methods include

(1) sedimentation,

TABLE 8. EXAMPLE PROBLEM EVALUATING CATCH-BASIN PERFORMANCE

EXAMPLE PROBLEM 7-3: ANNUAL POLLUTION ASSESSMENT OF CATCHBASIN PERFORMANCE[Δ]

Given the conditions expressed in the preceding problems, determine the aggregate effectiveness of the catchbasins over a period of years in terms of BOD_5 removed.

Specified Conditions

1. Total number of catchbasins = 25,000.
2. Curb length per catch basin = 0.10 curb mile (0.16 curb-km).
3. Annual precipitation = 35.1 in (89.2 cm).
4. Catchbasins are cleaned twice a year.
5. The pollution load displaced from each basin is 0.21 lb (.10 kg) BOD_5 for each of 50 storms occurring in a year.
6. The runoff coefficient = 50%.

Assumptions

1. The annual rainfall can be characterized as 50 equal 5-h storms.
2. BOD_5 removal by sedimentation will total 26.6% of the applied load.

Solution

1. Determine the annual loss of BOD_5 by liquid volume displacement.

   BOD_5 loss = 25,000 basins x 50 storms x 0.21 lb (.10 kg)

   = 262,500 lb/yr (125,000 kg/yr)

2. Compute the BOD_5 entering a catchbasin each storm (following procedures of earlier example).

   BOD_5 entering = 1.3 lb (.59 kg) available x 0.80 removed from streets

   = 1.04 lb (.47 kg)

3. Determine the annual removal of BOD_5 by sedimentation.

   BOD_5 removed = 25,000 basins x 50 storms x [1.04 lb (.47 kg) x 0.266]

   = 345,800 lb/yr (156,800 kg/yr)

4. Compare the net benefit ratio

   Benefit = 345,800 lb (156,800 kg) removed ÷ 262,500 lb (119,000 kg) lost

   = 1.32:1.

Comment

The problem illustrates that from a pollution abatement standpoint the benefits of catchbasins are marginal at best. Of course, with the cleaning frequency of twice per year, the liquid fraction pollution might average half the specified value, thereby doubling the benefit ratio; however, the gross impact is still small. This example is based on grossly synthesized data and real, long-term removal data from a few catchbasins are required for an actual assessment of catchbasins.

[Δ]Reference: Lager, J.A., and Smith, W.G., "Catchbasin Technology Overview and Assessment," USEPA Report (draft), 1977.

(2) swirl concentrators,

(3) microstrainers,

(4) dissolved air flotation,

(5) contact stabilization, and

(6) physical-chemical treatment.

The operation of a storage-treatment facility is a production process involving two inputs and one output. The inputs are the storage capacity and the maximum treatment rate and represent the input vector to the storage-treatment process, $\bar{X}_{ST}$. The output is the fraction of available BOD removed annually, Y_{ST}.

A methodology to derive the production function and total cost curve for storage-treatment in any urbanized area is discussed in an EPA publication by Heaney, Huber, and Nix [7]. Rather than summarize this methodology here, the reader is referred to this report for details. With relatively little data, the production function and total cost curve may be derived for any city in the U.S. As an example, a production function (in the two-dimensional isoquant form) for Atlanta, Georgia, is shown in Figure 12. Additionally, a total cost curve for the storm sewered areas of that city is shown in Figure 13.

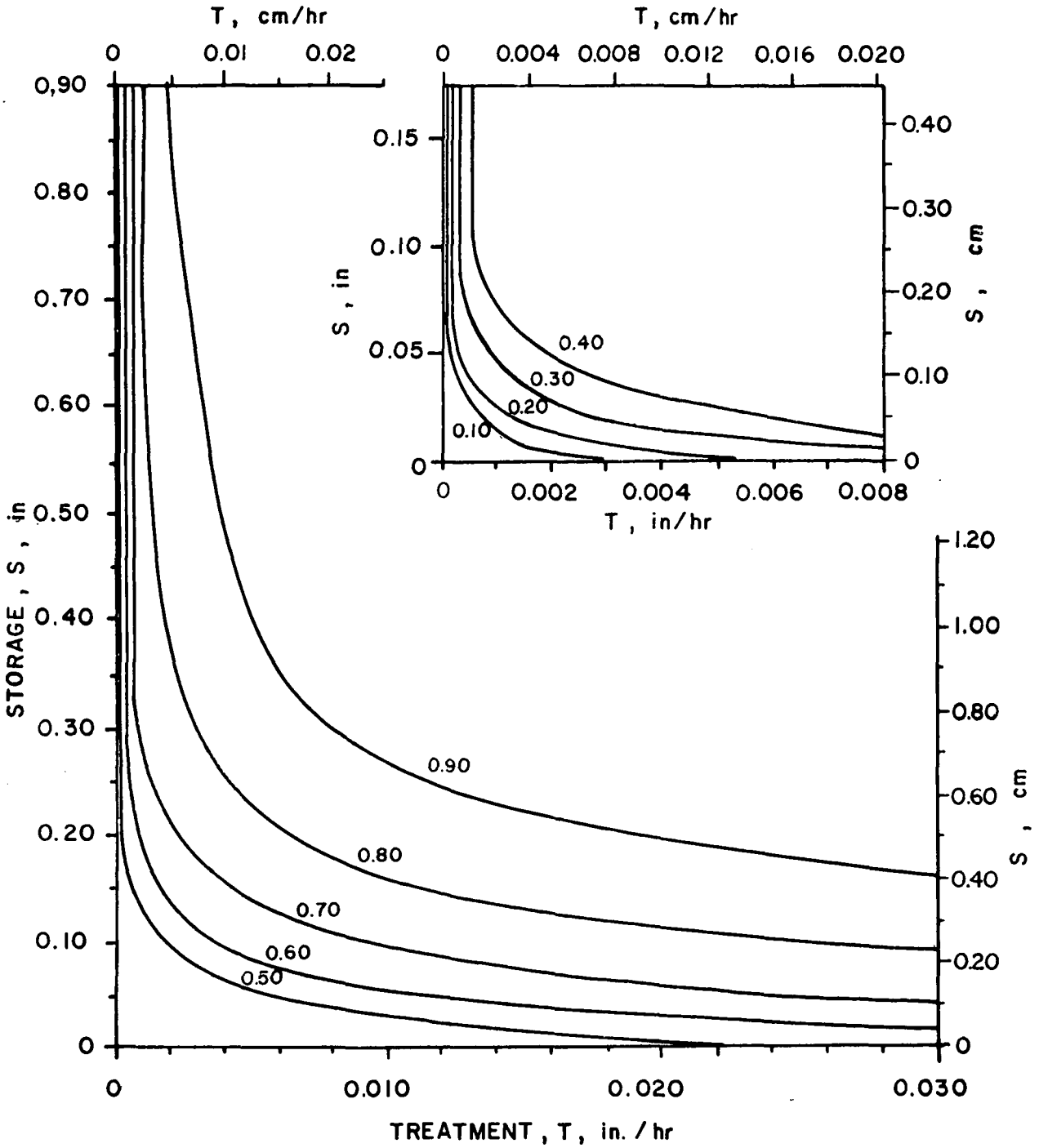


Figure 12. Production Function (Isoquants) for Storage-Treatment, Atlanta, Georgia [Heaney, Huber and Nix, 1976]

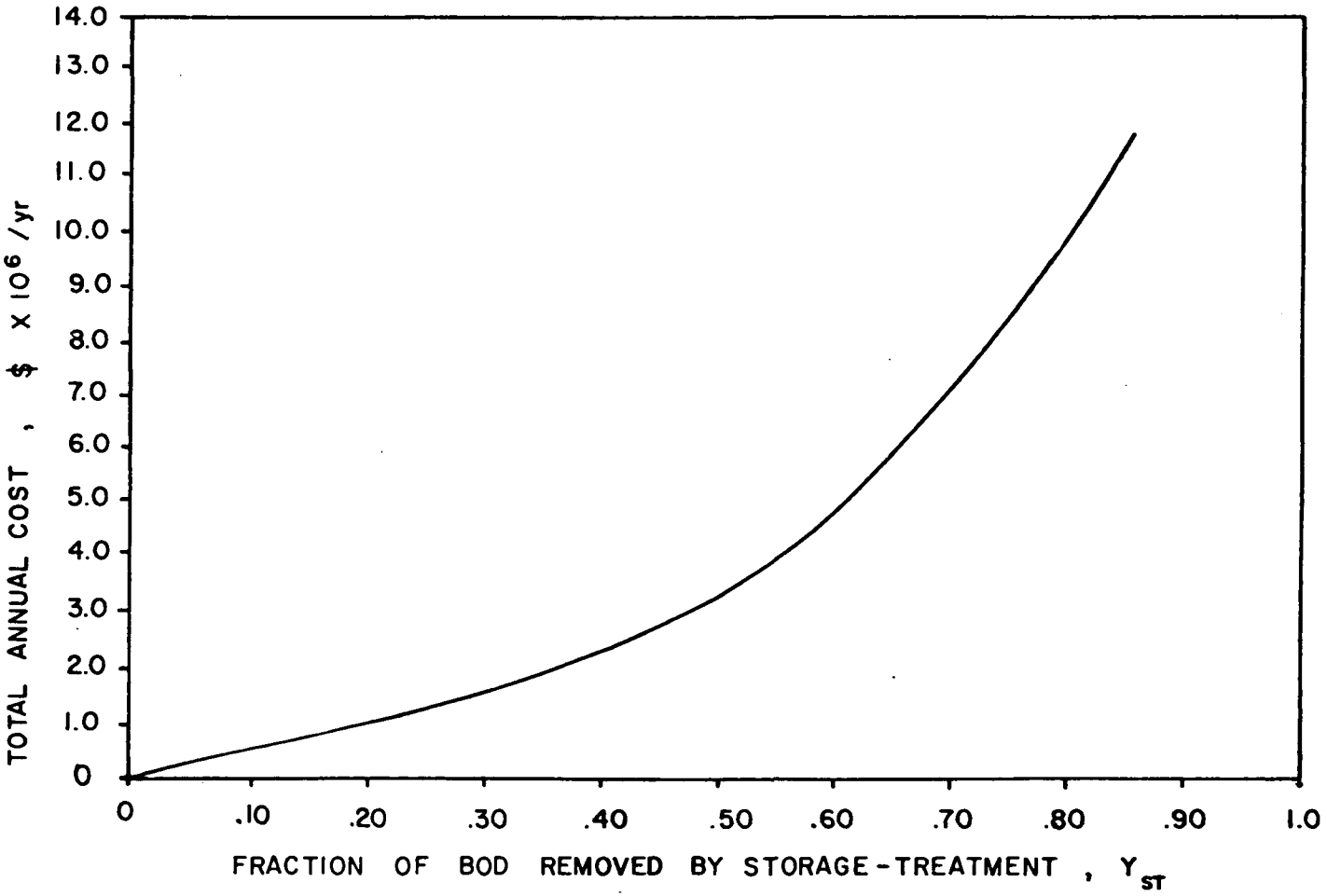


Figure 13. Total Cost Curve for Storage-Treatment, Storm Sewered Areas, Atlanta, Georgia

SECTION VII

APPLICATION TO ANYTOWN, U.S.A.

PROBLEM STATEMENT

In this section, the methodology presented in Section V is applied to a hypothetical urbanized area. The characteristics of this area, called Anytown, necessary to utilize this methodology are derived from average characteristics of the 248 urbanized areas in the U.S. [1]. These averages are:

(1) Population Density (urbanized area): 5.14 persons/ac (12.70 persons/ha).

(2) Mean Annual Precipitation: 33.4 in (84.8 cm).

(3) Land Use Percentage (urbanized area): residential, 31.4%; commercial, 4.6%; industrial, 8.0%; other developed, 9.8%; undeveloped, 46.2%.

(4) Land Use Percentage (developed areas only): residential, 58.4%; commercial, 14.8%; industrial, 8.6%; other developed, 18.2%.

(5) Percent of Developed Area Served by Type of Drainage System: combined sewers, 14.4%; storm sewers, 38.3%; unsewered, 47.3%.

(6) Population Density of the Developed Area by Type of Drainage System--persons/ac (persons/ha): combined, 16.7 (41.3); storm, 13.0 (32.1); unsewered, 4.6 (11.4); all developed areas, 9.6 (23.7).

Assuming a population of 1,000,000 persons and using the above values, the necessary land use, drainage system, and population characteristics for the developed areas in Anytown are derived and shown in Table 9. The land use percentages are assumed to be constant regardless of the drainage system (e.g., combined, storm, and unsewered areas each have 58.4 percent of the service area as residential). Additionally, it is assumed that the population density for each drainage system service area is constant regardless of the land use [e.g., residential, commercial, industrial, and other developed areas in the combined sewered areas all have a population density of 16.7 persons/ac (41.3 persons/ha)].

TABLE 9. LAND USE AND POPULATION CHARACTERISTICS OF ANYTOWN, U.S.A.

Drainage System	Land Use	Area, ac (ha)	Population Density, persons/ac (persons/ha)
Combined	Residential	8800 (3560)	16.7 (41.3)
	Commercial	1300 (530)	16.7 (41.3)
	Industrial	2200 (890)	16.7 (41.3)
	Other	2800 (1130)	16.7 (41.3)
	TOTAL or AVG.	15100 (6110)	16.7 (41.3)
Storm	Residential	23500 (9510)	13.0 (32.1)
	Commercial	3400 (1380)	13.0 (32.1)
	Industrial	5900 (2390)	13.0 (32.1)
	Other	7300 (2950)	13.0 (32.1)
	TOTAL or AVG.	40100 (16230)	13.0 (32.1)
Unsewered	Residential	29000 (11740)	4.6 (11.4)
	Commercial	4200 (1700)	4.6 (11.4)
	Industrial	7300 (2950)	4.6 (11.4)
	Other	9000 (3640)	4.6 (11.4)
	TOTAL or AVG.	49500 (20030)	4.6 (11.4)
TOTAL OR AVG.		104700 (42370)	9.6 (23.7)

The methodology is applied to the control network shown in Figure 14. In effect, the entire procedure is carried out for each drainage system area as though it were a separate entity. Within each system area, a specific control technique applied to one land use is regarded as a separate option. This is primarily due to the fact that the wet-weather BOD load is different for each land use. Therefore, in combined areas, the control network consists of eight options (four for street sweeping and four for sewer flushing) in parallel followed by storage-treatment in series. In the storm sewered areas, the control network consists of the four street sweeping options followed by storage-treatment in series. In the unsewered areas, the only control option is storage-treatment. The notation representing the various land uses and drainage systems found in Figure 14 will be used throughout the remainder of this section.

Before application of the methodology can begin, the wet-weather BOD loads (the pollutant of interest here), for each land use within each drainage system service area, must be estimated. Using the information provided on Table 9 and the relationships found in Appendix A, these values are computed and shown in Table 10. For comparative purposes, the annual wet-weather and dry-weather flows and the dry-weather BOD loads are also shown.

APPLICATION

Following the seven steps discussed and shown in Figures3, 4, 5 and 6 of Section V for each drainage system service area will give an optimal operating strategy applicable to that area. The production functions and total cost curves for the individual parallel options are derived from the information provided in Section VI and Table 9. The gutter length necessary to develop total cost curves for sweeping and flushing was computed from Equation 22 for residential areas and taken from Table 6 for other land uses. The production function and total cost curve for storage-treatment for each drainage system are found by applying the data found in Table 9 and the mean annual precipitation to the simplified assessment procedure discussed by Heaney, Huber, and Nix [7]. Anytown is assumed to be located in Region III [7]. The total cost curves for the fifteen options are presented in Appendix B.

Much uncertainty exists regarding the appropriate values to use for the availability factors for street sweeping (ϕ_{SW}) and sewer flushing (ϕ_{SF}). Thus, three different cases will be analyzed: (1) high availability ($\phi_{SF} = \phi_{SW} = 1.0$), medium availability ($\phi_{SF} = 0.8$, $\phi_{SW} = 0.7$), and low availability ($\phi_{SF} = 0.6$, $\phi_{SW} = 0.4$).

Street sweeping in the storm sewered areas will be used to illustrate how these four options shown in Figure 14 are combined into one equivalent option. The marginal cost curves for these four options and the composite marginal cost curve are shown in Figure 15. Recall that, for a given marginal cost, the pollutant removal by the four options is simply the sum of the individual removals. The removal by option SW4 is insignificant because the pollutant loading per acre is so small. The derived total cost curves for the three cases in the storm sewered area are shown in Figure 16.

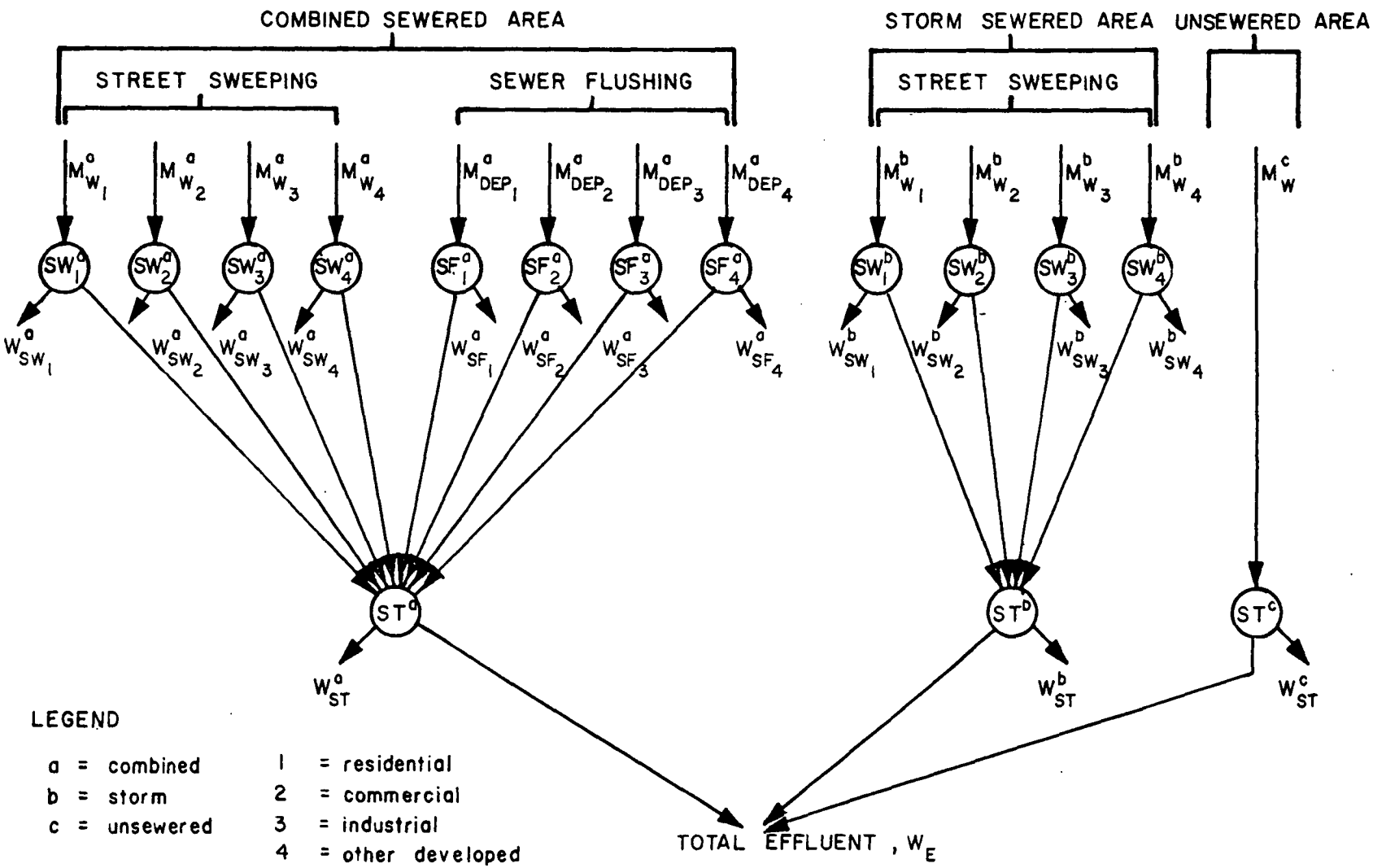


Figure 14. Stormwater Pollution Control Network for Anytown, U.S.A. - Availability Factors Equal 1.0

TABLE 10. ANNUAL WET- AND DRY-WEATHER FLOWS AND BOD LOADS FOR ANYTOWN, U.S.A.

Drainage System	Land Use	Wet-Weather Flow in (cm)	Dry-Weather Flow in (cm)	Dry-Weather BOD Load lb/ac (kg/ha)	Wet-Weather BOD Loads: Street Solids lb/ac (kg/ha)	Wet-Weather BOD Loads: Sewer Deposition lb/ac (kg/ha)	Wet-Weather BOD Loads: Street Solids 10^6lb (10^6kg)	Wet-Weather BOD Loads: Sewer Deposition 10^6lb (10^6kg)
Combined	Residential	13.7 (34.8)	22.4 (56.9)	942 (1057)	30 (34)	95 (107)	0.264 (0.120)	0.836 (0.380)
	Commercial	13.7 (34.8)	22.4 (56.9)	703 (789)	107 (120)	334 (375)	0.139 (0.063)	0.434 (0.197)
	Industrial	13.7 (34.8)	22.4 (56.9)	910 (1021)	40 (45)	127 (142)	0.088 (0.040)	0.279 (0.127)
	Other	13.7 (34.8)	22.4 (56.9)	1035 (1161)	0.5 (0.6)	1.7 (1.9)	0.001 (0.0005)	0.005 (0.002)
	TOTAL	-	-	-	-	-	0.492 (0.223)	1.554 (0.706)
Storm	Residential	12.5 (31.8)	17.4 (44.2)	807 (905)	27 (30)	-	0.634 (0.288)	-
	Commercial	12.5 (31.8)	17.4 (44.2)	807 (905)	107 (120)	-	0.364 (0.165)	-
	Industrial	12.5 (31.8)	17.4 (44.2)	807 (905)	40 (45)	-	0.236 (0.107)	-
	Other	12.5 (31.8)	17.4 (44.2)	807 (905)	0.5 (0.6)	-	0.004 (0.002)	-
	TOTAL	-	-	-	-	-	1.238 (0.562)	-
Unsewered	Residential	8.5 (21.6)	6.2 (15.7)	286 (321)	17 (19)	-	0.493 (0.224)	-
	Commercial	8.5 (21.6)	6.2 (15.7)	286 (321)	107 (120)	-	0.449 (0.204)	-
	Industrial	8.5 (21.6)	6.2 (15.7)	286 (321)	40 (45)	-	0.292 (0.133)	-
	Other	8.5 (21.6)	6.2 (15.7)	286 (321)	0.5 (0.6)	-	0.004 (0.002)	-
	TOTAL	-	-	-	-	-	1.238 (0.563)	
TOTAL							2.968 (1.347)	1.554 (0.706)
					TOTAL (all wet-weather sources)			4.522 (2.053)

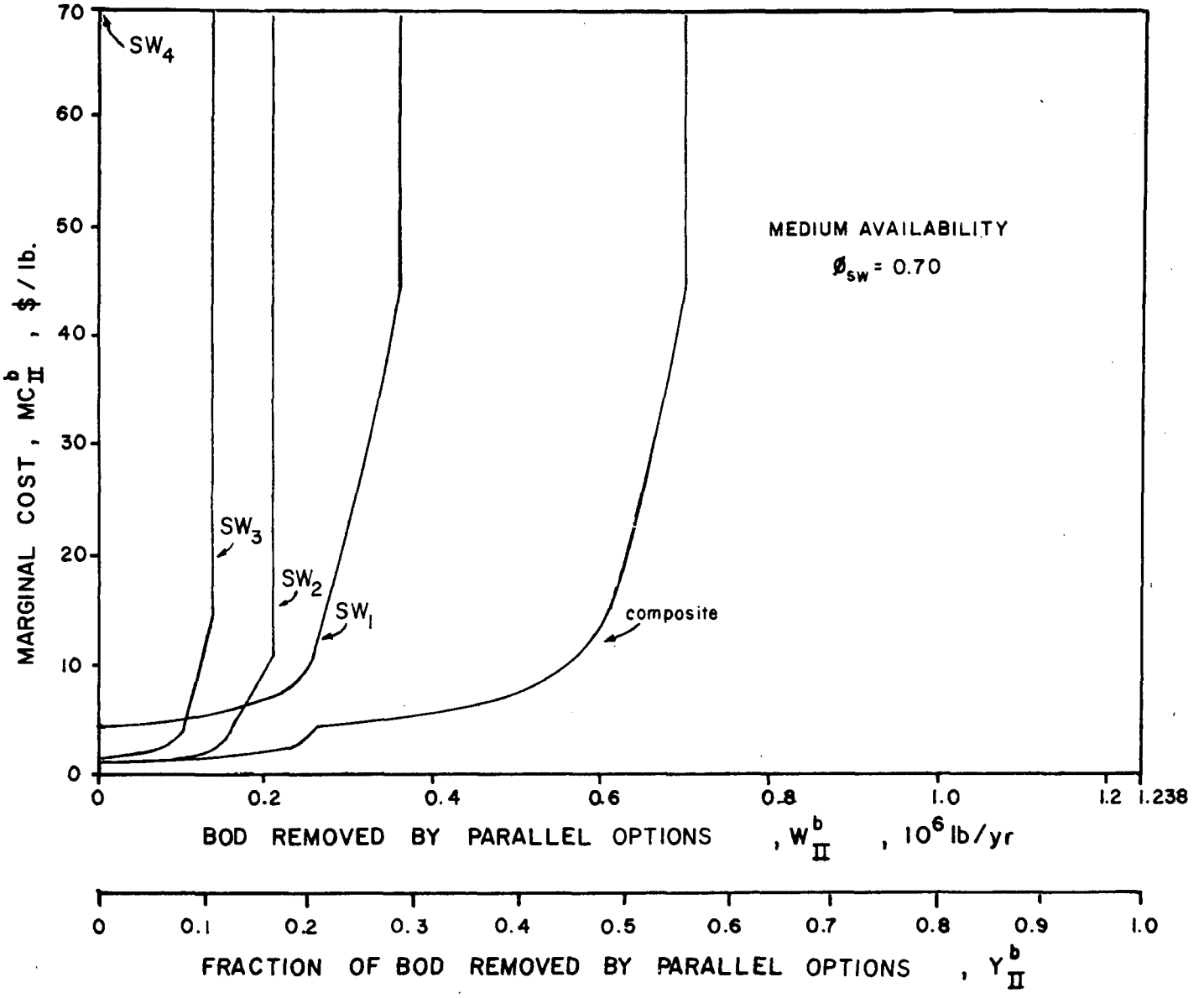


Figure 15. Marginal Cost Curves for the Parallel Options, Storm Areas - Medium Availability Factors

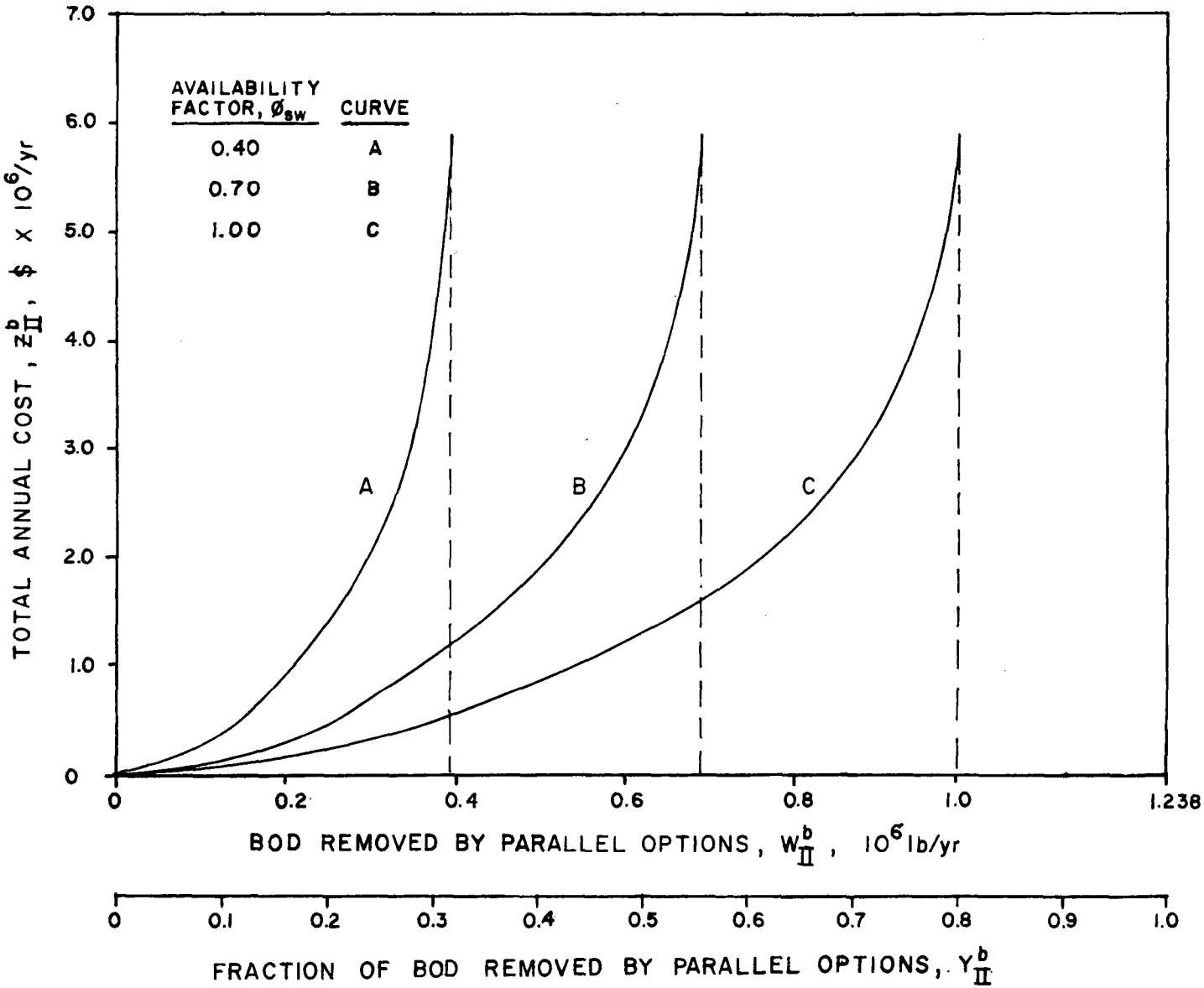


Figure 16. Total Cost Curves for the Parallel Options, Storm Areas

As expected, the total costs increase as the availability factor, ϕ_{SW}, decreases.

For the storm sewer area, the problem is now reduced to evaluating the optimal combination of the composite cost curve for the parallel options and the downstream total cost curve for storage-treatment shown in Figure 17. The resultant optimal solutions for the three assumed availability factors are shown in Figure 18 ($\phi_{SW} = 1.0$), Figure 19 ($\phi_{SW} = 0.7$), and Figure 20 ($\phi_{SW} = 0.4$). In each figure the ordinate and abscissa are scaled so that the maximum cost is used as the upper bound. Thus, the maximum overall pollutant removal is a point in the northwest corner of each figure, e.g., 97 percent in Figure 18. Lastly, the final results for the storm sewered area are shown in Figures 21 ($\phi_{SW} = 1.0$), 22 ($\phi_{SW} = 0.7$), and 23 ($\phi_{SW} = 0.4$).

Identical procedures are used to determine the optimal solutions for the combined sewered area and unsewered area. The result is now simply a case of three options in parallel. These three options are combined to obtain the final results which are presented, by availability factors, in Tables 11 (high), 12 (medium), and 13 (low). Lastly, the final total costs using storage-treatment only, and in conjunction with other management practices, are presented in Figure 24. If the availability factors are high, then the savings from using an integrated program are quite substantial.

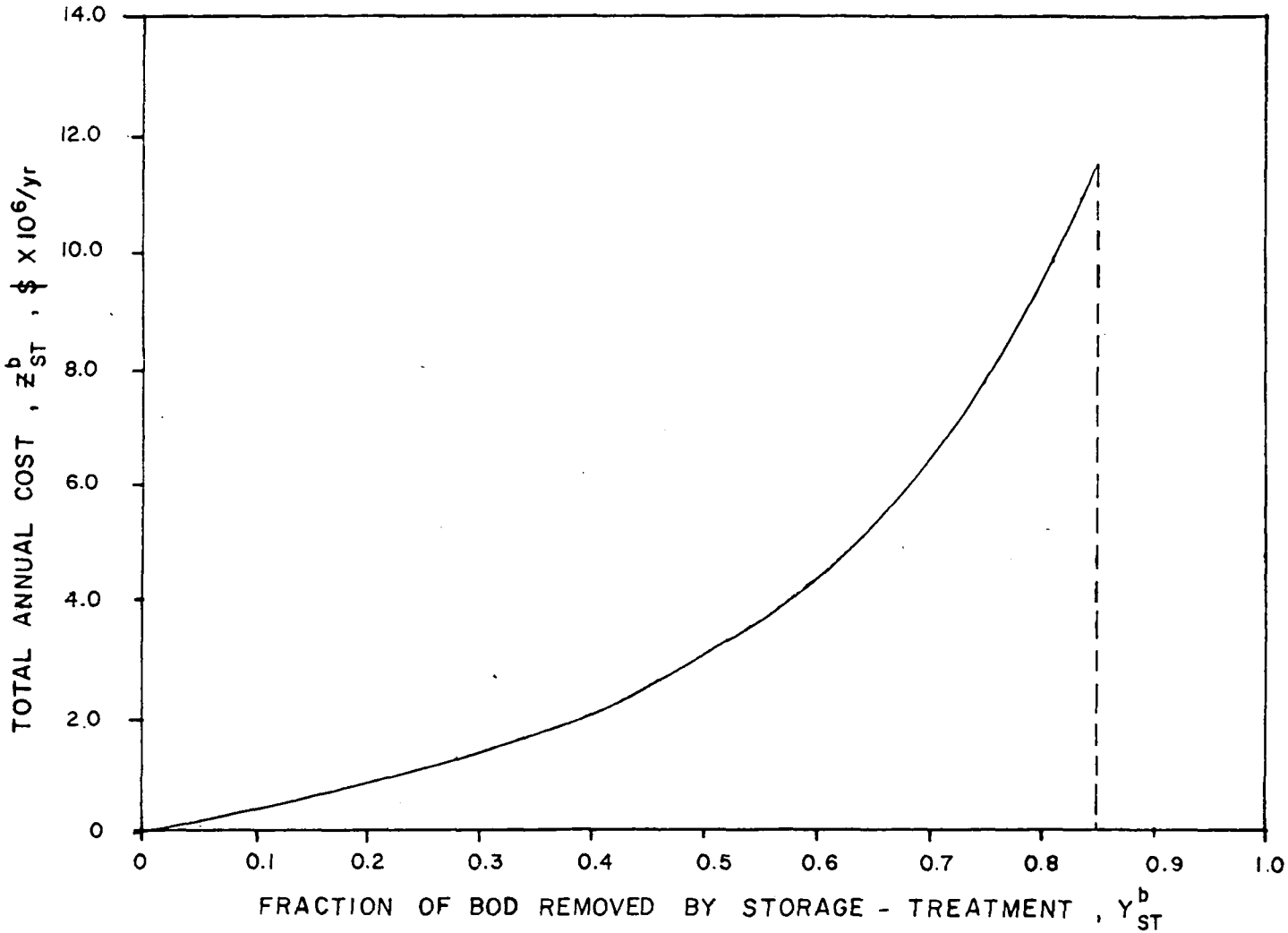


Figure 17. Total Cost Curve for Storage-Treatment, Storm Areas

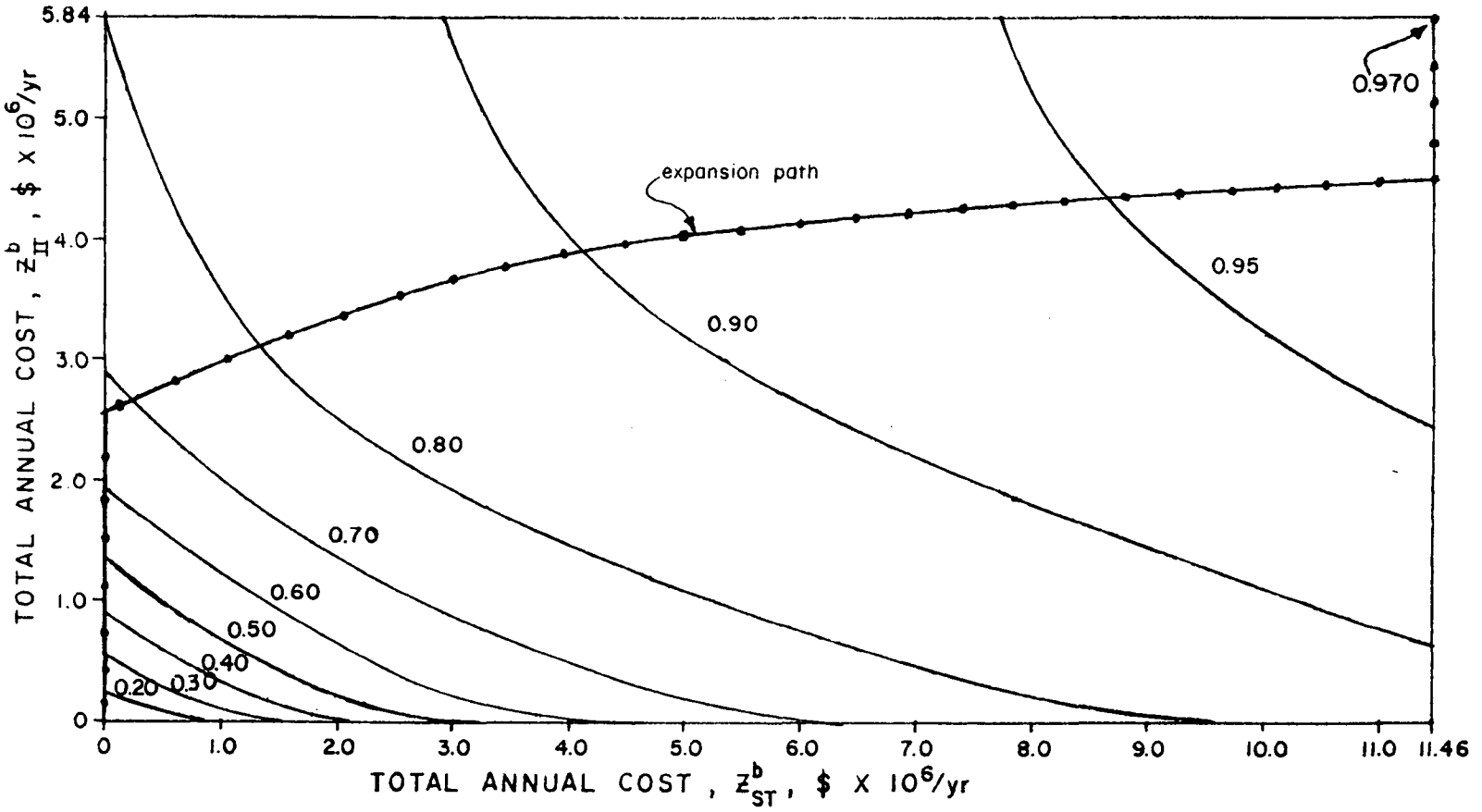


Figure 18. Isoquants of the Overall Fraction of BOD Removed, Storm Areas - High Availability Factors

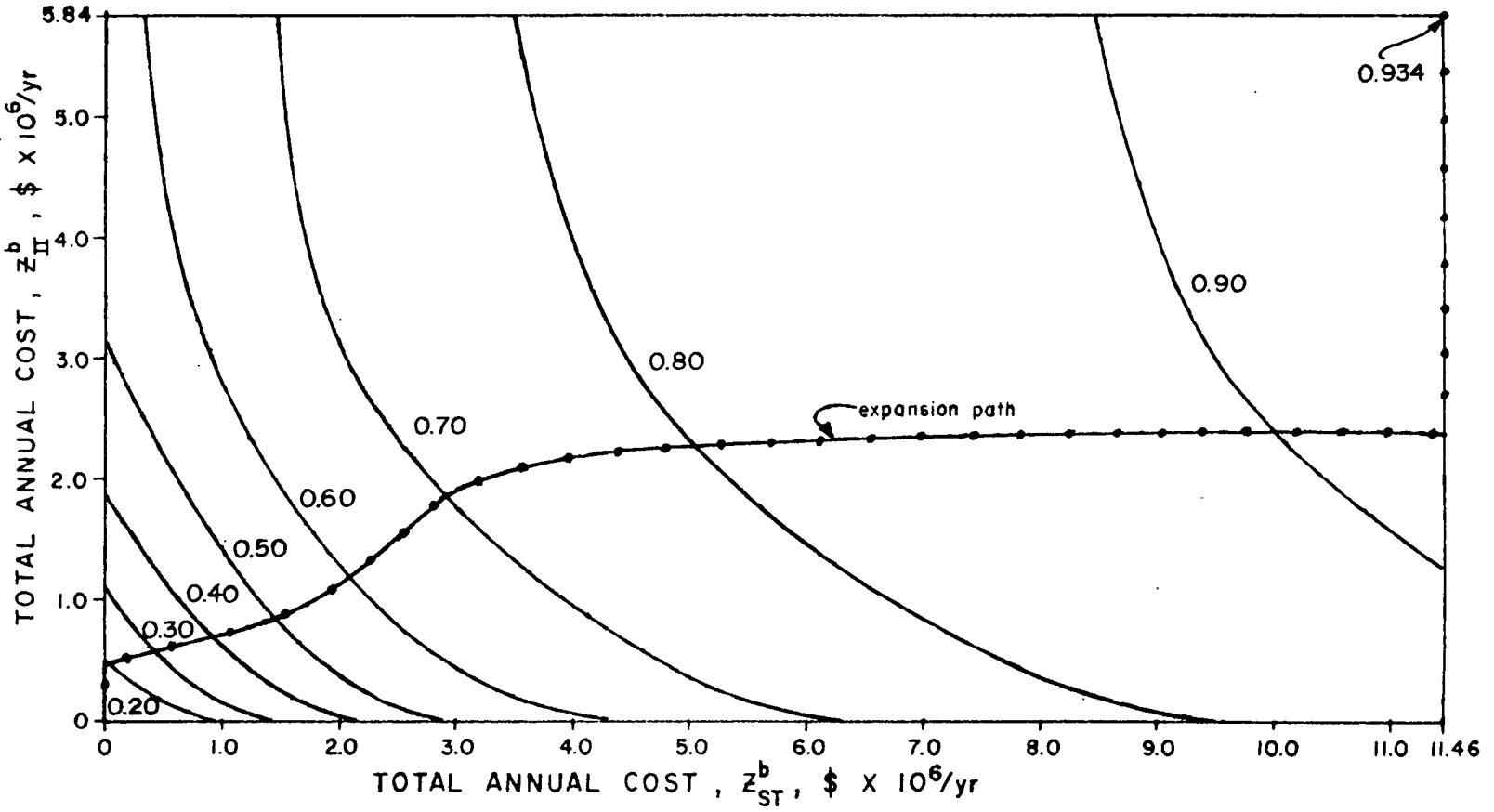


Figure 19. Isoquants of the Overall Fraction of BOD Removed, Storm Areas - Medium Availability Factors

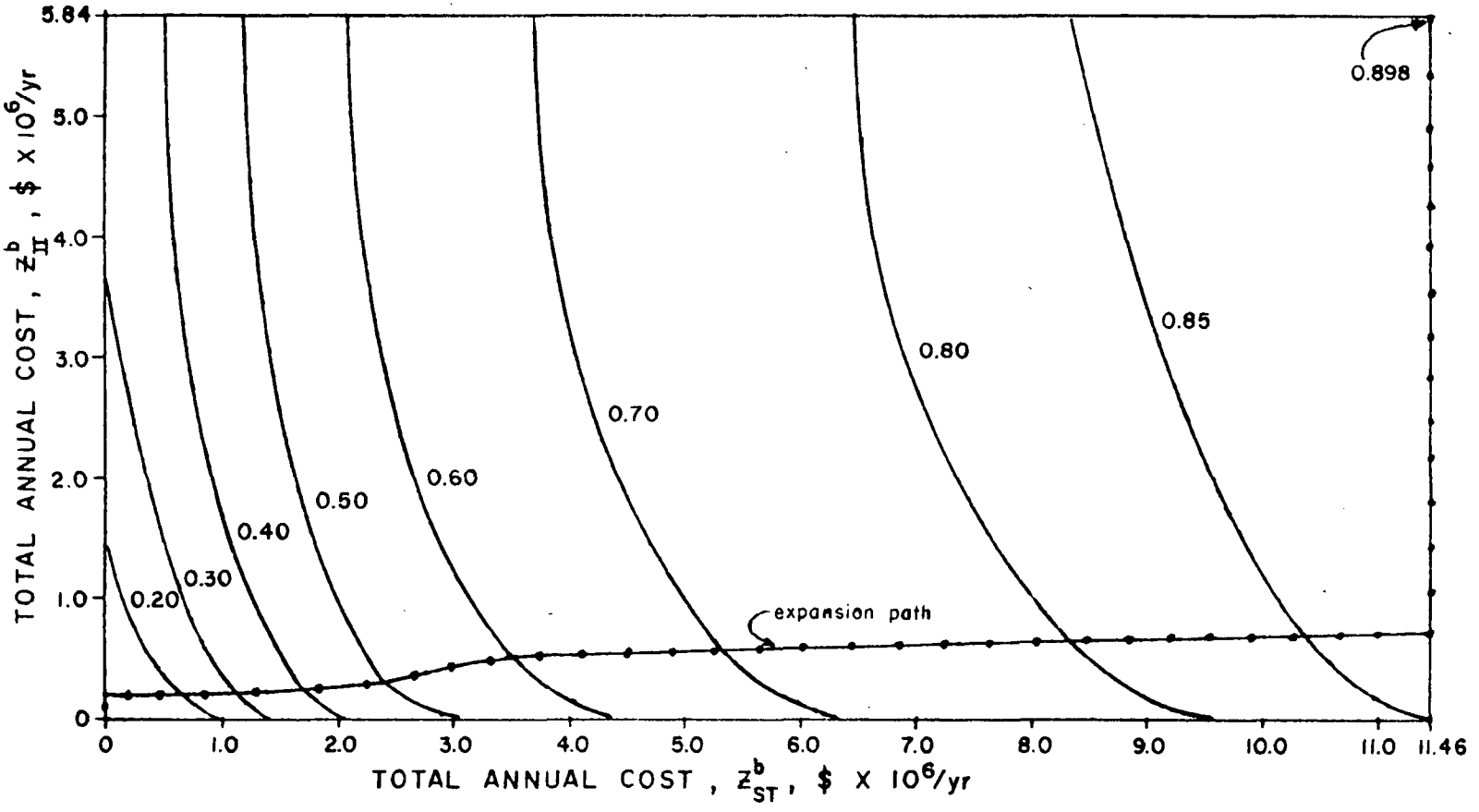


Figure 20. Isoquants of the Overall Fraction of BOD Removed, Storm Areas - Low Availability Factors

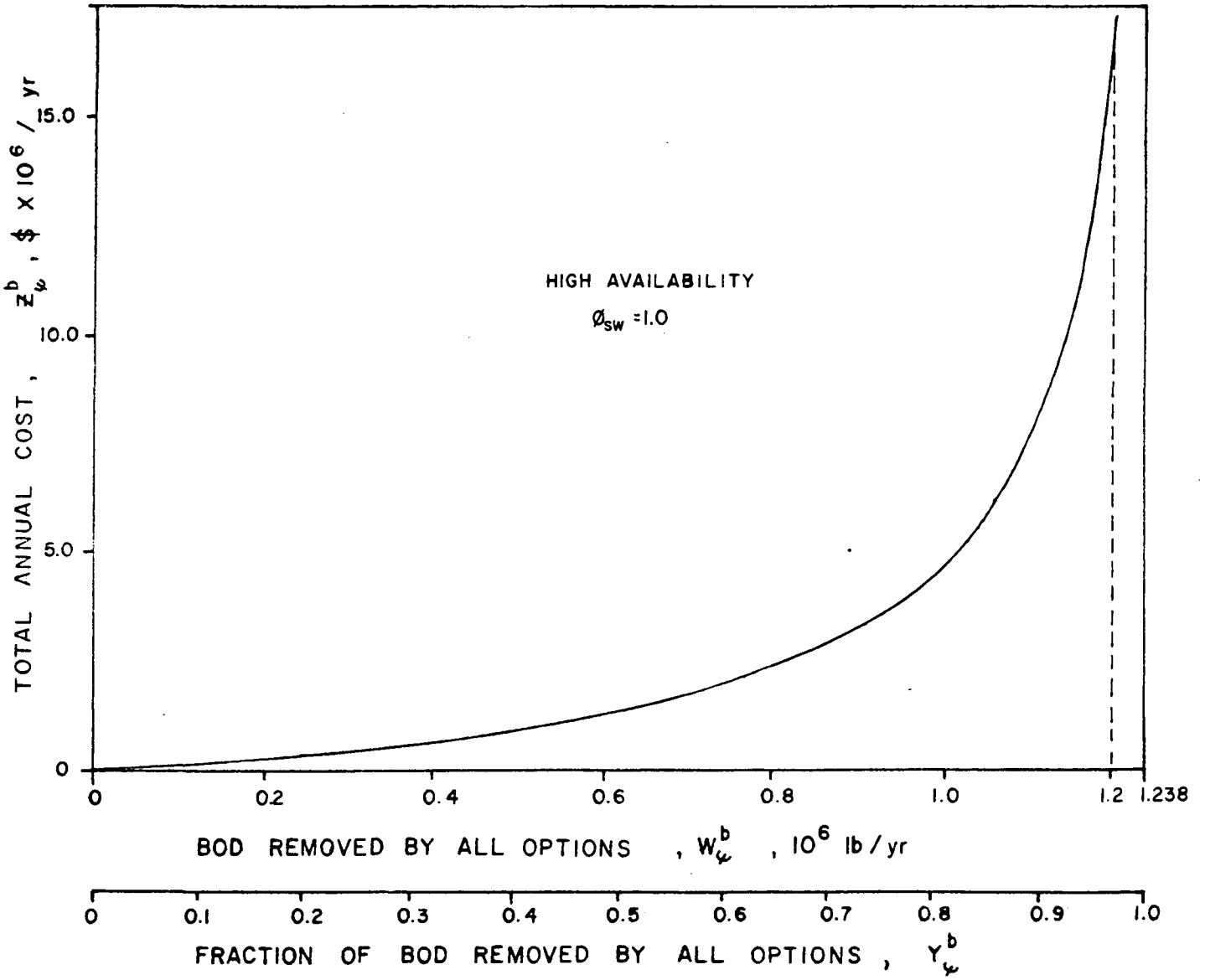


Figure 21. Total Cost Curve for All Options, Storm Areas - High Availability Factors

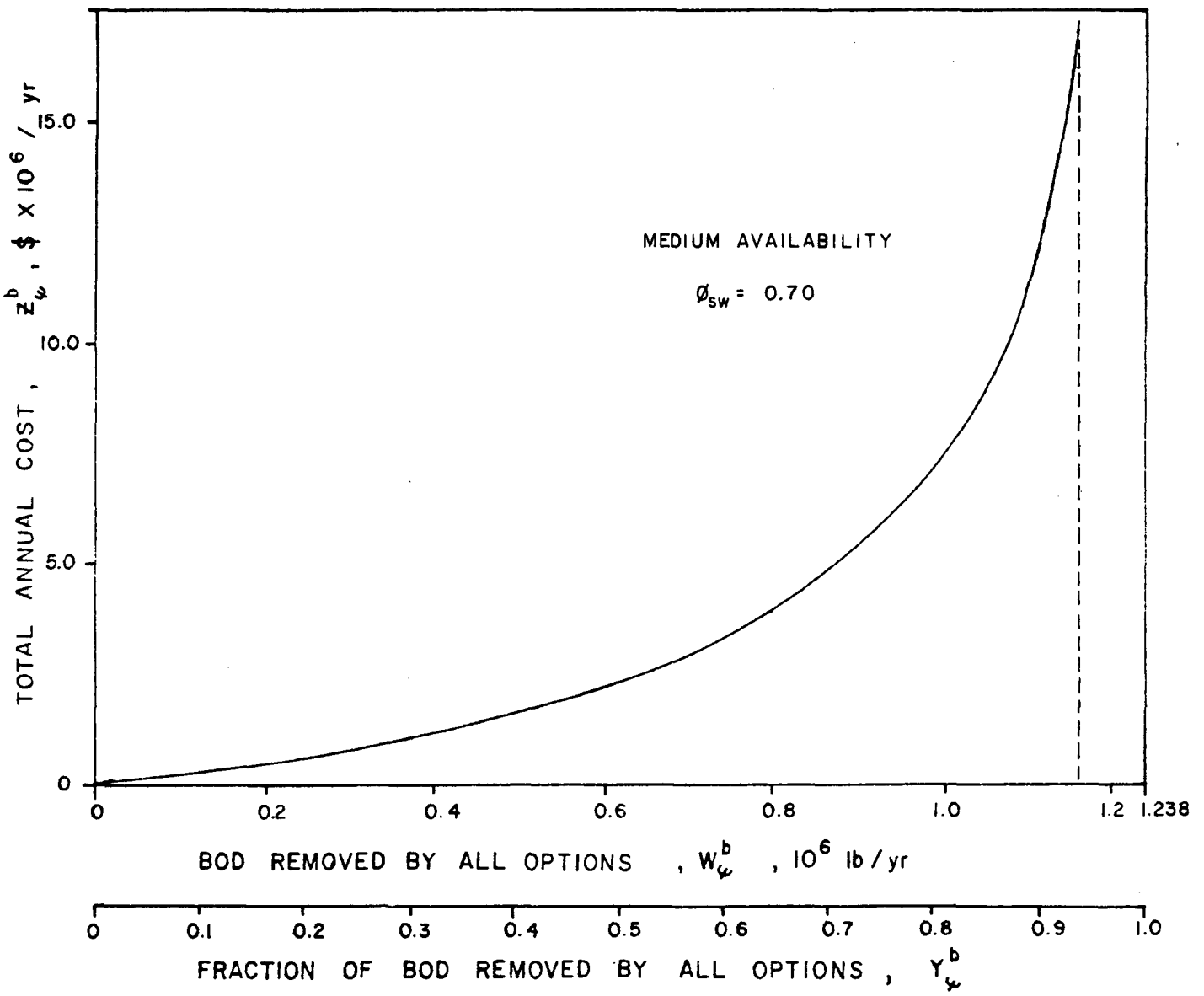


Figure 22. Total Cost Curve for All Options, Storm Areas - Medium Availability Factors

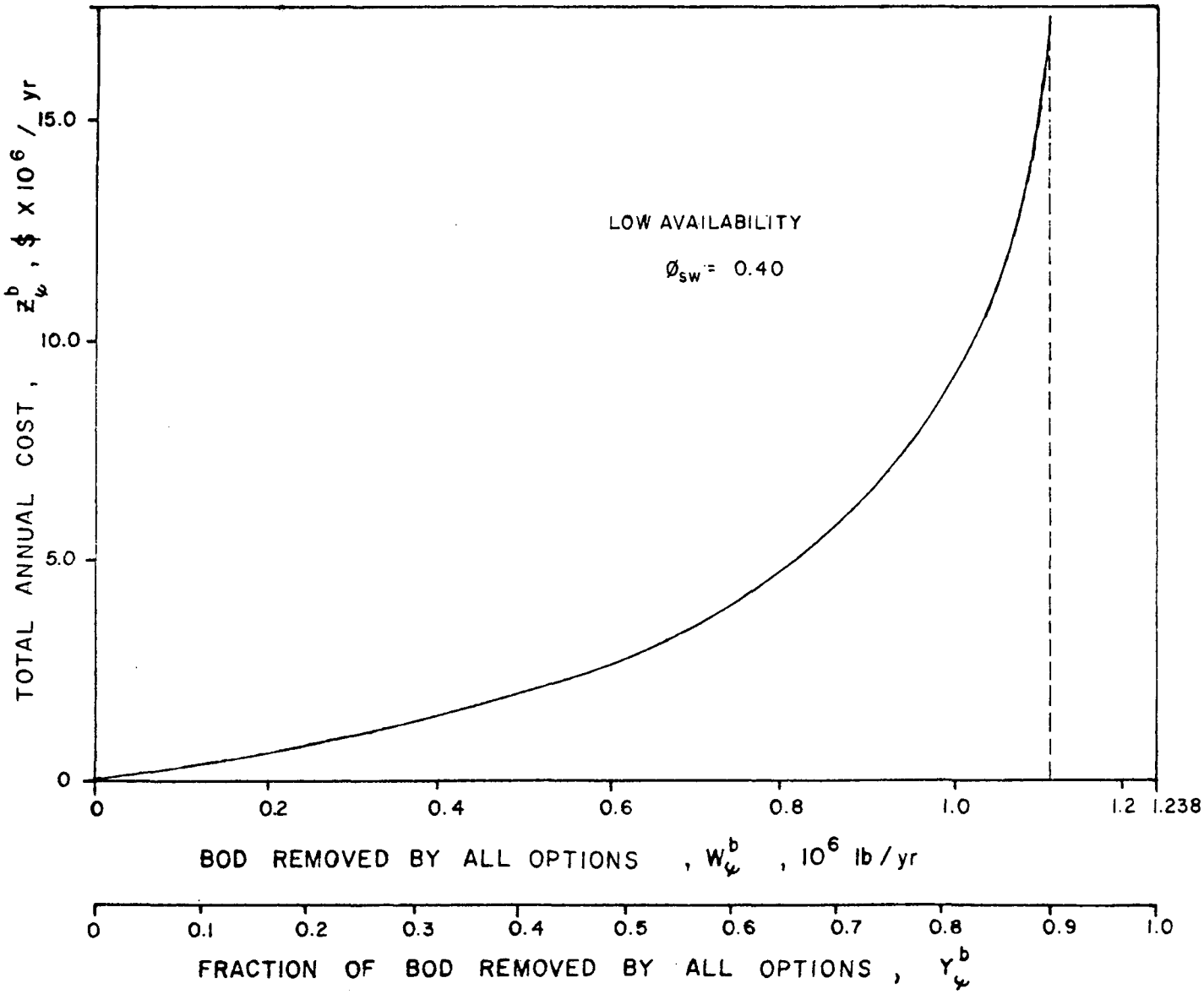


Figure 23. Total Cost Curve for All Options, Storm Areas - Low Availability Factors

TABLE 11. OPTIMAL STRATEGY FOR 25, 50, 75 AND 85 PERCENT OVERALL BOD REMOVAL, ANYTOWN, U.S.A., - HIGH AVAILABILITY FACTORS

Drainage System	Alternatives	Land Use	BOD Removed By Each Process (10^6 lb/yr)				Fraction of Influent BOD Removed By Each Process				Total Cost of Each Process ($ x 10^6/yr)			
			25%	50%	75%	85%	25%	50%	75%	85%	25%	50%	75%	85%
Combined	Street Sweeping	Residential	0	0	0.06	0.11	0	0	0.22	0.43	0	0	0.18	0.39
		Commercial	0.07	0.08	0.09	0.09	0.50	0.56	0.64	0.68	0.07	0.08	0.11	0.13
		Industrial	0.03	0.04	0.05	0.06	0.36	0.49	0.61	0.65	0.04	0.05	0.08	0.09
		Other	0	0	0	0	0	0	0	0	0	0	0	0
	Sewer Flushing	Residential	0.22	0.25	0.30	0.32	0.27	0.29	0.36	0.38	0.08	0.13	0.27	0.35
		Commercial	0.18	0.19	0.24	0.26	0.41	0.45	0.54	0.59	0.06	0.09	0.20	0.29
		Industrial	0.10	0.12	0.14	0.16	0.37	0.42	0.51	0.56	0.04	0.06	0.13	0.18
		Other	0	0	0	0	0	0	0	0	0	0	0	0
	Storage-Treatment		0	0.40	0.65	0.78	0	0.29	0.56	0.75	0	0.85	2.43	5.35
	TOTAL/AVG.		0.60	1.08	1.53	1.78	0.30	0.53	0.75	0.87	0.29	1.26	3.40	6.78
Storm	Street Sweeping	Residential	0	0.09	0.35	0.41	0	0.14	0.56	0.64	0	0.28	1.48	2.07
		Commercial	0.14	0.23	0.29	0.29	0.39	0.63	0.79	0.80	0.12	0.27	0.59	0.61
		Industrial	0.01	0.14	0.18	0.19	0.02	0.59	0.75	0.80	0.01	0.20	0.40	0.51
		Other	0	0	0	0	0	0	0	0	0	0	0	0
	Storage-Treatment		0	0	0	0.12	0	0	0	0.33	0	0	0	1.55
	TOTAL/AVG.		0.15	0.46	0.82	1.01	0.12	0.37	0.66	0.82	0.13	0.75	2.47	4.74
Unsewered TOTAL/AVG.			0.36	0.70	1.05	1.05	0.29	0.57	0.85	0.85	0.43	1.10	2.82	2.82
TOTAL/AVG.			1.11	2.24	3.40	3.84	0.25	0.50	0.75	0.85	0.85	3.11	8.69	14.34

TABLE 12. OPTIMAL STRATEGY FOR 25, 50, 75 AND 85 PERCENT OVERALL BOD REMOVAL, ANYTOWN, U.S.A. - MEDIUM AVAILABILITY FACTORS

Drainage System	Alternatives	Land Use	BOD Removed By Each Process (10^6 lb/yr)				Fraction of Influent BOD Removed By Each Process				Total Cost of Each Process ($ x 10^6/yr)			
			25%	50%	75%	85%	25%	50%	75%	85%	25%	50%	75%	85%
Combined	Street Sweeping	Residential	0	0	0	0.04	0	0	0	0.22	0	0	0	0.18
		Commercial	0.03	0.05	0.06	0.06	0.32	0.48	0.63	0.65	0.04	0.06	0.11	0.11
		Industrial	0	0.02	0.04	0.04	0	0.33	0.58	0.60	0	0.03	0.07	0.08
		Other	0	0	0	0	0	0	0	0	0	0	0	0
	Sewer Flushing	Residential	0.16	0.18	0.24	0.25	0.24	0.27	0.35	0.37	0.06	0.09	0.25	0.30
		Commercial	0.13	0.15	0.19	0.20	0.38	0.42	0.54	0.57	0.05	0.08	0.20	0.25
		Industrial	0.09	0.09	0.11	0.12	0.35	0.38	0.51	0.52	0.03	0.04	0.12	0.14
		Other	0	0	0	0	0	0	0	0	0	0	0	0
	Storage-Treatment		0.18	0.65	0.93	1.08	0.11	0.41	0.66	0.80	0.30	1.35	3.75	6.80
	TOTAL/AVG.		0.58	1.14	1.57	1.79	0.29	0.55	0.77	0.87	0.48	1.65	4.50	7.86
Storm	Street Sweeping	Residential	0	0.01	0.14	0.24	0	0.02	0.33	0.54	0	0.04	0.71	1.39
		Commercial	0.08	0.16	0.17	0.19	0.31	0.62	0.67	0.76	0.10	0.26	0.33	0.49
		Industrial	0.01	0.10	0.10	0.12	0.07	0.59	0.62	0.71	0.02	0.20	0.23	0.33
		Other	0	0	0	0	0	0	0	0	0	0	0	0
	Storage-Treatment		0	0.07	0.36	0.45	0	0.08	0.43	0.65	0	0.32	2.29	5.22
	TOTAL/AVG.		0.09	0.34	0.77	1.00	0.07	0.28	0.63	0.81	0.12	0.82	3.56	7.43
Unsewered TOTAL/AVG.			0.45	0.80	1.05	1.05	0.36	0.65	0.85	0.85	0.56	1.42	2.82	2.82
TOTAL/AVG.			1.12	2.28	3.39	3.84	0.25	0.50	0.75	0.85	1.16	3.89	10.88	18.11

TABLE 13. OPTIMAL STRATEGY FOR 25, 50, 75 AND 85 PERCENT OVERALL BOD REMOVAL, ANYTOWN, U.S.A. - LOW AVAILABILITY FACTORS

Drainage System	Alternatives	Land Use	BOD Removed By Each Process (10^6 lb/yr)				Fraction of Influent BOD Removed By Each Process				Total Cost of Each Process ($ x 10^6/yr)			
			25%	50%	75%	85%	25%	50%	75%	85%	25%	50%	75%	85%
Combined	Street Sweeping	Residential	0	0	0	0.02	0	0	0	0.16	0	0	0	0.14
		Commercial	0	0.01	0.02	0.04	0	0.22	0.43	0.65	0	0.02	0.05	0.11
		Industrial	0	0	0.01	0.02	0	0	0.23	0.57	0	0	0.02	0.07
		Other	0	0	0	0	0	0	0	0	0	0	0	0
	Sewer Flushing	Residential	0.12	0.13	0.14	0.20	0.24	0.26	0.28	0.40	0.06	0.07	0.11	0.47
		Commercial	0.10	0.11	0.12	0.16	0.38	0.40	0.45	0.61	0.05	0.06	0.08	0.34
		Industrial	0.06	0.06	0.07	0.09	0.36	0.39	0.42	0.57	0.03	0.04	0.07	0.20
		Other	0	0	0	0	0	0	0	0	0	0	0	0
	Storage-Treatment		0.37	0.82	1.23	1.29	0.21	0.47	0.73	0.85	0.57	1.80	4.97	8.14
	TOTAL/AVG.		0.65	1.13	1.59	1.82	0.32	0.55	0.78	0.89	0.71	1.99	5.30	9.47
Storm	Street Sweeping	Residential	0	0	0	0.02	0	0	0	0.06	0	0	0	0.12
		Commercial	0	0.06	0.09	0.09	0	0.39	0.64	0.65	0	0.13	0.29	0.29
		Industrial	0	0.02	0.06	0.06	0	0.23	0.59	0.60	0	0.06	0.20	0.21
		Other	0	0	0	0	0	0	0	0	0	0	0	0
	Storage-Treatment		0	0.24	0.61	0.82	0	0.21	0.56	0.77	0	0.89	3.71	8.35
	TOTAL/AVG.		0	0.32	0.76	0.99	0	0.26	0.61	0.80	0	1.08	4.20	8.97
Unsewered TOTAL/AVG.			0.49	0.83	1.05	1.05	0.39	0.67	0.85	0.85	0.61	1.56	2.82	2.82
TOTAL/AVG.			1.14	2.28	3.40	3.86	0.25	0.50	0.75	0.85	1.32	4.63	12.32	21.26

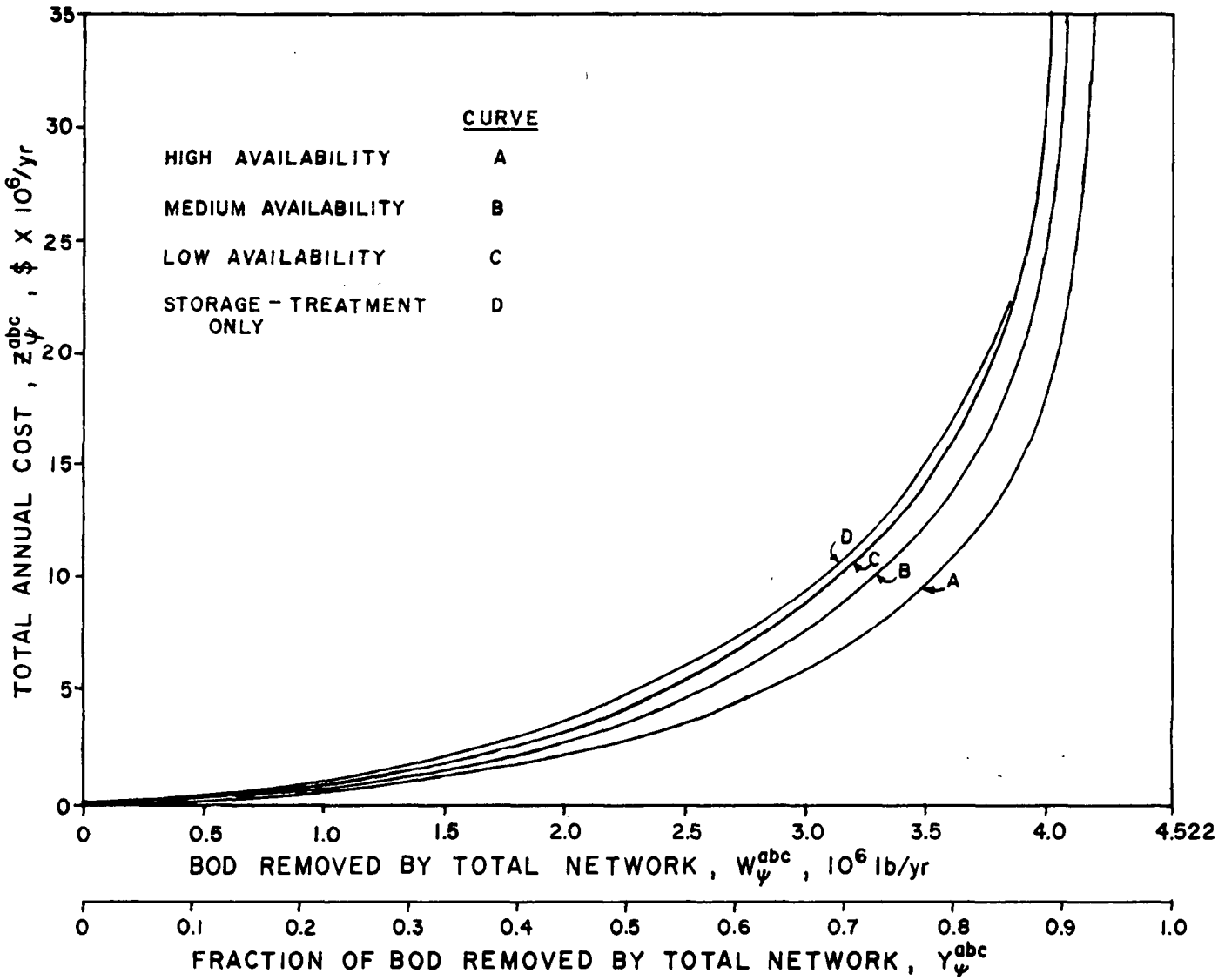


Figure 24. Total Cost Curves for All Drainage System Service Areas

REFERENCES

1. Heaney, J.P., Huber, W.C., Medina, M.A., Murphy, M.P., Nix, S.J., and Hasan, S.M., "Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Volume II: Cost Assessment," USEPA Report EPA-600/2-77-064B, March 1977.

2. Pisano, W.A., "Cost Effective Approach for Combined and Storm Sewer Cleanup," in Proc. Urban Stormwater Management Seminars, USEPA Report WPD-03-76-04, Jan. 1976.

3. FMC Corporation, "A Flushing System for Combined Sewer Cleansing," USEPA Report 11020 DNO, March 1972.

4. Sartor, J.D. and Boyd, G.B., "Water Pollution Aspects of Street Surface Contaminants," USEPA Report EPA-22-72-081, November 1972.

5. American Public Works Association, "Water Pollution Aspects of Urban Runoff," USEPA Report 11030 DNS 01/69, January 1969.

6. Metcalf and Eddy, Inc., "Urban Stormwater Management and Technology: An Assessment," USEPA Report EPA-670/2-74-040, December 1974.

7. Heaney, J.P., Huber, W.C., and Nix, S.J., "Stormwater Management Model: Level I--Preliminary Screening Procedures," USEPA Report EPA-600/2-76-275, October 1976.

8. Federal Water Pollution Control Act Amendments of 1972, PL 92-500.

9. Guidelines for Areawide Waste Treatment Management Planning, USEPA, 1975.

10. U.S. Bureau of the Census, County and City Data Book, 1972 (A Statistical Abstract Supplement), Social and Economic Statistic Administration, U.S. Department of Commerce, 1973.

11. Baumol, W.J., Economic Theory and Operations Analysis, Prentice-Hall, Inc., Englewood Clifs, New Jersey, 1965.

12. James, L.D., and Lee, R.R., Economics of Water Resources Planning, McGraw-Hill, Inc., New York, 1971.

13. Huber, W.C., Heaney, J.P., Medina, M.A., Peltz, W.A., Hasan, S.M., and Smith, G.F., "Storm Water Management Model; User's Manual--Version II," USEPA Report EPA-670/2-75-017, March 1975.

14. Hydrologic Engineering Center, Corps of Engineers, "Urban Storm Water Runoff: STORM," Generalized Computer Program 723-58-L2520, May 1975.

15. Graham, R.H., Costello, L.S., and Mallon, H.J., "Estimation of Imperviousness and Specific Curb Length for Forecasting Stormwater Quality and Quantity," Journal of the Water Pollution Control Federation, Vol. 46, No. 4, April 1974, pp. 717-725.

16. Unpublished data from American Public Works Association, 1976.

17. Northeastern U., "Characterization of Solids Behavior in and Variability Testing of Selected Control Techniques for Combined Sewer Systems," EPA Grant No. R-804578, R. Field, Project Officer, Edison, N.J., 1977.

18. Metcalf and Eddy, Inc., "Catchbasin Technology: Overview and Assessment," USEPA Contract No. 68-03-0274 (Draft), September 1976.

GLOSSARY

Combined sewage: Sewage containing both domestic sewage and surface water or stormwater, with or without industrial wastes. Includes flow in heavily infiltrated sanitary sewer systems as well as combined sewer systems.

Combined sewer: A sewer receiving both intercepted surface runoff and municipal sewage.

Combined sewer overflow: Flow from a combined sewer in excess of the interceptor capacity that is discharged into a receiving water.

Depression storage: Amount of precipitation which can fall on an area without causing runoff.

Detention: The slowing, dampening, or attenuating of flows either entering the sewer system or within the sewer system by temporarily holding the water on a surface area, in a storage basin, or within the sewer itself.

Domestic sewage: Sewage derived principally from dwellings, business buildings, institutions, and the like. It may or may not contain groundwater.

Isocost lines: Lines of equal cost.

Isoquants: Curves representing combinations of the inputs yielding the same amount of output.

Marginal cost: The rate of change of total cost.

Precipitation event: A precipitation event terminates if zero rainfall has been recorded for the previous specified time interval.

Production function: Locus of technologically efficient combinations of inputs and outputs.

Runoff coefficient: Fraction of rainfall that appears as runoff after subtracting depression storage and interception. Typically accounts for infiltration into ground and evaporation.

Storm flow: Overland flow, sewer flow, or receiving stream flow caused totally or partially by surface runoff or snowmelt.

Storm sewer: A sewer that carries intercepted surface runoff, street wash and other wash waters, or drainage, but excludes domestic sewage and industrial wastes.

Storm sewer discharge: Flow from a storm sewer that is discharged into a receiving water.

Stormwater: Water resulting from precipitation which either percolates into the soil, runs off freely from the surface, or is captured by storm sewer, combined sewer, and to a limited degree sanitary sewer facilities.

Surface runoff: Precipitation that falls onto the surfaces of roofs, streets, ground, etc., and is not absorbed or retained by that surface, thereby collecting and running off.

APPENDIX A

QUANTITY AND QUALITY ANALYSIS

In order to develop an optimal stormwater pollution control strategy, the magnitude of the problem must be estimated. Several methods are available to estimate the quantity and quality of urban runoff. A simplied method to assess stormwater pollution loads and control costs by Heaney, Huber, and Nix can be used to compute these parameters for any urbanized area [A1]. In addition to the runoff estimations, equations are presented to determine the corresponding dry-weather (sanitary sewage) flows and quality. This methodology is briefly described below. The following equations may be applied to any land use or sewerage system service area.

Annual runoff may be estimated by the following equation:

$$AR = (0.15 + 0.75\ (I/100))\ P - 5.234\ DS^{0.5957} \qquad (A1)$$

where AR = annual runoff, in;

I = total imperviousness, percent;

P = annual precipitation, in; and

DS = annual depression storage, in.

The annual depression storage is an index of the available areas capable of retaining precipitation. This parameter is determined by the following relationship,

$$DS = 0.25 - 0.1875\ (I/100). \qquad (A2)$$

The equation used to estimate imperviousness is

$$I = 9.6\ PD_d^{(0.573-0.0391\ \log_{10}PD_d)} \qquad (A3)$$

where PD_d = population density in the developed area, persons/ac.

Knowing the population density of the area allows the annual runoff to be quickly determined.

The dry-weather flow may be estimated by the following equation,

$$DWF = 1.34\ PD_d \qquad (A4)$$

where DWF = dry weather flow, in/hr.

This relationship is based on an assumed dry-weather flow of 100 gallons/capita-day (378ℓ/capita-day).

Estimating the quality of urban runoff presents a more difficult task. Available data indicate wide variation in estimated pollutant loads. If annual pollutant loads are assumed to vary as a function of population density, precipitation, land use and type of sewerage system, the following relationships may be used:

$$m_W = \beta(i,j) \cdot P \cdot f_i(PD_d) \quad \text{for combined sewered areas,} \qquad (A5)$$

and

$$m_W = \alpha(i,j) \cdot P \cdot f_i(PD_d) \quad \text{for storm and unsewered areas,} \qquad (A6)$$

where m_W = annual wet weather pollutant load, lb/ac-yr;

P = annual precipitation, in/yr;

$f_i(PD_d)$ = population density function for land use i;

$\alpha(i,j)$ = coefficient for storm and unsewered areas for pollutant j on land use i, lb/ac-yr-in; and

$\beta(i,j)$ = coefficient for combined sewered areas for pollutant j on land use i, lb/ac-yr-in.

Values of $\alpha(i,j)$, $\beta(i,j)$ and $f_i(PD)$ are shown in Table AI.

The equation used to estimate dry-weather quality, in terms of BOD, is

$$m_{Dw} = 62.1\ PD_d - m_{DEP} \qquad (A7)$$

where m_{Dw} = annual dry-weather BOD load, lb/ac-yr; and

m_{DEP} = annual BOD load of combined sewer deposition, lb/ac-yr.

This estimate assumes a per capita BOD discharge of 0.17 lbs (77 gm)/day.

Combined sewer deposition is defined as that portion of the dry-weather pollutant load that is deposited in the combined sewers, usually due to inadequate carrying velocities. Often, this load is flushed from the sewers during runoff periods and becomes part of the stormwater discharge. The deposition may be estimated by computing the difference between combined and storm sewered area BOD loadings derived for the combined area of concern.

TABLE A1. POLLUTANT LOADING FACTORS

Land Uses: i = 1 Residential
i = 2 Commercial
i = 3 Industrial
i = 4 Other (assume $PD_d = 0$)

Pollutants: j = 1 BOD_5, Total
j = 2 Suspended Solids (SS)
j = 3 Volatile Solids, Total (VS)
j = 4 Total PO_4 (as PO_4)
j = 5 Total N

Population Function: $i = 1 \quad f_i(PD_d) = 0.142 + 0.218 \cdot PD_d^{0.54}$
$i = 2,3 \quad f_i(PD_d) = 1.0$
$i = 4 \quad f_i(PD_d) = 0.142$

α and β Factors for Equations: Storm factors, α, and combined factors, β, have units lb/ac-yr-in.

			Pollutant, j				
			1. BOD_5	2. SS	3. VS	4. PO_4	5. N
Storm Areas, α	1.	Residential	0.799	16.3	9.45	0.0336	0.131
	2.	Commercial	3.20	22.2	14.0	0.0757	0.296
	3.	Industrial	1.21	29.1	14.3	0.0705	0.277
	4.	Other	0.113	2.70	2.6	0.00994	0.0605
Combined Areas, β	1.	Residential	3.29	67.2	38.9	0.139	0.540
	2.	Commercial	13.2	91.8	57.9	0.312	1.22
	3.	Industrial	5.00	120.0	59.2	0.291	1.14
	4.	Other	0.467	11.1	10.8	0.0411	0.250

Source: Heaney, J.P., Huber, W.C., and Nix, S.J., "Storm Water Management Model: Level I--Preliminary Screening Procedures," USEPA Report EPA-600/2-76-275, October 1976, p. 17.

This assumes that the greater loads experienced by combined areas are due to the deposition of dry-weather solids. Thus, combined sewer deposition is estimated by the following equation:

$$m_{DEP} = (\beta(i,j)-\alpha(i,j)) \cdot P \cdot f_i(PD_d). \qquad (A8)$$

Of course, for storm sewered and unsewered areas, deposition from dry-weather sources is not computed unless there are illicit connections of sewage to the storm sewers.

REFERENCE

A1. Heaney, J. P., Huber, W. C., and S. J. Nix, "Storm-Water Management Model: Level I--Preliminary Screening Procedures," EPA-600/2-76-275, Cincinnati, Ohio, Oct. 1976.

APPENDIX B

WORKING CURVES FOR GRAPHICAL SOLUTION

The basic curves for the analysis consist of one curve for each option showing total cost as a function of pounds of pollutant removed, and level of pollutant control. The scaling of each curve is set by the total cost as a a function of the level of pollution control which ranges from 0 to 1. The actual pounds removed differ as a function of the availability factor, ϕ, and the total load, M. Thus, the scaling on this abscissa is set up as a function of ϕ. The curves are arranged as follows:

Figures		Pages
B1-B4	Total Cost Curves for Combined Sewered Areas, by Land Use, Street Sweeping	74-77
B5-B8	Total Cost Curves for Combined Sewered Areas, by Land Use, Sewer Flushing	78-81
B9-B12	Total Cost Curves for Storm Sewered Areas, by Land Use, Street Sweeping	82-85
B13-B15	Total Cost Curves for Storage-Treatment, by Type of Sewerage System	86-88

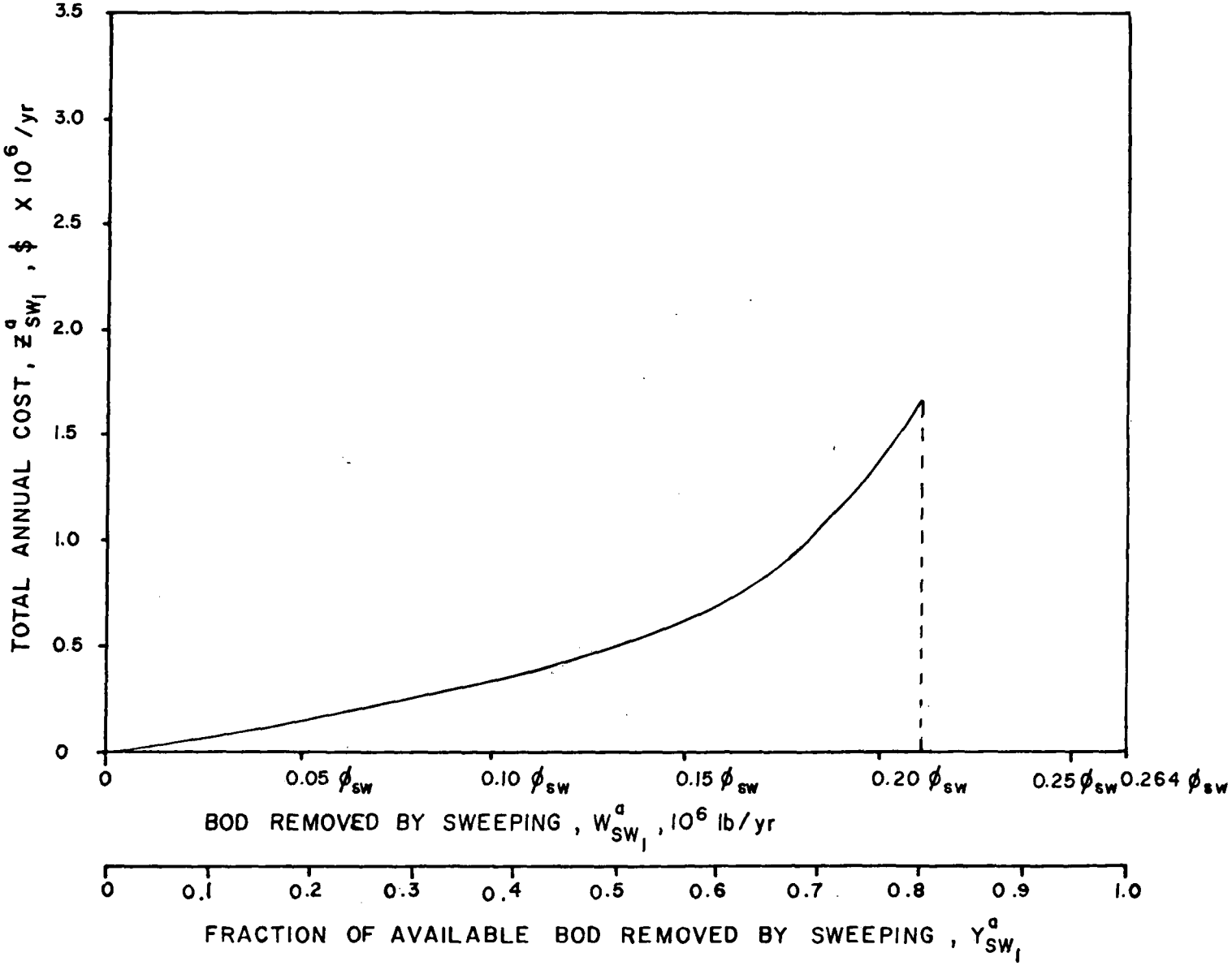


Figure B1. Total Cost Curve for Residential Portion of Combined Sewered Areas, Street Sweeping

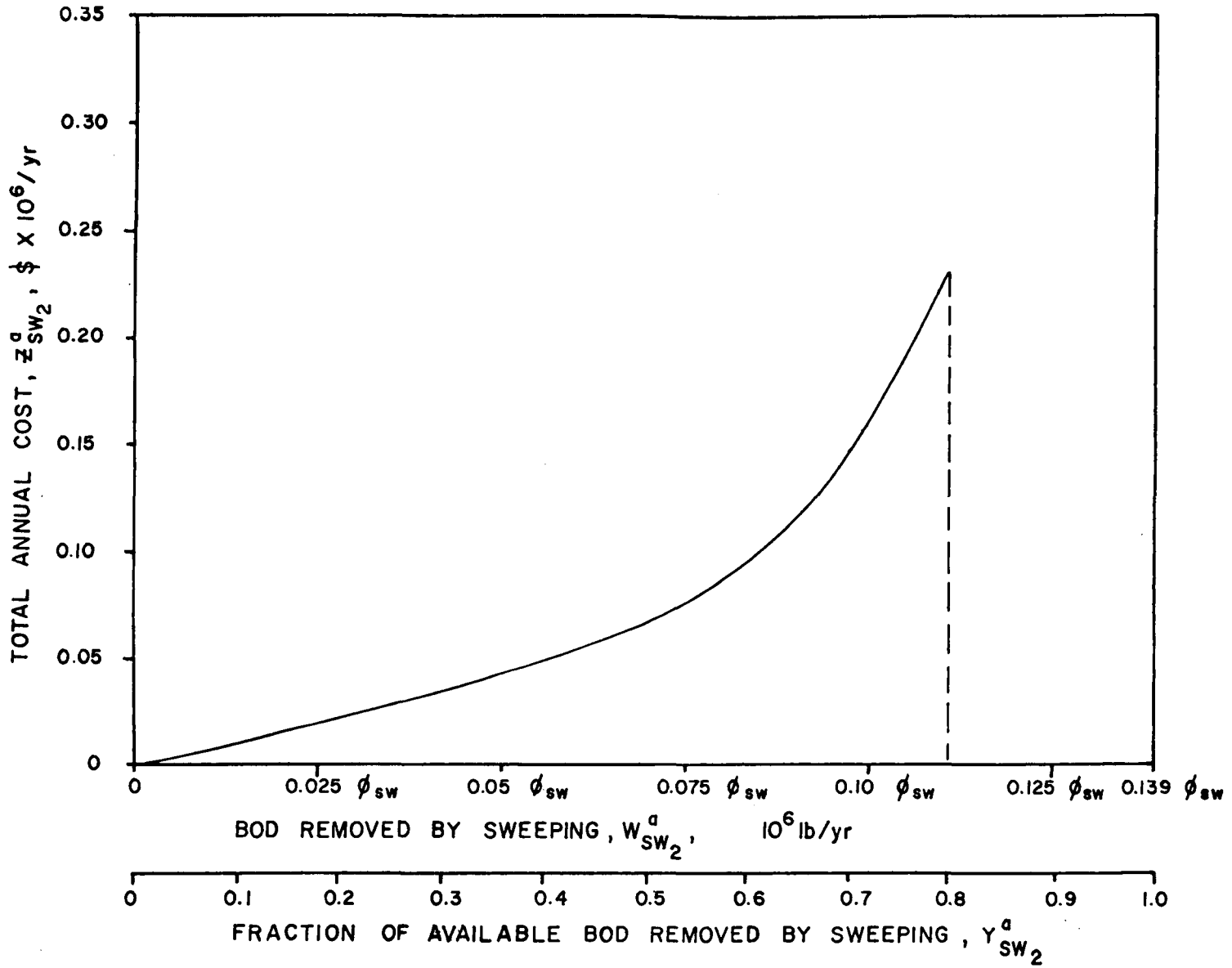


Figure B2. Total Cost Curve for Commercial Portion of Combined Sewered Areas, Street Sweeping

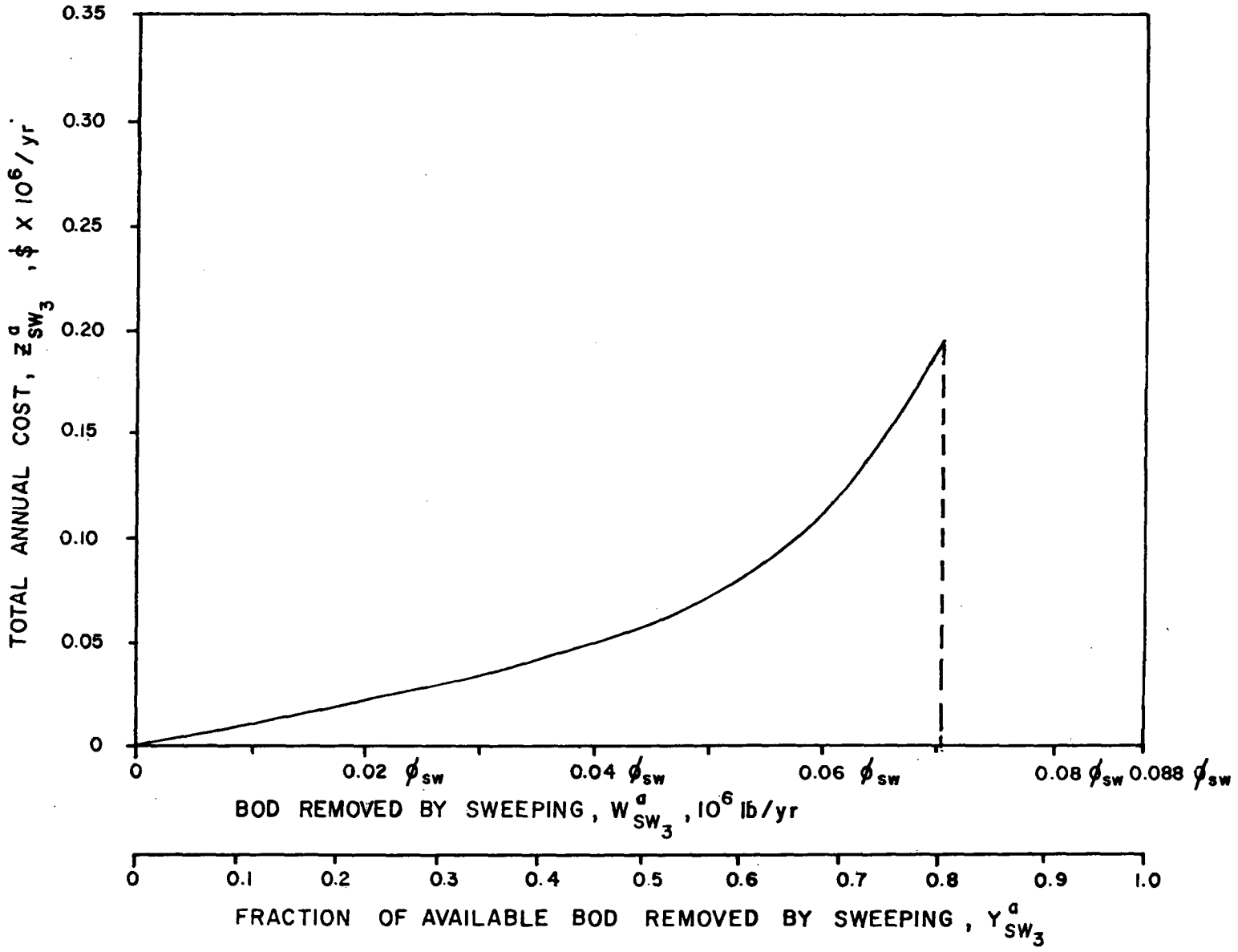


Figure B3. Total Cost Curve for Industrial Portion of Combined Sewered Areas, Street Sweeping

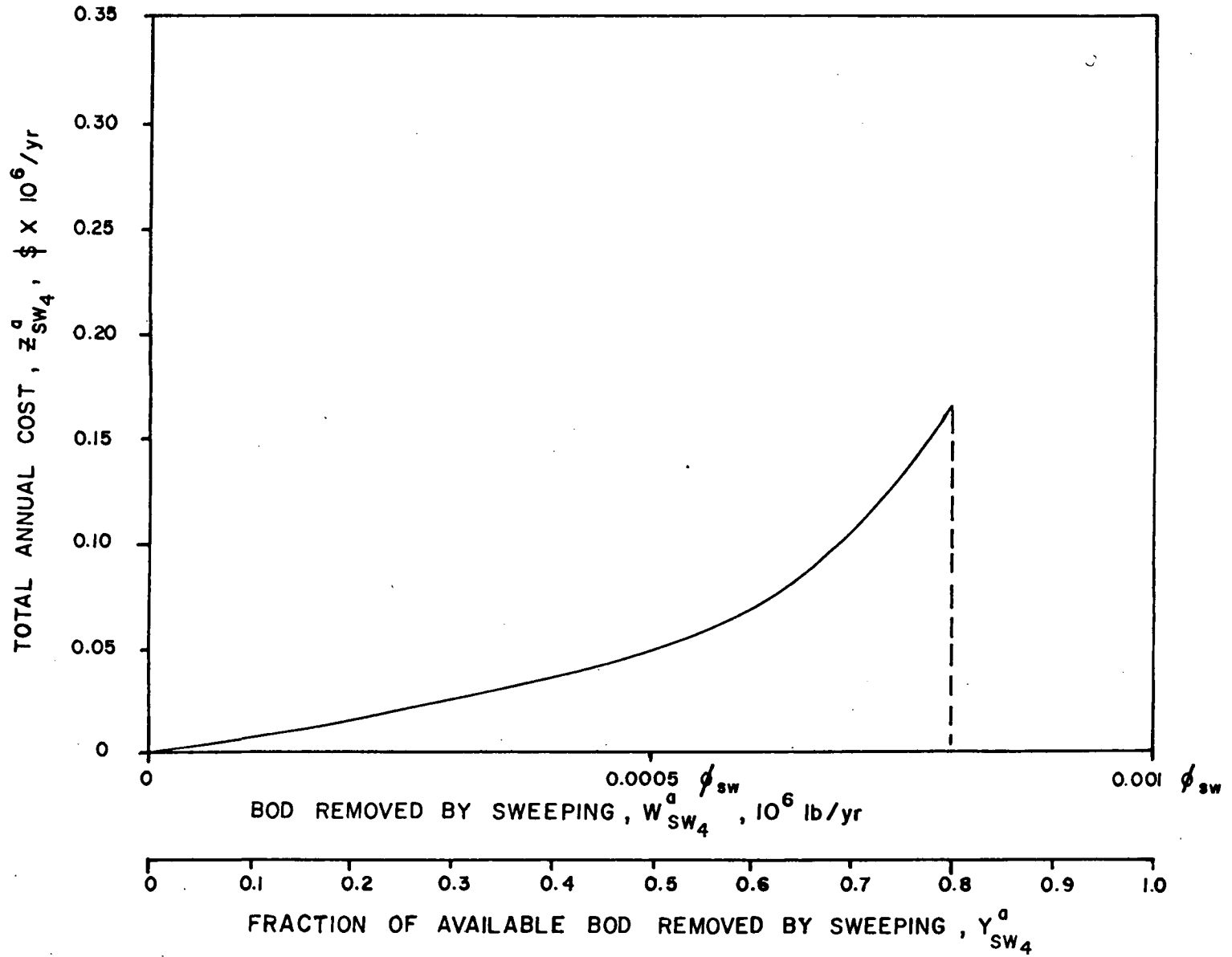


Figure B4. Total Cost Curve for Other Portion of Combined Sewered Areas, Street Sweeping

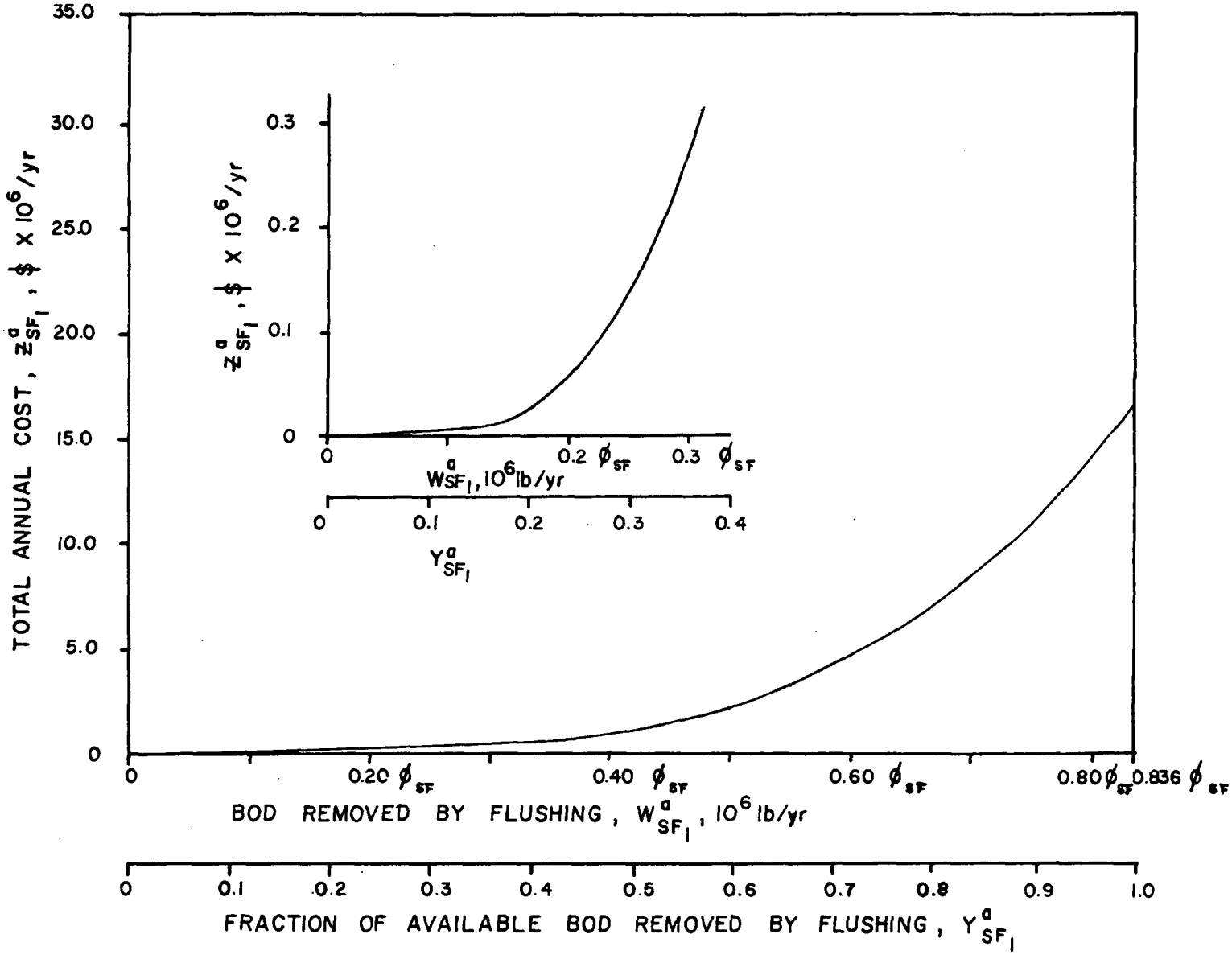


Figure B5. Total Cost Curve for Residential Portion of Combined Sewered Areas, Sewer Flushing

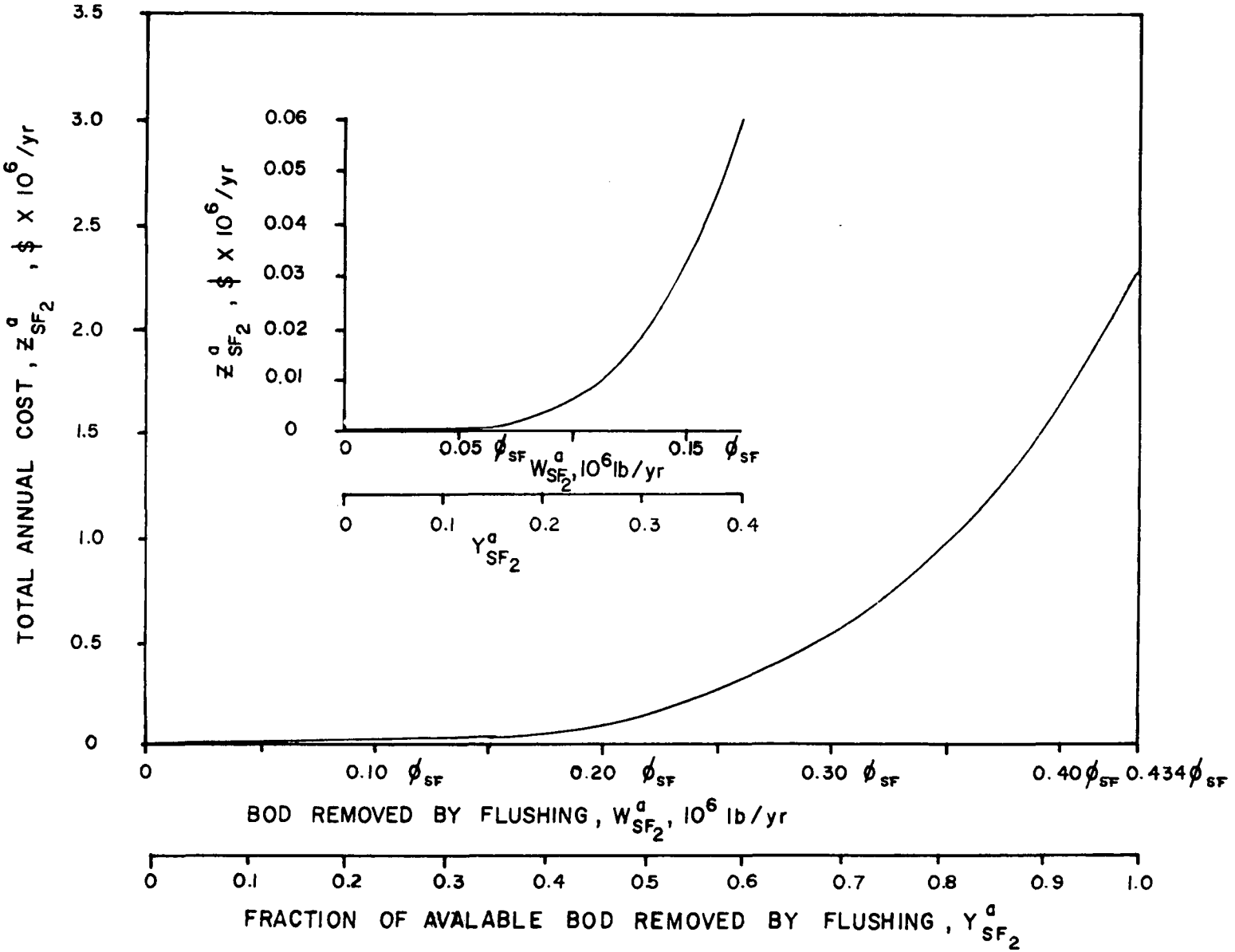


Figure B6. Total Cost Curve for Commercial Portion of Combined Sewered Areas, Sewer Flushing

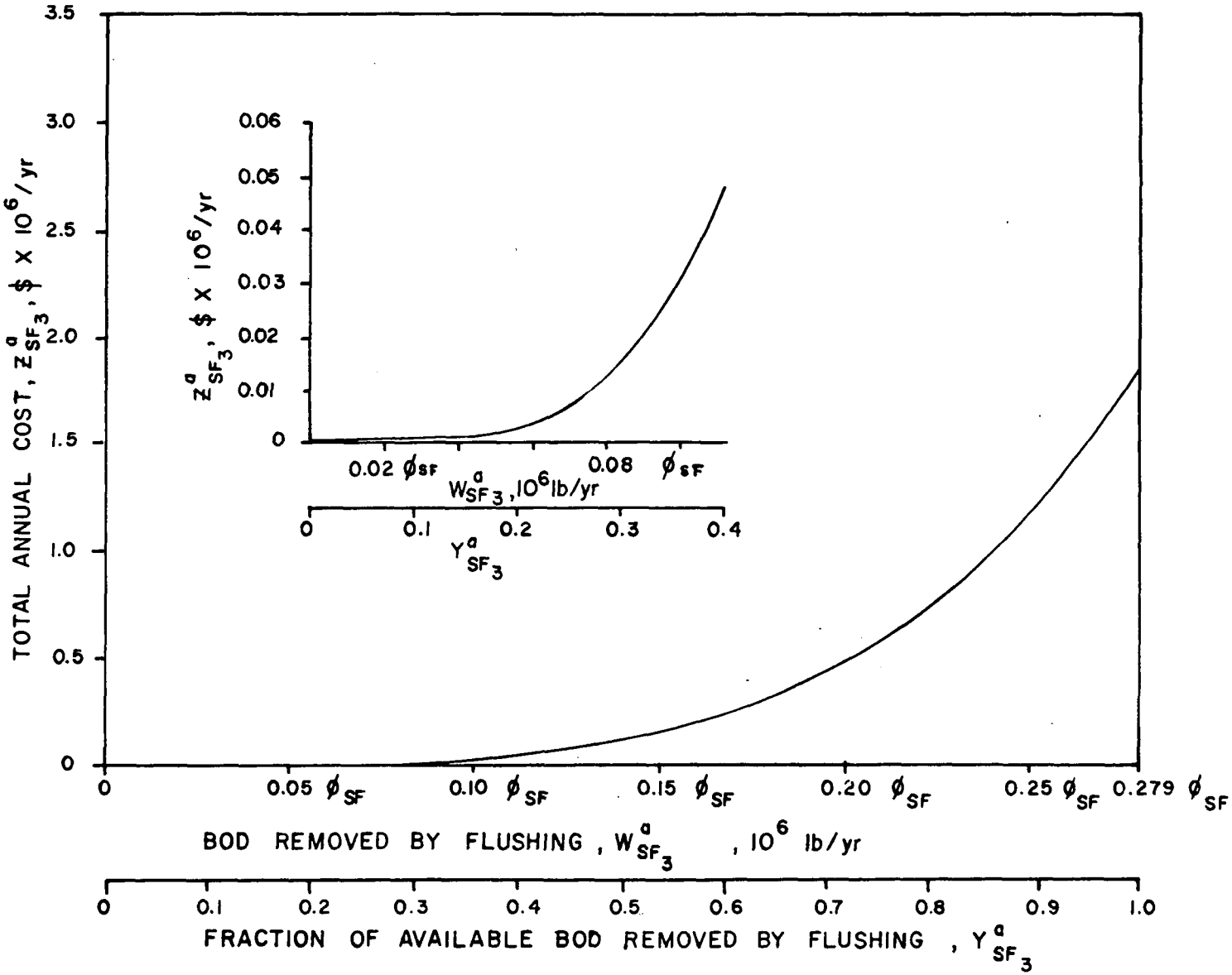


Figure B7. Total Cost Curve for Industrial Portion of Combined Sewered Areas, Sewer Flushing

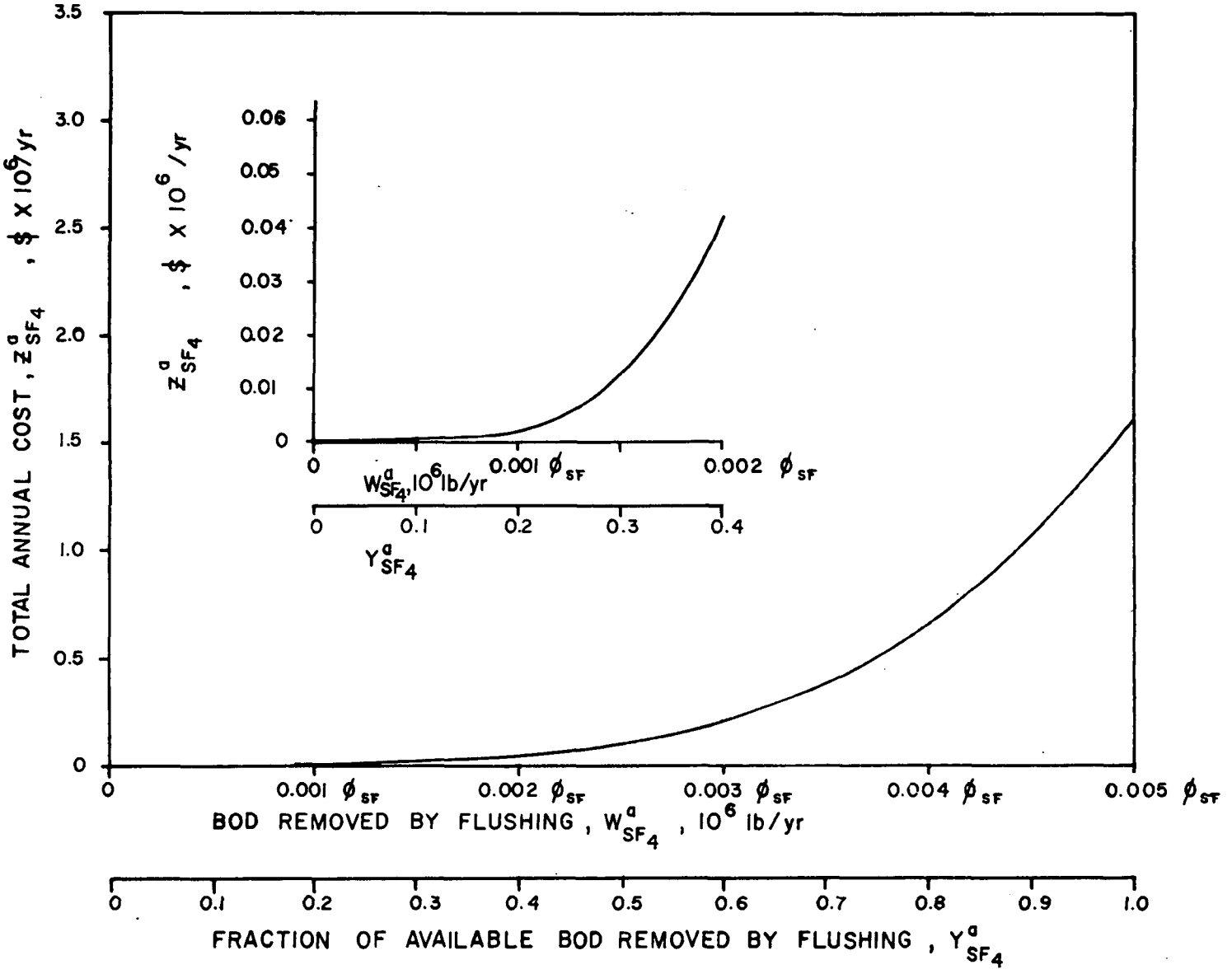


Figure B8. Total Cost Curve for Other Portion of Combined Sewered Areas, Sewer Flushing

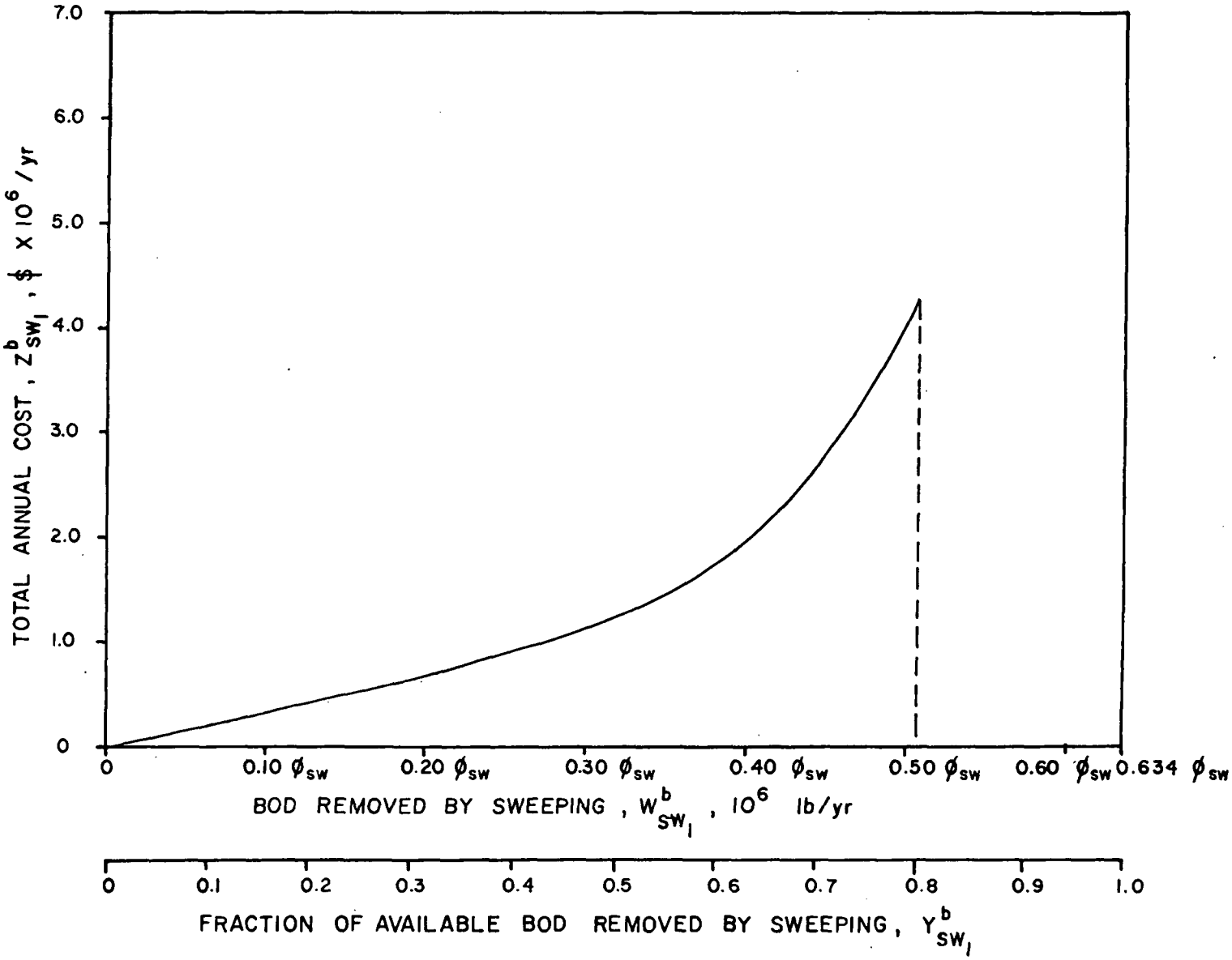


Figure B9. Total Cost Curve for Residential Portion of Storm Sewered Areas, Street Sweeping

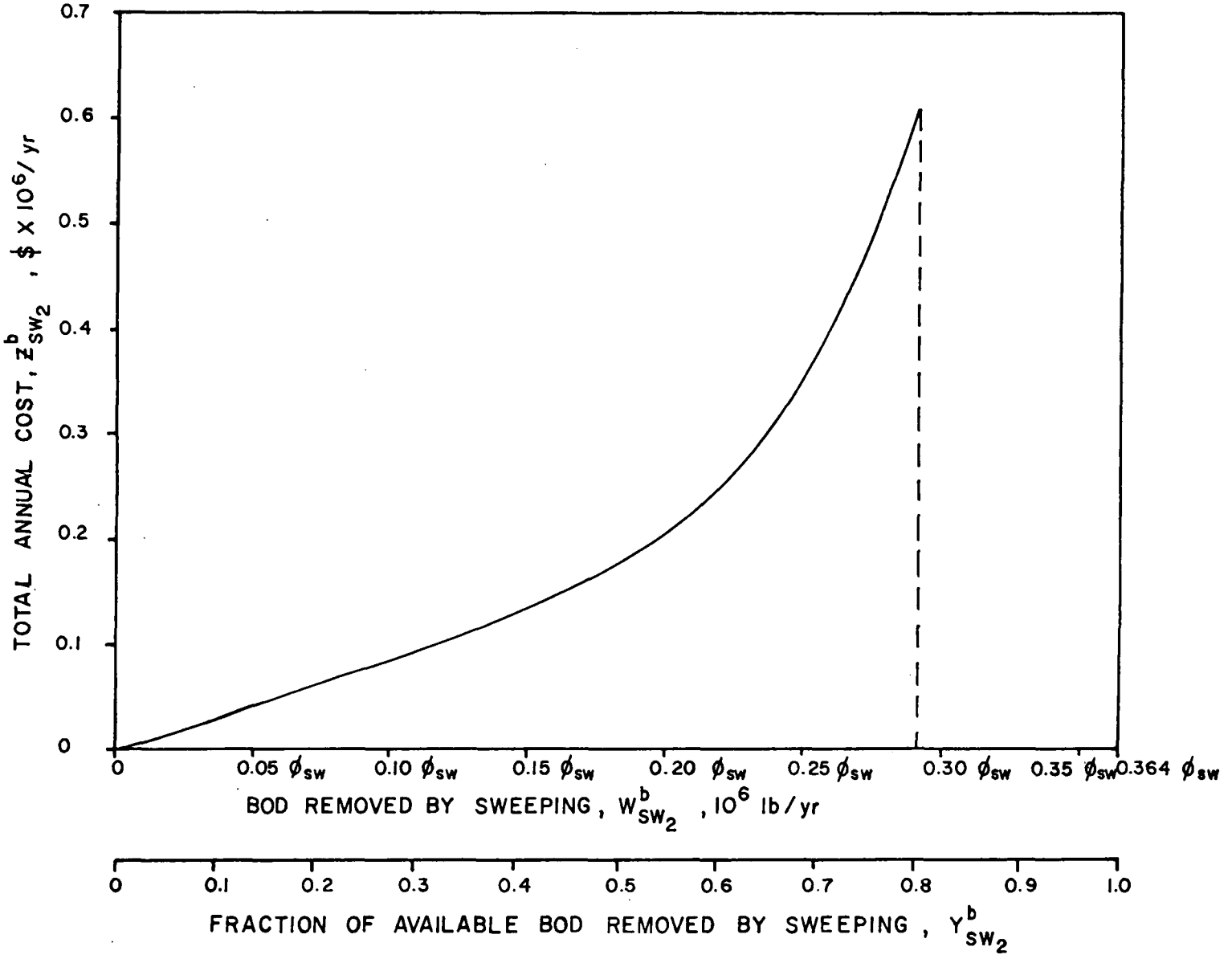


Figure B10. Total Cost Curve for Commercial Portion of Storm Sewered Areas, Street Sweeping

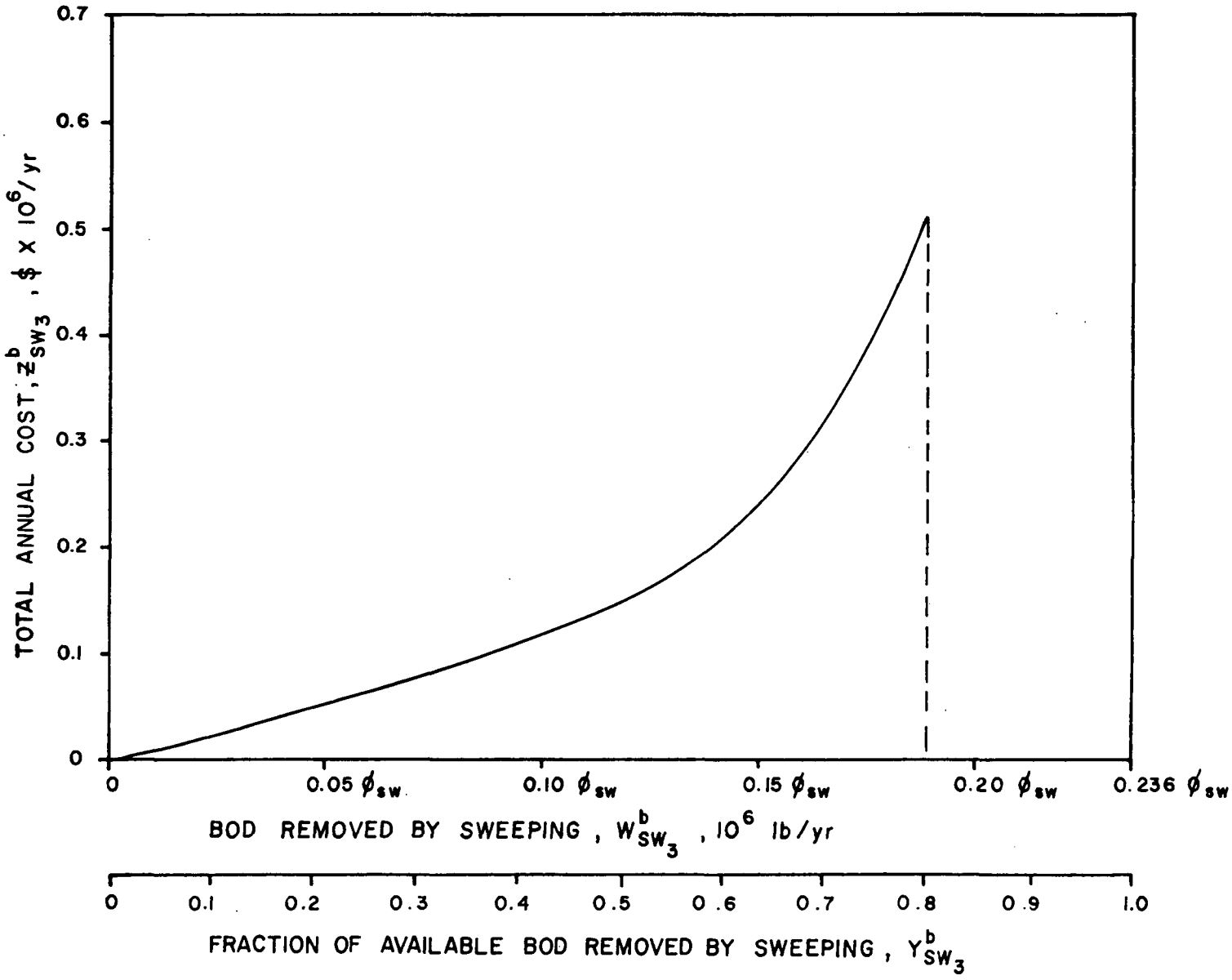


Figure B11. Total Cost Curve for Industrial Portion of Storm Sewered Areas, Street Sweeping

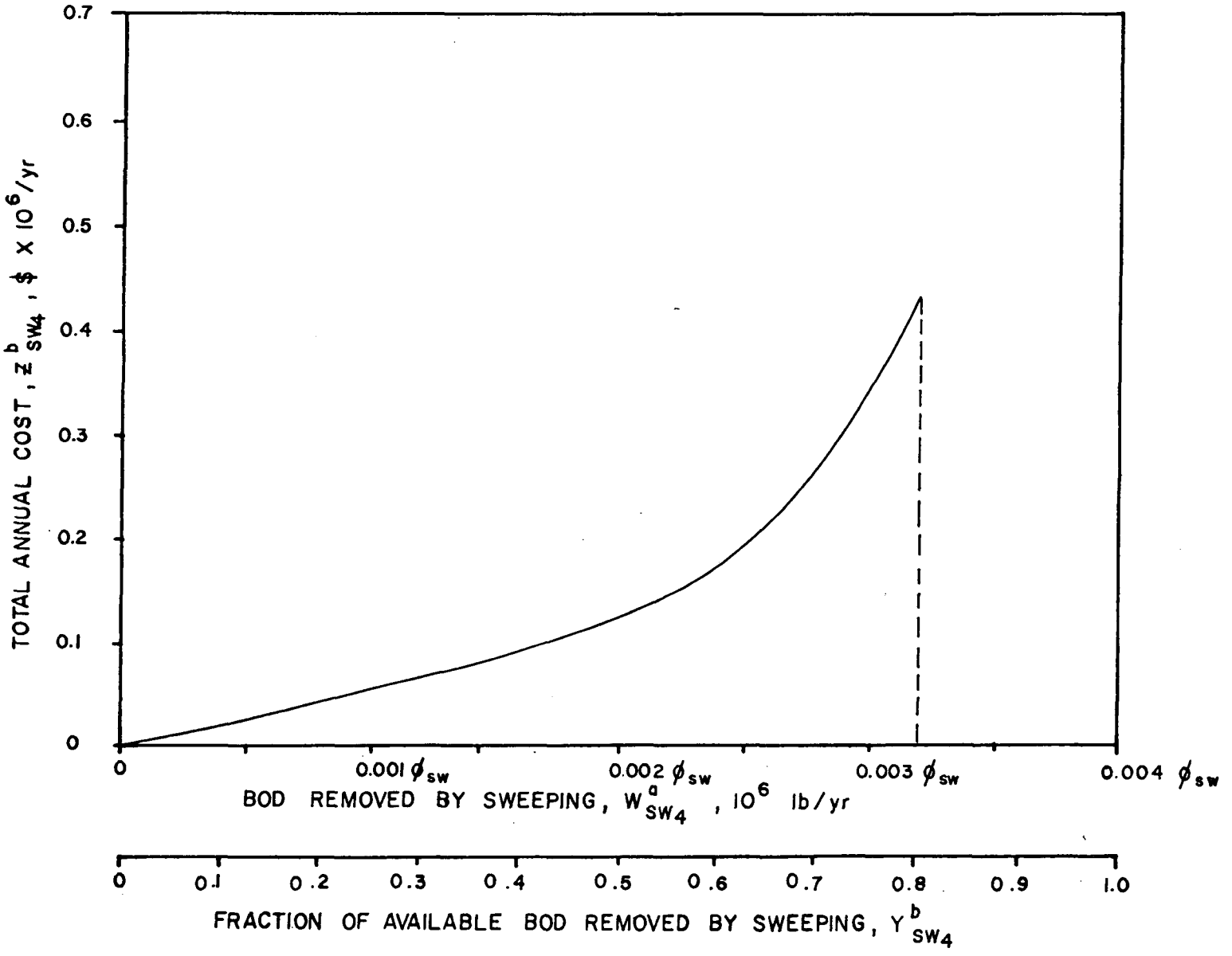


Figure B12. Total Cost Curve for Other Portion of Storm Sewered Areas, Street Sweeping

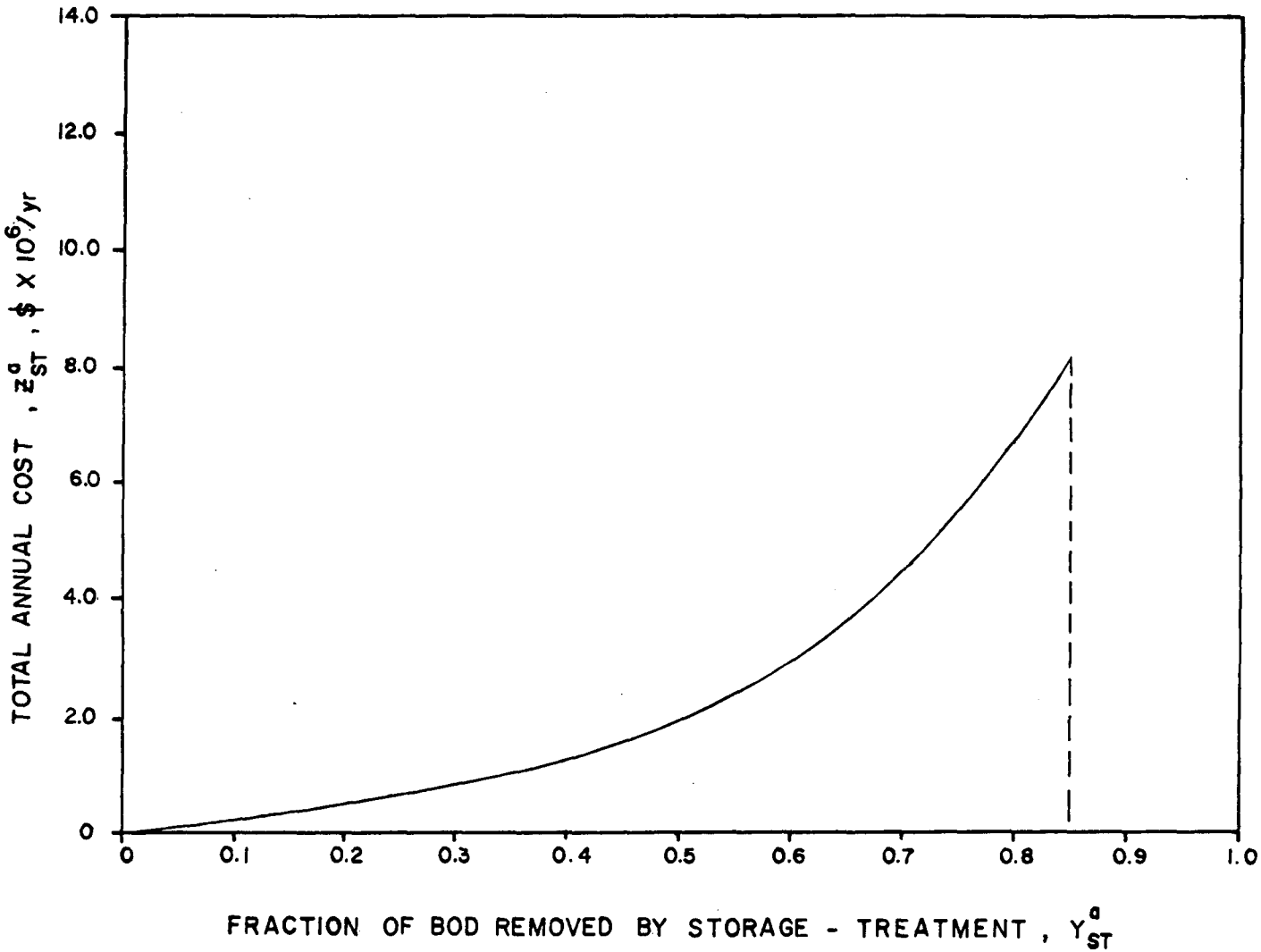


Figure B13. Total Cost Curve for Storage-Treatment, Combined Areas

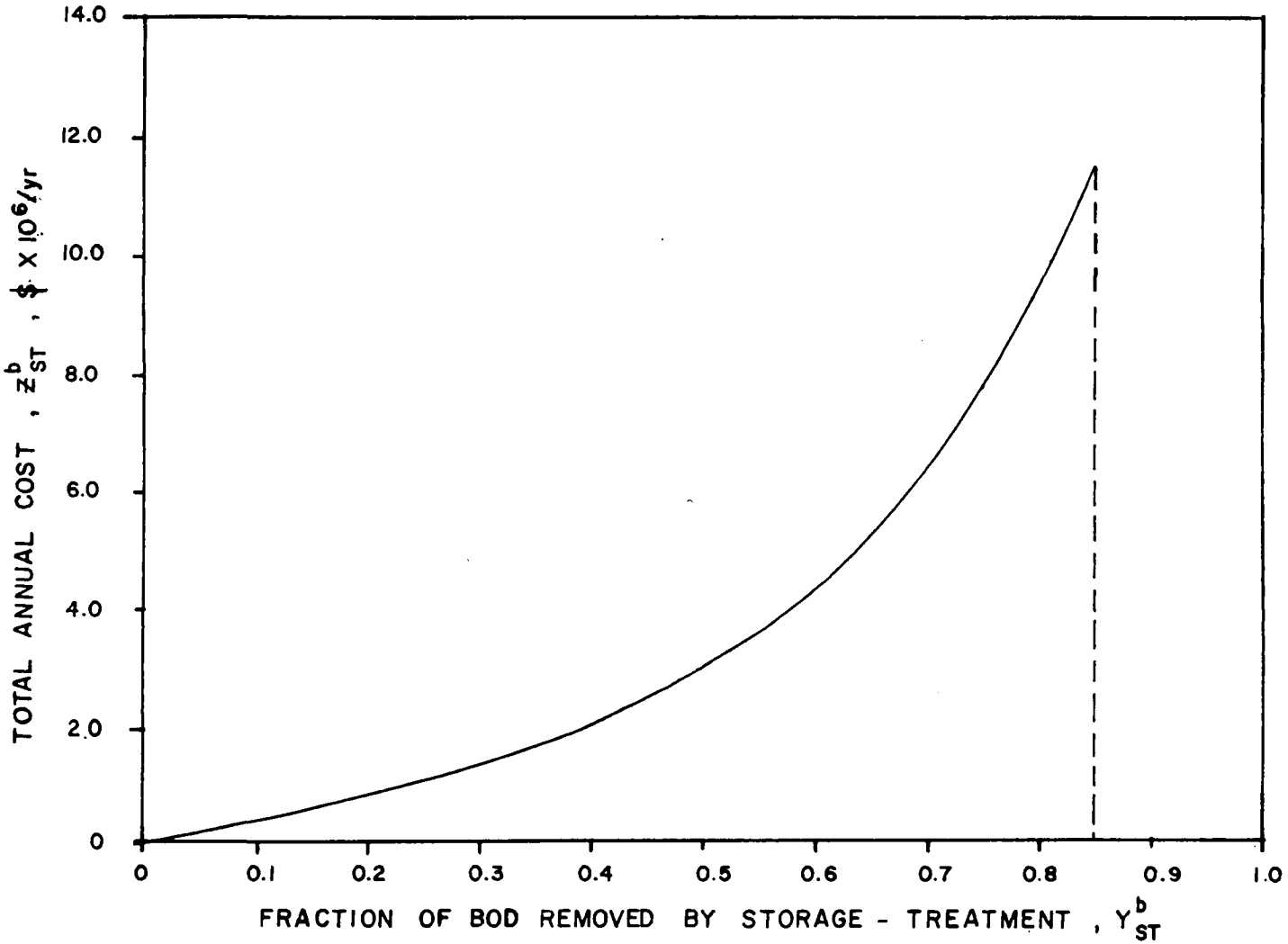


Figure B14. Total Cost Curve for Storage-Treatment, Storm Areas

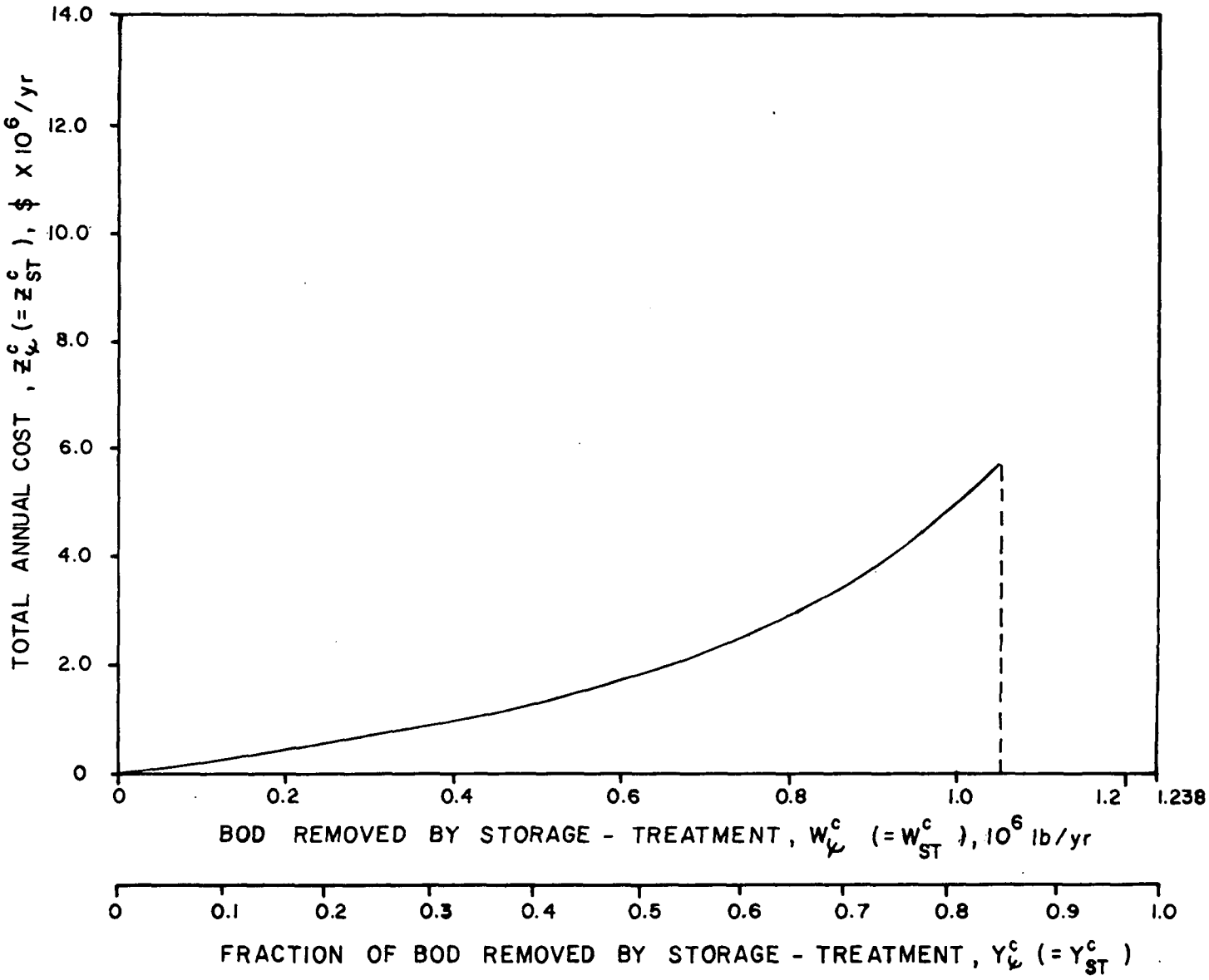


Figure B15. Total Cost Curve for all Options (Storage-Treatment), Unsewered Areas

TECHNICAL REPORT DATA *(Please read Instructions on the reverse before completing)*		
1. REPORT NO. EPA-600/2-77-083	2.	3. RECIPIENT'S ACCESSION NO.
4. TITLE AND SUBTITLE STORM WATER MANAGEMENT MODEL: LEVEL I - COMPARATIVE EVALUATION OF STORAGE-TREATMENT AND OTHER MANAGEMENT PRACTICES		5. REPORT DATE April 1977 (Issuing Date) 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) James P. Heaney Stephan J. Nix		8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS Department of Environmental Engineering Sciences University of Florida Gainesville, FL 32611		10. PROGRAM ELEMENT NO. 1BC611 11. CONTRACT/GRANT NO. R-802411
12. SPONSORING AGENCY NAME AND ADDRESS Municipal Environmental Research Laboratory--Cin.,OH Office of Research and Development U.S. Environmental Protection Agency Cincinnati, Ohio 45268		13. TYPE OF REPORT AND PERIOD COVERED 14. SPONSORING AGENCY CODE EPA/600/14

15. SUPPLEMENTARY NOTES
Project Officer: Richard Field, Phone: 201/548-3347 x503 (8-342-7503)
See also EPA-600/2-76-275, Storm Water Management Model: Level I - Preliminary Screening Procedures

16. ABSTRACT

The original USEPA Storm Water Management Model (SWMM) provides a detailed simulation of the quantity and quality of stormwater during a specified precipitation event lasting a few hours. This model is widely used. However, it is too detailed for many purposes. Indeed, a wide range of evaluation techniques ranging from simple to complex procedures are needed. In particular, the 208 planning effort needs simplified procedures to permit preliminary screening of alternatives. In response to this need, four levels of stormwater management models are being prepared. This volume presents a "desktop" procedure to compare selected alternative control technologies.

A graphical procedure is described which permits the analyst to examine a wide variety of control options operating in series with one another or in parallel. The final result is presented as a control cost function for the entire study area which is the optimal (least costly) way of attaining any desired level of control. Given a specification regarding the desired overall level of control the user can determine the appropriate amount of each control to apply.

This methodology is applied to Anytown, U.S.A., a hypothetical community of 1,000,000 people. The results indicate the mix of treatment, storage, street sweeping, and sewer flushing which attains the specified pollution control level at minimum cost.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
*Storm sewers, *Water pollution, Control simulation, *Cost effectiveness, *Waste treatment, *Sewage treatment, *Surface water runoff, *Runoff, *Combined sewers, *Mathematical models, Storage tanks, Methodology, Economics, Flushing, Catch basins	Simplified evaluation, Sewer flushing, Street sweeping, Catch-basin cleaning	13B

18. DISTRIBUTION STATEMENT	19. SECURITY CLASS *(This Report)*	21. NO. OF PAGES
RELEASE TO PUBLIC	UNCLASSIFIED	105
	20. SECURITY CLASS *(This page)*	22. PRICE
	UNCLASSIFIED	

EPA Form 2220-1 (9-73)

Areawide Assessment Procedures Manual
Appendix H
POINT SOURCE CONTROL ALTERNATIVES: Performance and Cost

Prepared for
Municipal Environmental Research Laboratory
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

TABLE OF CONTENTS

Appendix		Page
H	POINT SOURCE CONTROL ALTERNATIVES: PERFORMANCE AND COST	H-1
	H.1 Introduction	H-1
	H.2 Relative Reliability of Performance and Cost Information	H-2
	H.3 Cost Components	H-4
	H.3.1 Construction Costs	H-5
	H.3.2 Operation and Maintenance Costs	H-9
	H.3.2.1 Labor Costs	H-11
	H.3.2.2 Material Costs	H-11
	H.3.2.3 Energy Costs	H-11
	H.3.2.4 Chemical Cost	H-11
	H.4 Cost Format	H-12
	H.5 Treatment System Performance Matrix	H-13
	H.6 Municipal Wastewater Treatment Systems	H-15
	H.7 Wet-Weather Treatment Processes	H-46
	H.8 Municipal Wastewater Treatment Processes	H-72
	H.8.1 Support Personnel	H-72
	H.8.2 Miscellaneous Structures	H-72
	H.8.3 Wastewater Treatment Processes	H-76
	H.8.4 Sludge Treatment and Disposal Processes	H-154
	H.8.5 Wastewater Transportation Methods	H-196
	H.8.6 Sludge Transportation Methods	H-200
	H.9 References	H-210

LIST OF FIGURES

Figure No.	Title	Page
H-1	Upgrading/Expansion Factor	H-10
H-2	Treatment System 1 - Primary Treatment	H-16
H-3	Treatment System 2 - Primary Treatment With Metal-Salt Addition ($FeCl_3$)	H-18
H-4	Treatment System 3 - Trickling Filter	H-20
H-5	Treatment System 4 - Trickling Filter With Metal-Salt Addition ($FeCl_3$)	H-22
H-6	Treatment System 5 - Activated Sludge	H-24
H-7	Treatment System 6 - Activated Sludge With Metal-Salt Addition (Alum)	H-26
H-8	Treatment System 7 - Activated Sludge/ Nitrification (Single Stage)	H-28
H-9	Treatment System 8 - Activated Sludge/ Nitrification/Denitrification (Three Stages)	H-30
H-10	Treatment System 9 - Activated Sludge/ Nitrification - Filtration (With Alum)	H-32
H-11	Treatment System 10 - Physical/Chemical	H-34
H-12	Treatment System 11 - Activated Sludge/ Nitrification/Denitrification with Alum (Three Stages)	H-36
H-13	Treatment System 12 - Activated Sludge/ Nitrification/Denitrification/Activated Carbon (With Alum)	H-38
H-14	Treatment System 13 - Small Flow Treatment Systems	H-40
H-15	Treatment System 14 - Oxidation Ditch (Designed for Nitrification)	H-42
H-16	Treatment System 15 - Land Application of Wastewater (Non-Underdrained)	H-44
H-17	Concrete Stormwater Storage Reservoir	H-46

LIST OF FIGURES
(continued)

Figure No.	Title	Page
H-18	Earthen Storage Basin	H-50
H-19	Stationary Screen	H-52
H-20	Horizontal-Shaft Rotary Screen	H-54
H-21	Vertical-Shaft Rotary Screen	H-56
H-22	Chemical Coagulation (Ferric Chloride and Polymer)	H-60
H-23	Stormwater Sedimentation	H-62
H-24	Dissolved Air Flotation	H-64
H-25	Swirl Regulator/Concentrator	H-66
H-26	High-Intensity Mixing/Chlorine Contact Basin	H-68
H-27	High-Rate Filtration (Gravity)	H-70
H-28	Support Personnel	H-72
H-29	Miscellaneous Structures	H-74
H-30	Low-Lift Pump Station	H-76
H-31	Preliminary Treatment	H-78
H-32	Flow Equalization	H-80
H-33	Primary Clarifier With Sludge Pumps	H-84
H-34	Conventional Activated Sludge Aeration With Diffused Air	H-86
H-35	Activated Sludge Aeration With Pure Oxygen	H-88
H-36	High-Rate Trickling Filter	H-90
H-37	Low-Rate Trickling Filter	H-92
H-38	Aerated Lagoons	H-94
H-39	Rotating Biological Contactors	H-96

LIST OF FIGURES
(continued)

Figure No.	Title	Page
H-40	Facultative Lagoons	H-100
H-41	High-Rate Activated Sludge (Complete Mix)	H-104
H-42	Final Clarifier (Flocculator Type)	H-106
H-43	Clarifier for High-Rate Trickling Filter (Includes Recycle Pumps)	H-108
H-44	Activated Sludge/Nitrification (Single Stage)	H-110
H-45	Separate Nitrification (With Clarifier)	H-112
H-46	Separate Denitrification (With Clarifier)	H-114
H-47	Breakpoint Chlorination	H-116
H-48	Ammonia Stripping	H-118
H-49	Ion Exchange (For Ammonia Removal)	H-120
H-50	Phostrip	H-122
H-51	Polymer Feed System	H-124
H-52	Mineral Addition	H-126
H-53	Two-Stage Tertiary Lime Treatment (Without Recalcination)	H-130
H-54	Lime Recalcination	H-132
H-55	Microscreening	H-134
H-56	Gravity Filtration (Dual-Media)	H-136
H-57	Activated Carbon Adsorption	H-138
H-58	Chlorination (Disinfection)	H-142
H-59	Dechlorination Using Sulfur Dioxide	H-144
H-60	Ozonation (Air- and O_2-Generated)	H-146

LIST OF FIGURES
(continued)

Figure No.	Title	Page
H-61	Post Aeration	H-150
H-62	Groundwater Recharge (Infiltration)	H-152
H-63	Sludge Pumping	H-154
H-64	Gravity Thickener	H-156
H-65	Dissolved Air Flotation Thickener	H-158
H-66	Centrifugation (Lime Sludge Dewatering)	H-160
H-67	Centrifugation (Biological Sludge Dewatering)	H-162
H-68	Vacuum Filtration (Biological Sludge)	H-164
H-69	Vacuum Filtration (Lime Sludge)	H-166
H-70	Filter Press (Lime Sludge)	H-168
H-71	Filter Press (Biological Sludge)	H-170
H-72	Sludge Drying Beds	H-172
H-73	Two-Stage Anaerobic Digesters	H-174
H-74	Aerobic Digesters	H-176
H-75	Heat Treatment of Sludge	H-178
H-76	Composting	H-180
H-77	Incineration (Fluidized Bed)	H-182
H-78	Incineration (Multiple Hearth)	H-184
H-79	Lime Stabilization	H-186
H-80	Sludge Storage	H-188
H-81	Landfilling (Biological Sludge Excluding Transportation)	H-190
H-82	Landfilling (Lime Sludge Excluding Transportation)	H-192

LIST OF FIGURES
(continued)

Figure No.	Title	Page
H-83	Land Application of Sludge	H-194
H-84	Gravity Sewers	H-196
H-85	Transmission Force Main	H-198
H-86	Liquid Sludge Transport (Rail)	H-200
H-87	Dewatered Sludge Transport (Rail)	H-202
H-88	Dewatered Sludge Transport (Truck)	H-204
H-89	Liquid Sludge Transport (Truck)	H-206
H-90	Sludge Transport (Pipeline)	H-208

APPENDIX H

POINT SOURCE CONTROL ALTERNATIVES:

Performance and Cost

H.1 Introduction

Estimation of the monetary cost of the various control alternatives requires the ready availability of comprehensive construction and operating cost data. To facilitate use in a 208 program, these data must be in a convenient format applicable to the wide range of situations that may be encountered. The cost data should be useful to planners and engineers with varying degrees of technical background, and should be useful both for developing and for reviewing the costs associated with structural control alternatives.

Cost data presented in this appendix are generalized rather than site-specific, and are intended primarily for comparative analysis. Also, the data in this appendix represent a technology which is by no means static. Therefore, this information should be updated to reflect technological advances or other factors affecting comparative costs.

The user of this appendix is cautioned against treating any cost estimates based on the data contained herein as absolute. The estimates are valid only for comparative purposes, and even these comparisons must be performed with caution because of the possible differences in the reliability of the performance and cost information from various sources. There may be considerable difference between cost estimates for comparative purposes and the actual construction costs of facilities. Frequently, planners and engineers err in not emphasizing this fact enough to the local communities involved in the 208 program. The comparative numbers somehow become cast in stone during the planning phase. Later, during the implementation phase of the program, the communities express their concern when they find that the actual costs of implementation are vastly different than those implied by the comparative cost estimates.

H.2 Relative Reliability of Performance and Cost Information

Monetary-cost comparisons should not be the sole basis for selecting control alternatives, as discussed in Chapter 6; but, at some point in the selection process, the relative costs of various alternatives must be considered. Therefore, it is very important for the engineer or planner to understand the reliability of the information upon which the cost figures are based.

The reliability of performance data must also be considered, because this type of information frequently is the basis for determining the size of treatment units or other control devices needed for a particular control alternative. This section covers the use of the performance and cost information of Appendix G (Best Management Practices) and Appendix H (Point Source Control Alternatives), and provides a method for assuring adequate consideration of the relative reliability of the various applicable performance and cost information needed for comparison of applicable control alternatives.

The performance data and cost information in this and the Best Management Practices Appendix are utilized to determine the monetary cost of feasible control alternatives for addressing water quality problems in 208 planning areas. Performance data for a particular control alternative are compared to a required standard to assess the alternative's capability to meet the standard. Then the monetary costs of those alternatives which meet the performance requirements are determined by utilizing the cost curves. Since performance and cost information for one alternative may be based on much more extensive data and experience than the information for another alternative, the reliability of both the cost information and the performance information should be taken into account when considering relative cost and performance capabilities.

The reliability of information is especially important in developing, evaluating, and selecting control alternatives for areawide water quality management, because there are highly varying degrees of experience with the various control alternatives. For example, an abundance of cost figures and estimates are available to substantiate performance and cost curves for an activated sludge treatment plant. Less data and experience are available to substantiate performance and cost relationships for street sweeping as

an alternative in control of pollution from urban runoff. Even less data and experience may be available for various land management alternatives such as zoning. Although each of these three alternatives is known to be effective in reducing pollutant levels, determination of the best combination will require careful deliberation, good judgment, and full recognition of the reliability of each type of information at the time the decision is made.

The concept of "relative reliability" is presented here to aid the user of this manual in comparing the monetary cost of control alternatives. Five levels of relative reliability are used to identify the nature and extent of the experience and data upon which the cost and performance information is based:

- Level A indicates estimates based on detailed breakdowns of all pertinent cost elements and is supported by detailed engineering data. This level of reliability is always based on site-specific information. The relative reliability of information in this level is ± 15%. For example, facilities-planning estimates (Section 201 of P.L. 92-500) represent Level A information reliability.

- Level B indicates that the data and experience on a particular control alternative are sufficient only to establish a relationship, as expressed by a table of data or a single curve or family of curves. The relative reliability of information at this level is ± 30%. For example, general cost curves such as the wastewater treatment systems curves and process curves presented in Appendix H represent Level B information reliability.

- Level C indicates that the data and experience are sufficient only to establish a range of values for cost or performance. The relative reliability of information at this level is ± 50%. For example, street sweeping estimates, such as those in the Best Management Practices Appendix, represent Level C information reliability.

- Level D indicates that the data and experience are sufficient only to establish the relative order of magnitude of the cost and performance characteristics.

- Level E indicates that the data or experience is insufficient to establish any level of cost or performance estimate, or that site-specific factors are so critical to the performance and cost that a general estimate should not be made.

This manual does not present guidelines on the application of the relative reliability concept for particular situations. Rather, the application by the user will be a function of the control alternatives that are being compared, the closeness of the cost or performance estimates, the background of the user, the consequences of error, and other factors.

The "relative reliability" concept is introduced to emphasize to the user that comparisons of cost or performance estimates prepared using this manual are only as reliable as the lowest level of reliability assigned to the control alternatives being considered. The concept is particularly well suited to compare more traditional engineering approaches to load reduction with "emerging" non-structural control techniques whose costs and relative effectiveness have not been satisfactorily evaluated or sufficiently documented.

H.3 Cost Components

Information in a convenient format has been developed to provide a concise, yet complete, set of cost data that enable the user of the manual to produce cost estimates of adequate quality for use in comparing strategies in Area-wide Assessment Planning. Many sources were used in the development of the cost data, but in all cases the cost data were converted to a uniform basis. Where possible, these cost data were checked against actual construction costs.

The method used in developing the cost curves was to plot all available cost data uniformly, including actual construction costs. When differences in cost occurred, every effort was made to determine the reasons; and usually the significant differences were corrected, such as inclusion of buildings and pumps in some estimates and not in others. The curves included in this appendix were then fitted to these representative data.

Some of the costs presented in this appendix will be more reliable than others. Higher reliability costs generally occur where a large amount of historical

data exists, such as clarifiers and activated sludge. However, all cost data presented should be adequate to provide the user with the information needed to make judgments necessary in Areawide Assessment Planning.

Table H-1 provides the user with the general basis of all cost curves contained in this appendix.

H.3.1 Construction Costs

Construction-cost curves have been developed both for process components and for treatment systems. These construction-cost curves represent the cost of unit processes or treatment systems for a wide capacity range. Specific items not included in the construction cost are:

1. Land Costs (except land application and landfill).
2. Site Work.
3. Piping.
4. Electrical Work.
5. Engineering and Construction Supervision.
6. Project Contingencies.
7. Miscellaneous Structures.

The additional costs for Items 1 through 6 are generally represented as a percentage of the installed construction cost of the components specific to the unit or system being evaluated for comparative cost estimates of the type generally developed by using these curves. Additional cost curves are included for Item 7 (Miscellaneous Structures) and should be included where appropriate. Note that the cost curves developed for municipal wastewater treatment systems (Figures H-2 through H-16) already include the costs for miscellaneous structures. Table H-2 presents a general format for identifying the costs, with representative percentages for the items indicated. Land costs depend on local conditions and should be developed for each individual case. However, it should be noted that development of land costs, except in land application and land disposal of sludge, may not be necessary for comparison purposes. Most alternatives will have similar land requirements.

TABLE H-1

GENERAL COST AND DESIGN BASIS FOR COST CURVES

Basis of Costs

1. ENR = 2475, September 1976

2. Labor rate, including fringe benefits = $7.50/hr

   Note: Labor costs are based on a man-year of 1,500 hours. This represents: a 5-day work week; an average of 29 days for holidays, vacations, and sick leave; and 6½ hours of productive work time per day.

3. Energy Costs

a.	Electric Power	=	$0.02/kwh
b.	Fuel Oil	=	$0.37/gallon
c.	Gasoline	=	$0.60/gallon

4. Land = $1,000/acre

5. Chemical Costs

a.	Liquid Oxygen	=	$65/Ton
b.	Methanol	=	$0.50/gallon
c.	Chlorine 150-lb cylinder	=	$360/Ton
	1-ton cylinder	=	$260/Ton
	Tank Car	=	$160/Ton
d.	Quicklime	=	$25/Ton
e.	Hydrated Lime	=	$30/Ton (as CaO)
f.	Polymer (Dry)	=	$1.50/lb
g.	Ferric Chloride	=	$100/Ton
h.	Alum	=	$72/Ton
i.	Activated carbon (granulated)	=	$0.50/lb
j	Sulfuric Acid (66° Be')	=	$50/Ton
k.	Sodium hexametaphosphate	=	$0.25/lb
l.	SO_2 150-lb cylinders	=	$450/Ton
	1-Ton cylinder	=	$215/Ton
	Tank Car	=	$155/Ton

Design Basis

1. Construction costs and operation and maintenance costs are based on design average flow unless otherwise noted.

2. Operation and maintenance costs include:

   a. Labor costs for operation, preventive maintenance, and minor repairs.
   b. Materials costs to include replacement parts and major repair work (normally performed by outside contractors).
   c. Chemical costs.
   d. Fuel costs.
   e. Electrical power costs.

3. Construction costs do not include external piping, electrical, instrumentation, site work, contingency, or engineering and fiscal fees.

TABLE H-2

DEVELOPMENT OF CAPITAL COSTS

Component Installed Construction Costs				
			$_____	
(Unit processes specific to each Cost Estimate)			$_____	
Misc. Structures (Figure H-29)*			$_____	
Subtotal 1				$_____
	Avg.	Range**		
Piping	10%	8-15%	$_____	
Electrical	8%	5-12%	$_____	
Instrumentation	5%	3-10%	$_____	
Site Preparation	5%	1-10%	$_____	
Subtotal 2				$_____
Engineering and Construction -Supervision @ 15%	***		$_____	
Contingencies @ 15%	***		$_____	
TOTAL CAPITAL COST				$_____

*Not to be included when municipal wastewater treatment system or wet weather treatment process curves are used. These include miscellaneous structures in their construction costs.

**Range due to level of complexity, degree of instrumentation, subsoil conditions, configuration of site, etc.

***Percentage of Subtotal 2.

All construction costs have been indexed to September, 1976. To adjust for other time periods, appropriate cost indexes, such as those in the following list, should be used where appropriate:

- ENR Building Cost Index - appears weekly in Engineering News Record, McGraw-Hill, Inc.
- ENR Construction Cost Index - appears weekly in Engineering News Record, McGraw-Hill, Inc.
- EPA Treatment Plant and Sewer Construction Cost Index - appears monthly in the WPCF Journal.
- BLS Labor Cost Index - appears monthly in Employment and Earnings, Bureau of Labor Statistics.
- BLS Wholesale Price Index - appears monthly in Wholesale Prices and Prices Indexes, Bureau of Labor Statistics.

To facilitate use of the cost data presented in this appendix, each construction-cost curve includes a summary of the design basis, assumptions, and cost basis. Representative assumptions were made for the cost-curve development, but these can be easily modified to reflect site-specific conditions.

A generalized adjustment factor can be used to develop costs for mass or hydraulic loading factors, detention times, periods of operation, or solids concentrations other than those shown as the design basis. The cost curves are entered at an effective flow (Q_E), where

$$Q_E = Q_{DESIGN} \times \text{Cumulative Adjustment Factor}$$

where:

$$Q_{DESIGN} = \text{the flow shown on the curve}$$

and

$$\text{Cumulative Adjustment Factor} = \frac{\text{Design Mass Loading}}{\text{New Mass Loading}} \times \frac{\text{Design Solids Concentration}}{\text{New Solids Concentration}} \times \frac{\text{Design Time Operation}}{\text{New Time of Operation}} \times \text{Etc.}$$

This generalized adjustment factor will permit development of costs for site specific situations not identical to the design basis used for development of the curves.

Ideally, the cost curves would have been developed to compare a design parameter such as Surface Area or Volume vs Cost rather than Plant Flow vs Cost. For the purposes of this appendix, however, the latter approach was selected to screen the many alternatives required in Areawide Assessment.

A common occurrence when developing comparative cost estimates for areawide assessment plans is considering upgrading and/or expanding existing facilities. Often there are many economies to be realized in this practice. However, there are also economic disadvantages that should not be overlooked. Some of the additional costs that are often associated with upgrading and/or expanding an existing facility include interface costs associated with connecting pieces of equipment together, additional costs incurred by the contractor due to working in and around process equipment and yard piping, and problems of interfacing with old or obsolete equipment. Obviously, the actual additional costs associated with upgrading and/or expanding existing facilities will vary considerably depending on plant layout, contractor experience, age of existing plant, and relative size of the upgrade and/or expansion. However, a review of plant upgrade and expansions did provide a trend which can be useful in developing construction costs. Figure H-1 provides an Upgrade/Expansion Factor. This factor represents a percentage increase to be applied to the construction cost of the Upgrade/Expansion Alternative. As the size of the upgrade/expansion increases, the construction cost approaches the cost of a new facility. However, as the size of the upgrade/expansion decreases, the factor increases.

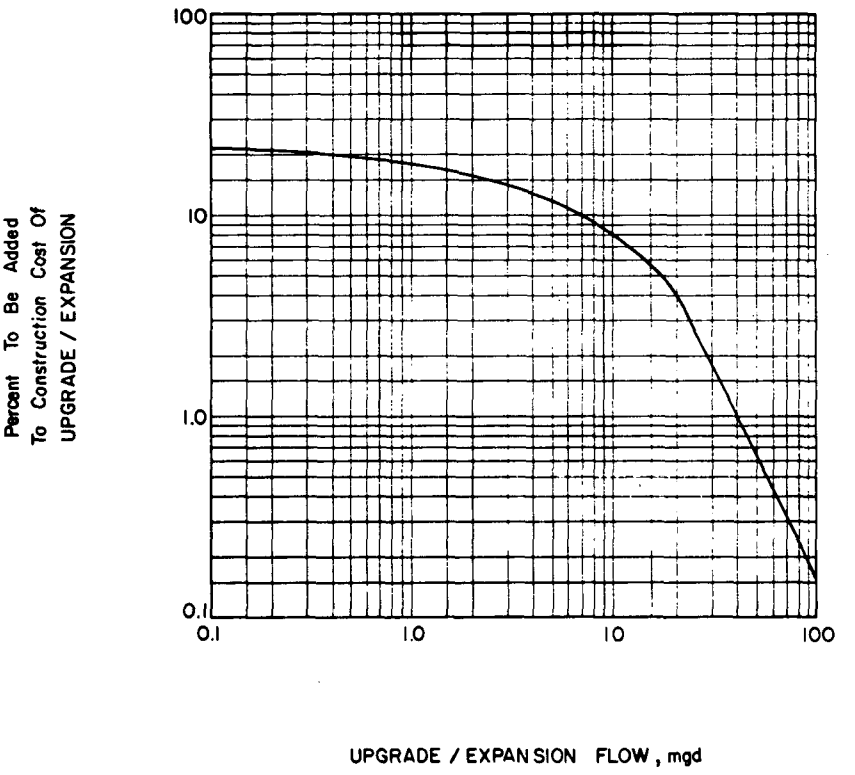
H.3.2 Operation and Maintenance Costs

Operation and maintenance costs consist of labor, material, energy, and chemical components. Each is detailed below. For each construction-cost curve, there is a corresponding O & M cost curve covering each of the O & M cost components and a total O & M cost.

FIGURE H-1

UPGRADING/EXPANSION FACTOR

FIGURE H-1 UPGRADING / EXPANSION FACTOR



H.3.2.1 Labor Costs

Labor costs include the manpower required to operate and maintain the facility or system, plus such support tasks as supervision and administration, clerical work, laboratory work, and yard work. Labor costs are based on a labor rate including fringe benefits of $7.50/hr; a man-year of 1,500 hours incorporating a five-day work week; an average of 29 days for holidays, vacations, and sick leave; and 6½ hours productive work time per day.

H.3.2.2 Material Costs

Material costs include the various materials required for routine maintenance of facilities. Examples are paint, grease, and replacement parts. Structural equipment (buildings, roads, and basins) generally has lower material costs than does mechanical equipment (pumps and aerators). Material costs have a base date of September, 1976. The wholesale price index should be used to adjust for other time periods.

H.3.2.3 Energy Costs

Each mechanical operation at a treatment facility, such as pumping, mixing, and aeration, consumes energy. The horsepower requirements have been converted into electrical units (KWH). A corresponding cost has been developed using a unit price of 2¢ per KWH. Adjustments to this unit cost for site-specific applications can be made by direct ratio.

Energy other than electrical is required for some processes (such as incineration). In these cases, the amount of energy (fuel) required was computed and a cost was determined on the basis of an assumed cost per unit; this assumed unit cost appears with the cost curves. Adjustments to this unit cost again may be made on a direct-ratio basis.

H.3.2.4 Chemical Cost

Various unit processes require chemicals. The quantities of chemicals required were based on assumed loading rates. The costs were then computed using a unit price for each chemical. The assumed unit chemical costs are included in Table H-1. Adjustments to the unit costs may be made by direct ratio within the same bulk quantity ranges.

H.4 Cost Format

The cost data in this appendix are presented in a format designed to facilitate use at the various levels of the planning process. The general presentation of the cost data is as follows:

- Treatment System Performance Matrix
  - Table H-3. Provides selected Municipal Wastewater Treatment Systems that will achieve varying degrees of treatment.
- Municipal Wastewater Treatment Systems: Performance and Costs
  - Figures H-2 through H-16, Section H.6
- Wet-Weather Treatment Processes: Costs
  - Figures H-17 through H-27, Section H.7
- Support Personnel: Costs
  - Figure H-28, Section H.8.1
- Miscellaneous Structures: Costs
  - Figure H-29, Section H.8.2
- Wastewater Treatment Processes: Costs
  - Figures H-30 through H-62, Section H.8.3
- Sludge Treatment and Disposal Processes: Costs
  - Figures H-63 through H-83, Section H.8.4
- Wastewater Transportation Methods: Costs
  - Figures H-84 and H-85, Section H.8.5
- Sludge Transportation Methods: Costs
  - Figures H-86 through H-90, Section H.8.6

The Treatment System curves will probably be most useful in the 208 process, because they provide a relatively quick and efficient means of developing comparative costs of various treatment facilities. If costs which are more specific to the site or facility involved are desired, the process curves in Sections H.7 and H.8 can be used to synthesize specific treatment systems as appropriate.

The process curves generally include individual unit processes, so that the system configuration for both treatment and sludge handling can be more flexible.

H.5 Treatment System Performance Matrix

The Treatment System Performance Matrix, Table H-3, links the point source load allocations and the Municipal Wastewater Treatment System cost curves.

Water quality throughout the planning period is defined by applying 20-year projected wastewater effluent characteristics from municipal wastewater treatment plants. Should an impact analysis indicate unacceptable stream water quality with these effluent characteristics, additional treatment would be required. The wastewater allocation needed to meet the water quality objectives establishes the effluent restrictions and, therefore, the required level of treatment. The control level refers to the projected effluent characteristics. Comparing these requirements to the Treatment System Performance Matrix indicates the system curve that most closely approximates the desired level of control. This should be done with caution because, when using the matrix, the system selected will be controlled by a single parameter. For example, System 11 may be required to meet the desired nitrogen level; therefore, total nitrogen is the controlling parameter. In order to achieve the required removal of this pollutant, System 11 might remove more BOD_5 than necessary. If this type of situation appears extreme, the user should evaluate the configuration of the required system to determine if the imbalance is unavoidable. If the manual user can conceive an alternative system that will produce a more balanced effluent with respect to the required effluent objective, then a new system curve should be developed from the component curves (Sections H.7 and H.8). In the foregoing example, a treatment system consisting of System 5 with the addition of breakpoint chlorination may provide a more cost-effective alternative to System 11. Should this be the case, the user of the manual can develop his own systems curves on the forms provided. The component municipal treatment process cost curves will provide the basis for the hybrid treatment system curves. The system configuration should be developed and the cumulative costs developed from the component municipal treatment process curves. The cumulative costs may be plotted on the forms provided.

TABLE H-3

TREATMENT SYSTEM PERFORMANCE MATRIX

System Number	Figure No	EFFLUENT CHARACTERISTICS BOD5	TSS	COD	Total P	NH3-N	NO3-N	UOD[1]	Description of System
-	-	210	230	400	11	20	0	406	Raw Waste Characteristics
1	H-2	130	100	250	9	20	0	286	Primary
2	H-3	100	50	185	2	20	0	241	Primary and Metal Salt Addition ($FeCl_3$)
3	H-4	45	60	90	8	18	0	150	Primary and Trickling Filter
4	H-5	25	30	50	2	18	0	120	Primary and Trickling Filter with Metal Salt ($FeCl_3$)
5	H-6	20	20	45	7	17	0	107	Primary and Activated Sludge
6	H-7	15	15	35	2	17	0	100	Primary, Activated Sludge, Metal Salt (Alum)
7	H-8	10	20	35	8	2	18	24	Primary, Activated Sludge/Nitrification
8	H-9	10	20	45	8	1	1	20	Primary, Activated Sludge, Nitrification, Denitrification
9	H-10	10	10	45	1	2	18	24	Primary Metal Salt Addition (Alum), Activated Sludge/Nitrification Filtration
10	H-11	5	5	25	1	20	0	99	Preliminary, Two-Stage Lime, Filtration, Carbon Adsorption
11	H-12	5	5	30	0.5	1	1	12	Primary Metal Salt Addition (Alum), Activated Sludge, Nitrification, Denitrification, Polymer, Filtration
12	H-13	3	5	10	0.5	1	1	12	Primary Metal Salt Addition (Alum), Activated Sludge, Nitrification-Denitrification, Polymer, Filtration, Carbon Adsorption
13	H-14	20	20	45	7	17[2] 2[3]	0	107	Small-Flow Treatment Systems, i.e., Package Plants - Extended Aeration Plant 0.01-0.1 mgd; contact stabilization plant 0.1-1.0 mgd.
14	H-15	15	15	46	6	1.0	8	27	Oxidation Ditch; 0.05-10 mgd (designed for nitrogen removal)
15	H-16	-	-	-	-	-	-	-	Land Application

Note: Treatment systems 1-12 include disinfection, sludge handling, miscellaneous structures, and support personnel. Treatment Systems 13-15 DO NOT include sludge dewatering, miscellaneous structures and support personnel.

[1] UOD = Ultimate Oxygen Demand = $(1.5 \times BOD_5) + (4.57 \times NH_3\text{-}N)$

[2] Contact Stabilization.

[3] Extended Aeration.

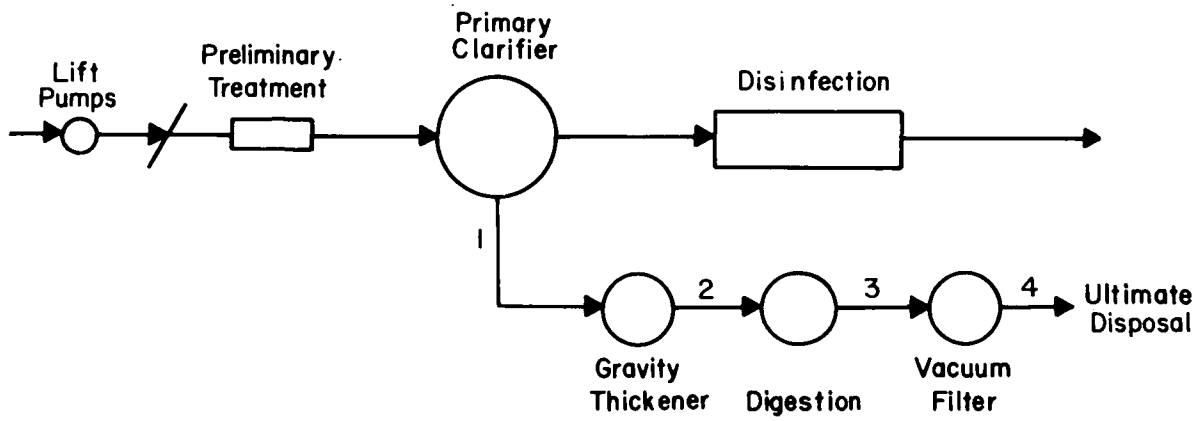
H.6 Municipal Wastewater Treatment Systems

Municipal wastewater treatment systems that will meet various effluent criteria are presented in the Treatment System Performance Matrix (Table H-3). These systems are not presented as a comprehensive list of the only technological methods available to meet the effluent criteria, but rather to provide the user of this manual with an expedient compilation of monetary costs for selected systems. Construction and operation & maintenance cost curves have been developed for each of these systems. The cost curves and related effluent quality, sludge audit, process, service life, design, and other data are presented in Figures H-2 through H-16.

Municipal Wastewater Treatment Systems:
Performance and Costs:

Figures H-2 through H-16

FIGURE H-2 TREATMENT SYSTEM 1-
PRIMARY TREATMENT



Processes Included: Lift Pumps, Preliminary Treatment, Primary Clarifiers, Chlorination, Gravity Thickener, Sludge Digestion, Vacuum Filters, Miscellaneous Structures, Support Personnel.

Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	130
COD, mg/l	400	265
TSS, mg/l	230	100
Total-P, mg/l	11	9
NH_3-N, mg/l	20	20
NO_3-N, mg/l	0	0
UOD, mg/l	406	286

Sludge Audit:

Point No.	Sludge Quantity lbs/mg	Concentration %
1	1,080	4
2	1,080	8
3	540	4.5
4	540	20

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. Aerobic digestion at plant sizes less than 3 mgd; two-stage anaerobic digestion at plant sizes greater than 3 mgd.

FIGURE H-2 TREATMENT SYSTEM 1- PRIMARY TREATMENT

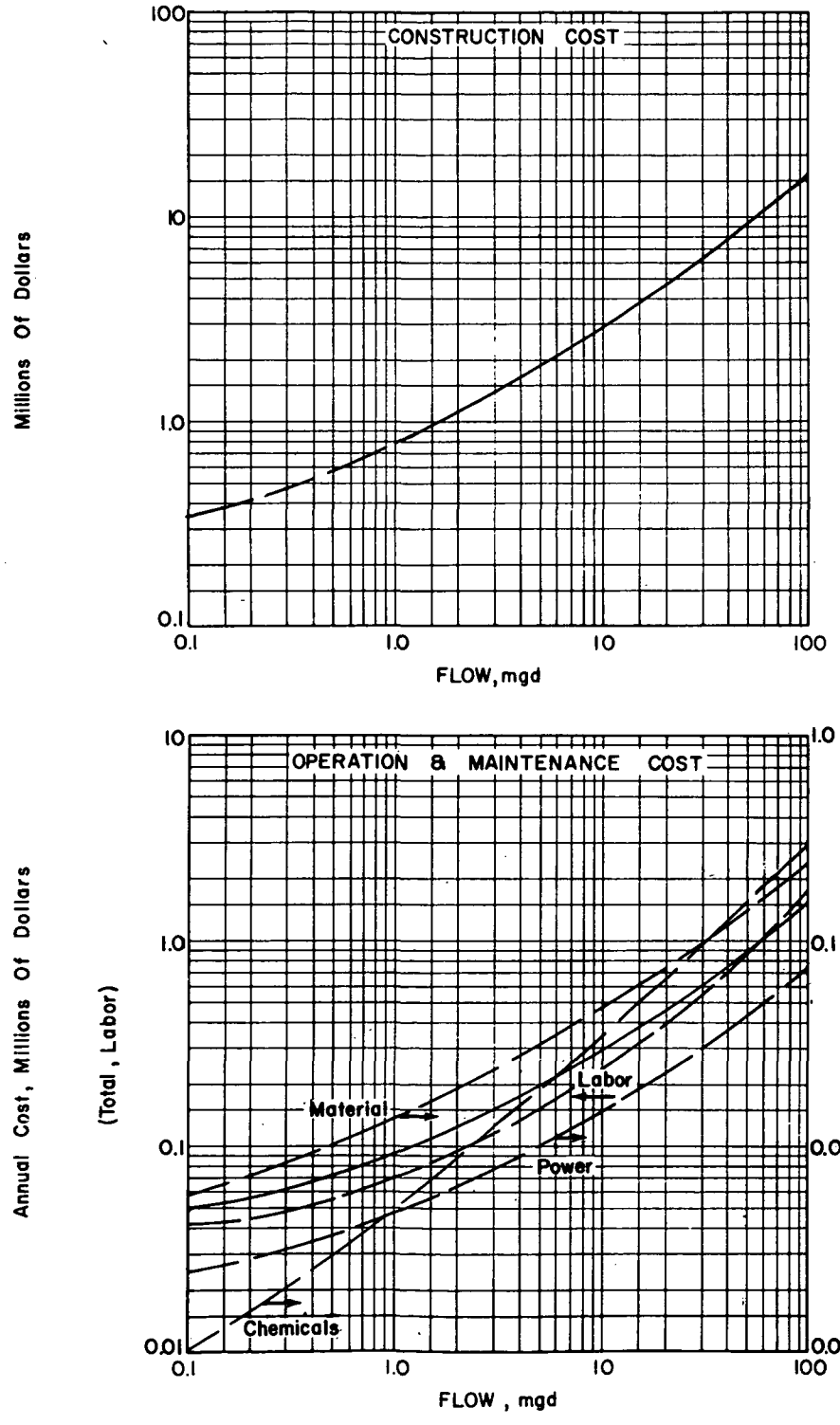
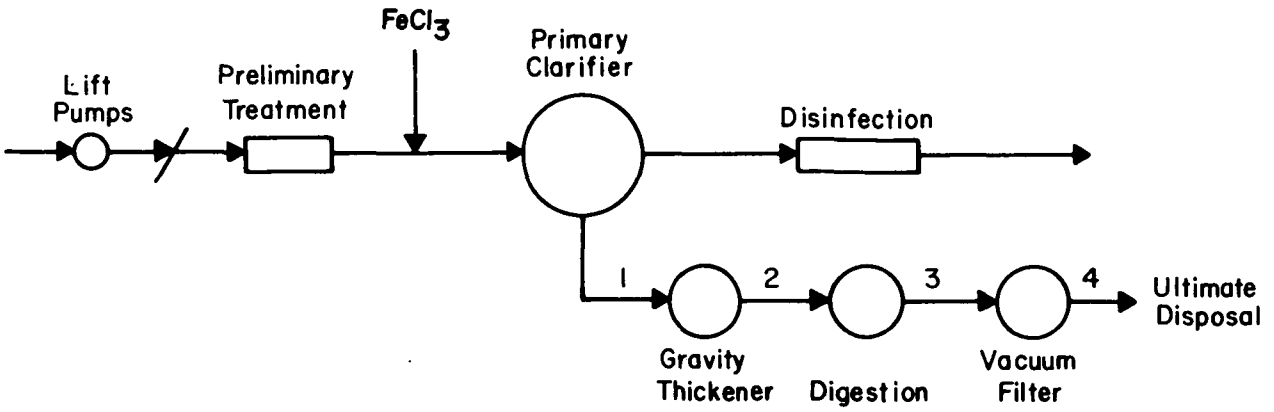


FIGURE H-3 TREATMENT SYSTEM 2-
PRIMARY TREATMENT WITH METAL-SALT ADDITION ($FeCl_3$)



Processes Included: Lift Pumps, Preliminary Treatment, Ferric Chloride Addition, Primary Clarifiers, Chlorination, Gravity Thickener, Sludge Digestion, Vacuum Filters, Miscellaneous Structures, Support Personnel.

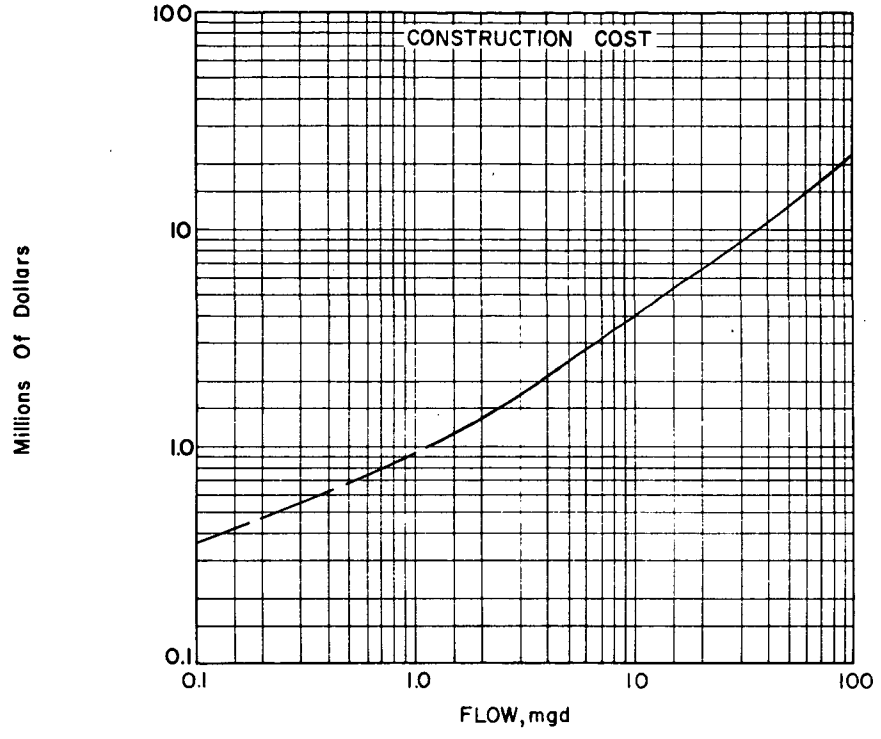
Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	100
COD, mg/l	400	185
TSS, mg/l	230	50
Total-P, mg/l	11	2
NH_3-N, mg/l	20	20
NO_3-N, mg/l	0	0
UOD, mg/l	406	241

Sludge Audit:

Point No.	Sludge Quantity lbs/mg	Concentration %
1	2,160	2
2	2,160	4
3	1,410	3
4	1,410	20

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. $FeCl_3$ dosage at 100 mg/l.
4. Aerobic digestion at plant sizes less than 3 mgd; two-stage anaerobic digestion at plant sizes greater than 3 mgd.

FIGURE H-3 TREATMENT SYSTEM 2- PRIMARY TREATMENT WITH METAL-SALT ADDITION ($FeCl_3$)



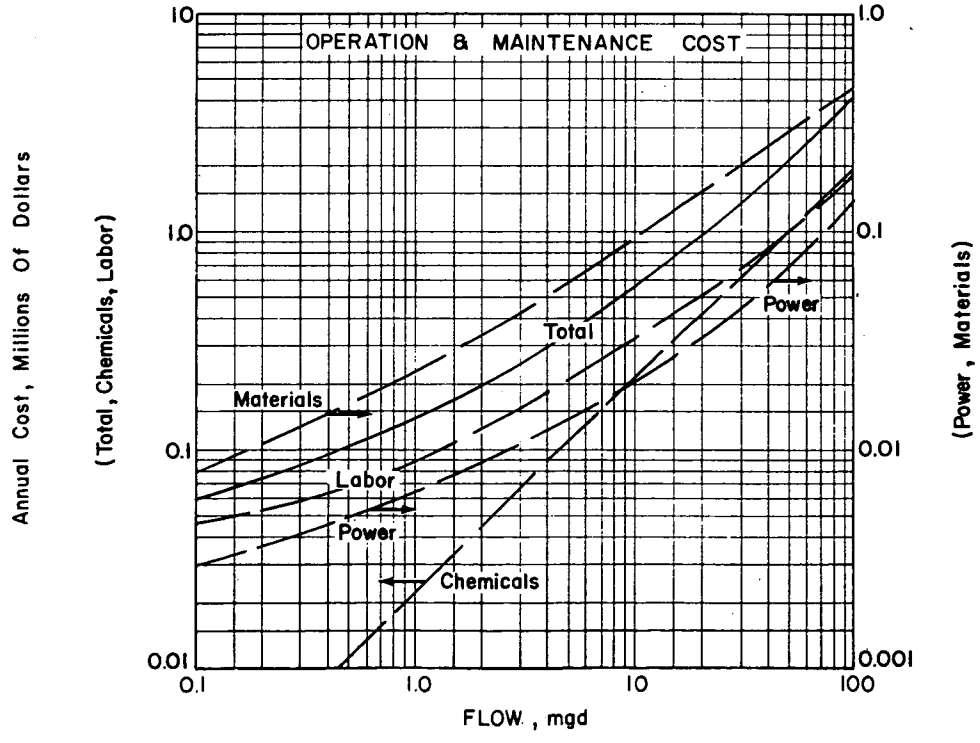
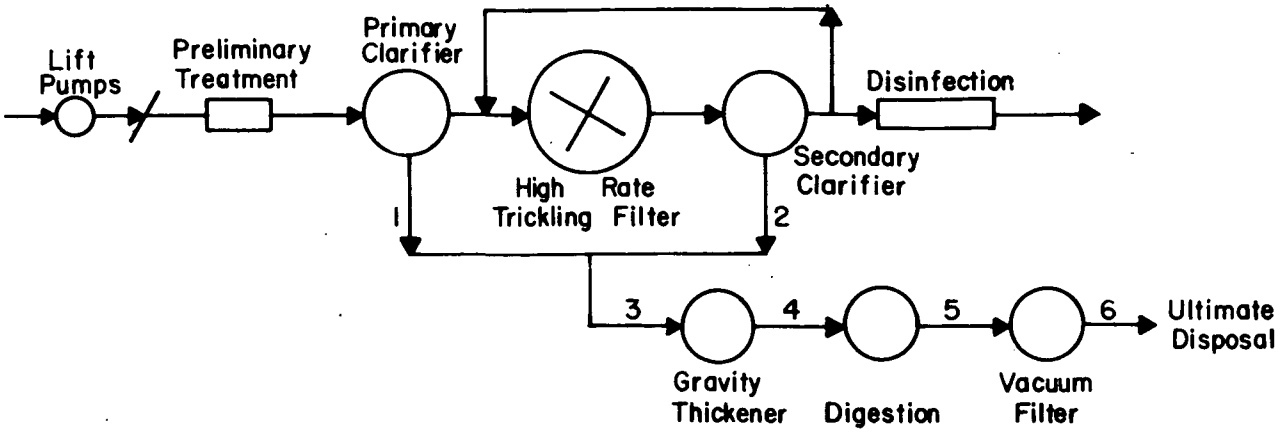


FIGURE H-4 TREATMENT SYSTEM 3-
TRICKLING FILTER



Processes Included: Lift Pumps, Preliminary Treatment, Primary Clarifiers, High Rate Trickling Filter, Secondary Clarifiers, Chlorination, Gravity Thickener, Sludge Digestion, Vacuum Filters, Miscellaneous Structures, Support Personnel.

Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	45
COD, mg/l	400	90
TSS, mg/l	230	60
Total-P, mg/l	11	8
NH_3-N, mg/l	20	18
NO_3-N, mg/l	0	0
UOD, mg/l	406	150

Sludge Audit:

Point No.	Sludge Quantity lbs/mg	Concentration %
1	1,080	4
2	450	3
3	1,530	4
4	1,530	5
5	765	3
6	765	20

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. Aerobic digestion at plant sizes less than 3 mgd; two-stage anaerobic digestion at plant sizes greater than 3 mgd.

FIGURE H-4 TREATMENT SYSTEM 3- TRICKLING FILTER

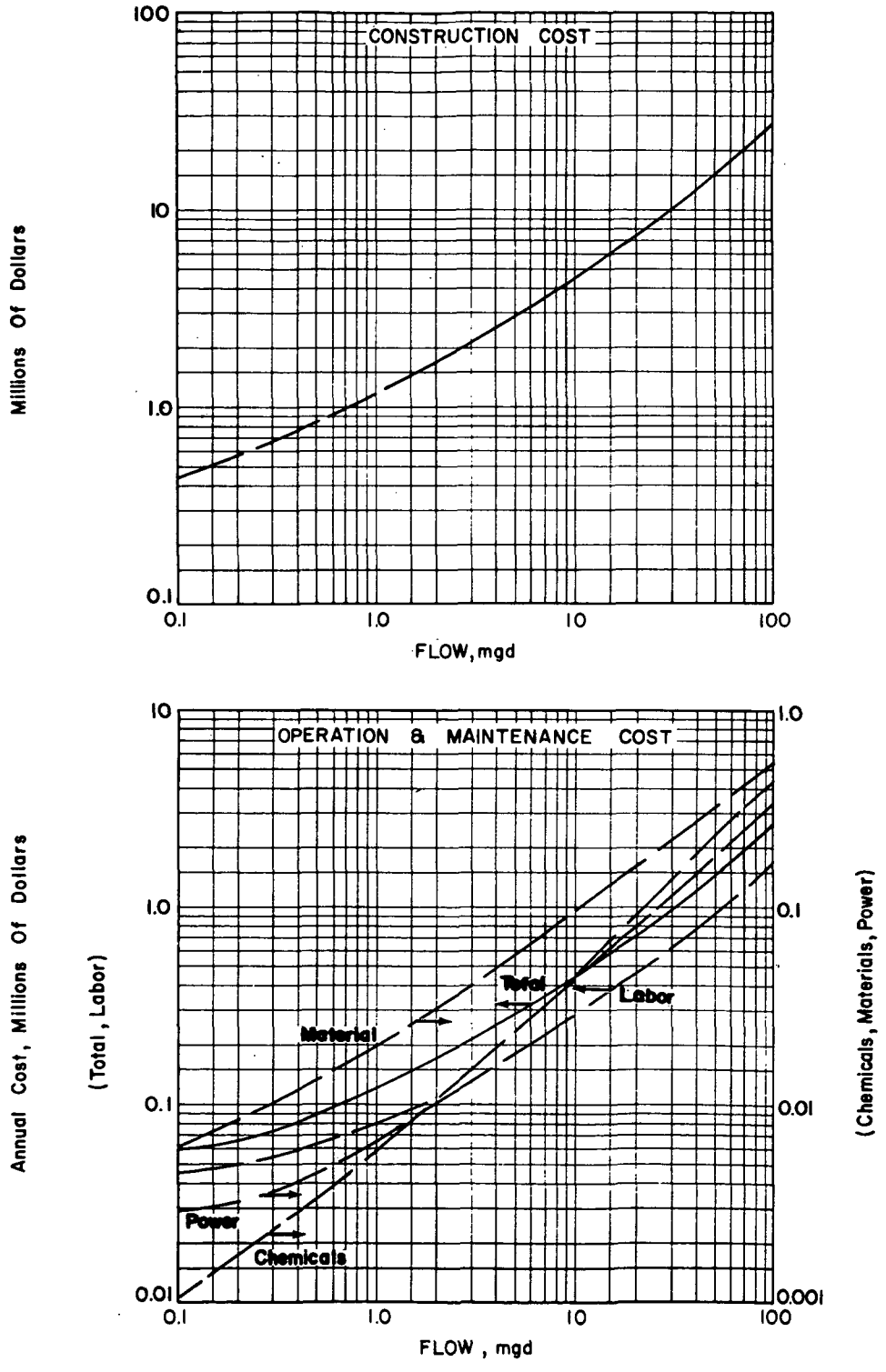
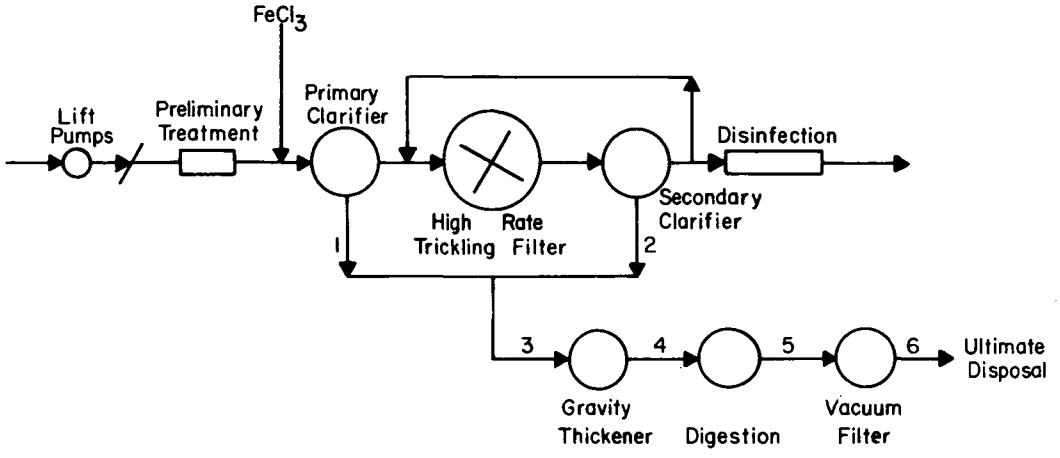


FIGURE H-5 TREATMENT SYSTEM 4-
TRICKLING FILTER WITH METAL- SALT ADDITION ($FeCl_3$)



Processes Included: Lift Pumps, Preliminary Treatment, Ferric Chloride Addition, Primary Clarifiers, High Rate Trickling Filters, Secondary Clarifiers, Chlorination, Gravity Thickener, Sludge Digestion, Vacuum Filters, Miscellaneous Structures, Support Personnel.

Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	25
COD, mg/l	400	50
TSS, mg/l	230	30
Total-P, mg/l	11	2
NH_3-N, mg/l	20	18
NO_3-N, mg/l	0	0
UOD, mg/l	406	120

Sludge Audit:

Point No.	Sludge Quantity lbs/mg	Concentration %
1	2,160	2
2	330	3
3	2,490	2
4	2,490	4
5	1,580	3
6	1,580	20

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. $FeCl_3$ dosage at 100 mg/l.
4. Aerobic digestion at plant sizes less than 3 mgd; two-stage anaerobic digestion at plant sizes greater than 3 mgd.

FIGURE H-5 TREATMENT SYSTEM 4- TRICKLING FILTER WITH METAL-SALT ADDITION ($FeCl_3$)

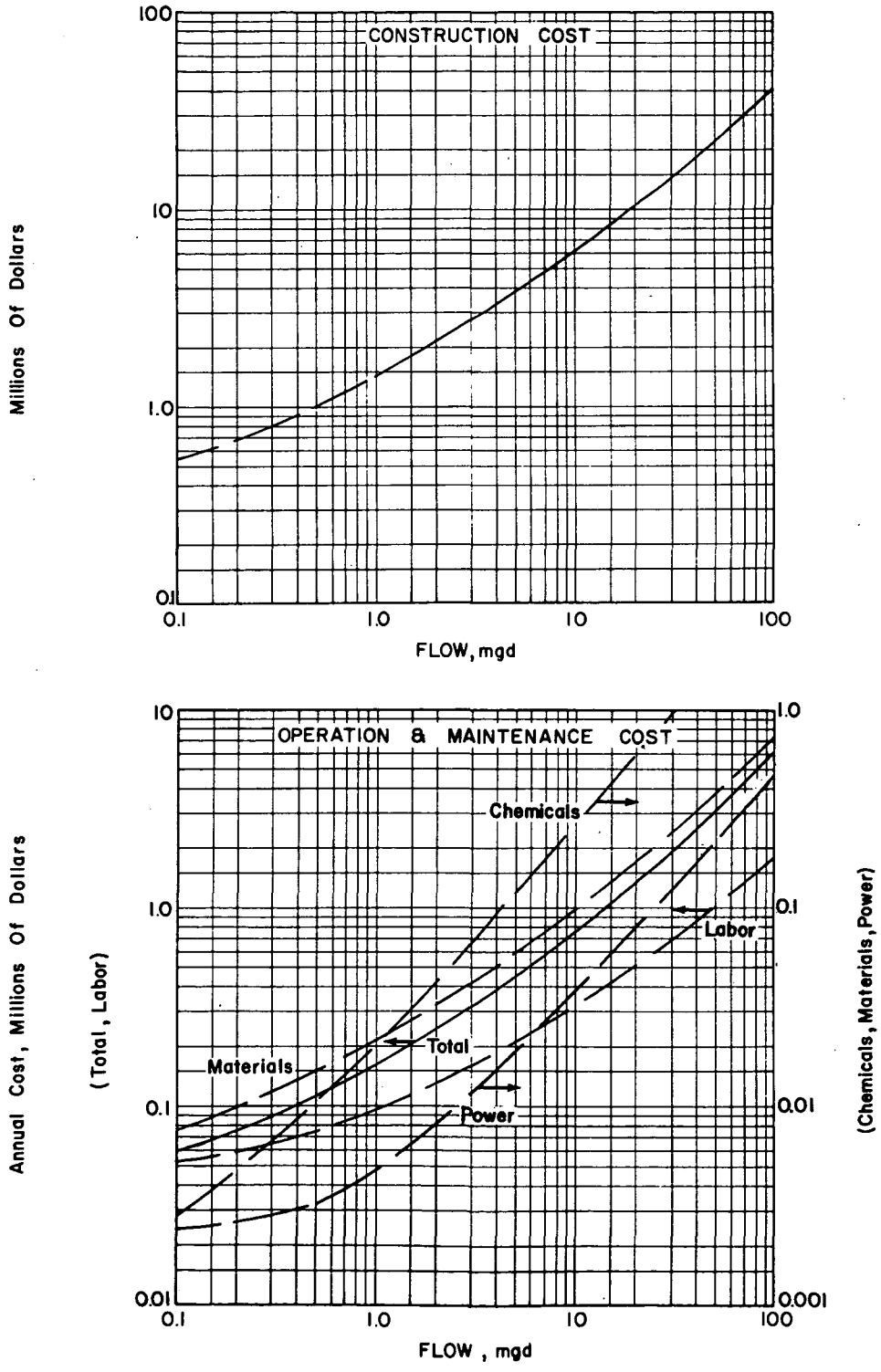
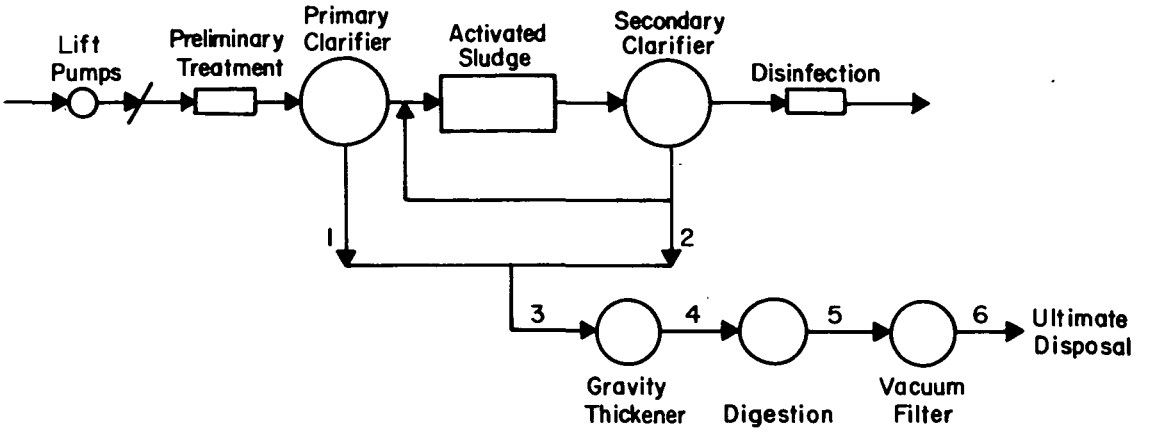


FIGURE H-6 TREATMENT SYSTEM 5-
ACTIVATED SLUDGE



Processes Included: Lift Pumps, Preliminary Treatment, Primary Clarifiers, Activated Sludge, Secondary Clarifiers, Chlorination, Gravity Thickener, Sludge Digestion, Vacuum Filters, Miscellaneous Structures, Support Personnel.

Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	20
COD, mg/l	400	45
TSS, mg/l	230	20
Total-P, mg/l	11	7
NH_3-N, mg/l	20	17
NO_3-N, mg/l	0	0
UOD, mg/l	406	107

Sludge Audit:

Point No.	Sludge Quantity lbs/mg	Concentration %
1	1,080	4
2	820	0.8
3	1,900	2.6
4	1,900	8
5	950	5
6	950	20

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. Aerobic digestion at plant sizes less than 3 mgd; two-stage anaerobic digestion at plant sizes greater than 3 mgd.

FIGURE H-6 TREATMENT SYSTEM 5- ACTIVATED SLUDGE

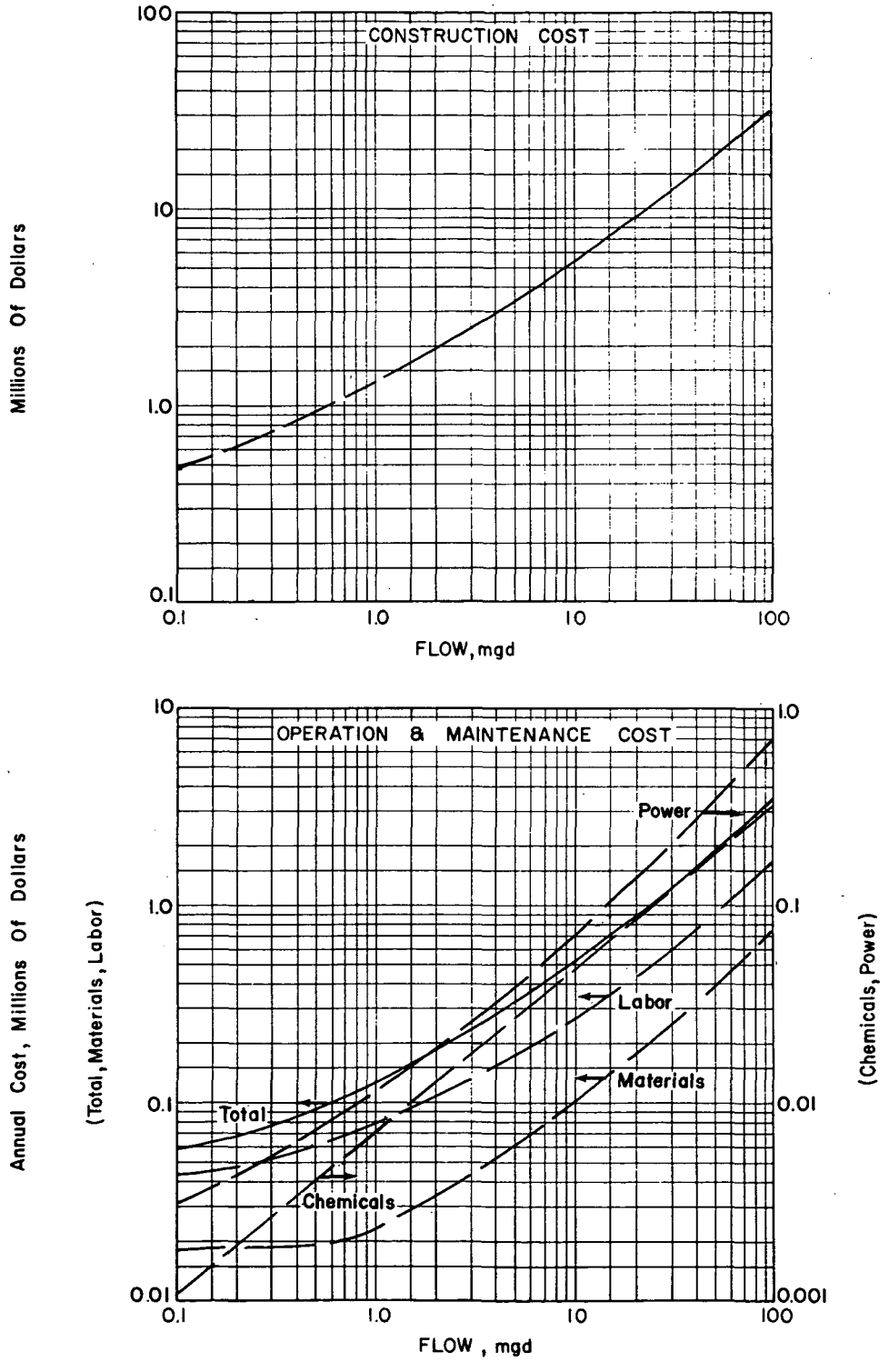
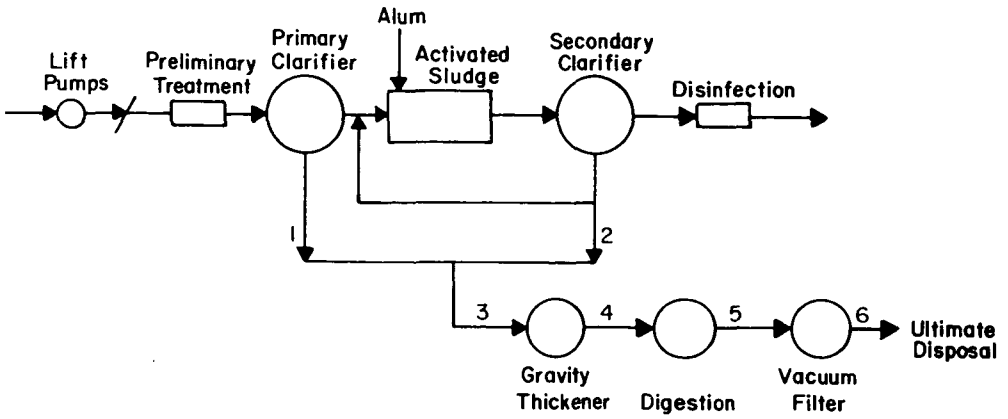


FIGURE H-7 TREATMENT SYSTEM 6-
ACTIVATED SLUDGE WITH METAL-SALT ADDITION (ALUM)



Processes Included: Lift Pumps, Preliminary Treatment, Primary Clarifiers, Alum Addition, Activated Sludge, Secondary Clarifiers, Chlorination, Gravity Thickener, Sludge Digestion, Vacuum Filters, Miscellaneous Structures, Support Personnel.

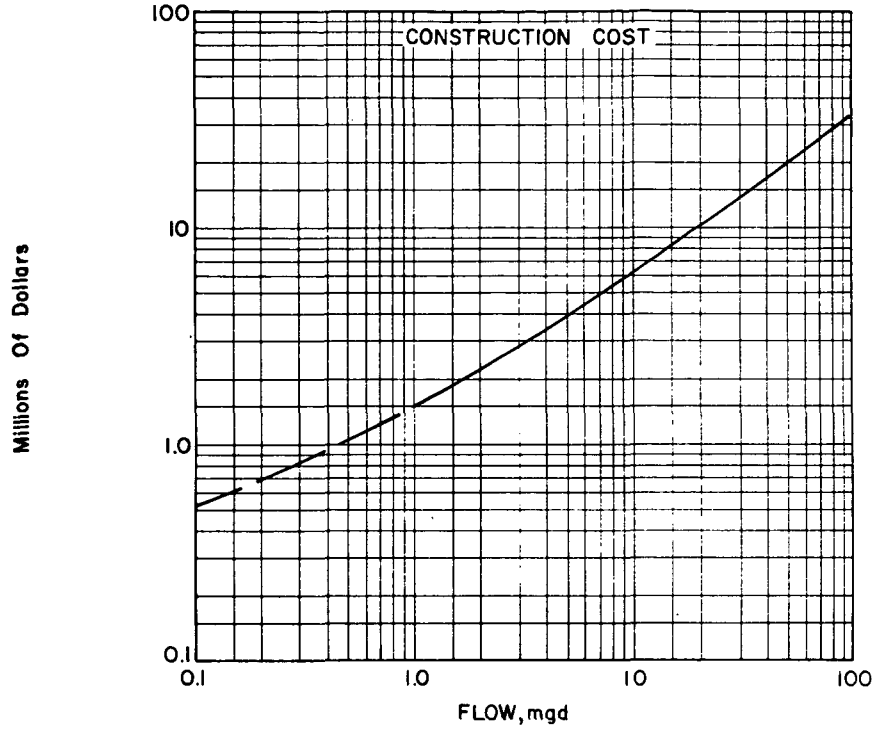
Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	15
COD, mg/l	400	35
TSS, mg/l	230	15
Total-P, mg/l	11	2
NH_3-N, mg/l	20	17
NO_3-N, mg/l	0	0
UOD, mg/l	406	100

Sludge Audit:

Point No.	Sludge Quantity lbs/mg	Concentration %
1	1,080	4
2	1,150	0.8
3	2,230	2.3
4	2,230	6
5	1,280	3.5
6	1,280	20

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. Alum dosage at 100 mg/l.
4. Aerobic digestion at plant sizes less than 3 mgd; two-stage anaerobic digestion at plant sizes greater than 3 mgd.

FIGURE H-7 TREATMENT SYSTEM 6 - ACTIVATED SLUDGE WITH METAL-SALT ADDITION (ALUM)



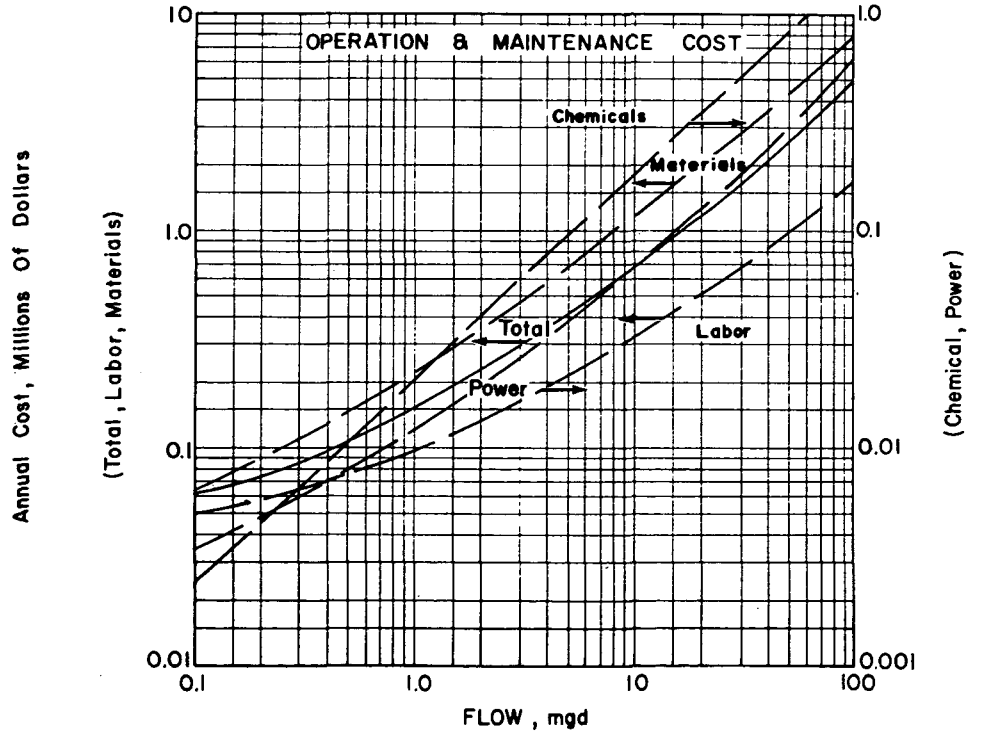
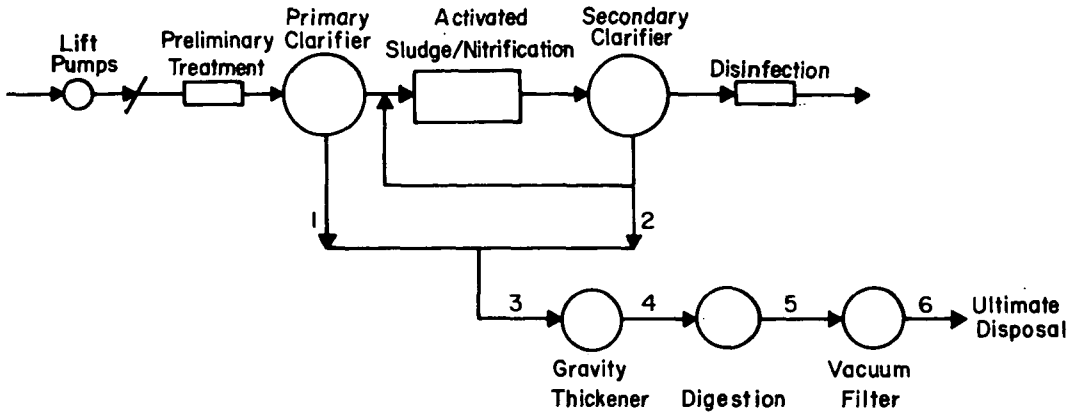


FIGURE H-8 TREATMENT SYSTEM 7-
ACTIVATED SLUDGE/NITRIFICATION (SINGLE STAGE)



Processes Included: Lift Pumps, Preliminary Treatment, Primary Clarifiers, Activated Sludge with Nitrification, Secondary Clarifiers, Chlorination, Gravity Thickener, Sludge Digestion, Vacuum Filters, Miscellaneous Structures, Support Personnel.

Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	10
COD, mg/l	400	35
TSS, mg/l	230	20
Total-P, mg/l	11	8
NH_3-N, mg/l	20	2
NO_3-N, mg/l	0	18
UOD, mg/l	406	24

Sludge Audit:

Point No.	Sludge Quantity lbs/mg	Concentration %
1	1,080	4
2	820	0.8
3	1,900	2.6
4	1,900	4
5	950	2.5
6	950	20

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. Aerobic digestion at plant sizes less than 3 mgd; two-stage anaerobic digestion at plant sizes greater than 3 mgd.

FIGURE H-8 TREATMENT SYSTEM 7-
ACTIVATED SLUDGE / NITRIFICATION
(SINGLE STAGE)

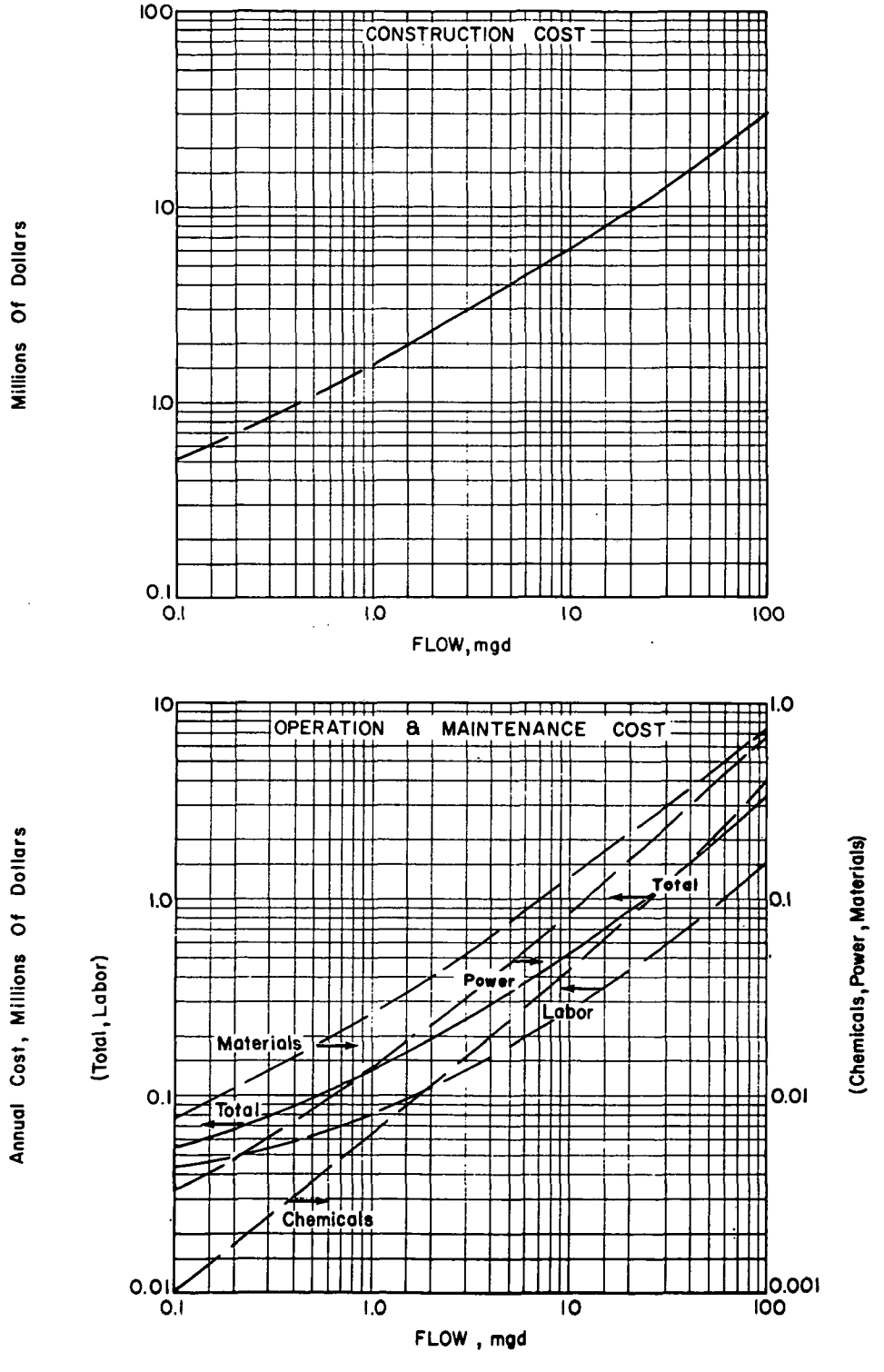
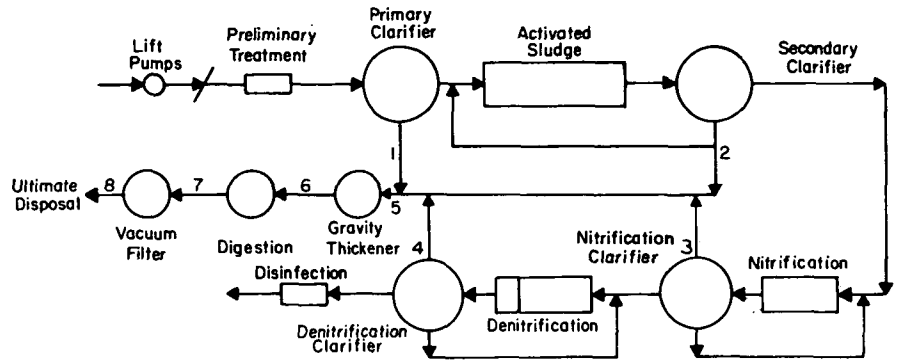


FIGURE H-9 TREATMENT SYSTEM 8-
ACTIVATED SLUDGE/NITRIFICATION/DENITRIFICATION (THREE STAGES)



Processes Included: Lift Pumps, Preliminary Treatment, Primary Clarifiers, Activated Sludge, Secondary Clarifiers, Nitrification, Denitrification, Chlorination, Gravity Thickener, Sludge Digestion, Vacuum Filters, Miscellaneous Structures, Support Personnel.

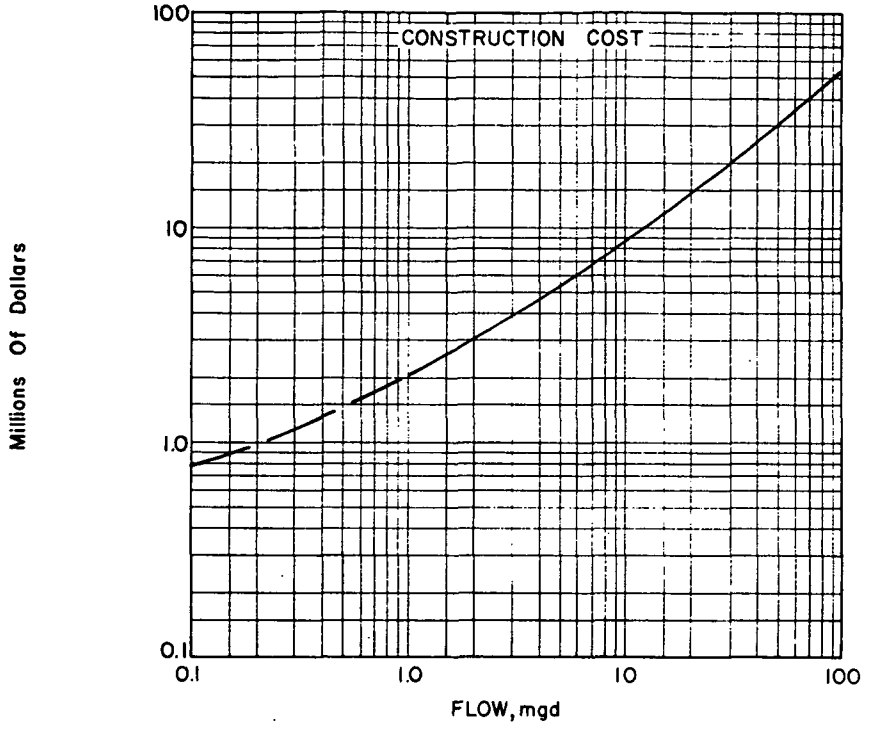
Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	10
COD, mg/l	400	45
TSS, mg/l	230	20
Total-P, mg/l	11	8
NH_3-N, mg/l	20	1
NO_3-N, mg/l	0	1
UOD, mg/l	406	20

Sludge Audit:

Point No.	Sludge Quantity lbs/mg	Concentration %
1	1,080	4
2	720	0.8
3	100	0.8
4	280	0.8
5	2,180	2.4
6	2,180	4
7	1,090	2.5
8	1,090	20

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. Aerobic digestion at plant sizes less than 3 mgd; two-stage anaerobic digestion at plant sizes greater than 3 mgd.

FIGURE H-9 TREATMENT SYSTEM 8-
ACTIVATED SLUDGE/NITRIFICATION/DENITRIFICATION
(THREE STAGES)



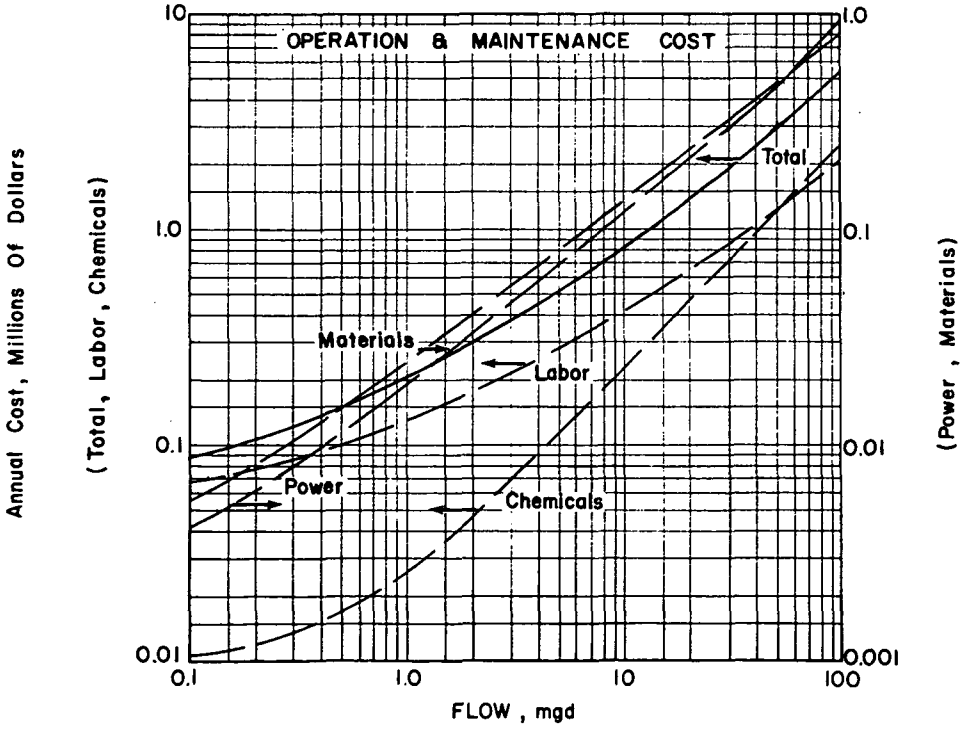
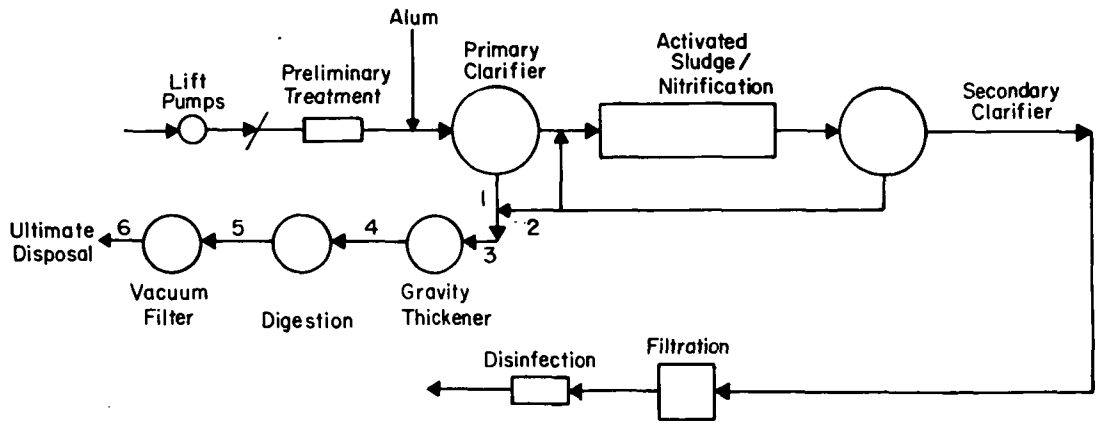


FIGURE H-10 TREATMENT SYSTEM 9-
ACTIVATED SLUDGE/NITRIFICATION - FILTRATION (WITH ALUM)



Processes Included: Lift Pumps, Preliminary Treatment, Alum Addition, Primary Clarifiers, Activated Sludge with Nitrification, Secondary Clarifiers, Filtration, Chlorination, Gravity Thickener, Sludge Digestion, Vacuum Filters, Miscellaneous Structures, Support Personnel.

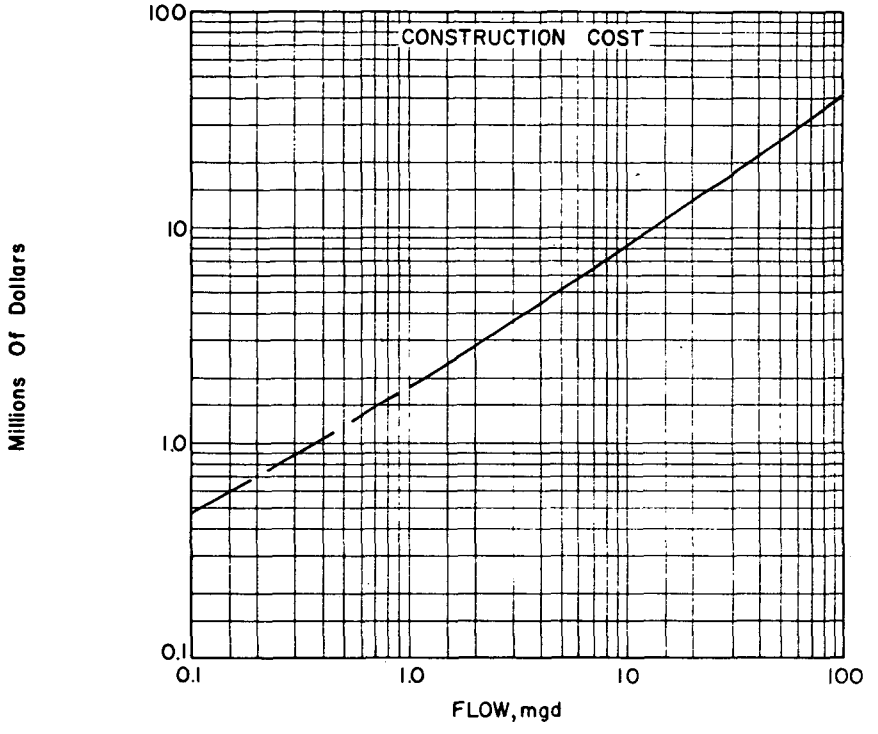
Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	10
COD, mg/l	400	45
TSS, mg/l	230	10
Total-P, mg/l	11	1
NH_3-N, mg/l	20	2
NO_3-N, mg/l	0	18
UOD, mg/l	406	24

Sludge Audit:

Point No.	Sludge Quantity lbs/mg	Concentration %
1	1,800	3
2	600	0.8
3	2,430	2.5
4	2,430	4
5	1,380	2.5
6	1,380	20

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. Aerobic digestion at plant sizes less than 3 mgd; two-stage anaerobic digestion at plant sizes greater than 3 mgd.

FIGURE H-10 TREATMENT SYSTEM 9-
ACTIVATED SLUDGE/NITRIFICATION - FILTRATION
(WITH ALUM)



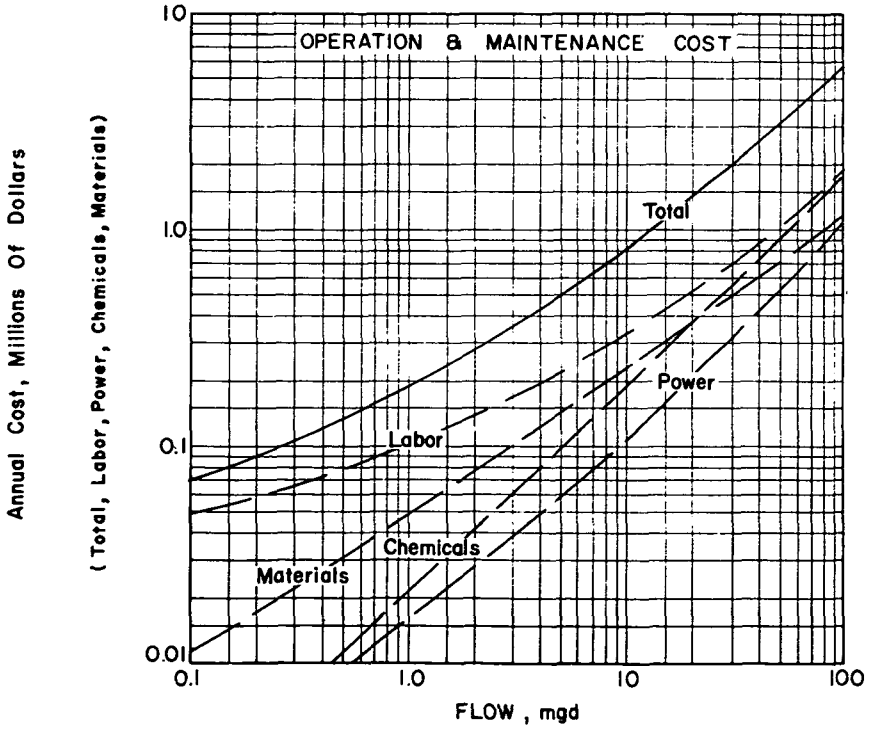
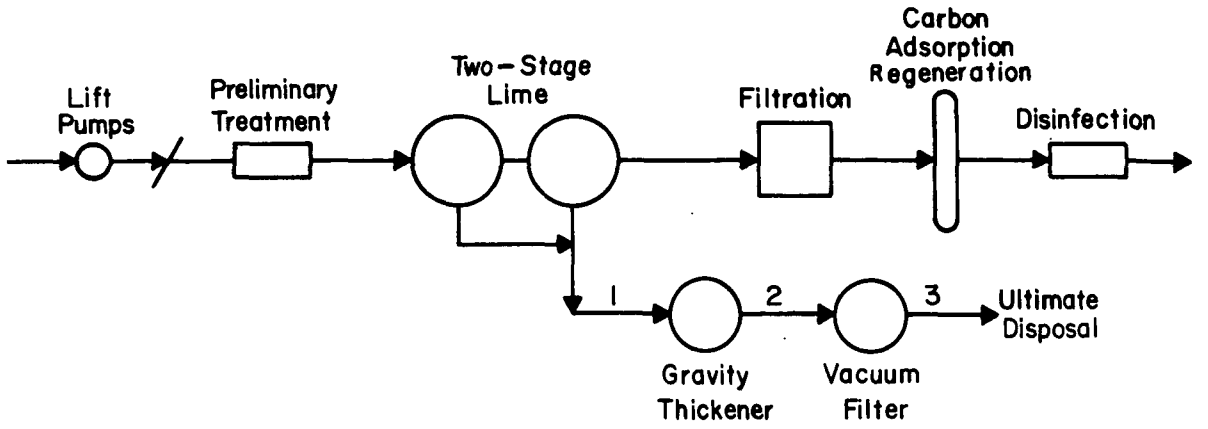


FIGURE H-11 TREATMENT SYSTEM 10-
PHYSICAL/CHEMICAL



Processes Included: Lift Pumps, Preliminary Treatment, Two-Stage Lime, Gravity Filtration, Interstage Pumping, Carbon Adsorption, Chlorination, Gravity Thickener, Vacuum Filters, Miscellaneous Structures, Support Personnel.

Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	5
COD, mg/l	400	25
TSS, mg/l	230	5
Total-P, mg/l	11	1
NH_3-N, mg/l	20	20
NO_3-N, mg/l	0	0
UOD, mg/l	406	99

Sludge Audit:

Point No.	Sludge Quantity lbs/mg	Concentration %
1	7,500	5
2	7,500	10
3	7,500	30

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. Lime dosage at 400 mg/l as CaO.
4. Carbon adsorption without regeneration at plant sizes less than 3 mgd; carbon adsorption with regeneration at plant size greater than 3 mgd.

FIGURE H-11 TREATMENT SYSTEM 10- PHYSICAL/CHEMICAL

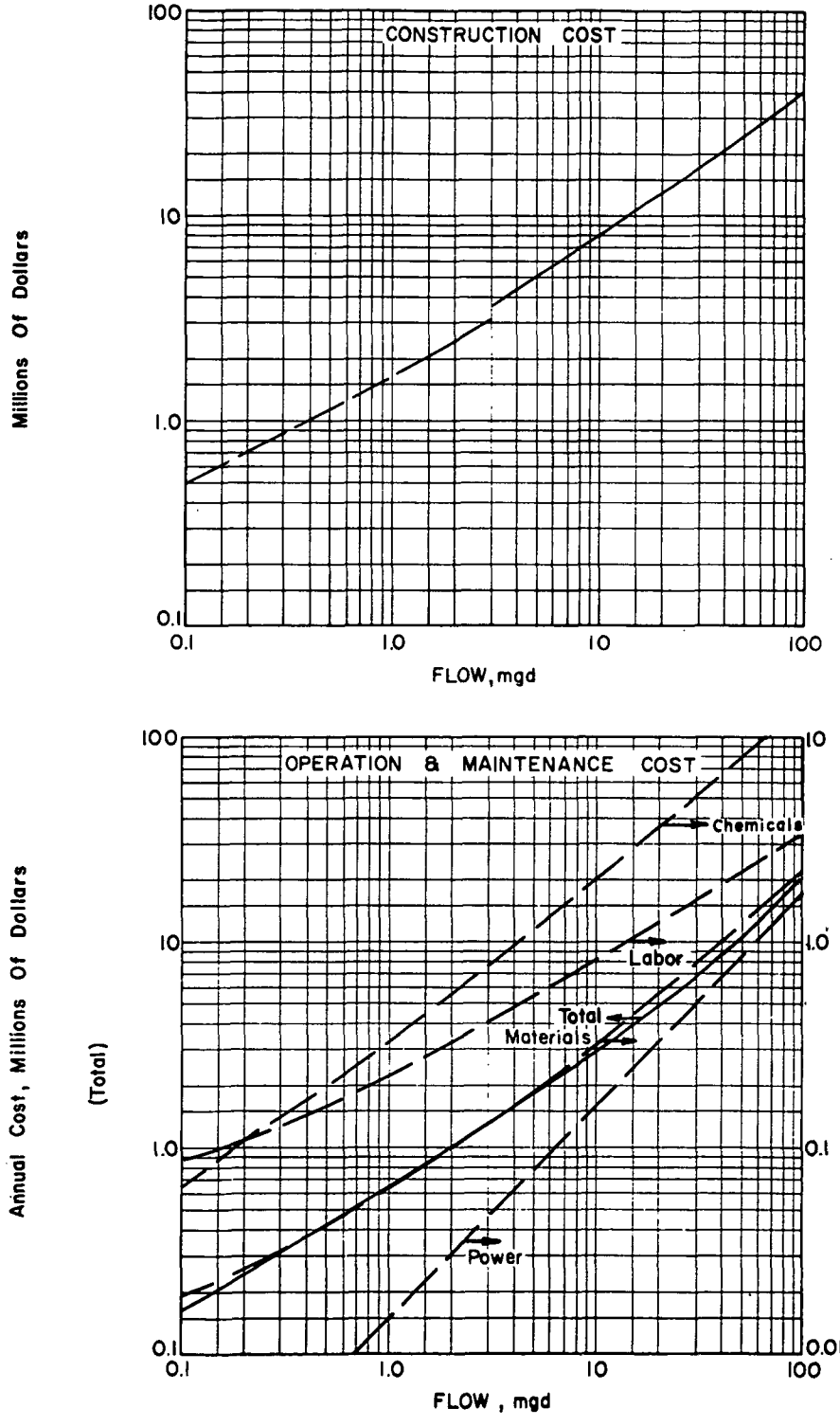
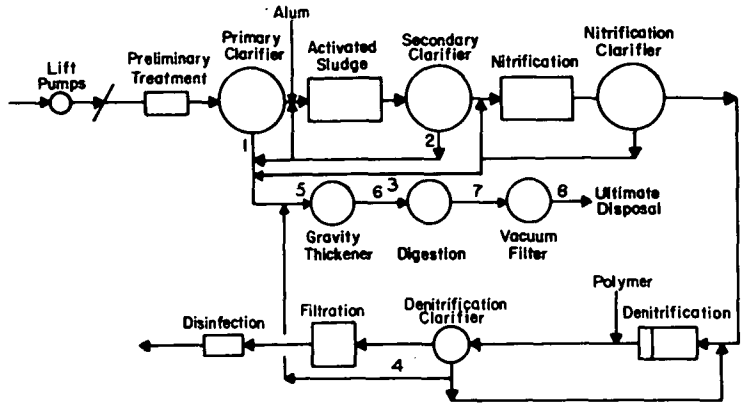


FIGURE H-12 TREATMENT SYSTEM 11-ACTIVATED SLUDGE/ NITRIFICATION/DENITRIFICATION/WITH ALUM (THREE STAGES)



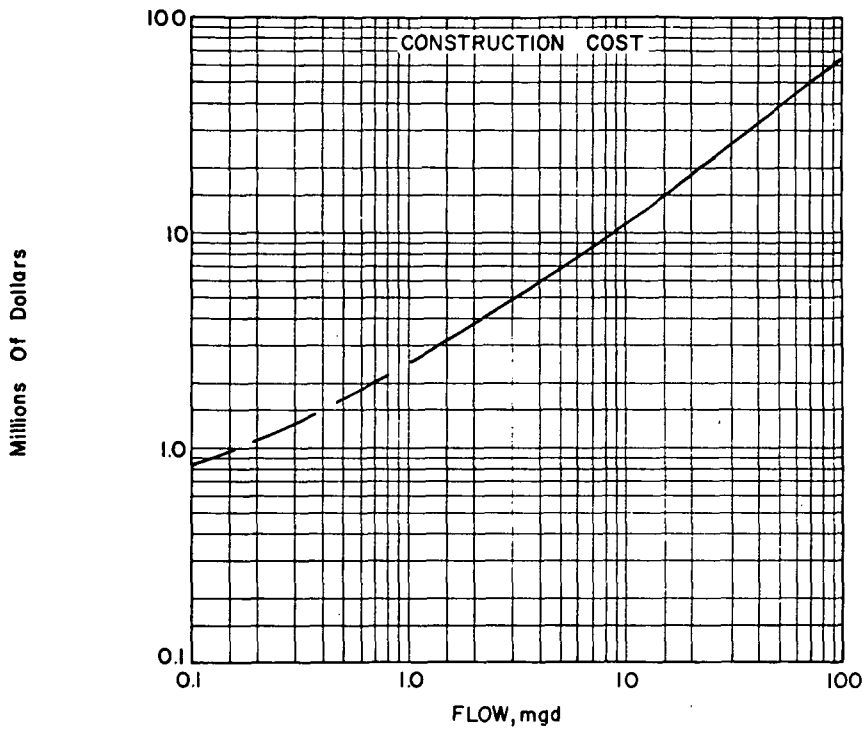
Processes Included: Lift Pumps, Preliminary Treatment, Primary Clarifiers, Alum Addition, Activated Sludge, Secondary Clarifiers, Nitrification, Nitrification Clarifier, Denitrification, Polymer Addition, Denitrification Clarifier, Filtration, Chlorination, Gravity Thickener, Sludge Digestion, Vacuum Filtration, Miscellaneous Structures, Support Personnel.

Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	5
COD, mg/l	400	30
TSS, mg/l	230	5
Total-P, mg/l	11	0.5
NH_3-N, mg/l	20	1
NO_3-N, mg/l	0	1
UOD, mg/l	406	12

Sludge Audit: Point No.	Sludge Quantity lbs/mg	Concentration %
1	1,080	4
2	1,050	0.8
3	100	0.8
4	280	0.8
5	2,510	2.8
6	2,510	4
7	1,420	3
8	1,420	20

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. Aerobic digestion at plant sizes less than 3 mgd; two-stage anaerobic digestion at plant sizes greater than 3 mgd.

FIGURE H-12 TREATMENT SYSTEM 11-
ACTIVATED SLUDGE / NITRIFICATION / DENITRIFICATION
WITH ALUM (THREE STAGES)



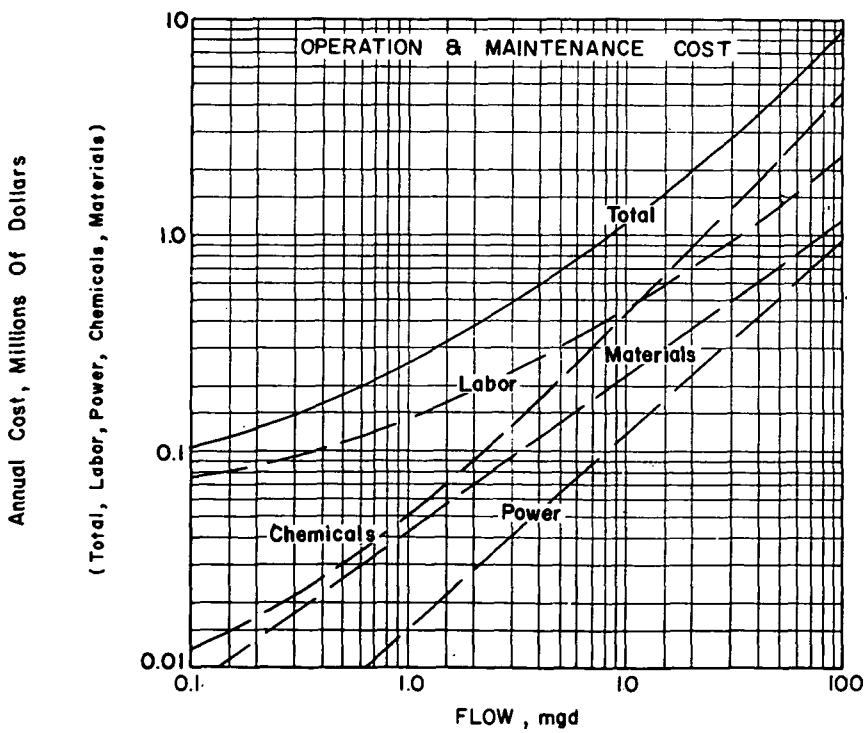
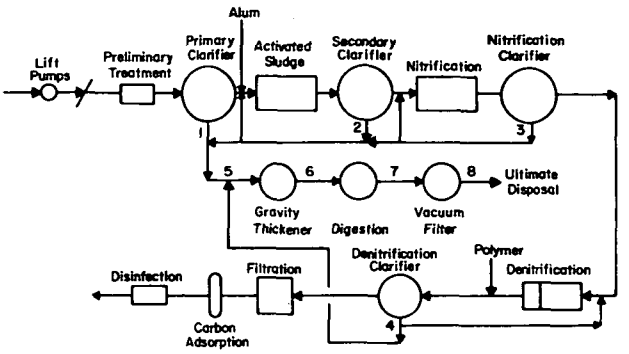


FIGURE H-13 TREATMENT SYSTEM 12-ACTIVATED SLUDGE/NITRIFICATION/ DENITRIFICATION/ACTIVATED CARBON (WITH ALUM)



Processes Included: Lift Pumps, Preliminary Treatment, Primary Clarifiers, Alum Addition, Activated Sludge, Secondary Clarifier, Nitrification, Nitrification Clarifier, Denitrification, Polymer Addition, Denitrification Clarifier, Filtration, Interstage Pumping, Carbon Adsorption, Chlorination, Gravity Thickener, Digestion, Vacuum Filter, Miscellaneous Structures, Support Personnel.

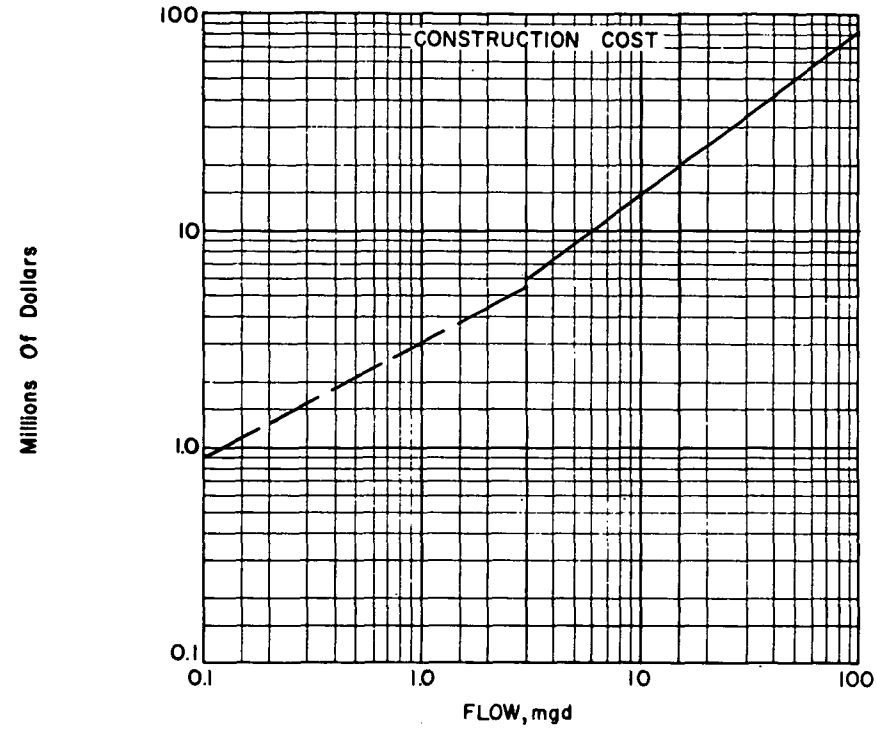
Wastewater Characteristics:	Influent	Effluent
BOD_5, mg/l	210	3
COD, mg/l	400	10
TSS, mg/l	230	5
Total-P, mg/l	11	0.5
NH_3-N, mg/l	20	1
NO_3-N, mg/l	0	1
UOD, mg/l	406	12

Sludge Audit:

Point No.	Sludge Quantity lbs/mg	Concentration %
1	1,080	4
2	1,050	1
3	100	0.8
4	280	0.8
5	2,510	2.8
6	2,510	4
7	1,420	3
8	1,420	20

Notes:
1. Dashed line on construction cost curve indicates size range of marginal applicability from a process and/or economic basis.
2. See individual treatment process curves for design basis of specific unit processes. (Adjustment factors should be used where applicable.)
3. Carbon adsorption without regeneration at plant sizes less than 3 mgd; carbon adsorption with regeneration at plant sizes greater than 3 mgd.

FIGURE H-13 TREATMENT SYSTEM 12-
ACTIVATED SLUDGE/NITRIFICATION/DENITRIFICATION/
ACTIVATED CARBON WITH ALUM ADDITION



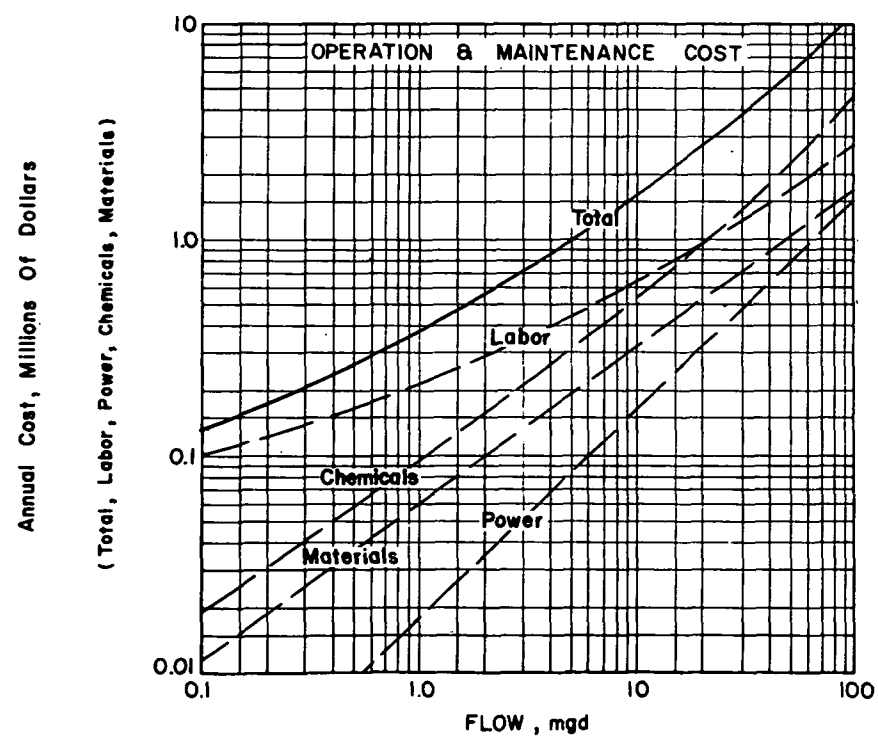


FIGURE H-14

TREATMENT SYSTEM 13
SMALL FLOW TREATMENT SYSTEMS

Service Life: 30 years

Relative Reliability: Level B ± 30%

Design Basis:

1. Construction costs include:

   a. Comminutor
   b. Aeration basin
   c. Clarifier
   d. Chlorine contact chamber
   e. Aerobic digester
   f. Chlorine feed facility
   g. Building
   h. Fencing

2. Extended aeration (E.A.) between 0.01 and 0.1 mgd.
   Detention time: 24 hours.

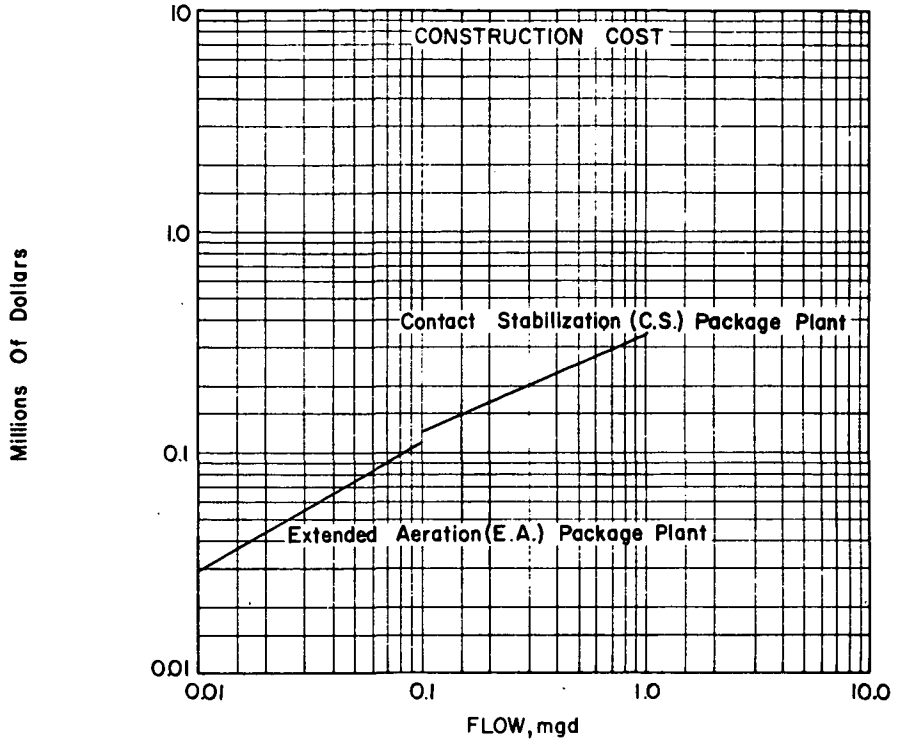
3. Contact stabilization (C.S.) between 0.1 and 1.0 mgd.
   Detention time: 3 hours contact; 6 hours stabilization.

4. Process Performance:

Wastewater Characteristics:	Influent	Effluent	
		C.S.	E.A.
BOD_5, mg/l	210	25	20
COD, mg/l	400	50	45
TSS, mg/l	230	25	20
Total-P, mg/l	11	7	7
NH_3-N, mg/l	20	17	2

References: 9, 43, 47

FIGURE H-14 TREATMENT SYSTEM 13-
SMALL FLOW TREATMENT SYSTEMS



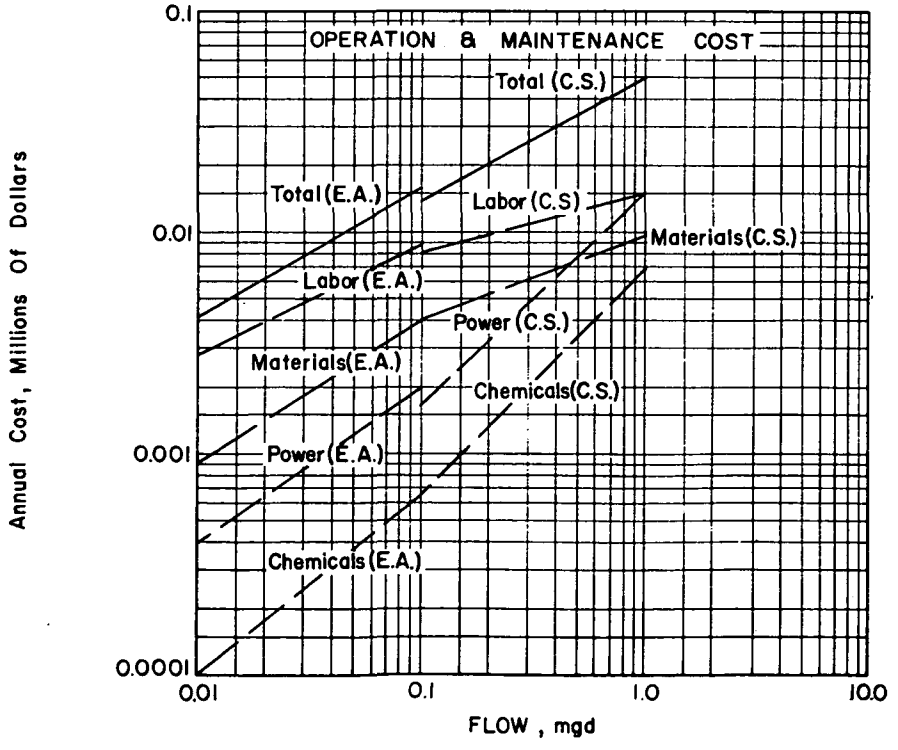


FIGURE H-15

TREATMENT SYSTEM 14
OXIDATION DITCH (DESIGNED FOR NITRIFICATION)

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:

   a. Comminutor
   b. Aeration basin
   c. Clarifier
   d. Chlorine contact chamber
   e. Chlorine feed facility
   f. Building
   g. Fencing

2. Detention time: 24 hours

3. Process Performance:

Wastewater Characteristics:	Influent	Effluent	
		w/N Removal	w/o N Removal
BOD_5, mg/l	210	15	15
COD, mg/l	400	40	40
TSS, mg/l	230	15	15
Total-P, mg/l	11	6	6
NH_3-N, mg/l	20	1	1
NO_3-N, mg/l	0	2	18

Note: O & M costs are w/o nitrogen removal. Add 5 - 10% annual O & M costs to those shown w/ nitrogen removal.

Reference: 47

FIGURE H-15 TREATMENT SYSTEM 14-
OXIDATION DITCH (DESIGNED FOR NITRIFICATION)

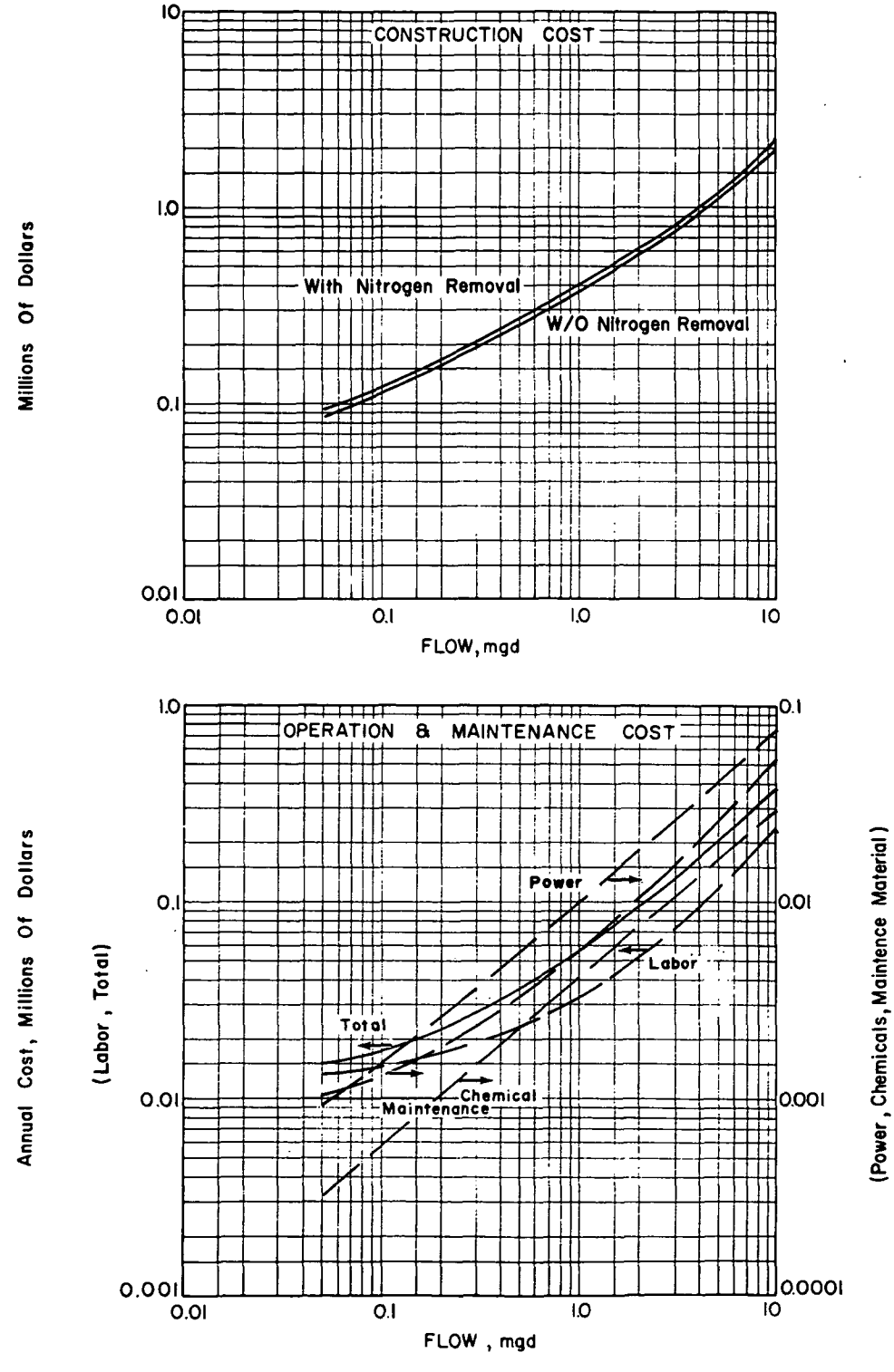


FIGURE H-16

TREATMENT SYSTEM 15
LAND APPLICATION OF WASTEWATER (NON-UNDERDRAINED)

Service Life: 30 years (equipment only)
(Land assumed to have infinite service life)

Relative Reliability: Level B

Design Basis:

1. Costs are for spray irrigation of wastewater.

2. Construction costs include:

   a. Storage - 60 days' retention (based on average flow of wastewater), earthen construction, asphalt-lined with riprap.

   b. Land - to include storage area; land required for 2"/wk application rate; 200 ft buffer zone.

   c. Field preparation - to include brush and tree removal and site leveling at 500 cu yd/acre.

   d. Distribution piping.

   e. Distribution pumping.

   f. Service roads and fencing.

   g. Monitoring wells.

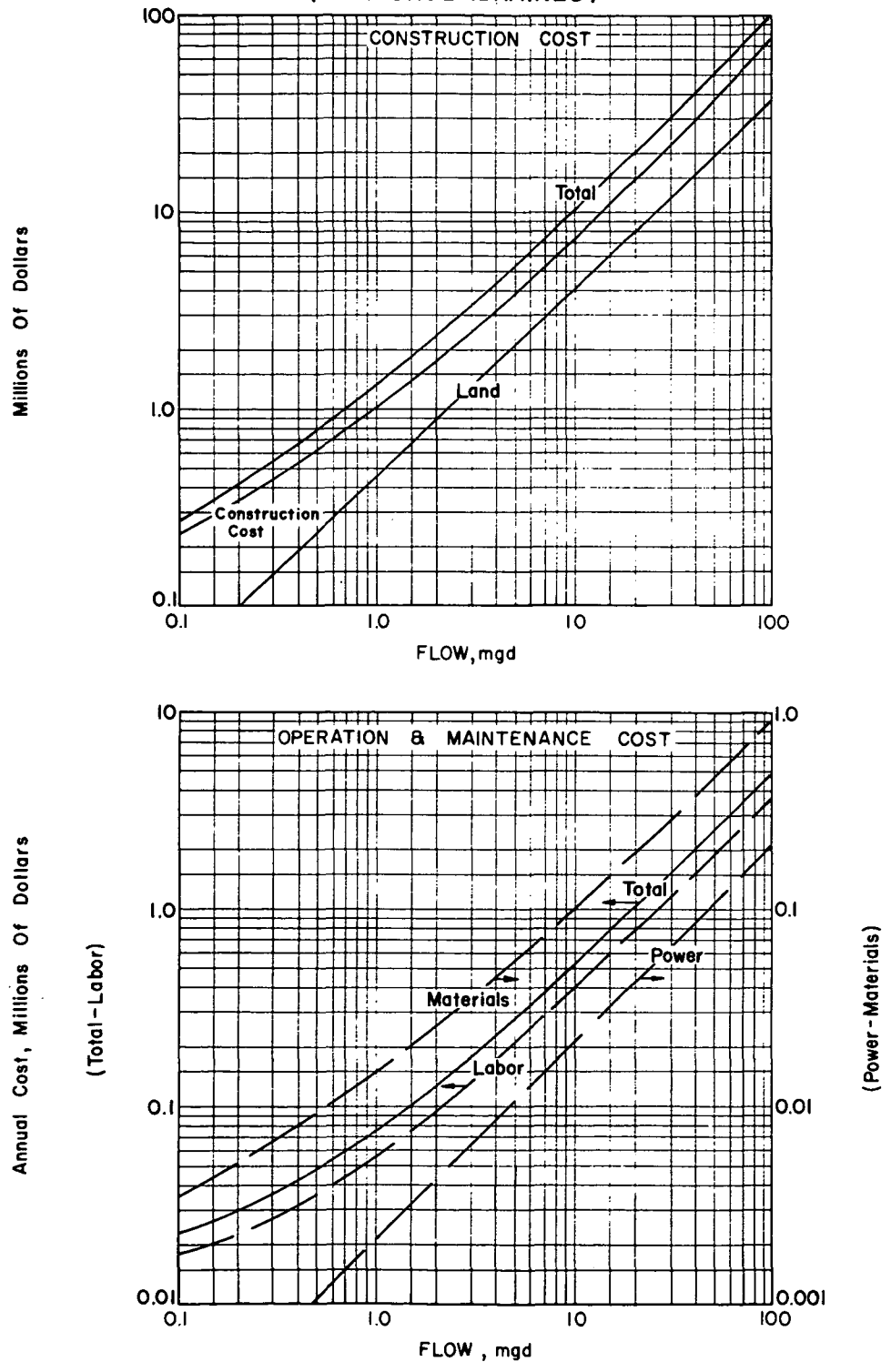
Adjustment Factor:

To adjust for application rate other than above, enter curve at effective flow (Q_E):

$$Q_E = Q_{DESIGN} \times \frac{2'' \text{ Week}}{\text{New Application Rate}}$$

Reference: 27

FIGURE H-16 TREATMENT SYSTEM 15-
LAND APPLICATION OF WASTEWATER
(NON-UNDERDRAINED)



H.7 Wet-Weather Treatment Processes Cost Curves:

Figures H-17 through H-27

FIGURE H-17

CONCRETE STORMWATER STORAGE RESERVOIR

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

1. Construction costs based on actual projects and on comparison with sludge aeration basins exclusive of aeration equipment ranging from 400,000 to 5,000,000 gallons.

2. Operation and maintenance costs include automatic washdown of basins after storms.

Reference: 36

FIGURE H-17[1] CONCRETE STORMWATER STORAGE RESERVOIR (WITHOUT COVER)

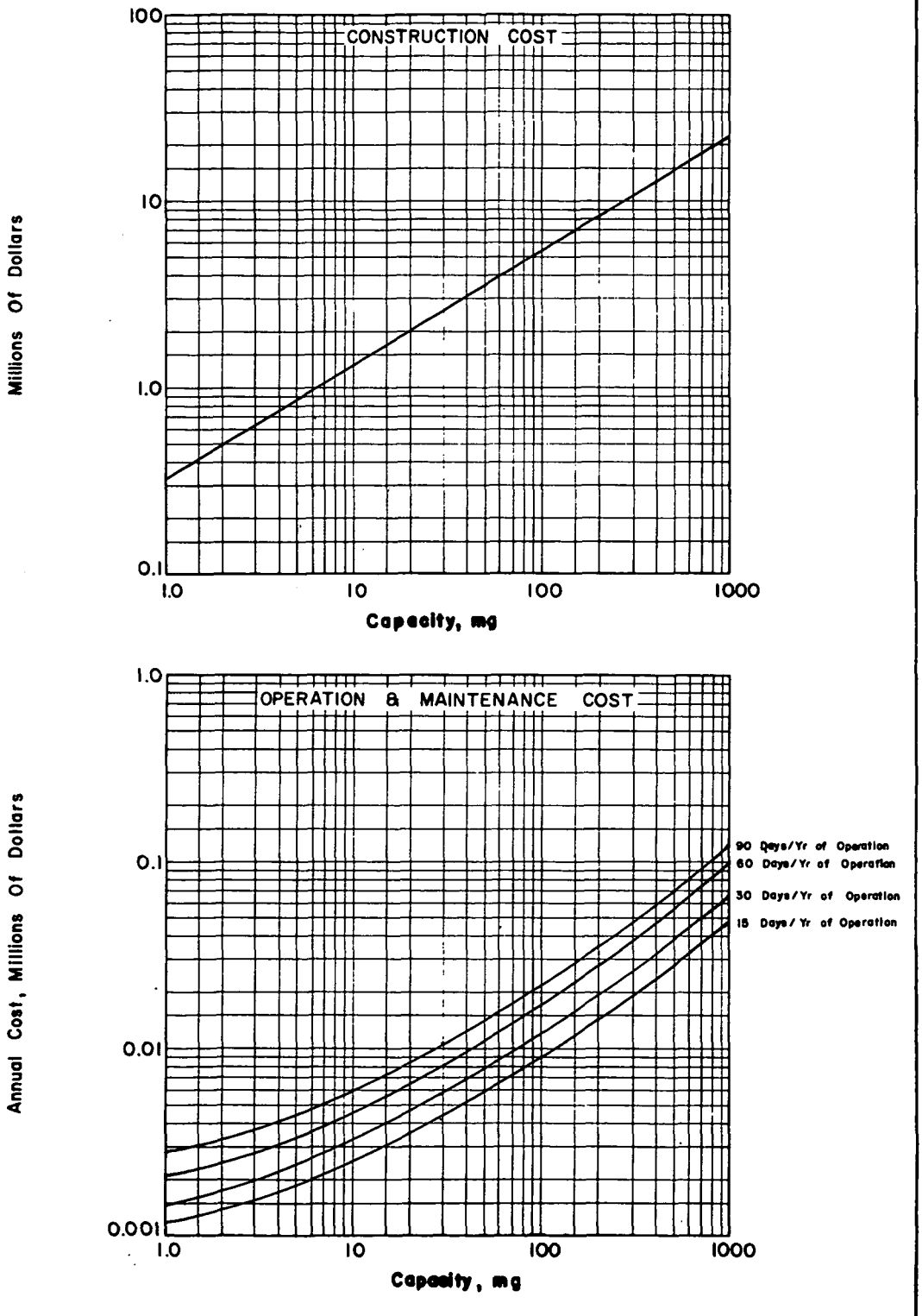


FIGURE H-17

CONCRETE STORMWATER STORAGE RESERVOIR

Service Life: 50 years

Relative Reliability: Level B

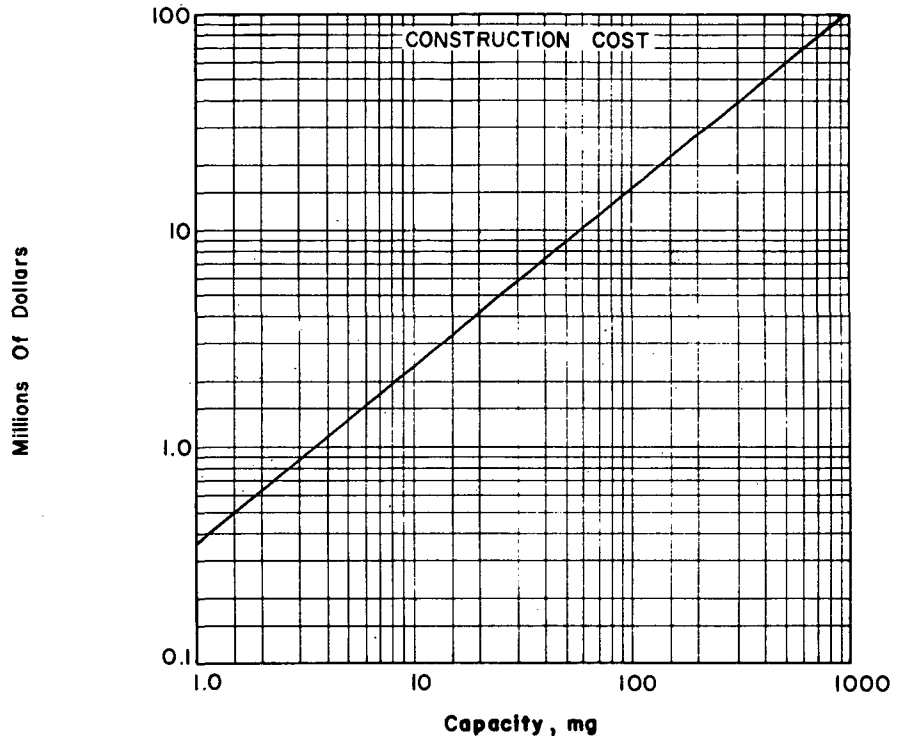
Design Basis:

1. Construction costs based on actual projects and on comparison with sludge aeration basins exclusive of aeration equipment ranging from 400,000 to 5,000,000 gallons.

2. Operation and maintenance costs include automatic washdown of basins after storms.

Reference: 36

FIGURE H-17[2] CONCRETE STORMWATER STORAGE RESERVOIR (WITH COVER)



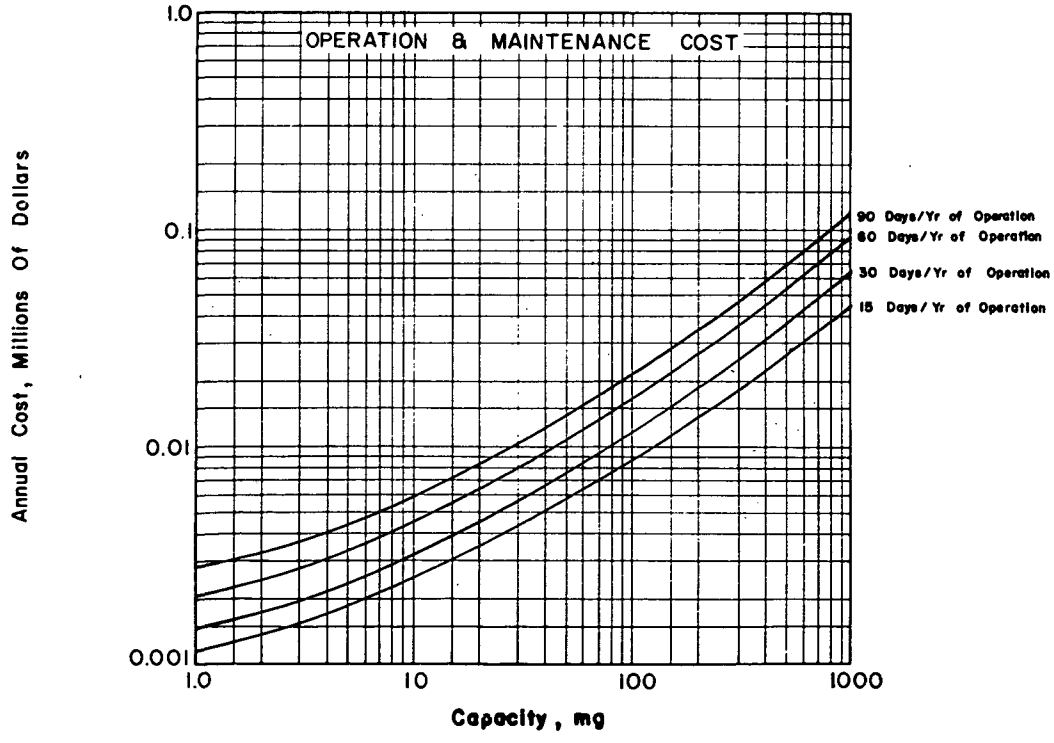


FIGURE H-18

EARTHEN STORAGE BASIN

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include liner, paving and fencing, on-site soil supply, and no groundwater problems or rock excavation.

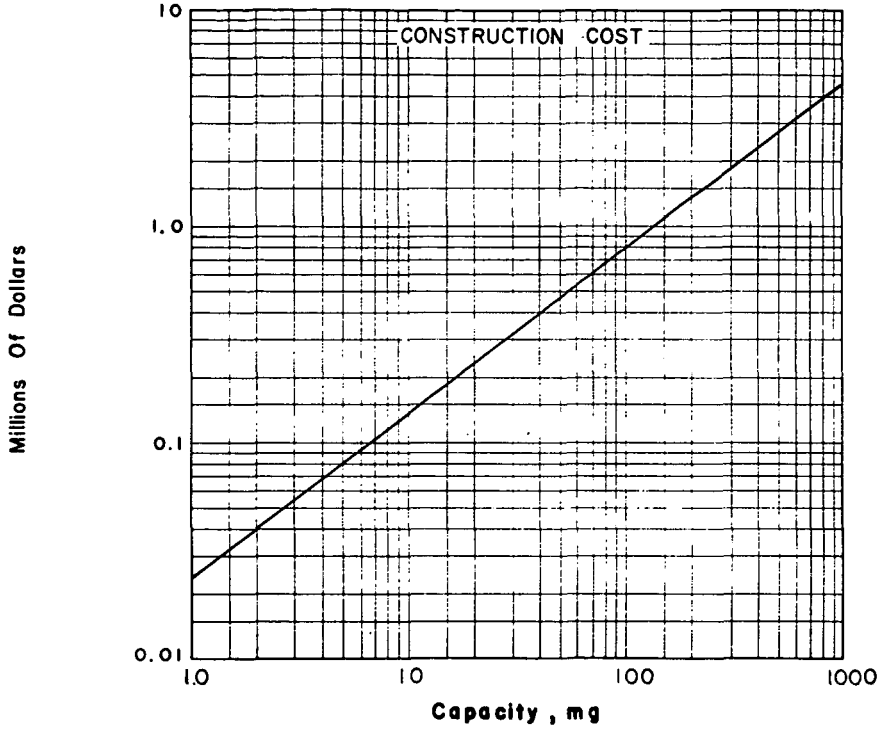
2. Interior slope is 2.5 to 1.
   Exterior slope is 3 to 1.
   Compaction loss = 20%.
   Levee top width = 16 feet.
   Reservoir is 18 feet deep.
   Length/width ratio is 2.

3. Manual cleanout twice a year based on 0.004 manhours/sq ft of basin water surface area.

4. No electrical costs.

Reference: 36

FIGURE H-18 EARTHEN STORAGE BASIN



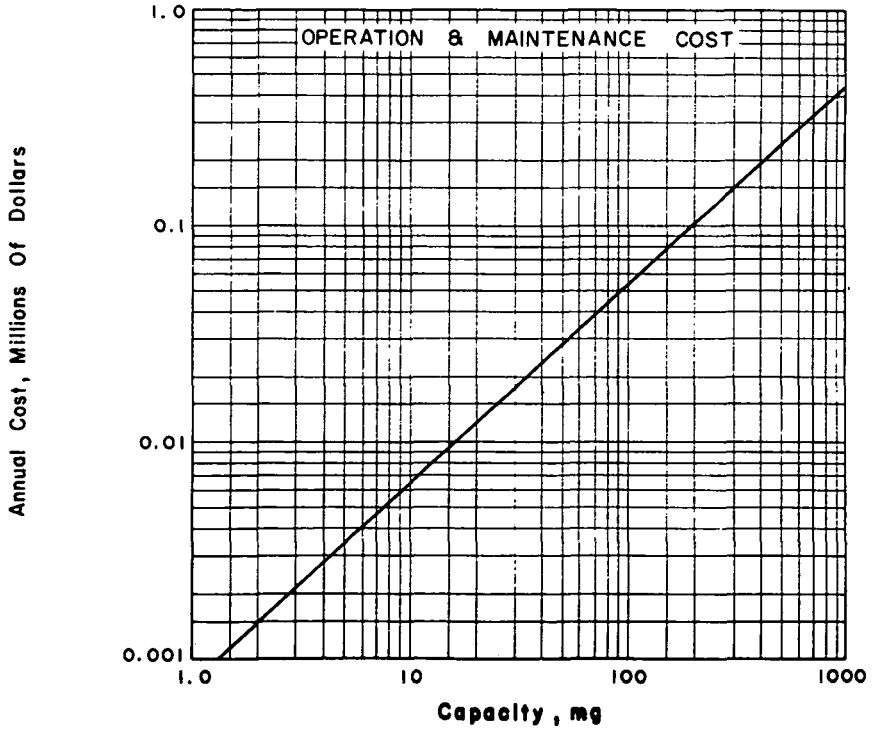


FIGURE H-19

STATIONARY SCREEN

Service Life: 15 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include collection flumes for sludge and screened effluent, and housing for screens.
2. Screens are wedgewire (stainless steel).
3. Costs based on screens rated at 4 mgd each. Total capacities range from 12 mgd (3 screens) to 192 mgd (48 screens).
4. Costs are related to hydraulic capacity.
5. Screen assembly requires 4 to 8 feet of head loss.
6. Costs include influent headers and metal weirs to distribute flow to each unit.
7. Labor costs include 24 hours/year for routine checks. Also for each overflow event: 1 hour startup/shutdown time, 1 hour wash time, 1 hour travel time, and 2 hours twice a day for operation.

Process Performance: (Swirl Concentrator Followed By Stationary Screen)

	Typical Pollutant Removals, %
BOD_5	0 - 25
TSS	0 - 25

References: 36, 51

FIGURE H-19 STATIONARY SCREEN

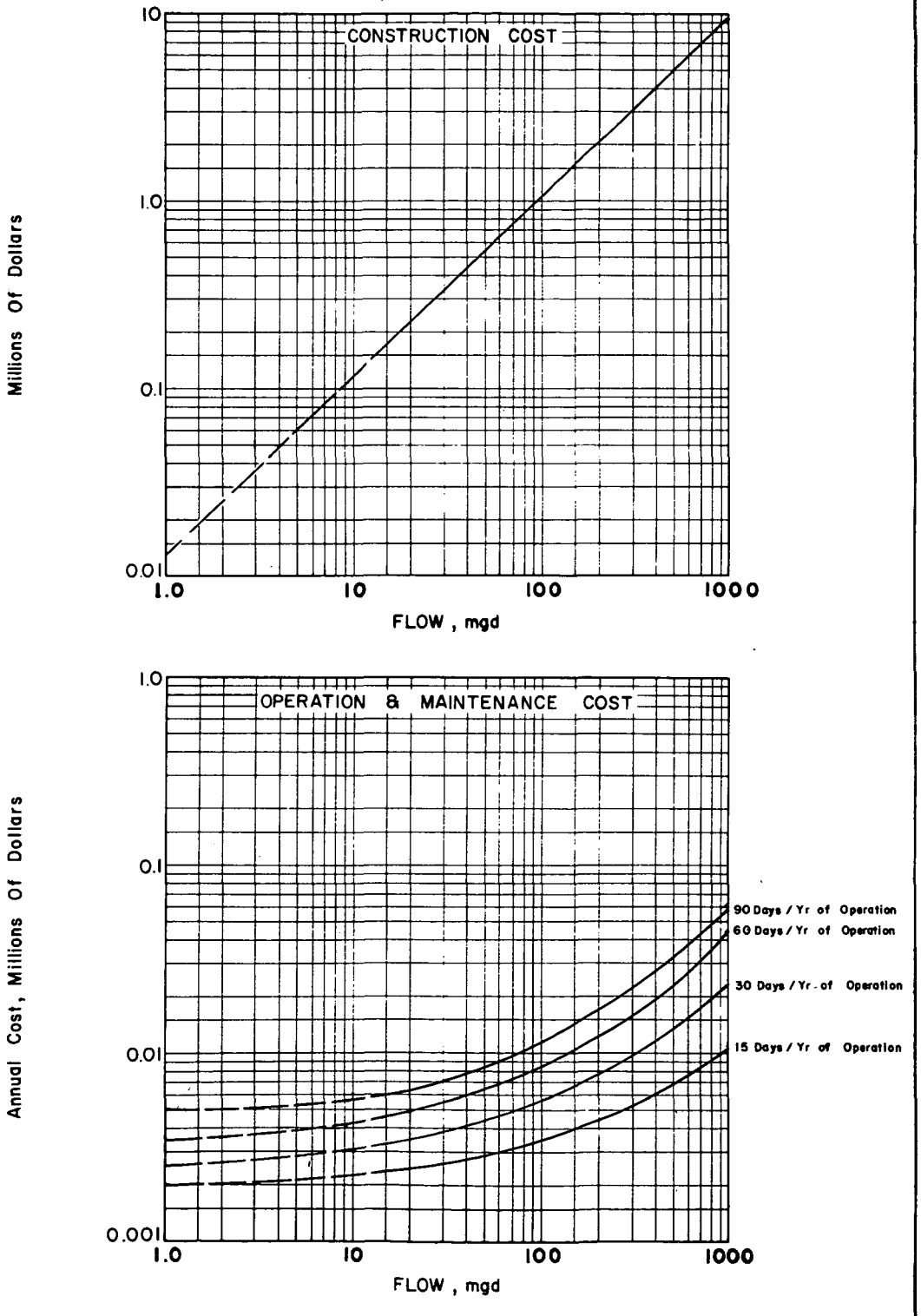


FIGURE H-20

HORIZONTAL-SHAFT ROTARY SCREEN

Service Life: 15 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include housing for gallery and screen chambers.
2. Loading rate = 14 sq ft screen/mgd.
3. Screen submergence varies from 74% to 83%.
4. Screen opening (Microns): Pretreatment 150-420; complete treatment 23-35.
5. Screen material is stainless steel.
6. Drum speed = 0-7 rpm.
7. Labor costs include 48 hours/yr for routine checks and 3 hr/yr/screen for maintenance. Also, for each overflow event: 1 hour setup/shutdown time, 1 hour travel time, 3 hours/screen for cleanup time, and 4 hours/day for operation.

Process Performance:

	Typical Pollutant Removals, %
BOD_5	40 - 60
SS	50 - 70

References: 36, 42, 51

FIGURE H-20 HORIZONTAL - SHAFT ROTARY SCREEN

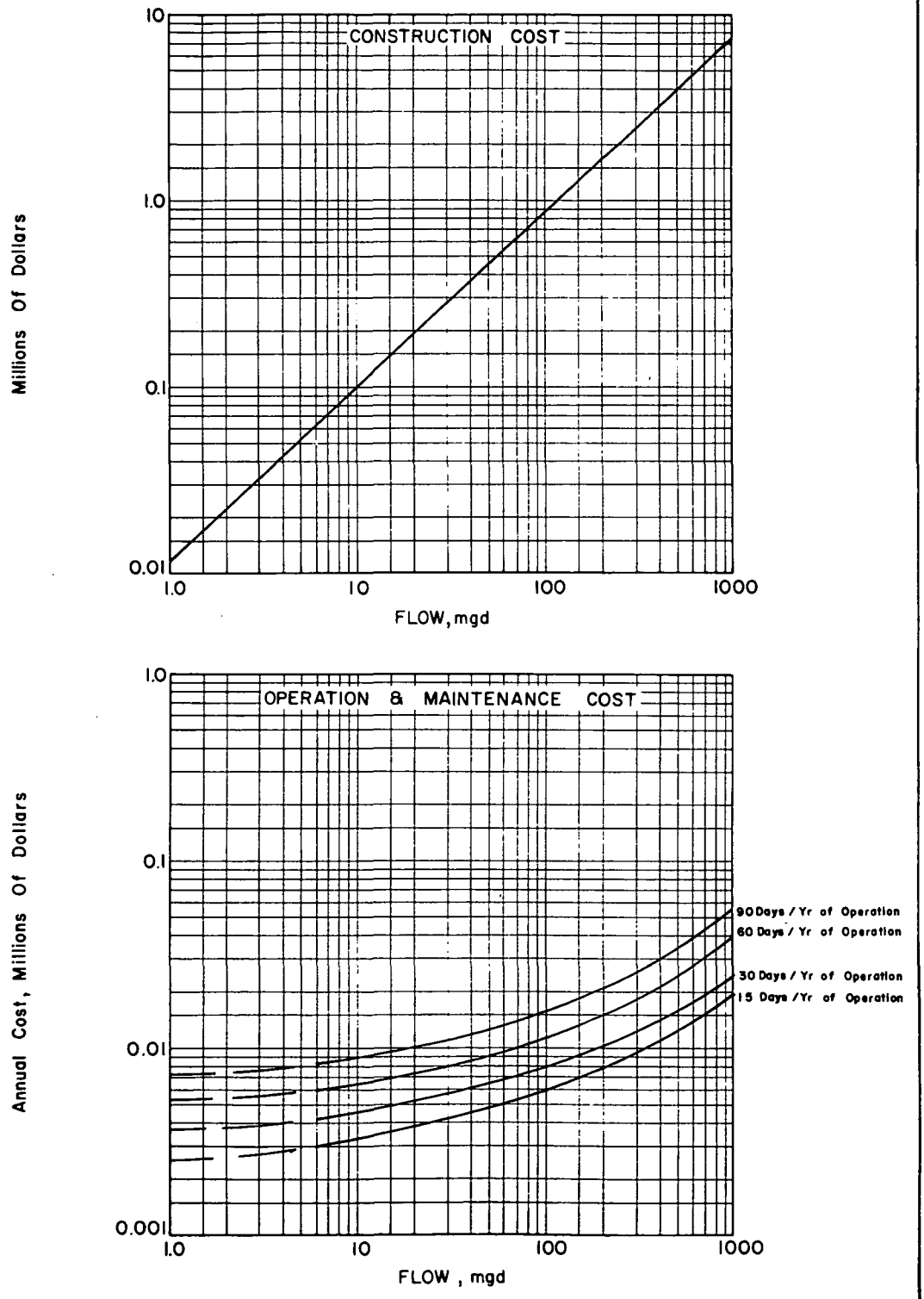


FIGURE H-21

VERTICAL-SHAFT ROTARY SCREEN

Service Life: 15 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include housing for equipment.
2. Cost data are developed for multiple use of 3-mgd, 60 inch screens.
3. Screen openings range from 74 to 230 microns.
4. Screen material is stainless steel.
5. Drum speed is 30-65 rpm.
6. Labor costs include 48 hours/yr for routine checks and 6 hours/yr/screen for maintenance. Also, for each overflow event: 1 hour/screen shutdown time for plant cleanup and 4 hours/day for operation.

Process Performance:

	Typical Pollutant Removals, %
BOD_5	( )
TSS	( )

References: 36, 42, 51

FIGURE H-21 VERTICAL-SHAFT ROTARY SCREEN

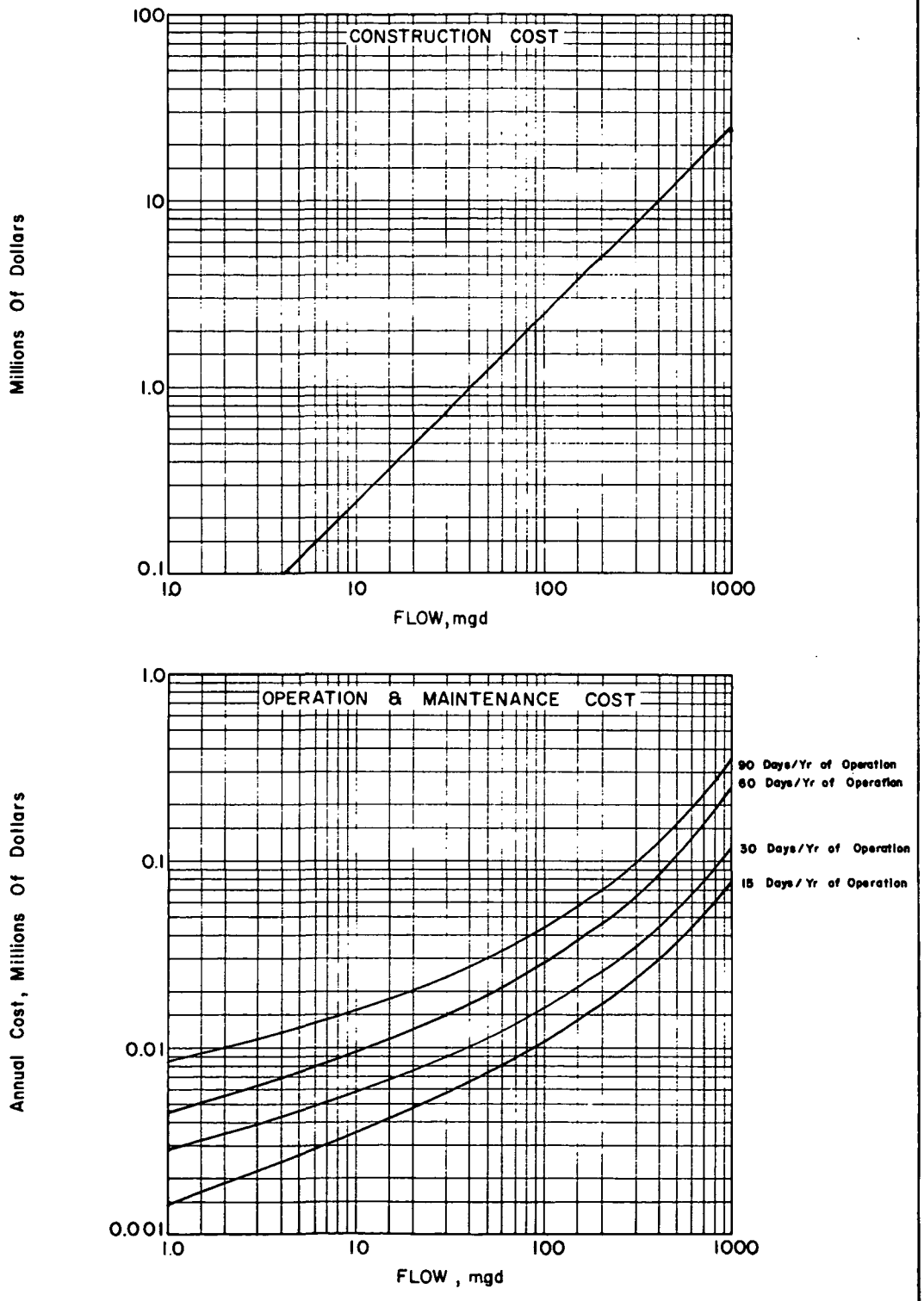


FIGURE H-22

CHEMICAL COAGULATION
(FERRIC CHLORIDE AND POLYMER)

Service Life: 35 years

Relative Reliability: Level B

Design Basis:

1. Velocity gradient, G = 20 to 70 sec^{-1} for flocculation, G = 300 sec^{-1} for rapid mix.

2. Detention time is 20 minutes for flocculation, 1 minute for rapid mix.

3. Reinforced concrete basins.

4. Turbine type mixers.

5. For ferric chloride:

   a. Pump costs based on two 25 gallon/hour hydraulic pumps to four 50 gallon/min pumps.

   b. Storage tanks range from two 3,000 gallon fiberglass reinforced plastic (FRP) tanks to ten 50,000 gallon underground concrete tanks with rubber linings.

   c. Ferric chloride dosage = 100 mg/l.

   For polymer:

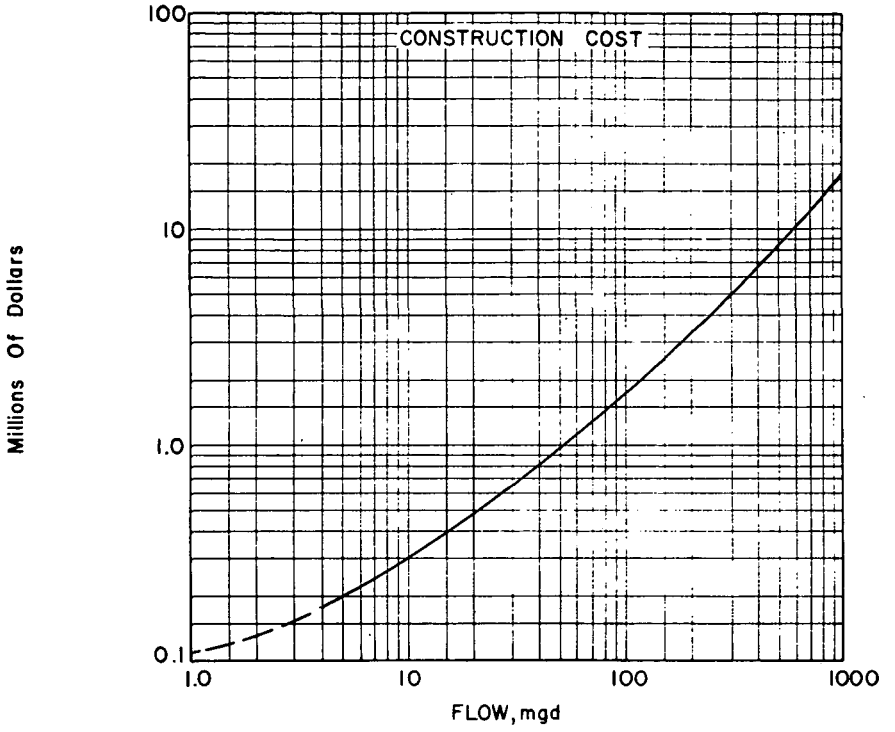
   a. Feed costs range from manual feed to four volumetric dry feeders, four mixing tanks and two holding tanks.

   b. Polymer dosage = 0.5 mg/l.

6. Operation and maintenance costs include 0.004 manhours/sq ft/storm overflow event and 0.7 hp/mgd.

Reference: 36

FIGURE H-22[1] CHEMICAL COAGULATION
(FERRIC CHLORIDE AND POLYMER)



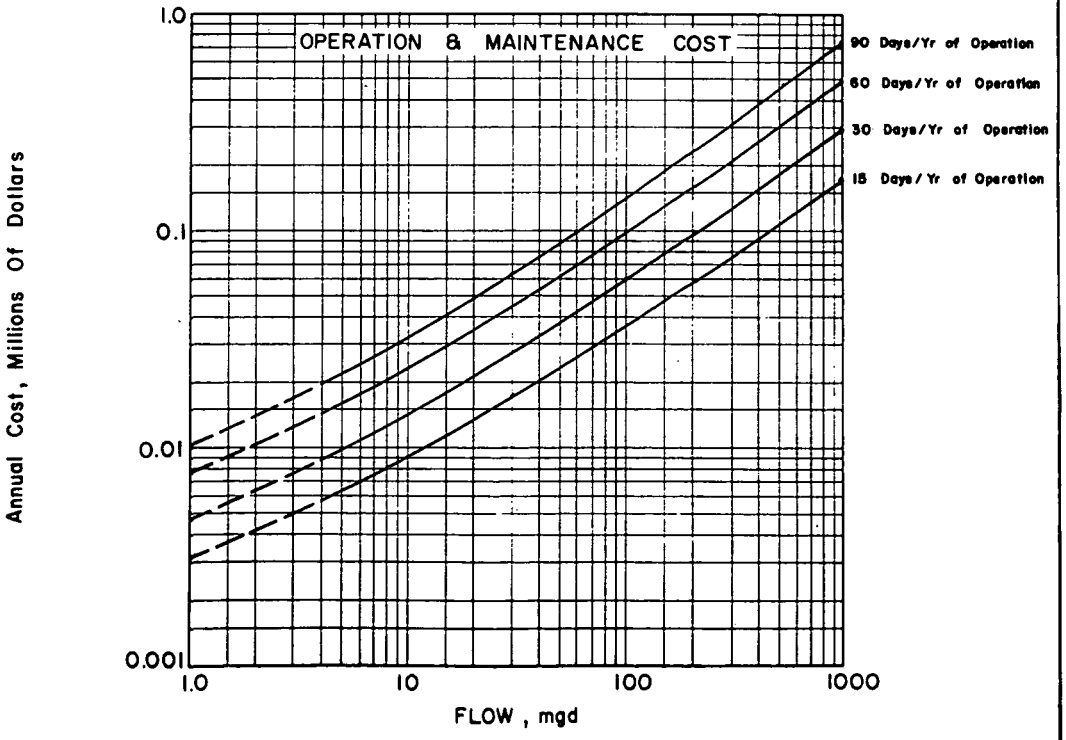


FIGURE H-22

CHEMICAL COAGULATION
(FERRIC CHLORIDE AND POLYMER)

Service Life: 35 years

Relative Reliability: Level B

Design Basis:

1. Velocity gradient, G = 20 to 70 sec^{-1} for flocculation, G = 300 sec^{-1} for rapid mix.

2. Detention time is 20 minutes for flocculation, 1 minute for rapid mix.

3. Reinforced concrete basins.

4. Turbine type mixers.

5. For ferric chloride:

   a. Pump costs based on two 25 gallon/hour hydraulic pumps to four 50 gallon/min pumps.

   b. Storage tanks range from two 3,000 gallon fiberglass reinforced plastic (FRP) tanks to ten 50,000 gallon underground concrete tanks with rubber linings.

   c. Ferric chloride dosage = 100 mg/l.

   For polymer:

   a. Feed costs range from manual feed to four volumetric dry feeders, four mixing tanks and two holding tanks.

   b. Polymer dosage = 0.5 mg/l.

6. Operation and maintenance costs include 0.004 manhours/sq ft/storm overflow event and 0.7 hp/mgd.

Reference: 36

FIGURE H-22[2] CHEMICAL COAGULATION (POLYMER)

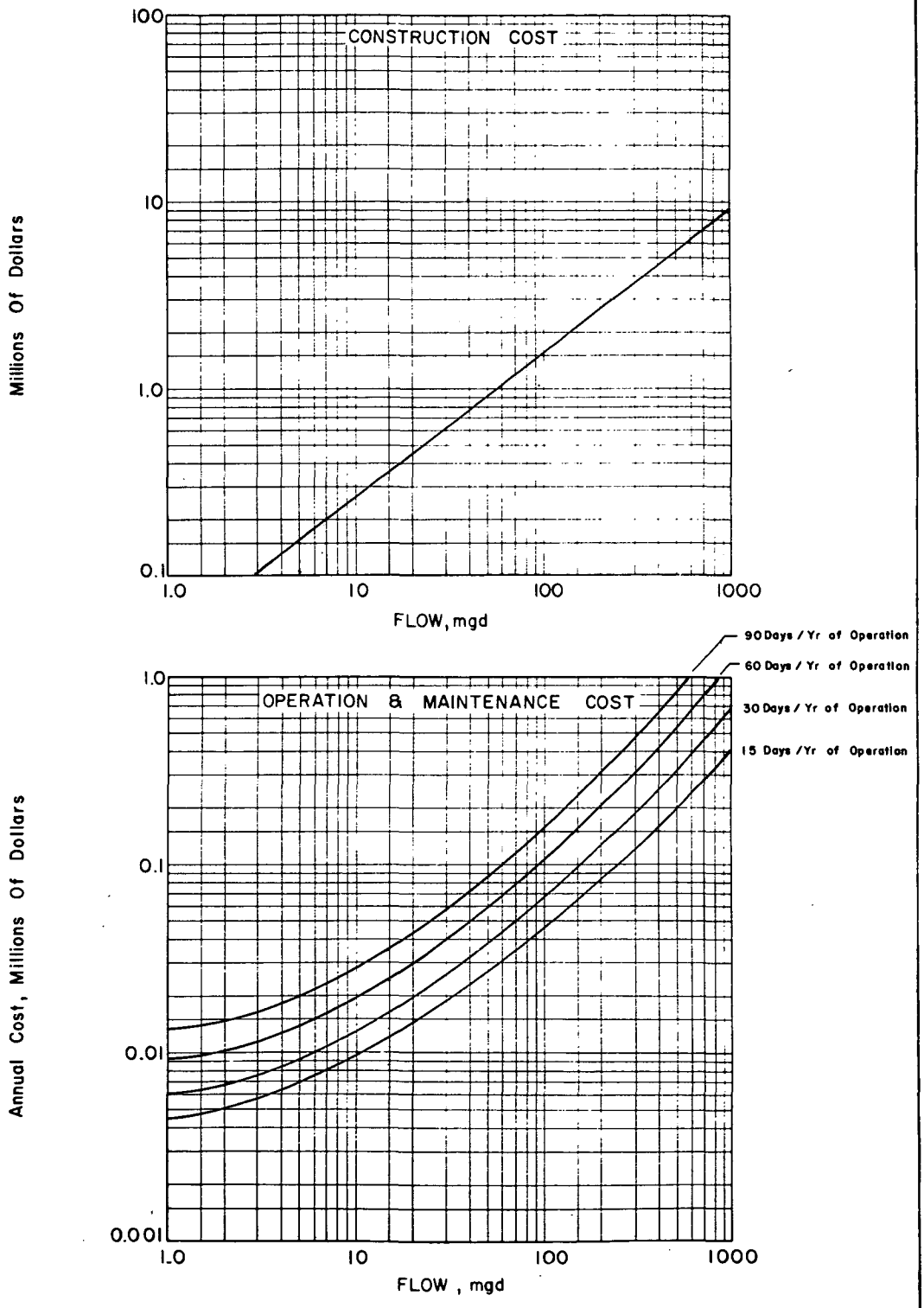


FIGURE H-23

STORMWATER SEDIMENTATION

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Cost is based on recommended design flow of 3,000 gpd/sq ft of surface area.

2. Basins have side water depth of 12 feet and freeboard of 1.5 feet.

3. Costs apply to circular sludge collection equipment in circular basins and large rectangular basins (20,000 sq ft). Costs do not apply to straight-line sludge collection equipment.

4. Costs do not include sludge removal or disposal.

5. Construction cost includes steel troughs and weirs.

Process Performance:

	Typical Pollutant Removals, %
BOD_5	25 - 40
SS	25 - 50

References: 36, 51

FIGURE H-23 STORMWATER SEDIMENTATION

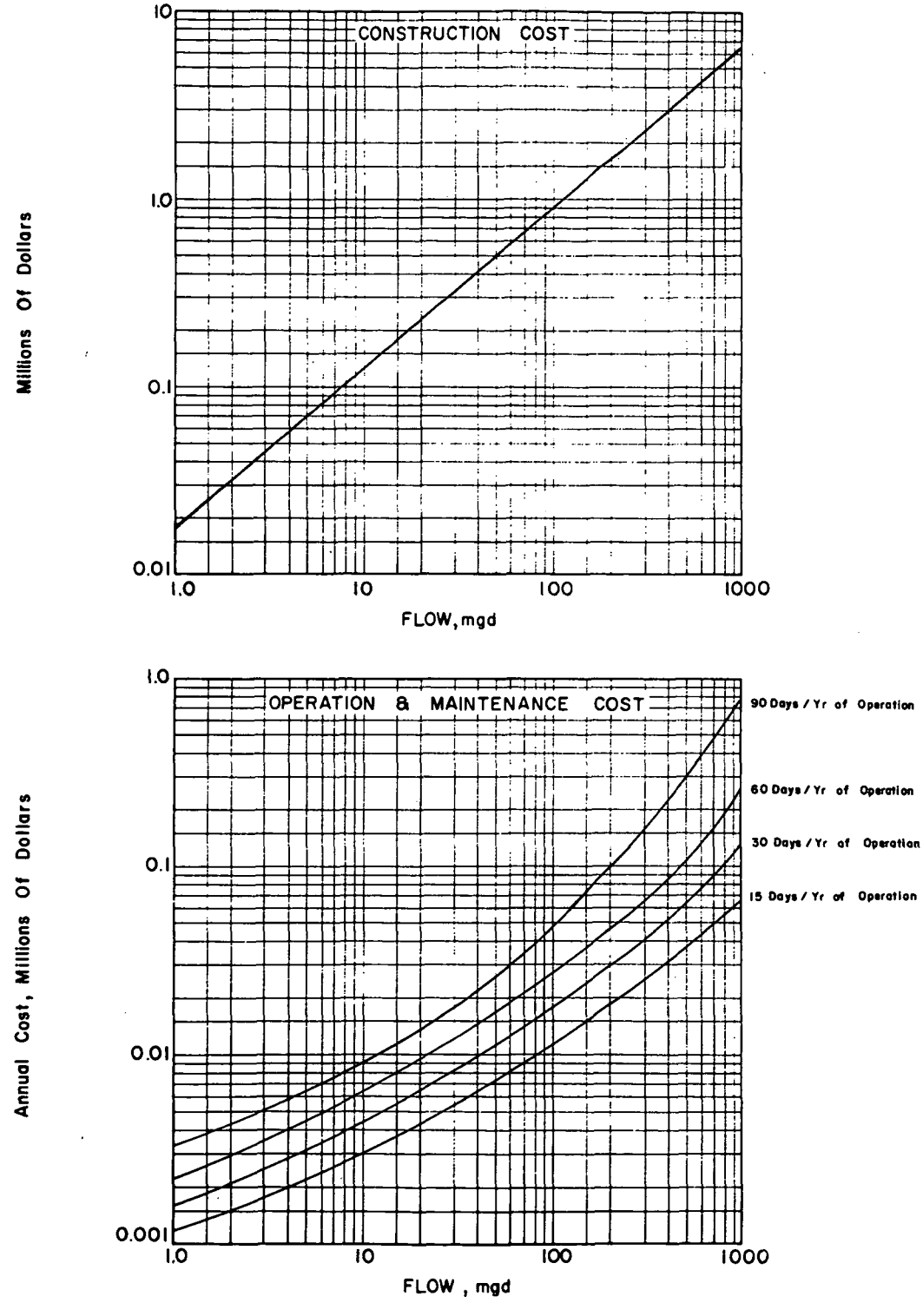


FIGURE H-24

DISSOLVED AIR FLOTATION

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Costs are based on an overflow rate of 4,440 gpd/sq ft of surface area.

2. Construction cost includes the basic unit, flow splitting device, and an enclosed piping and equipment gallery for drive equipment, pumps, air tanks and miscellaneous equipment.

3. Largest practical size for an individual unit is 20 ft X 100 ft.

4. Power cost is based on 0.10 kw hr/sq ft.

5. No chemical costs are included.

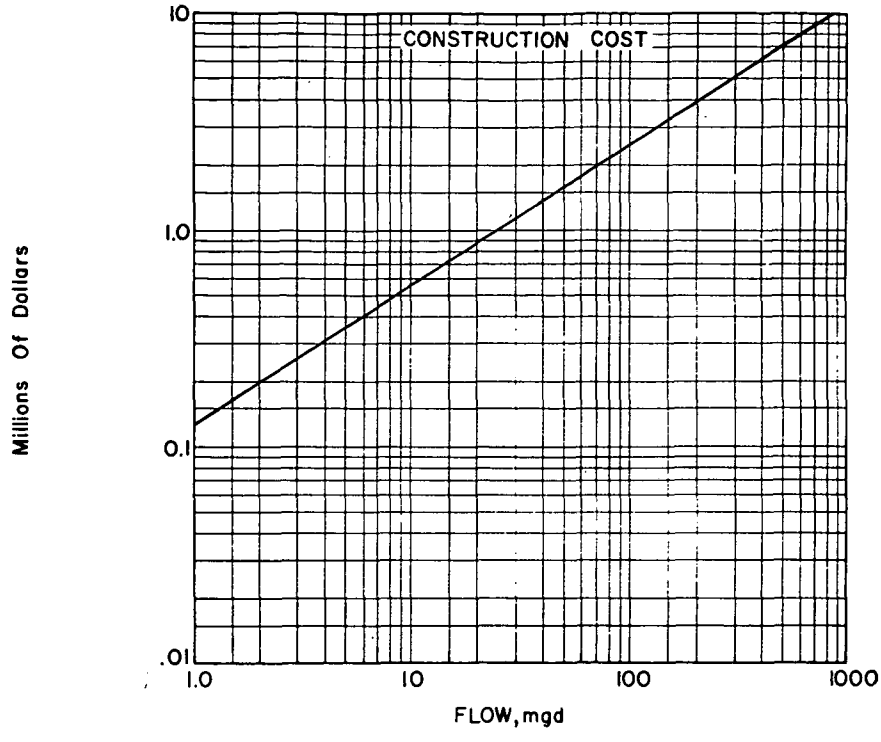
6. Operations and maintenance costs include 48 hours/yr for routine checks, 0.009 hours/sq ft for maintenance. Also, for each overflow event: 1 hour travel time, 1 hour setup/shutdown time, 0.004 hours/sq ft for washdown, and 4 hours/day operation.

Process Performance: (With Chemicals)

	Typical Pollutant Removals, %
BOD_5	40 - 60
SS	50 - 70

References: 36, 37, 51

FIGURE H-24 DISSOLVED AIR FLOTATION



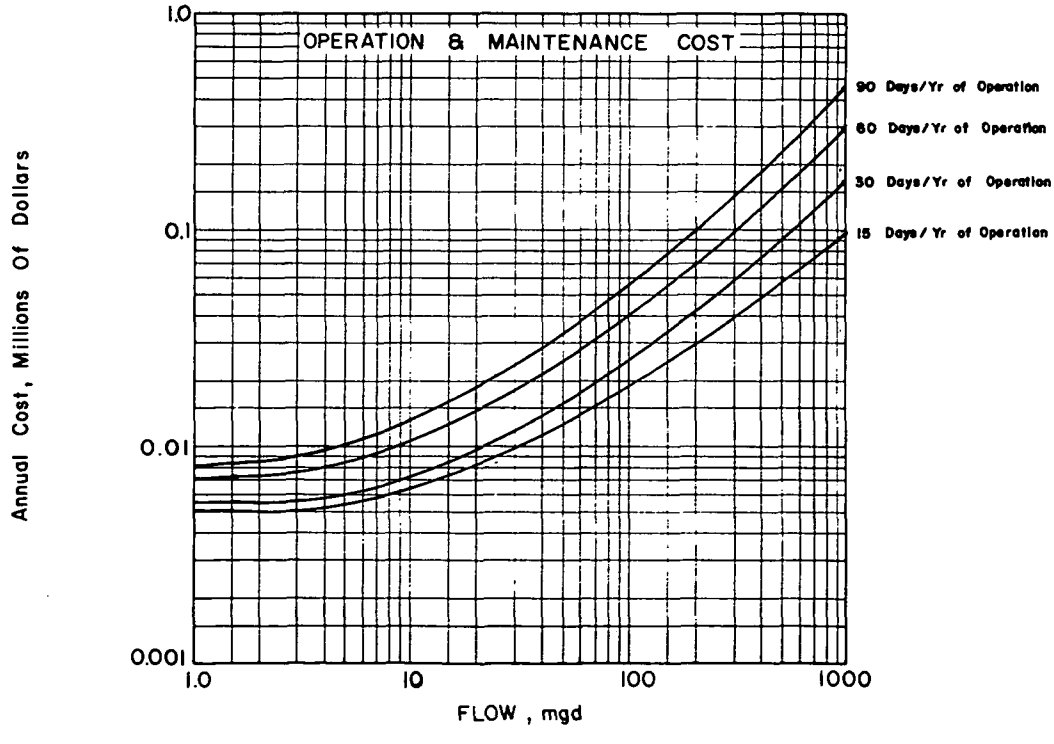


FIGURE H-25

SWIRL REGULATOR/CONCENTRATOR

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include basic chamber. Costs do not include buildings, pumping stations, flow measurement, or basin dewatering facilities.

2. Chamber diameters range from 12 ft to 48 ft.

3. Labor costs include 52 hours/yr for routine inspections, plus process cleanup after storm overflow events.

4. Process cleanup based on following schedule/storm event:

Flow mgd	Manhours/Storm
5	12
15	14.4
30	15.9
50	20.4
80	22.9
150	25.4
250	27.9

Process Performance: (Swirl Concentrator Followed By Stationary Screen)

	Typical Pollutant Removals, %
BOD_5	0 - 25
SS	0 - 25

References: 36, 37, 51

FIGURE H-25 SWIRL REGULATOR / CONCENTRATOR

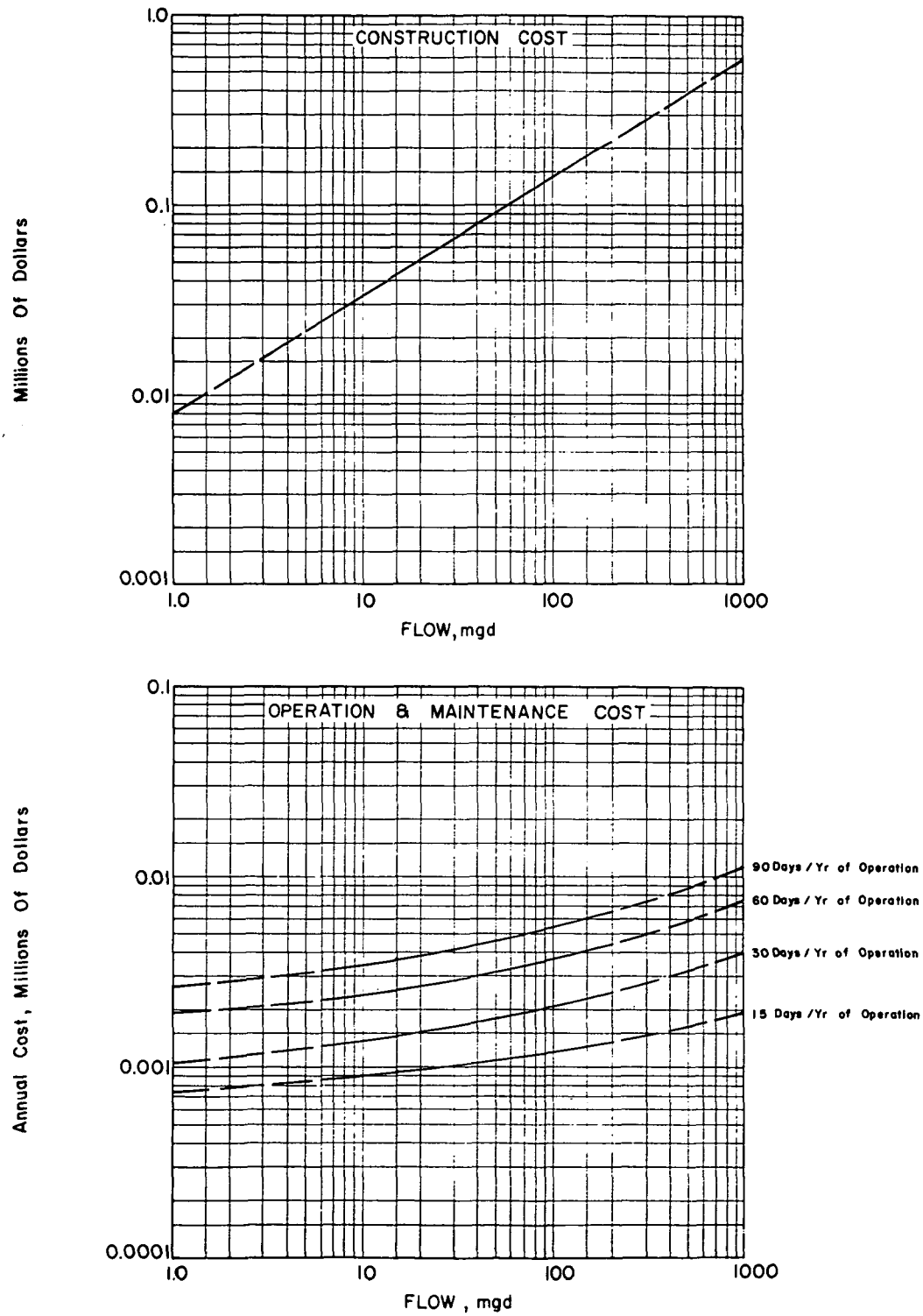


FIGURE H-26

HIGH-INTENSITY MIXING/CHLORINE CONTACT BASIN

Service Life: 50 years

Design Basis:

1. Costs are based on 2-minute detention time and on velocity gradient (G) of 300 sec^{-1}.

2. Construction costs include concrete mixing basin and stainless steel mixers.

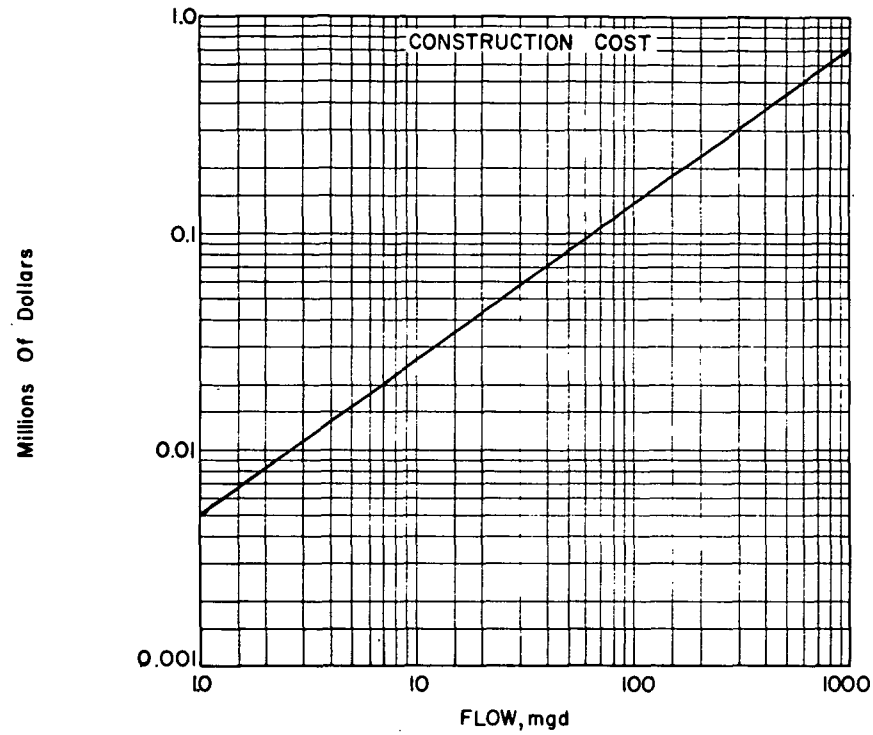
3. Miscellaneous costs such as chlorine storage area, hoist, and evaporator are not included.

4. Labor costs include 8 hours/yr/unit for routine maintenance. Also, for each overflow event: 8 hours for cleaning and 1 hour/day operation.

5. Power requirement is 1.25 hp/mgd.

References: 36, 37

FIGURE H-26 HIGH-INTENSITY MIXING / CHLORINE CONTACT BASIN



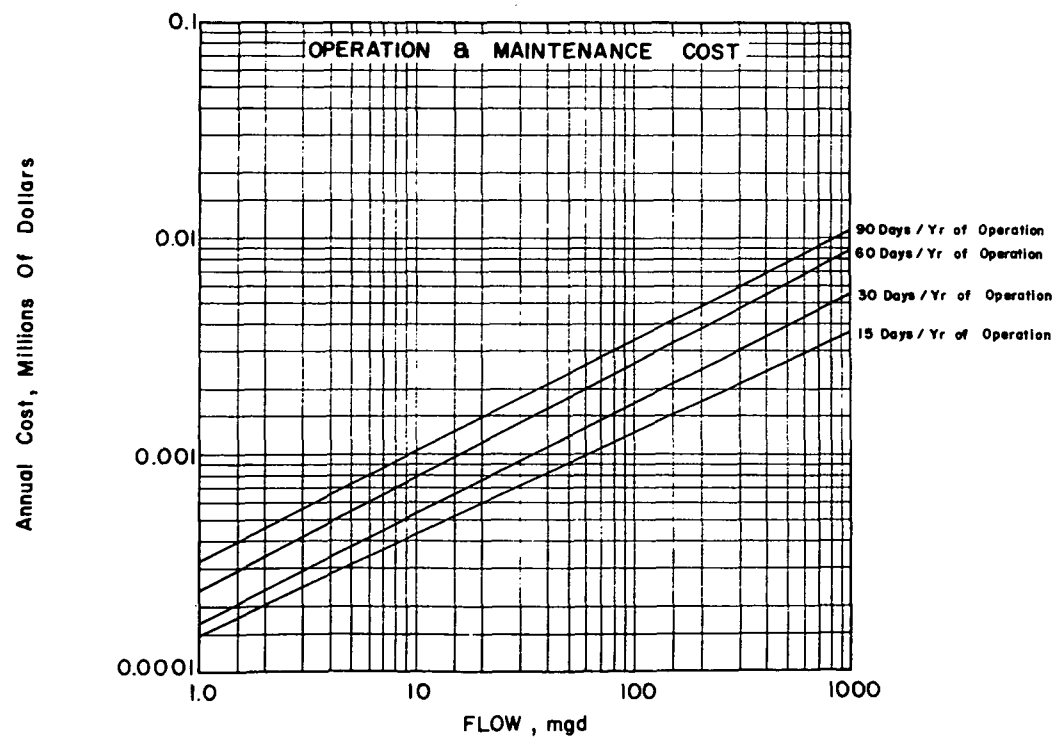


FIGURE H-27

HIGH-RATE FILTRATION (GRAVITY)

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

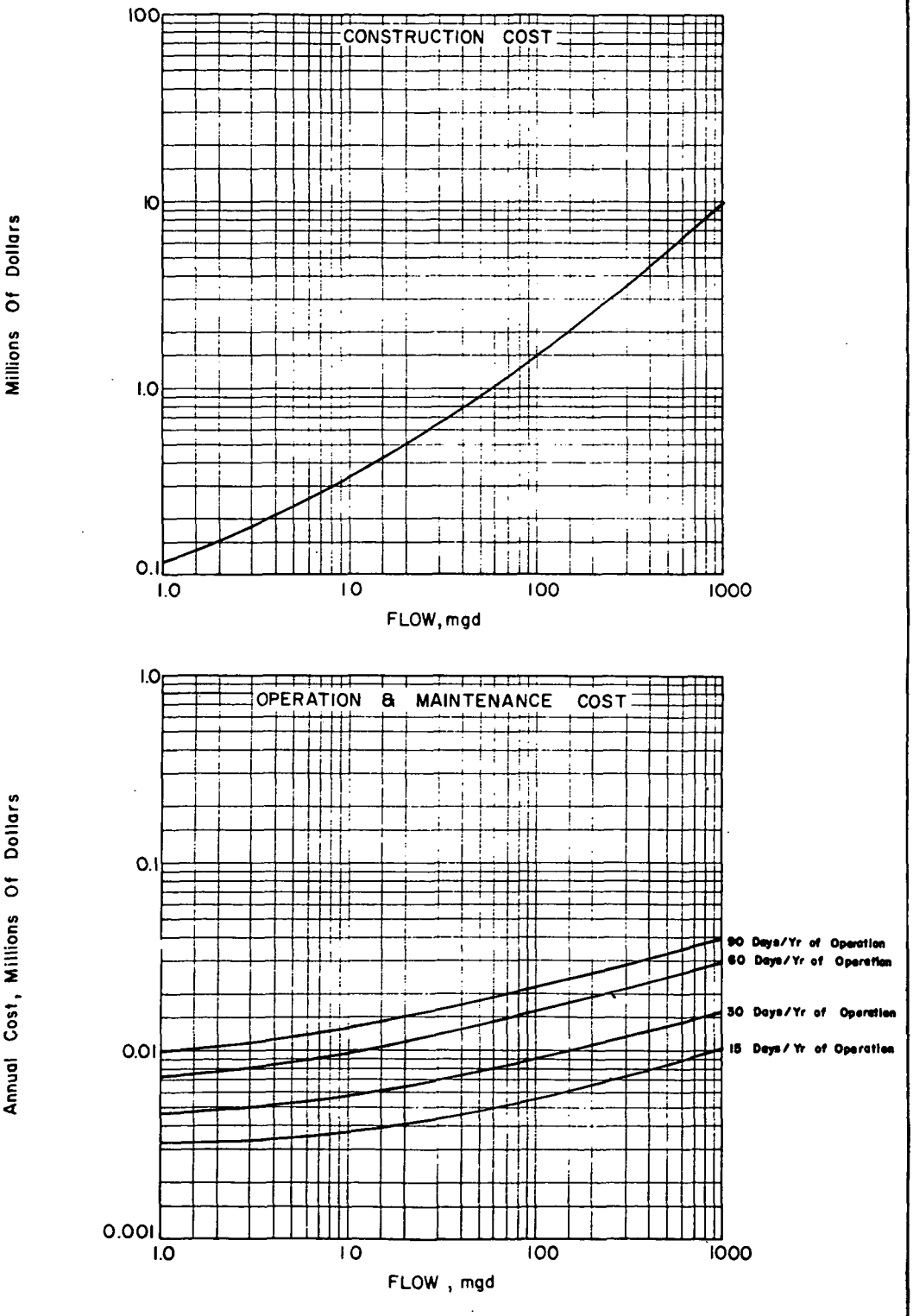
1. Construction cost includes the filters, dual non-proprietary filter media, and housing for filters and filter galleries.
2. Application rate is 16 gpm/sq ft.
3. Maximum headloss is 12 feet.
4. Pumping costs other than backwash and surface wash are not included.
5. Labor costs include 48 hours/yr for routine checks and 12 hours/yr/filter for maintenance. Also, for each overflow event: 1 hour startup and shutdown, 1 hour/filter cleanup, and 4 hours/day for operation.
6. No chemical costs are included.

Process Performance: (When Used in Combination With Chemical Addition, Flocculation/Sedimentation)

	Typical Pollutant Removals, %
BOD_5	60 - 80
SS	80 - 95

References: 36, 37, 51

FIGURE H-27 HIGH-RATE FILTRATION (GRAVITY)



H.8 Municipal Wastewater Treatment Processes

H.8.1 Support Personnel:
Cost Curves, Figure H-28

H.8.2 Miscellaneous Structures:
Cost Curves, Figure H-29

FIGURE H-28

SUPPORT PERSONNEL

Relative Reliability: Level B

Design Basis:

1. Costs include the manpower required to support the operation of a wastewater treatment facility. These support functions include supervision and administration, clerical work, laboratory work, and administrative costs.

2. The labor costs are based on 1500 hour man-year. This includes a five-day work week; an average of 29 days for holidays, vacations and sick leave; and 6 1/2 hours of productive work per 8 hour day.

Reference: 45

FIGURE H-28 SUPPORT PERSONNEL

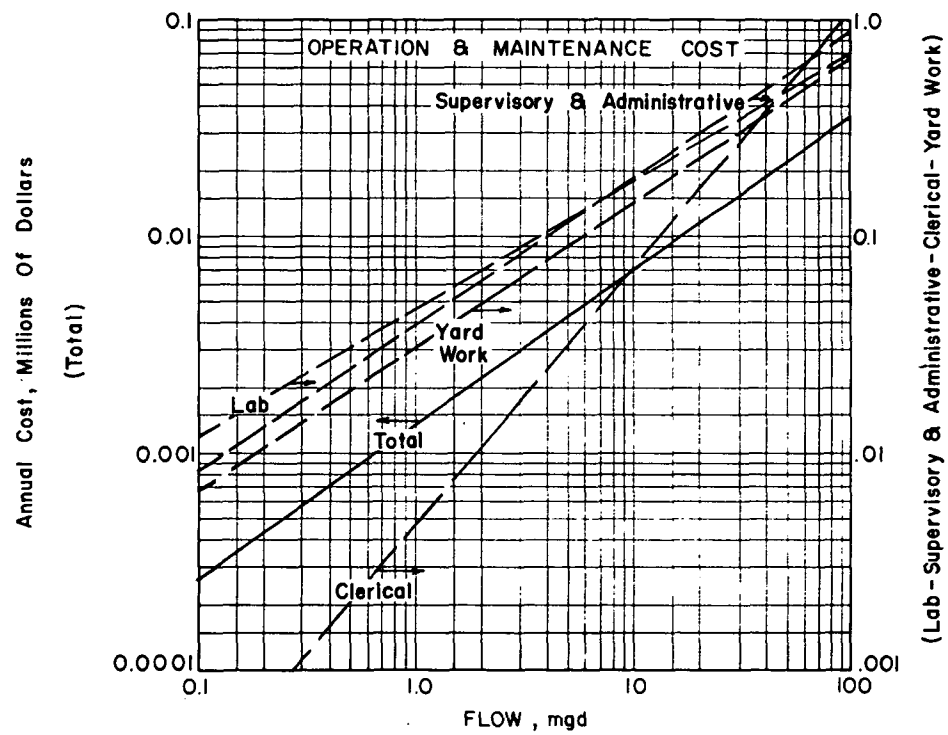


FIGURE H-29

MISCELLANEOUS STRUCTURES

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

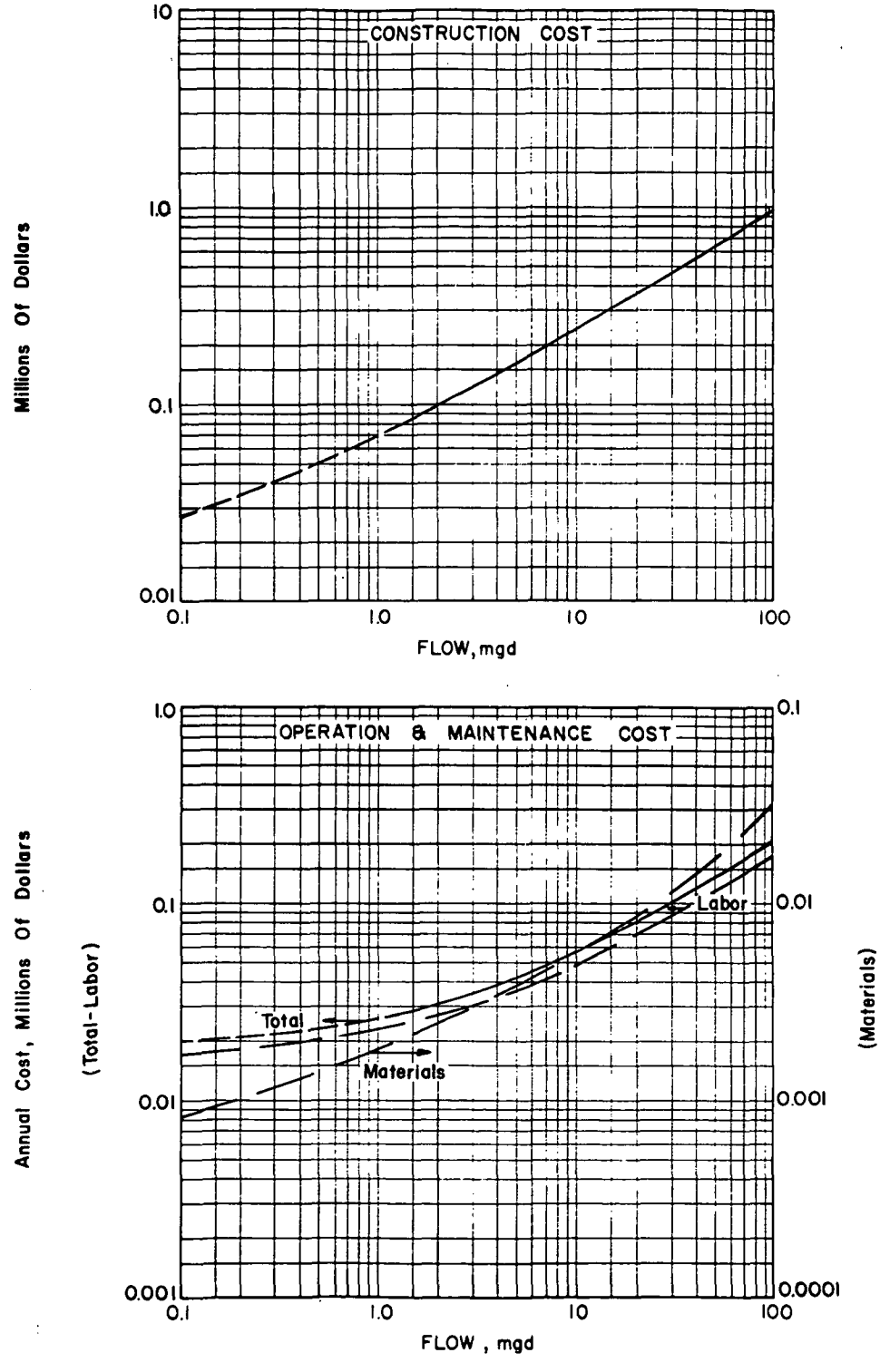
Construction cost includes:

a. Administrative offices.
b. Laboratories.
c. Shop and garage facilities.

Note: Full cost of miscellaneous structures might not be applicable for small-flow systems.

Reference: 3

FIGURE H-29 MISCELLANEOUS STRUCTURES



H.8.3 Wastewater Treatment Processes
Cost Curves:

Figures H-30 through H-62

Raw Wastewater Characteristics:

BOD_5	210 mg/1
COD	400 mg/1
TSS	230 mg/1
NH_3-N	20 mg/1
TKN	30 mg/1
NO_3-N	0 mg/1
UOD	406 mg/1
Total-P	11 mg/1
Alkalinity (as $CaCO_3$)	300 mg/1
pH	7.3

FIGURE H-30

LOW-LIFT PUMP STATION

Service Life: 15 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:

   a. Fully-enclosed wet well/dry well structure.

   b. Pumping equipment capable of meeting peak pumping requirements of 2 Q with largest unit out of service.

   c. Standby pumping facilities.

   d. Piping and valves within structure.

   e. Bar screens - mechanically cleaned.

2. Package pumping stations are used in a flow range of 50 to 1,200 gpm (0.072 to 1.73 mgd).

3. Curves developed for TDH = 10 feet.

Adjustment Factor:

To adjust operating power costs for TDH's other than 10 feet, enter curves at effective flow, Q_E

$$Q_E = Q_{DESIGN} \times \frac{TDH}{10}$$

References: 3, 7, 9

FIGURE H-30 LOW-LIFT PUMP STATION

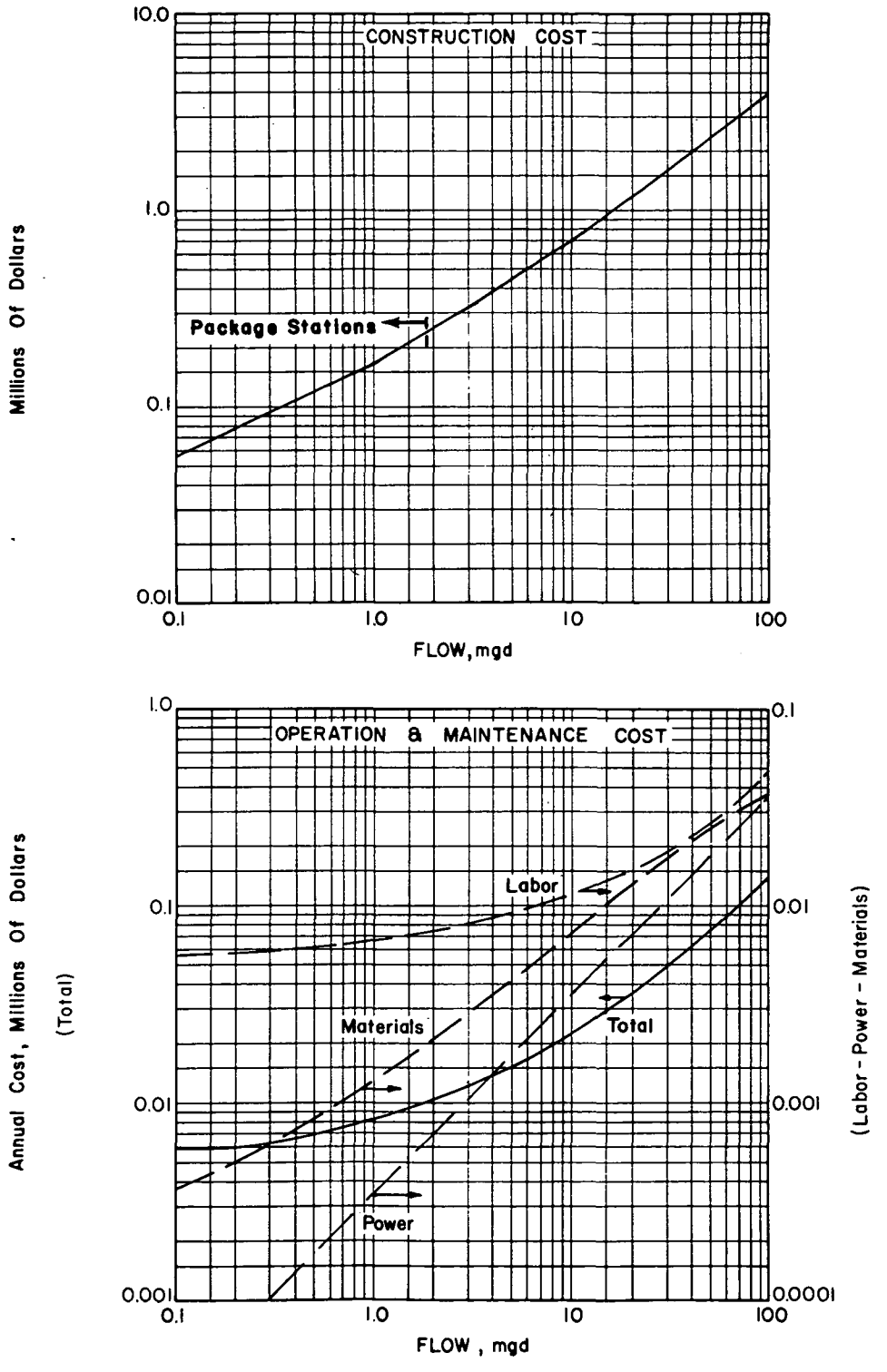


FIGURE H-31

PRELIMINARY TREATMENT

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:

   a. Flow channels and superstructures.
   b. Bar screens (mechanical).
   c. Grinders for screenings.
   d. Gravity grit chamber with mechanical grit-handling equipment.
   e. Parshall flume and flow-recording equipment.

2. Operation and maintenance costs do not include cost for grit disposal.

3. Screenings 1-3 cu ft/mg.

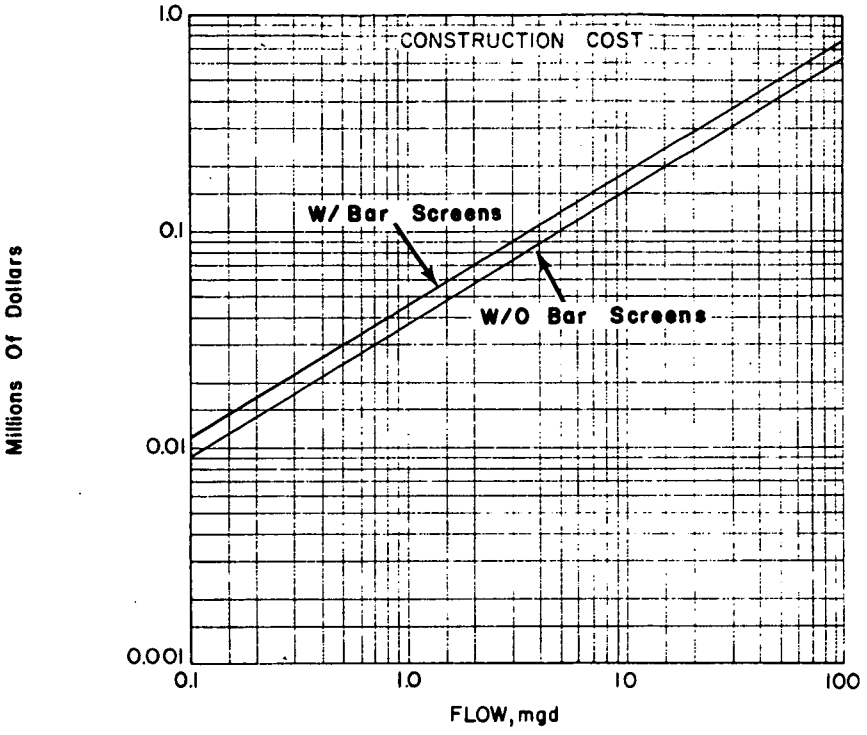
4. Grit 2-5 cu ft/mg.

5. If low-lift pumping is used prior to preliminary treatment, the cost for bar screens can be subtracted from the unit cost of preliminary treatment, because they are included in the low-lift pumping station.

Process Performance:

Suspended material greater than 5/8 inch and grit coarser than 65 mesh (0.208 mm) will be removed. However, removal of BOD and TSS by pre-treatment is assumed negligible.

FIGURE H-31 PRELIMINARY TREATMENT



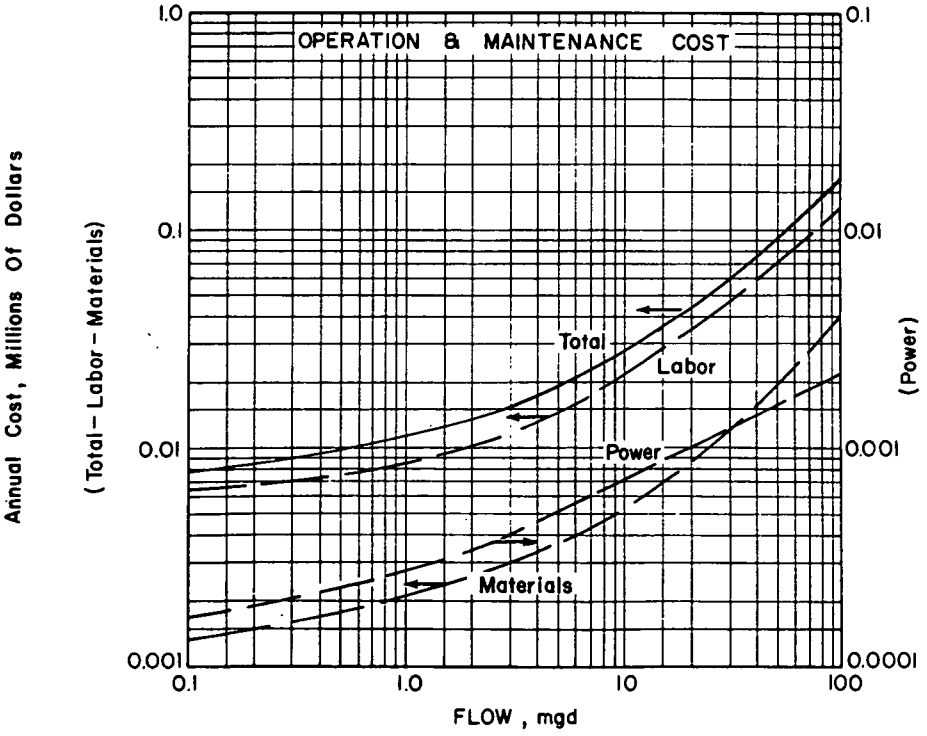


FIGURE H-32

FLOW EQUALIZATION

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Costs were derived for detention times of 0.5, 1.0, and 2.0 days based on offline storage at average flow conditions.

2. Mixing requirements = 20 to 40 hp/mg of storage volume.

3. Construction costs include basin and mechanical mixing equipment (pumping not included).

4. Construction costs are based on concrete basins for design flows less than 1 mgd and 6-inch concrete-lined earthen basins for design flows greater than 1 mgd.

References: 4, 5, 7

FIGURE H-32[1] FLOW EQUALIZATION
(DETENTION TIME = 0.5 DAYS)

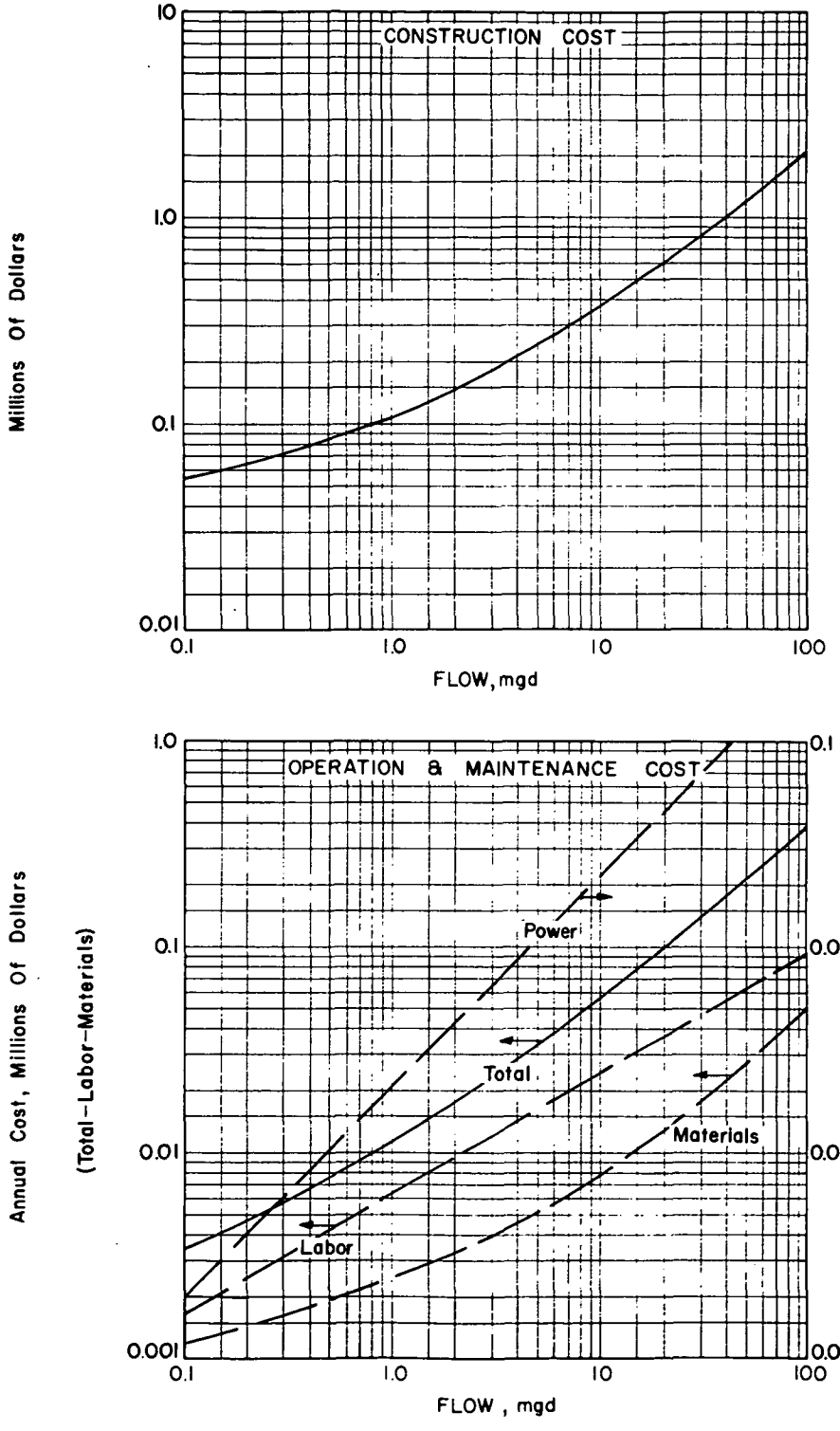
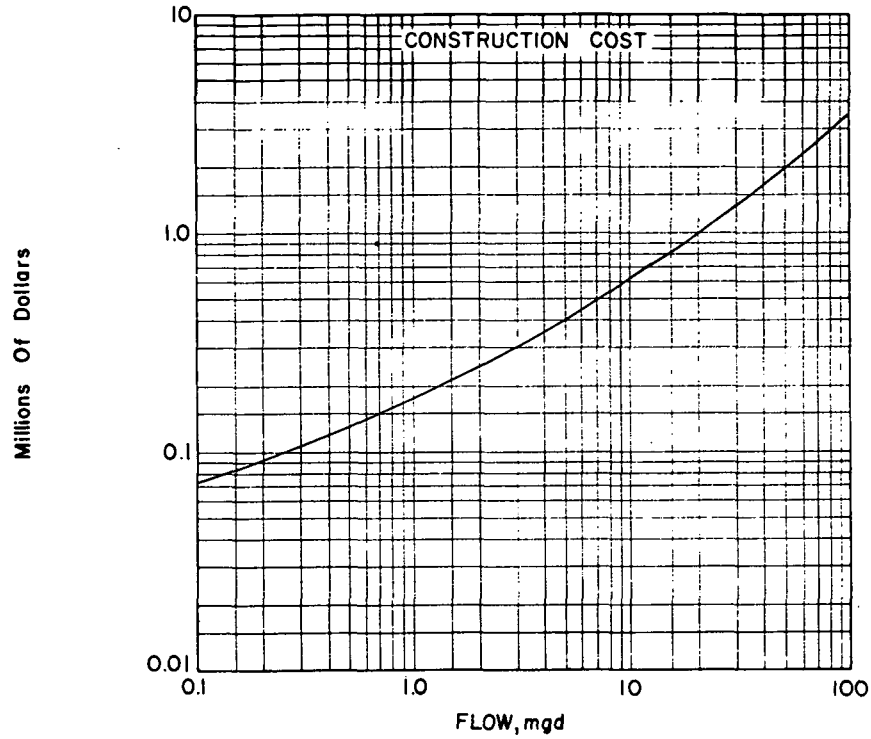


FIGURE H-32[2] FLOW EQUALIZATION
(DETENTION TIME = 1.0 DAYS)



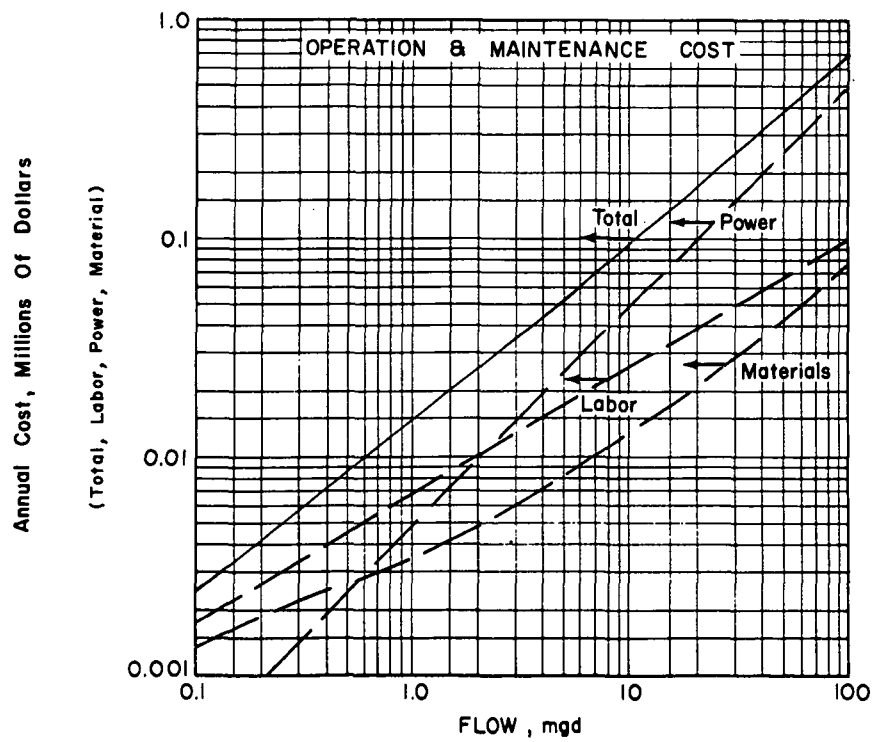
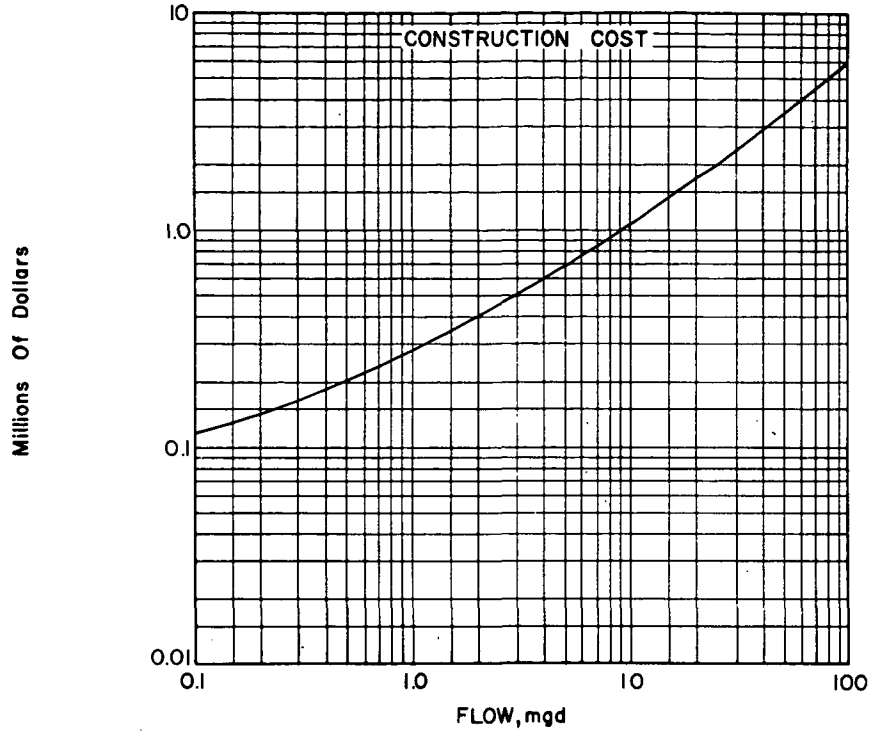


FIGURE H-32[3] FLOW EQUALIZATION
(DETENTION TIME = 2.0 DAYS)



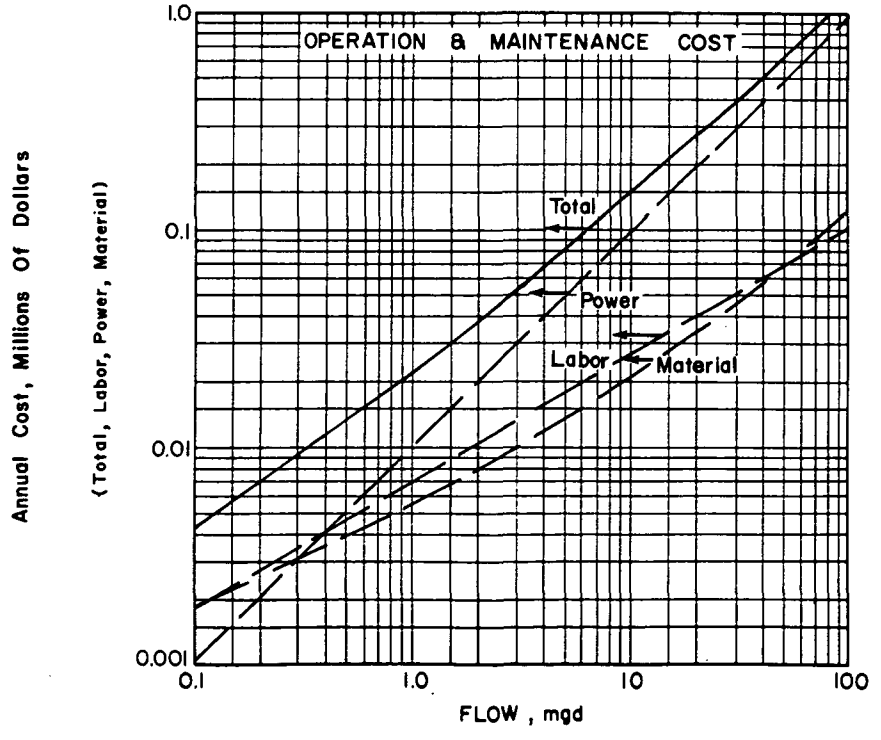


FIGURE H-33

PRIMARY CLARIFIER WITH SLUDGE PUMPS

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

1. Clarifier designed for surface overflow rate of 800 gpd/sq ft (based on average Q).

2. Costs include primary sludge pumps. Sludge concentration of 4% solids. Pump head assumed as 10 ft TDH.

3. Process performance:

Wastewater Characteristics

	In	Out
BOD_5, mg/l	210	130
COD, mg/l	400	265
TSS, mg/l	230	100
Total-P, mg/l	11	9

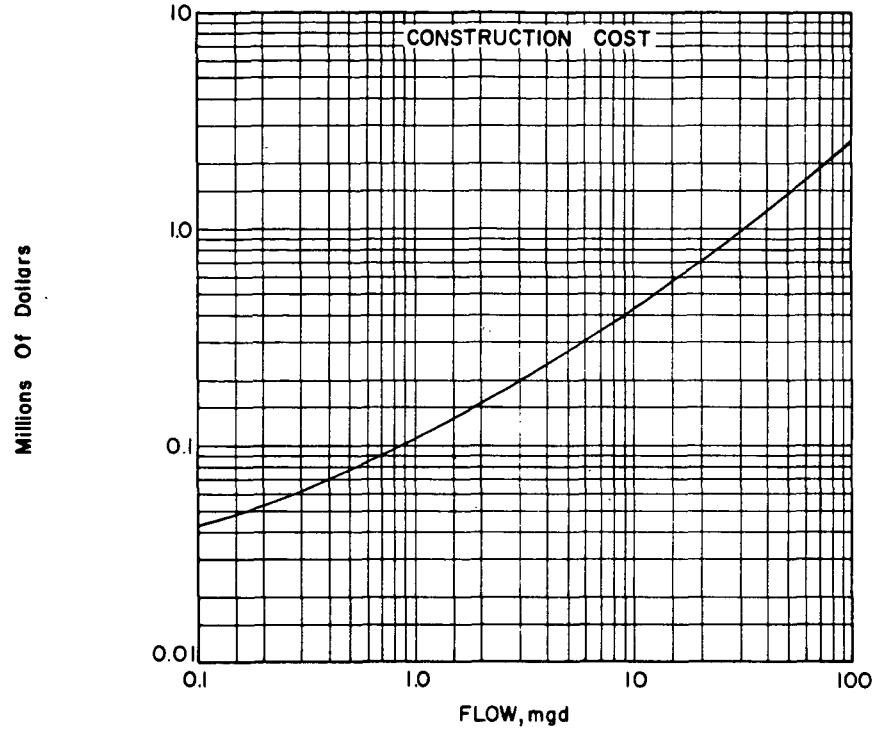
Adjustment Factor:

To adjust costs for alternative surface overflow rate, enter flow at effective flow (Q_E)

$$Q_E = Q_{DESIGN} \times \frac{800 \text{ gpd/sq ft}}{\text{New Design Surface Overflow Rate}}$$

References: 9, 24

FIGURE H-33 PRIMARY CLARIFIER WITH SLUDGE PUMPS



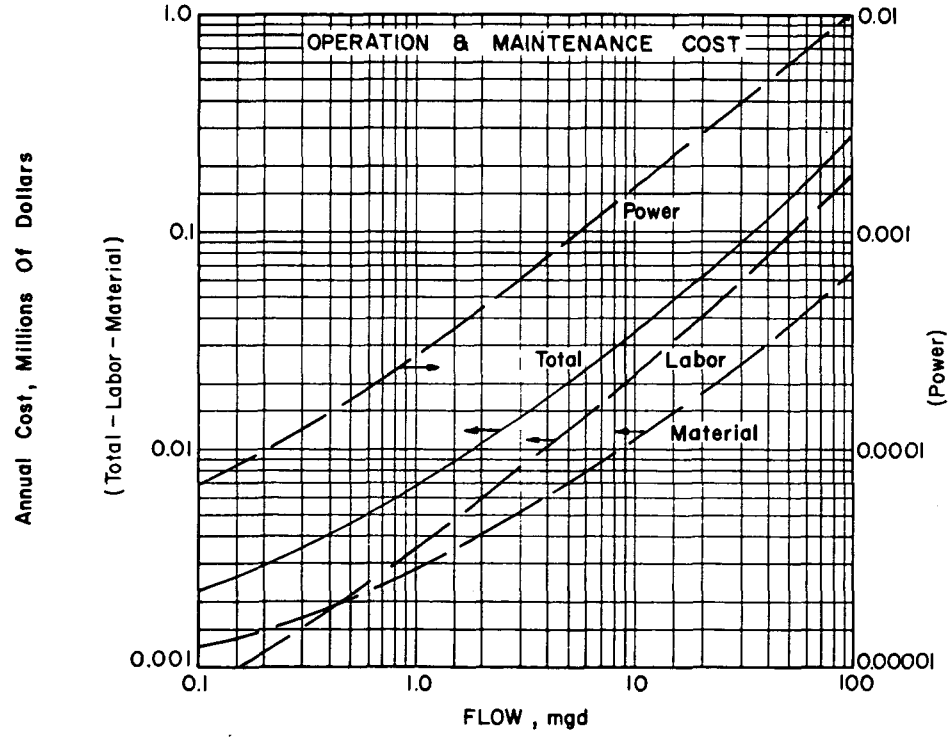


FIGURE H-34

CONVENTIONAL ACTIVATED SLUDGE AERATION WITH DIFFUSED AIR

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include costs for basins, air supply equipment and piping, and blower building. Clarifier and recycle pumps are not included. Clarifier costs are found in Figure H-42.
2. Diffused aeration.
3. 1.2 lbs O_2 supplied per pound of BOD_5 removed.
4. MLVSS = 2,000 mg/l.
5. F/M = 0.5.
6. Detention time = 6 hours.
7. Process performance:

Wastewater Characteristics

	In	Out*
BOD_5, mg/l	130	20
COD, mg/l	265	45
TSS, mg/l	100	20
Total-P, mg/l	9	7
NH_3-N, mg/l	20	17
TKN, mg/l	30	18

*Following clarification

References: 3, 7

FIGURE H-34 CONVENTIONAL ACTIVATED SLUDGE AERATION WITH DIFFUSED AIR

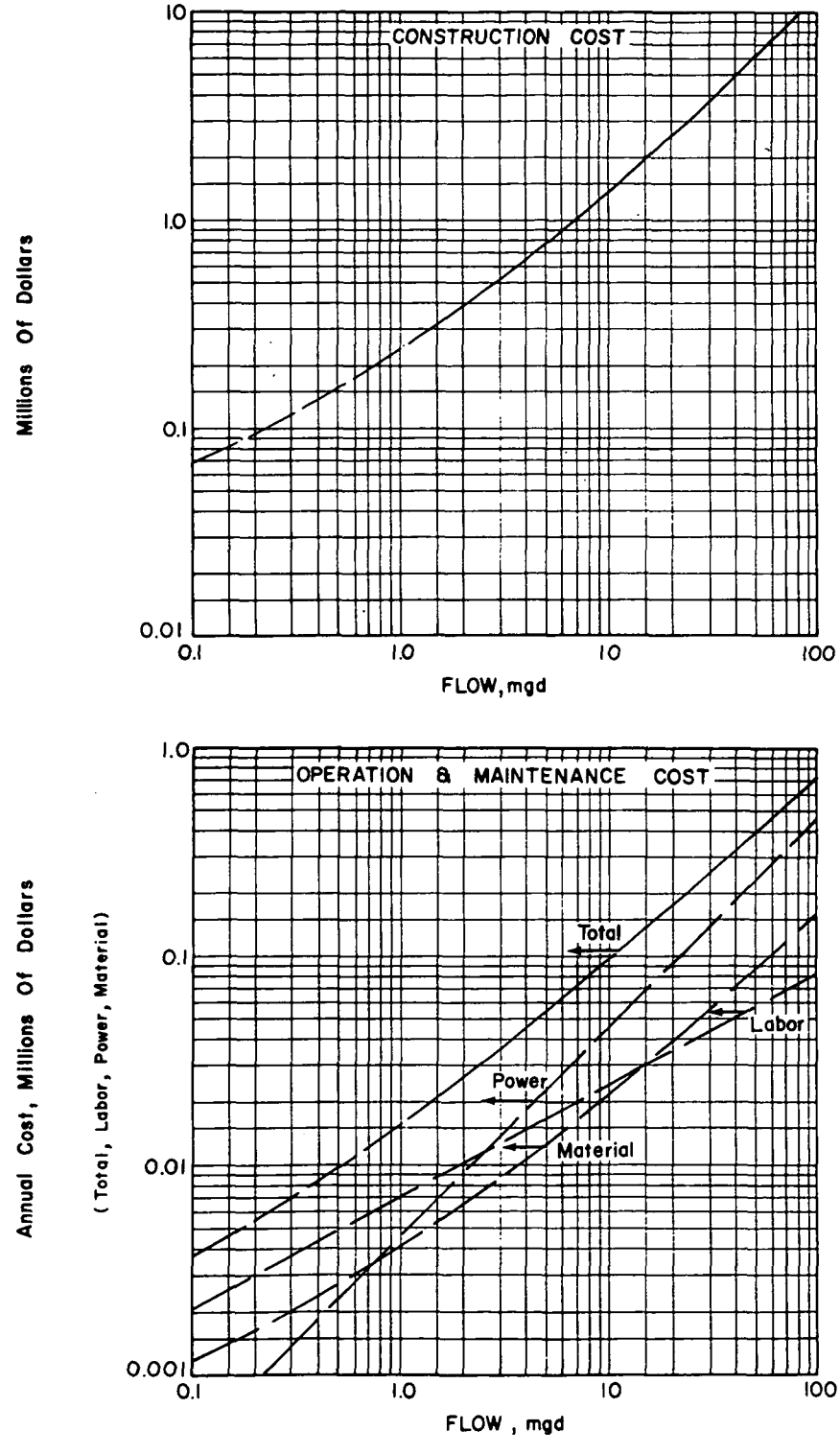


FIGURE H-35

ACTIVATED SLUDGE AERATION WITH PURE OXYGEN

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: dissolution equipment, oxygen generators and liquid oxygen feed/storage facilities instrumentation (where applicable), and licensing fees.

2. Oxygen was assumed to be delivered as liquid oxygen for plants from 0.1 to 1 mgd size. For plants from 1.0 to 100 mgd, oxygen was assumed to be generated on-site.

3. 1.2 lbs O_2 supplied per pound of BOD_5 removed.

4. MLVSS = 4,000 mg/l.

5. Detention time = 5 hours.

6. Process performance:

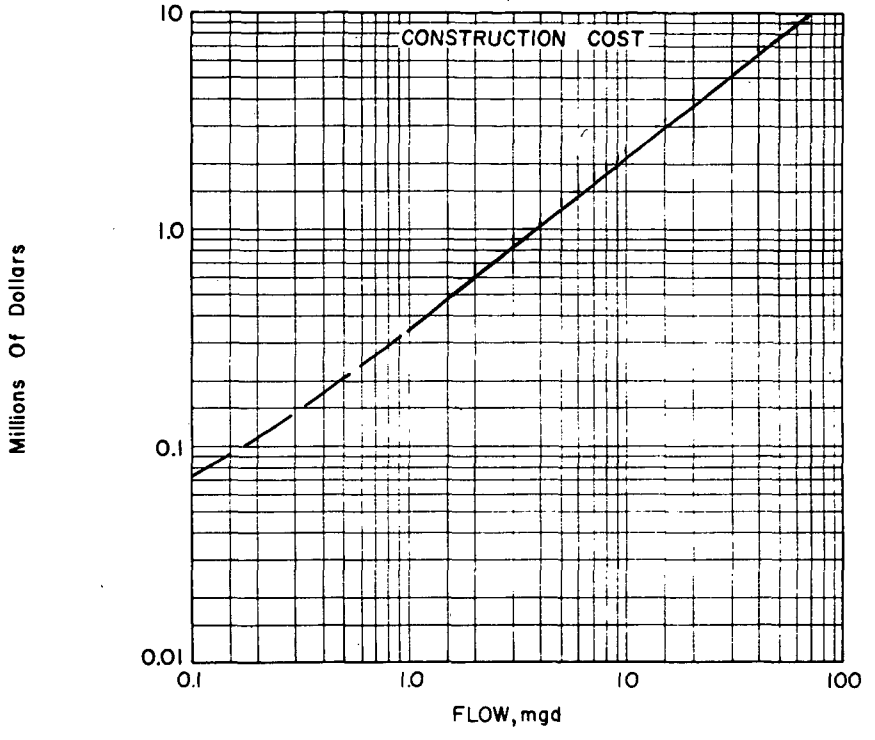
Wastewater Characteristics

	In	Out*
BOD_5, mg/l	130	20
COD, mg/l	265	40
TSS, mg/l	100	25
Total-P, mg/l	9	7
NH_3-N, mg/l	20	17
TKN, mg/l	30	18

*Following clarification

References: 39, 40, 41

FIGURE H-35 ACTIVATED SLUDGE AERATION WITH PURE OXYGEN



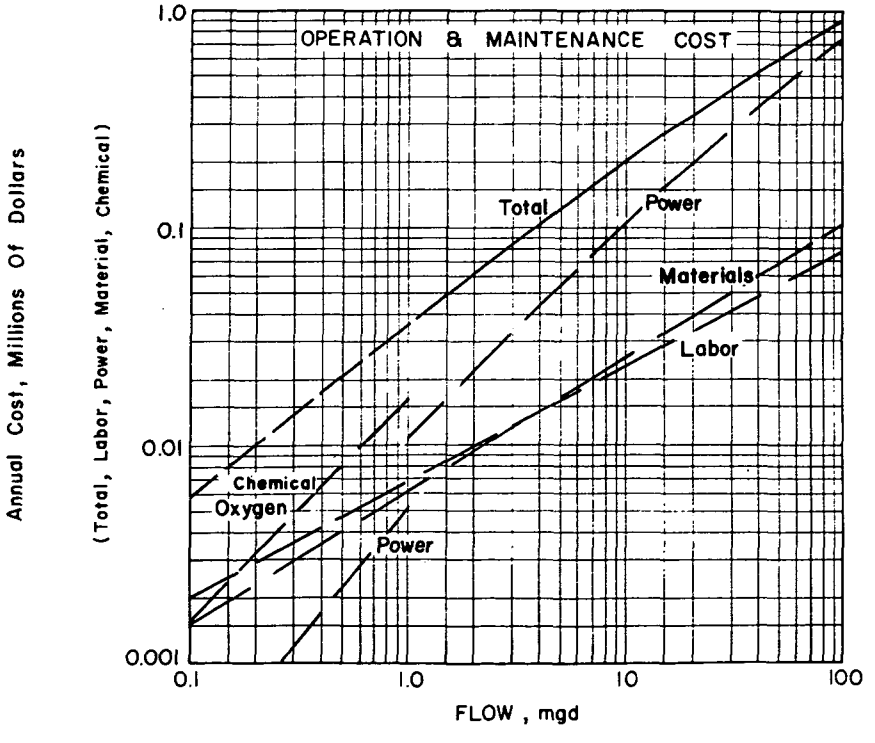


FIGURE H-36

HIGH-RATE TRICKLING FILTER

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: circular filter units with rotating distributor arms, synthetic media (6 feet-deep), and underdrains. Clarifiers and recycle equipment not included. (See Figure H-43.)

2. Organic loading rate: High Rate = 45 lb BOD_5/day/1,000 cu ft.

3. Hydraulic loading rate: High Rate = 30.2 mgad (693 gpd/sq ft) @ 3:1 recycle rate.

4. Electrical power not required (included in clarifier cost).

5. Process performance:

Wastewater Characteristics

	In	Out*
BOD_5, mg/l	130	45
COD, mg/l	265	90
TSS, mg/l	100	60
Total-P, mg/l	9	8
NH_3-N, mg/l	20	17
TKN, mg/l	30	18

*Following clarification

References: 3, 5, 6, 7, 30

FIGURE H-36 HIGH-RATE TRICKLING FILTER

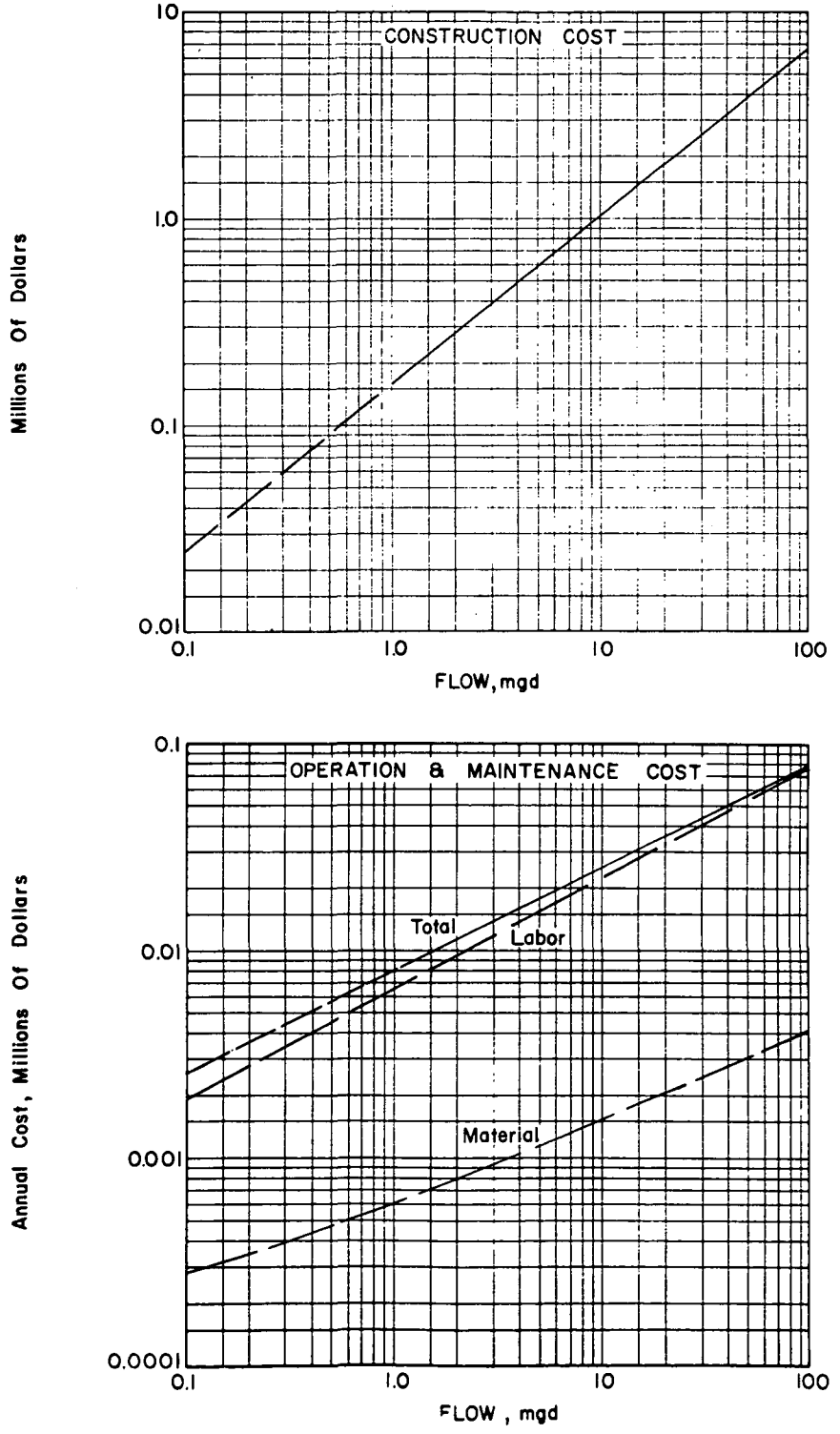


FIGURE H-37

LOW-RATE TRICKLING FILTER

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: circular filter units with rotating distributor arms, rock media (6 feet-deep), and underdrains. Clarifiers not included. (See Figure H-43.)

2. Organic loading rate: Low Rate = 15 lb BOD_5/day/1,000 cu ft.

3. Hydraulic loading rate: Low Rate = 2.4 mgad (55 gpd/sq ft).

4. Electrical power not required (included in clarifier cost).

5. No pump station included.

6. Process performance:

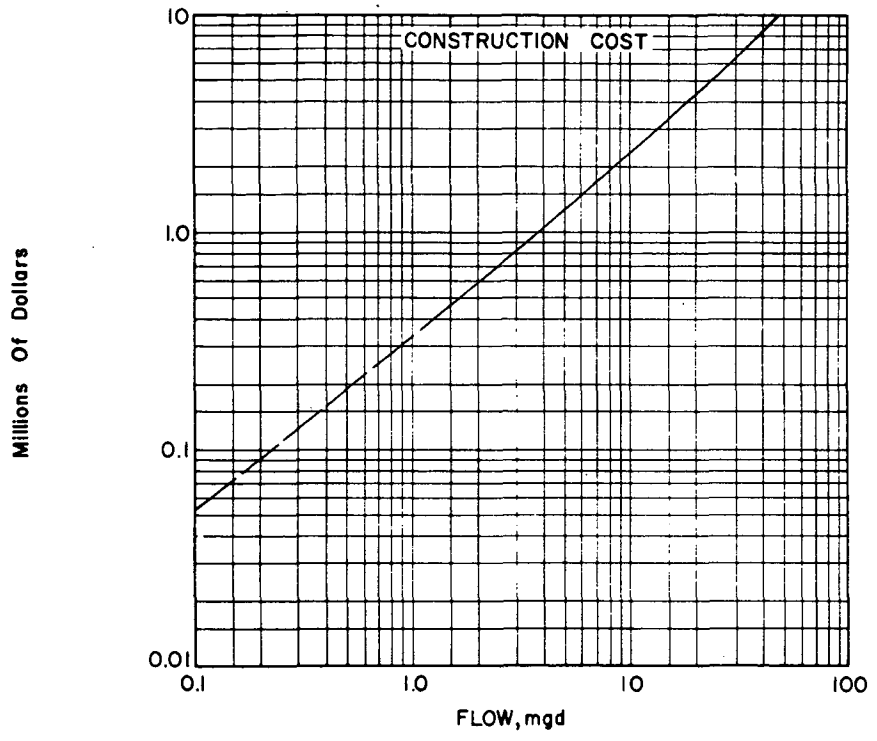
Wastewater Characteristics

	In	Out*
BOD_5, mg/l	130	30
COD, mg/l	265	60
TSS, mg/l	100	60
Total-P, mg/l	9	8
NH_3-N, mg/l	20	2
NO_3-N, mg/l	30	18

*Following clarification at temperature above 15°C.

References: 3, 5, 6, 7, 30

FIGURE H-37 LOW-RATE TRICKLING FILTER



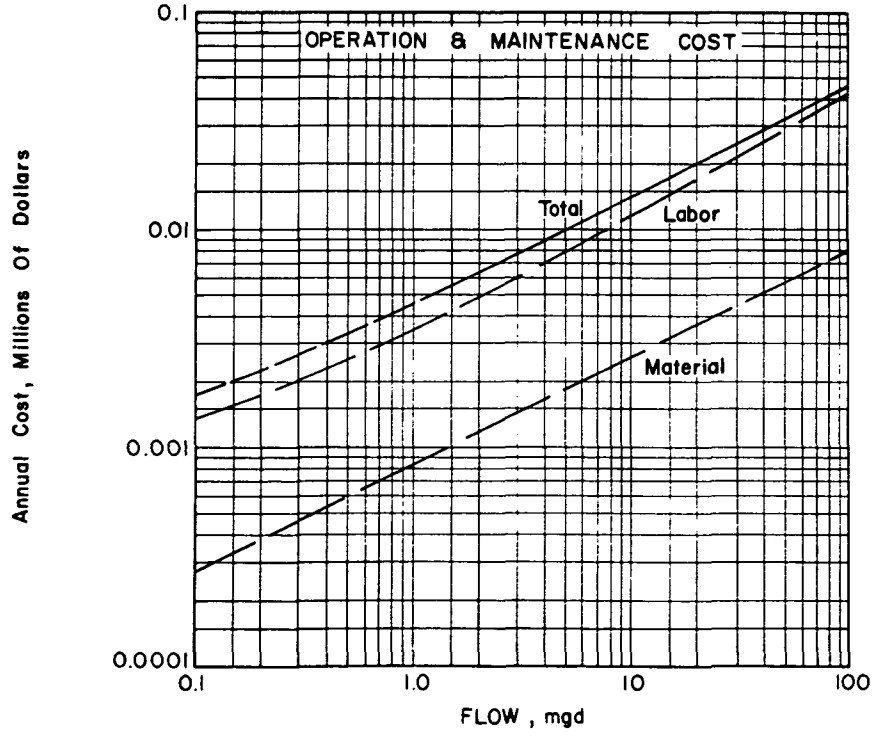


FIGURE H-38

AERATED LAGOONS

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Average detention time = 7 days.
2. 15-ft water depth.
3. Horsepower required: 30 to 40 hp/mg.
4. Floating mechanical aerators.
5. Construction cost includes:
   a. Excavation, embankment, and seeding of lagoon/slopes (3 cells).
   b. Service road and fencing.
   c. Riprap embankment protection.
   d. Hydraulic control works.
   e. Aeration equipment and electrical equipment.
6. Process performance:

Wastewater Characteristics

	In	Out
BOD_5, mg/l	210	25
COD, mg/l	400	50
TSS, mg/l	230	40
Total-P, mg/l	11	8
NH_3-N, mg/l	20	18

Adjustment Factor:

To adjust construction cost for detention time other than above, enter curve at effective flow (Q_E)

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Detention Time}}{7 \text{ Days}}$$

References: 27, 43

FIGURE H-38 AERATED LAGOONS

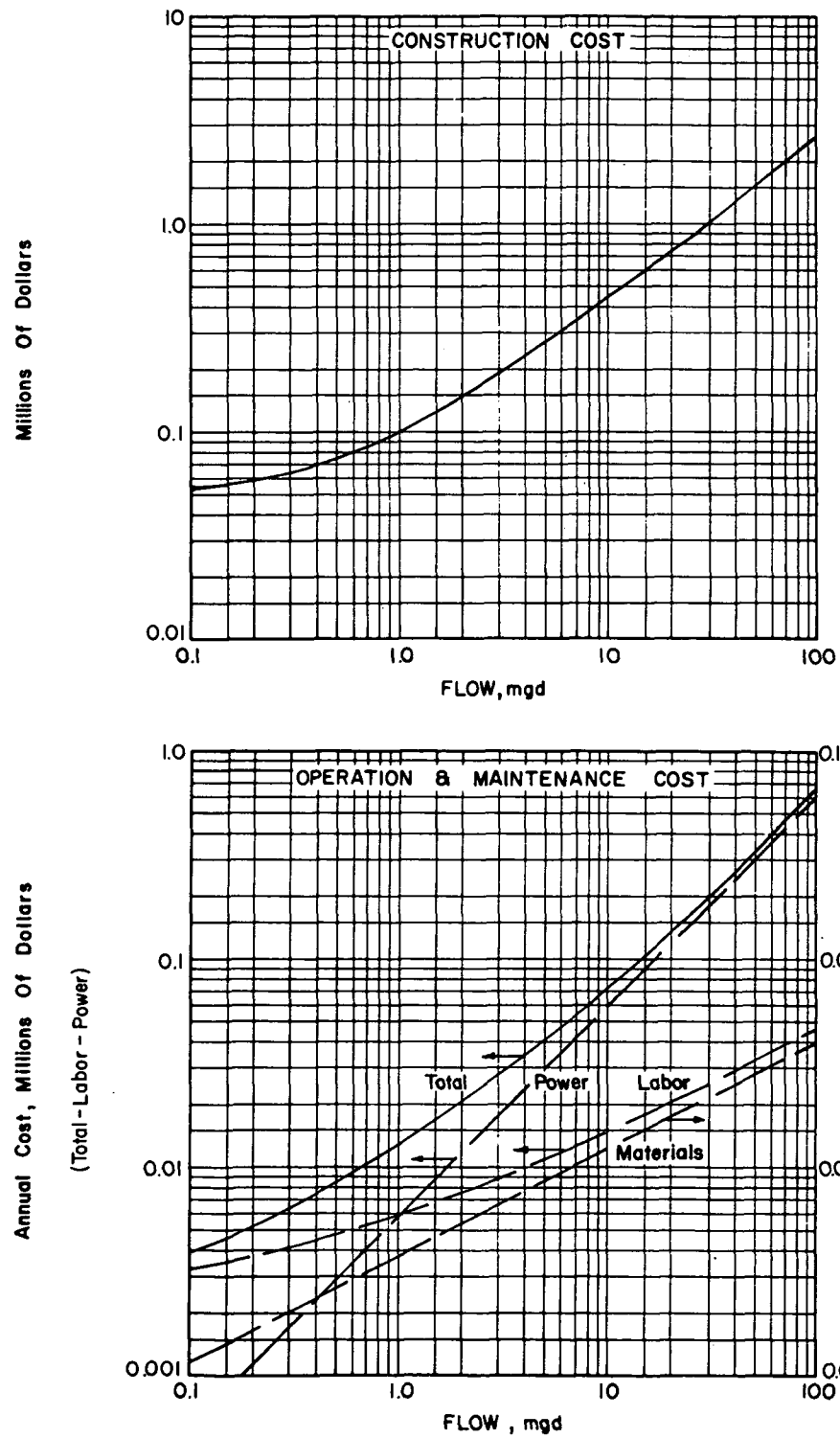


FIGURE H-39

ROTATING BIOLOGICAL CONTACTORS

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction cost includes:

   a. RBC shafts; standard media, 100,000 sq ft/shaft.
   b. Motor drives - 5 hp/shaft.
   c. Molded fiberglass covers.
   d. Reinforced concrete basins.

2. Cost does not include final clarifiers. (See Figure H-42.)

3. Loading rate:

   a. 1.0 gpd/sq ft, w/o nitrification.
   b. 0.5 gpd/sq ft, w/nitrification.

4. Process performance:

	Wastewater Characteristics		
	In	Out	
		w/o Nitrification	w/Nitrification
BOD_5, mg/l	130	20	15
COD, mg/l	265	45	35
TSS, mg/l	100	25	25
Total-P, mg/l	9	7	8
NH_3-N, mg/l	20	17	2

Reference: 48

FIGURE H-39[1] ROTATING BIOLOGICAL CONTACTOR (W/NITRIFICATION)

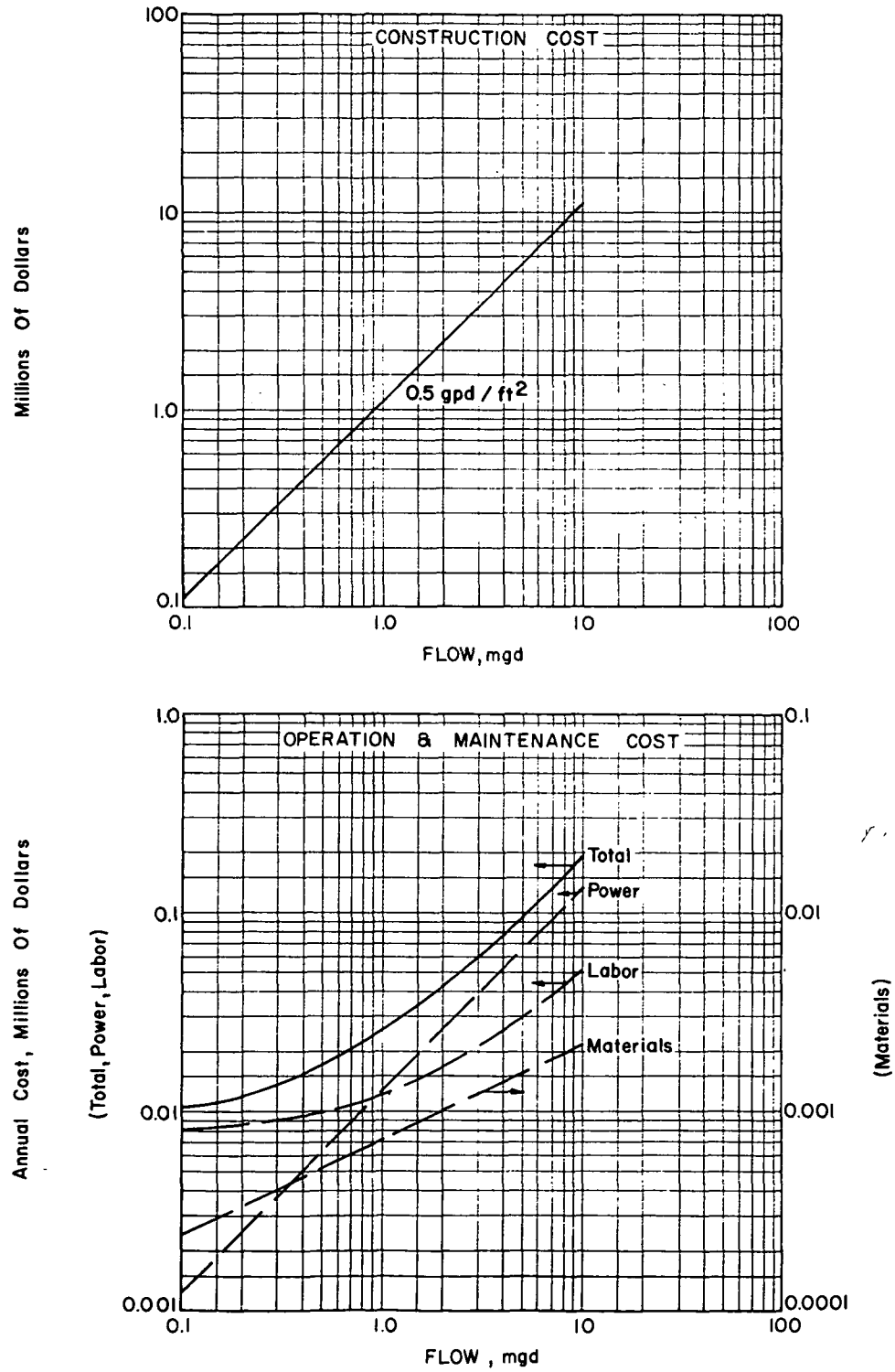


FIGURE H-39

ROTATING BIOLOGICAL CONTACTORS

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction cost incudes:

   a. RBC shafts; standard media, 100,000 sq ft/shaft.
   b. Motor drives - 5 hp/shaft.
   c. Molded fiberglass covers.
   d. Reinforced concrete basins.

2. Cost does not include final clarifiers. (See Figure H-42.)

3. Loading rate:

   a. 1.0 gpd/sq ft, w/o nitrification.
   b. 0.5 gpd/sq ft, w/nitrification.

4. Process performance:

	Wastewater Characteristics		
	In	Out	
		w/o Nitrification	w/Nitrification
BOD_5, mg/l	130	20	15
COD, mg/l	265	45	35
TSS, mg/l	100	25	25
Total-P, mg/l	9	7	8
NH_3-N, mg/l	20	17	2

Reference: 48

FIGURE H-39[2] ROTATING BIOLOGICAL CONTACTOR (W/O NITRIFICATION)

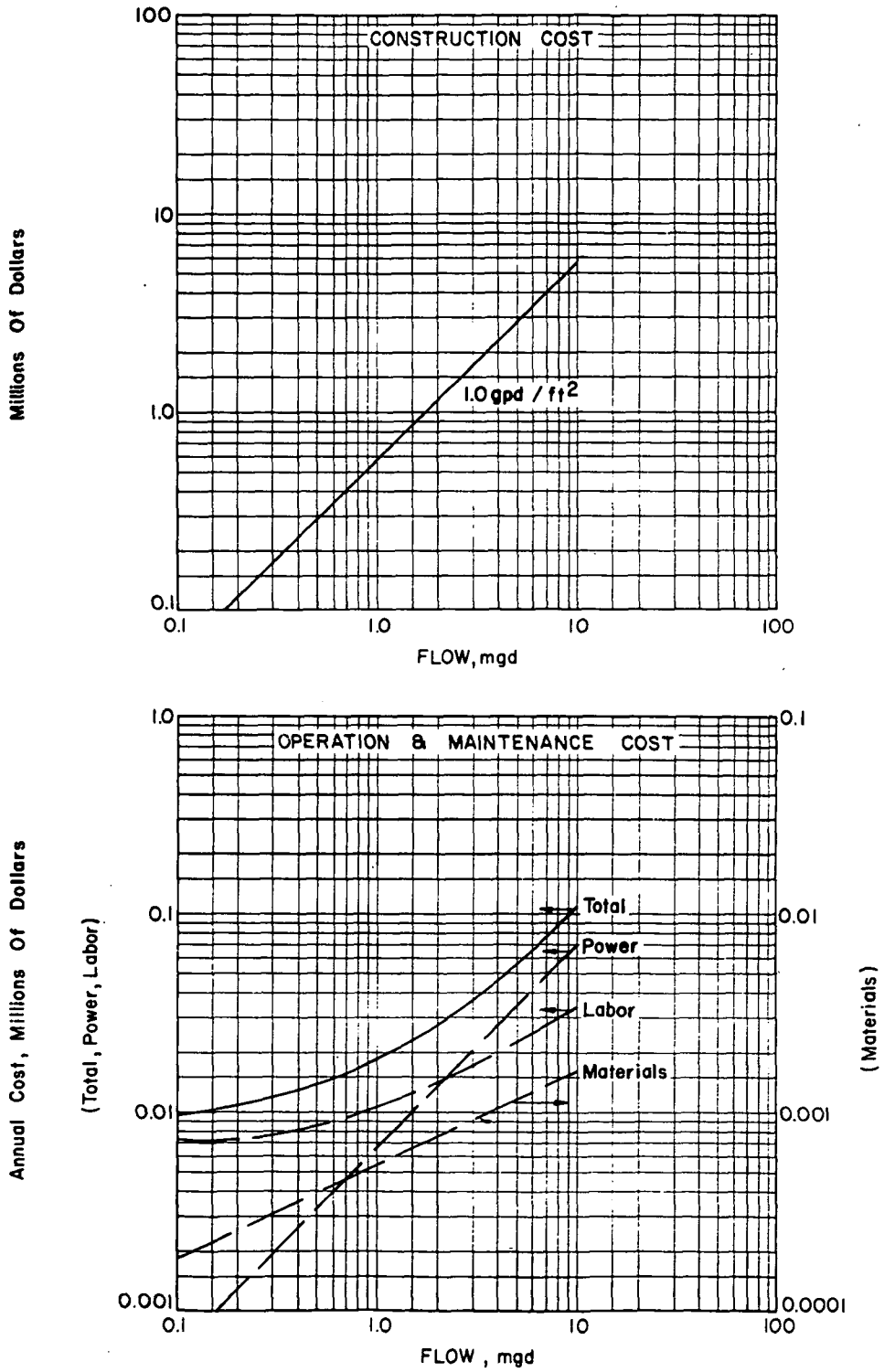


FIGURE H-40

FACULTATIVE LAGOONS

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

1. Warm climate - lagoon loading of 40 lb BOD_5/acre/day.
2. Cool climate (Northern U.S.) - lagoon loading of 20 lb BOD_5/acre/day.
3. Water Depth = 4 ft.
4. Construction costs include:
   a. Excavating, grading, and other earthwork required for normal subgrade preparation.
   b. Service roads.
   c. Costs do not include land and pumping.
5. Process performance:

Wastewater Characteristics

	In	Out	
BOD_5, mg/L	210	30	
COD, mg/l	400	100	
TSS, mg/l	230	60	
Total-P, mg/l	11	8	
NH_3-N, mg/l	20	15	(cool climate)
		1	(warm climate)

Adjustment Factor:

To adjust costs for loadings other than those above, enter curve at effective flow (Q_E)

Warm Climates:

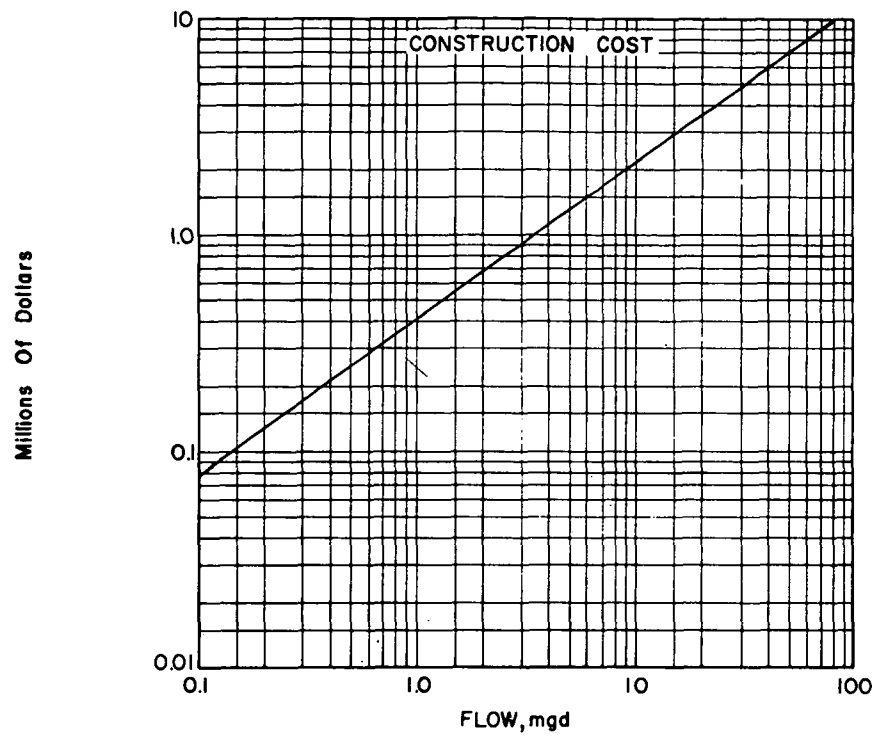
$$Q_E = Q_{DESIGN} \times \frac{40 \text{ lb } BOD_5/\text{acre/day}}{\text{New Design Loading}}$$

Cool Climates:

$$Q_E = Q_{DESIGN} \times \frac{20 \text{ lb } BOD_5/\text{acre/day}}{\text{New Design Loading}}$$

References: 3, 5, 28

FIGURE H-40[1] FACULATIVE LAGOON (Warm Climates)



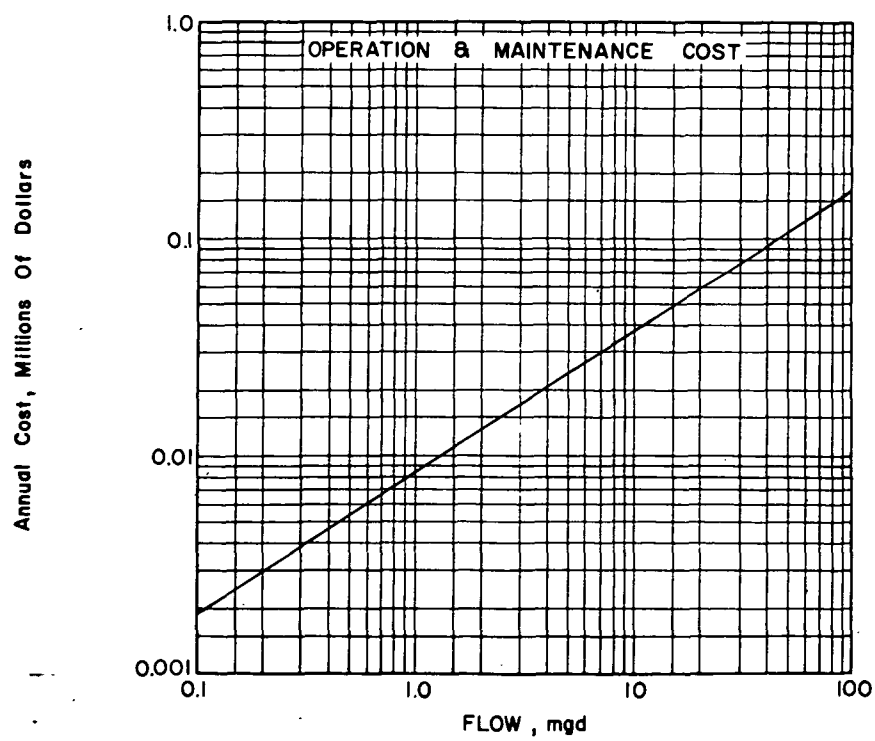


FIGURE H-40

FACULTATIVE LAGOONS

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

1. Warm climate - lagoon loading of 40 lb BOD_5/acre/day.
2. Cool climate (Northern U.S.) - lagoon loading of 20 lb BOD_5/acre/day.
3. Water Depth = 4 ft.
4. Construction costs include:
   a. Excavating, grading, and other earthwork required for normal subgrade preparation.
   b. Service roads.
   c. Costs do not include land and pumping.
5. Process performance:

Wastewater Characteristics

	In	Out	
BOD_5, mg/l	210	30	
COD, mg/l	400	100	
TSS, mg/l	230	60	
Total-P, mg/l	11	8	
NH_3-N, mg/l	20	15	(cool climate)
		1	(warm climate)

Adjustment Factor:

To adjust costs for loadings other than those above, enter curve at effective flow (Q_E)

Warm Climates:

$$Q_E = Q_{DESIGN} \times \frac{40 \text{ lb } BOD_5/\text{acre/day}}{\text{New Design Loading}}$$

Cool Climates:

$$Q_E = Q_{DESIGN} \times \frac{20 \text{ lb } BOD_5/\text{acre/day}}{\text{New Design Loading}}$$

References: 3, 5, 28

FIGURE H-40[2] FACULATIVE LAGOON (Cool Climates)

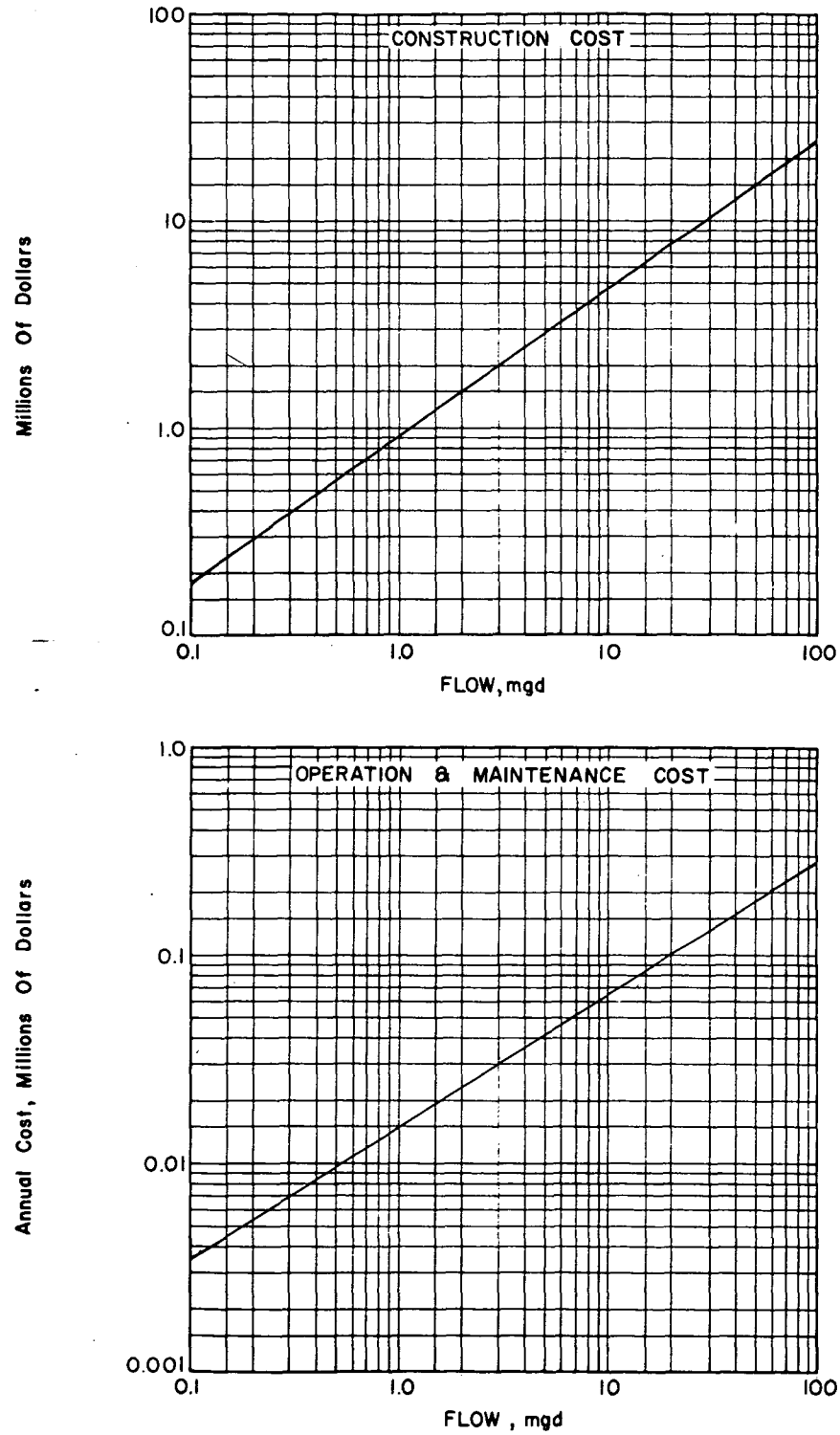


FIGURE H-41

HIGH-RATE ACTIVATED SLUDGE AERATION (COMPLETE MIX)

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

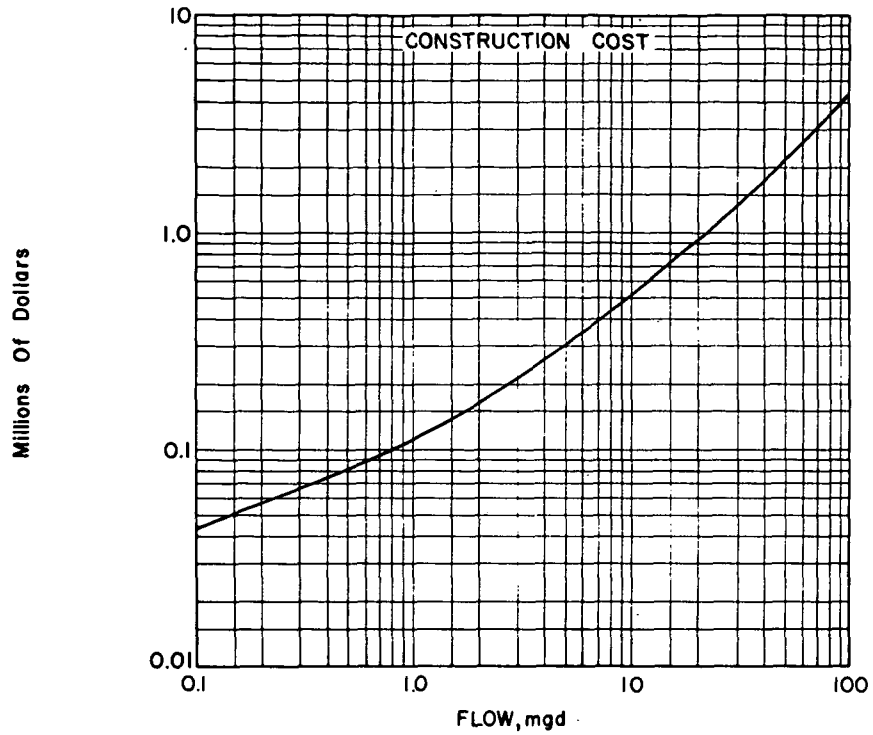
1. Construction cost includes cost for basins, air supply equipment and piping, and a blower building. Clarifier and recycle pumps are not included. (See Figure H-42.)
2. Basins sized with 50 percent recycle flow.
3. Detention time = 3 hours.
4. F/M = 1.5.
5. 0.7 lbs O_2 applied per pound BOD_5 removed.
6. MLVSS = 2,000 mg/l.
7. Process Performance:

Wastewater Characteristics

	In	Out
BOD_5, mg/l	130	40
COD, mg/l	265	80
TSS, mg/l	100	35
Total-P, mg/l	9	8
NH_3-N, mg/l	20	19

References: 3, 7

FIGURE H-41 HIGH-RATE ACTIVATED SLUDGE- AERATION - COMPLETE MIX
(Two - Hour Detention Time)



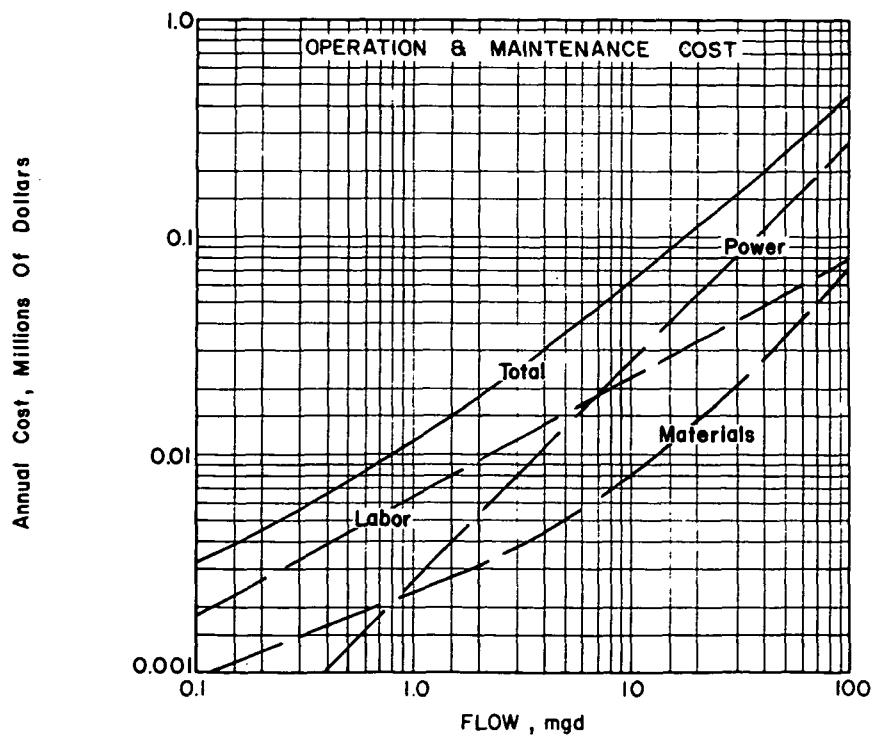


FIGURE H-42

FINAL CLARIFIER (FLOCCULATOR TYPE)

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Flocculator-type clarifier.

2. Overflow rate of 600 gpd/sq ft used for development of costs.

3. Costs include sludge return and waste pumps. Sludge concentration of 1 percent solids. Pump TDH at 10 ft. Spare pumps included as necessary. Pump capacity 350 gpm/mgd of plant capacity. Nonclog centrifugal pumps.

4. Rectangular units when surface area is less than 500 sq ft. Circular units when surface area is greater than 500 sq ft.

5. Maximum clarifier diameter = 200 ft.

6. Clarifier performance: See performance characteristics predicted for processes that precede clarifiers.

Adjustment Factor:

To adjust capital cost for alternative overflow rates, enter the curve at effective flow (Q_E)

$$Q_E = Q_{DESIGN} \times \frac{600 \text{ gpd/sq ft}}{\text{New Design Overflow Rate}}$$

References: 6, 7, 9, 24

FIGURE H-42 FINAL CLARIFIER (FLOCCULATOR TYPE)

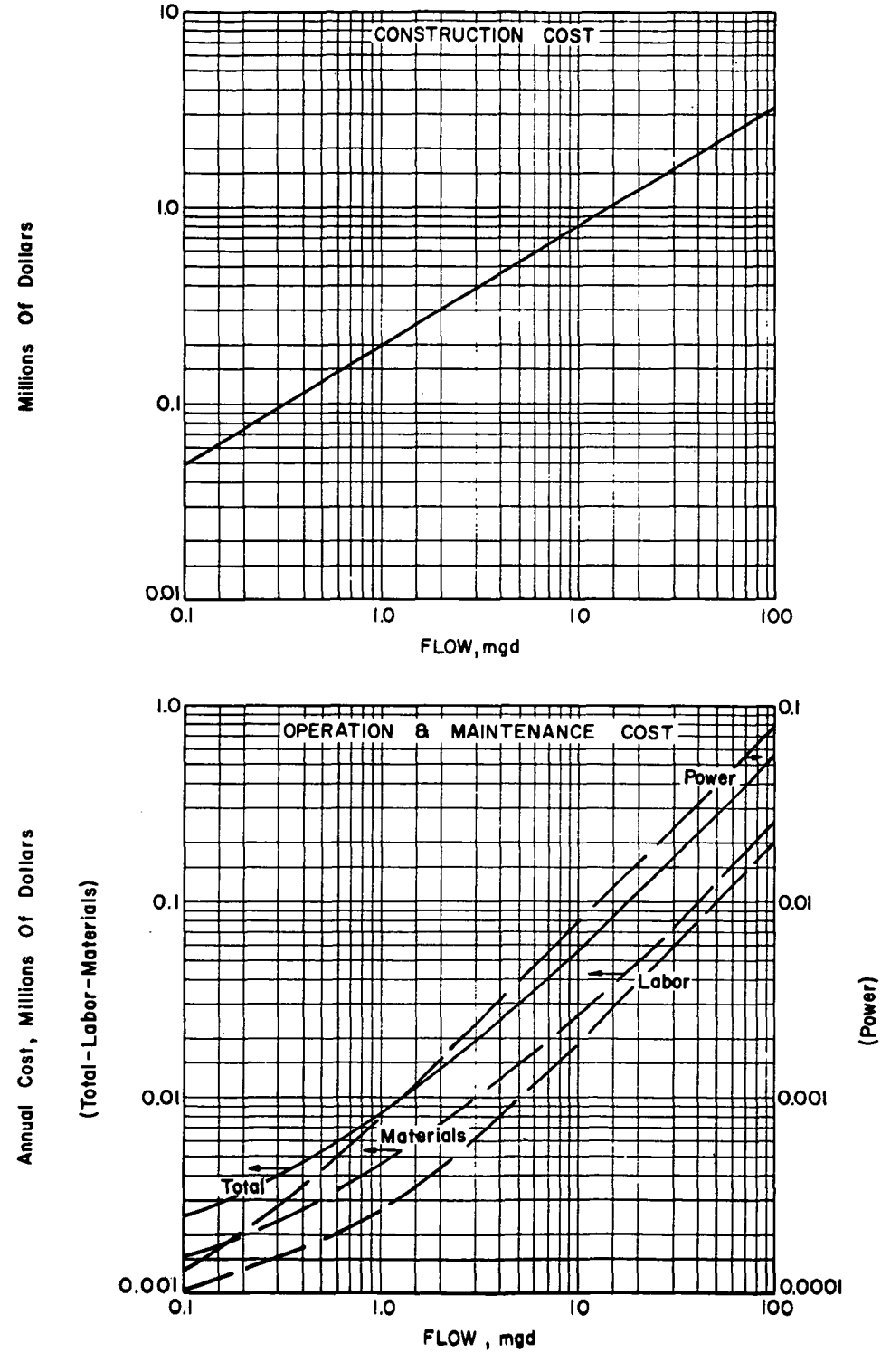


FIGURE H-43

CLARIFIER FOR HIGH-RATE TRICKLING FILTER
(INCLUDES RECYCLE PUMPS)

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: sludge pumps, effluent recycle pumps, clarifier mechanisms and internal piping.

2. Overflow rate = 800 gpd/sq ft at average design flow.

3. Recycle pumping capacity = 3 times average wastewater flow.

4. Curve is to be used in conjunction with high-rate trickling filter only.

5. Rectangular units when surface area is less than 500 sq. ft. Circular units when surface area flow is greater than 500 sq ft.

6. Maximum clarifier diameter = 200 ft.

7. Clarifier performance: see predicted performance of high-rate trickling filter.

Adjustment Factor:

To adjust construction cost for alternative loading rates, enter curve at effective flow (Q_E)

$$Q_E = Q_{DESIGN} \times \frac{800 \text{ gpd/sq ft}}{\text{New Design Overflow Rate}}$$

References: 3, 6

FIGURE H-43 CLARIFIER FOR HIGH RATE TRICKLING FILTER (INCLUDES RECYCLE PUMPS)

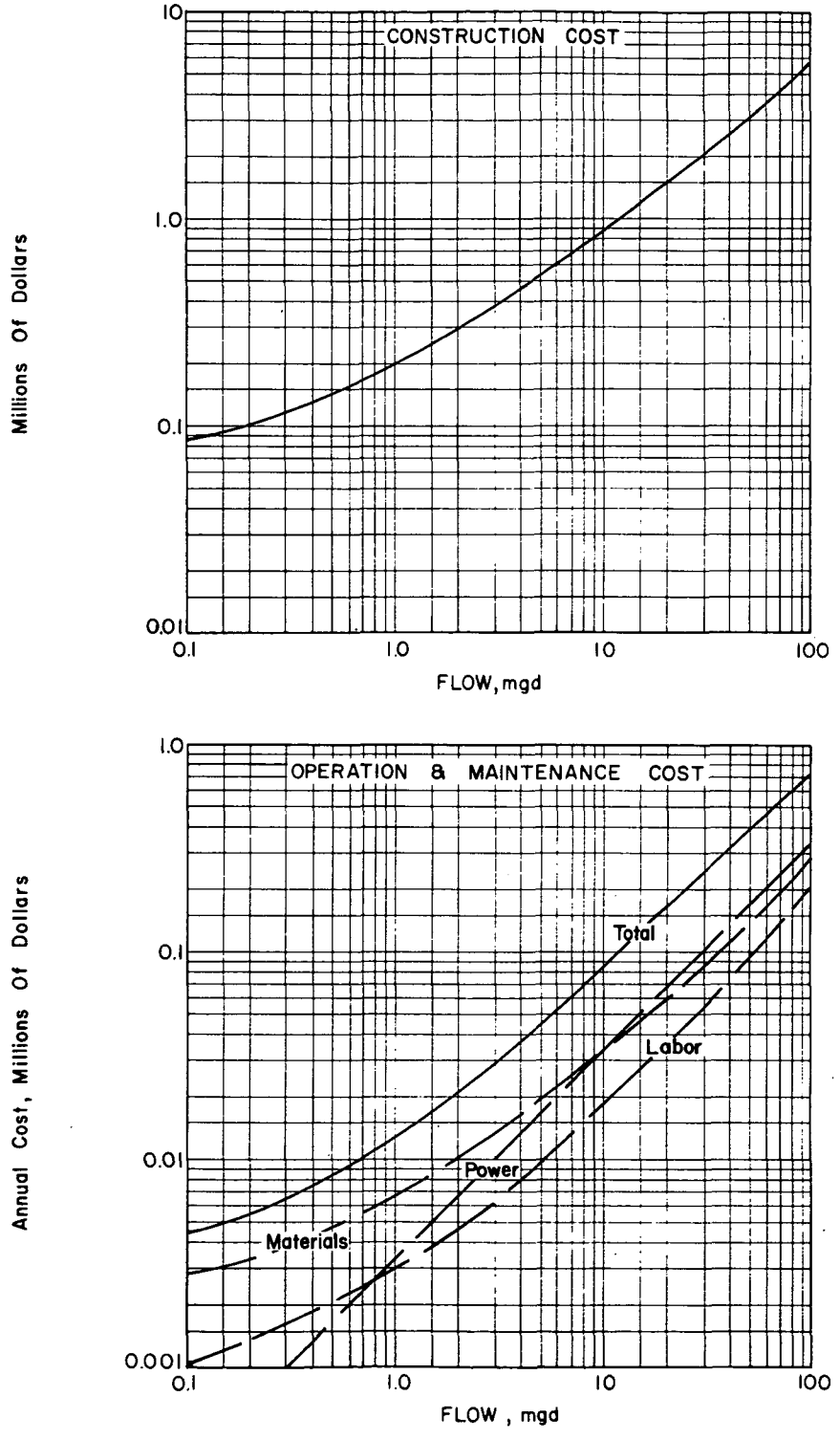


FIGURE H-44

ACTIVATED SLUDGE/NITRIFICATION (SINGLE STAGE)

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include aeration tanks and aeration devices, but not necessarily clarifiers and sludge pumps. (See Figure H-42.)

2. The activated sludge system is assumed to be a plug flow. The nitrification takes place in the latter portions of the basin.

3. The detention time is assumed to be 10 hours.

4. The aeration and mixing are accomplished by diffused air.

5. Oxygen requirements: 1.5 lbs O_2/lb BOD_5 removed plus 4.6 lbs O_2/lb NH_3 oxidized.

6. MLVSS = 1,500 mg/l.

7. SRT = 15 days.

8. Process performance:

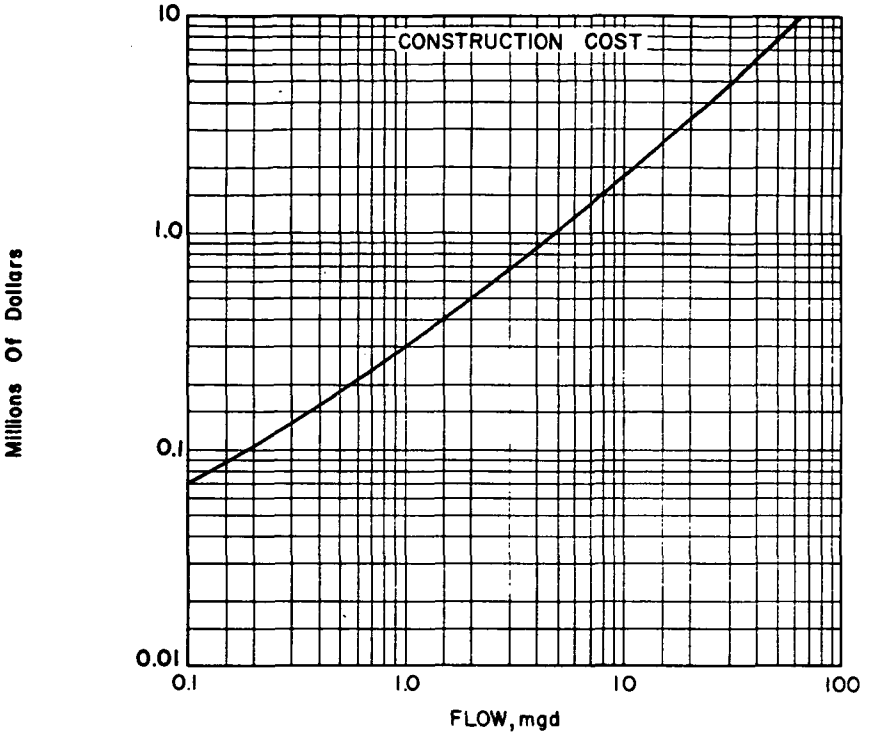
Wastewater Characteristics

	In	Out*
BOD_5, mg/l	140	10
COD, mg/l	265	35
TSS, mg/l	100	20
Total-P, mg/l	9	8
NH_3-N, mg/l	20	2
NO_3-N, mg/l	0	18

*Following clarification.

References: 3, 7, 9

FIGURE H-44 ACTIVATED SLUDGE / NITRIFICATION (SINGLE STAGE)



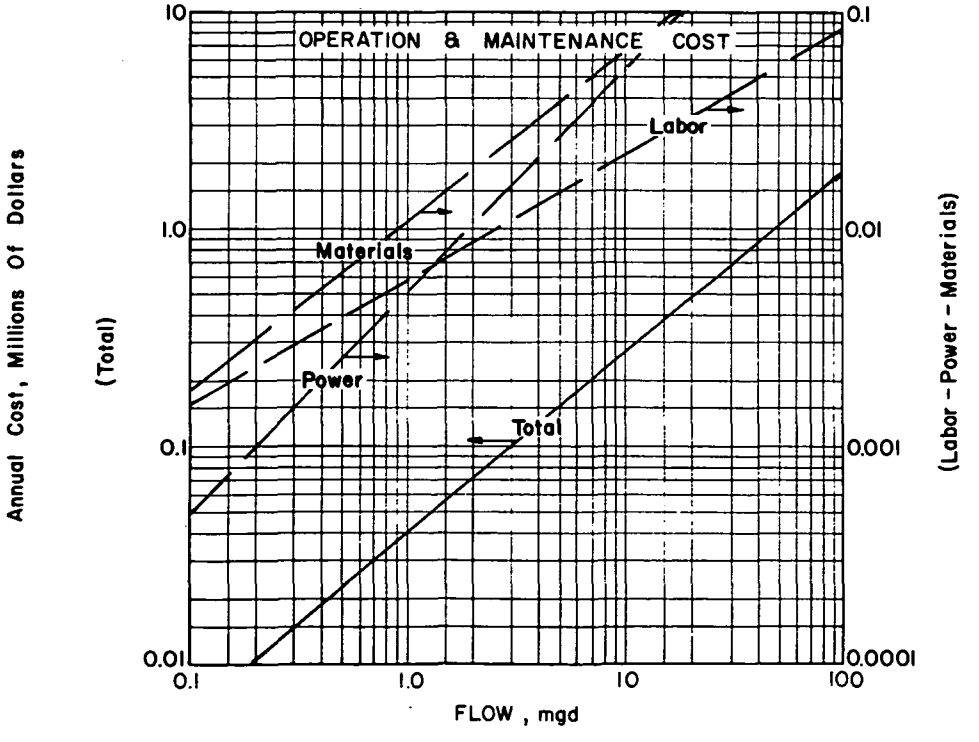


FIGURE H-45

SEPARATE NITRIFICATION (WITH CLARIFIER)

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include nitrification tanks, aeration devices, clarifiers, and sludge recycle and waste pumps, but not pH adjustment facilities.

2. System to follow high-rate activated sludge system.

3. Detention time = 3 hours.

4. O_2 requirements: 1.5 lbs O_2/lb BOD_5 removed plus 4.6 lbs O_2/lb NH_3-N oxidized.

5. Sludge return pumps sized for 100 percent recycle. Operated at 50 percent recycle.

6. Final clarifier overflow rate = 600 gpd/sq ft.

7. Process performance:

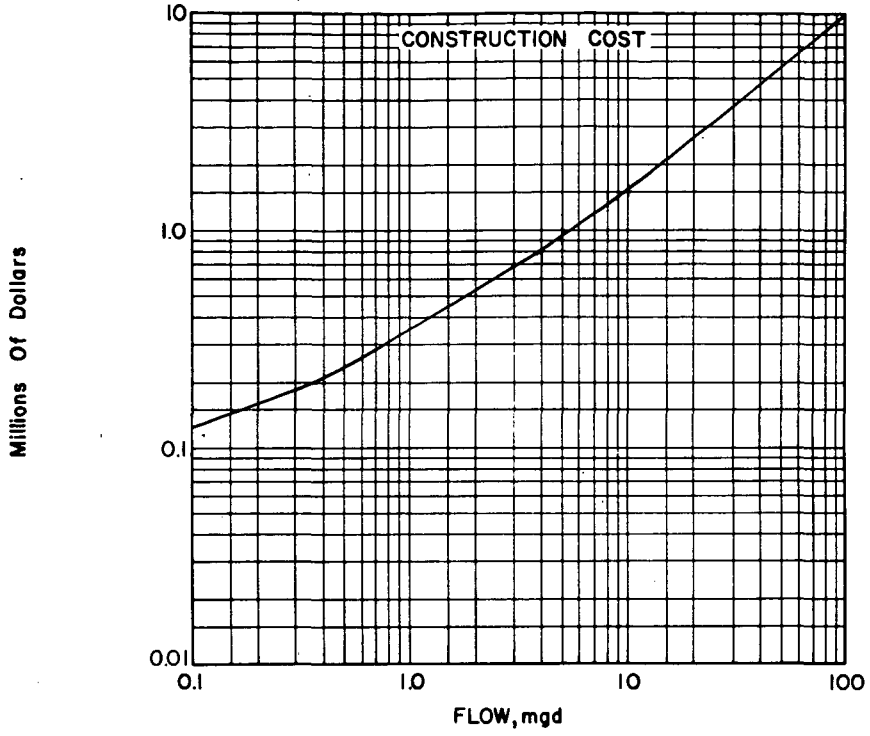
Wastewater Characteristics

	In	Out*
BOD_5, mg/l	40	10
COD, mg/l	80	35
TSS, mg/l	35	20
Total-P, mg/l	9	8
NH_3-N, mg/l	19	1
NO_3-N, mg/l	0	18

*Following clarification.

References: 2, 7

FIGURE H-45 SEPARATE NITRIFICATION (WITH CLARIFIER)



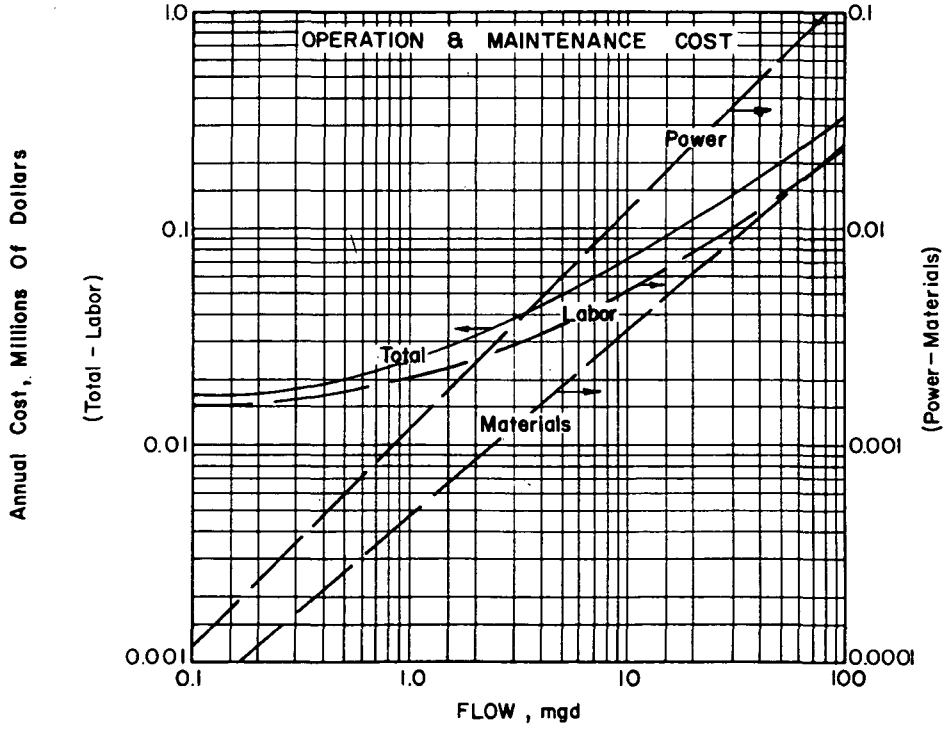


FIGURE H-46

SEPARATE DENITRIFICATION (WITH CLARIFIER)

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include denitrification tanks (uncovered), mixers, methanol feed, clarifiers, sludge recycle and waste pumps, but do not include reaeration facility. (See Figure H-61.)

2. Two hours detention time in denitrification tank.

3. MLVSS = 2,000 mg/l.

4. Denitrification recycle pumps sized for 100 percent recycle, operated at 50 percent recycle.

5. Final clarifier overflow rate = 600 gpd/sq ft.

6. 3 lbs methanol per lb of nitrate nitrogen removed.

7. Methanol storage - 30 days supply with a minimum tank size of 500 gallons.

8. Process performance:

Wastewater Characteristics

	In	Out
BOD_5, mg/l	10	10
COD, mg/l	35	45
TSS, mg/l	20	20
Total-P, mg/l	8	8
NH_3-N, mg/l	1	1
NO_3-N, mg/l	19	1

References: 2, 19

FIGURE H-46 SEPARATE DENITRIFICATION (WITH CLARIFIER)

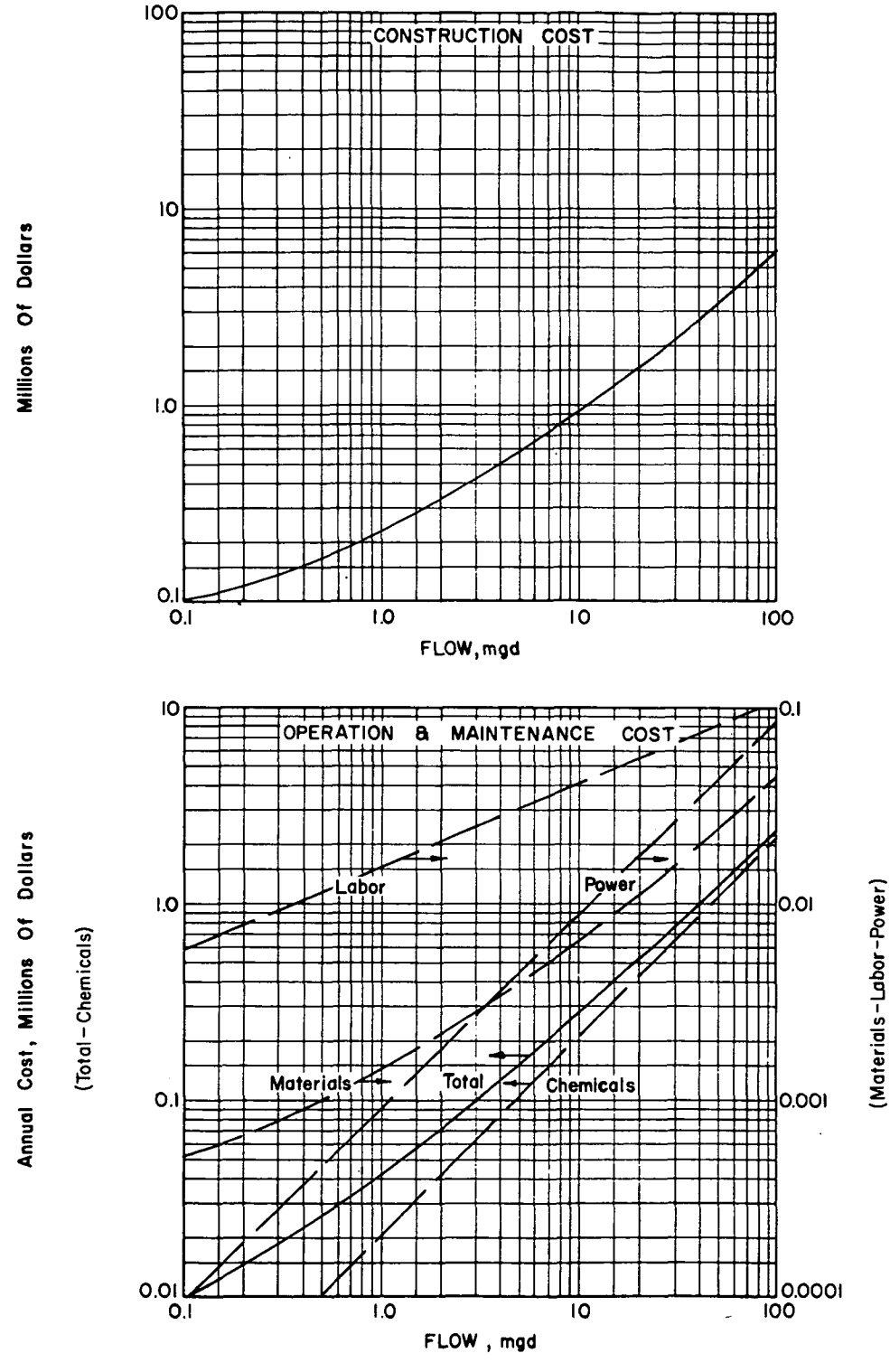


FIGURE H-47

BREAKPOINT CHLORINATION

Service Life: 15 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:

   a. Chlorine storage and feed system.

   b. Lime storage and feed system (1 hp/800 gal mixing).

   c. Chlorine contact tank (30 minutes detention).

   d. Costs do not include a dechlorination facility, but this should generally be included whenever breakpoint chlorination is used. (See Figure H-59.)

2. Chemical costs based on 10 lbs Cl_2/lb NH_3-N in feed (1,500 lbs/mg) and 9 lbs CaO/lb Cl_2 (1,350 lbs/mg).

3. Process performance:

Wastewater Characteristics

	In	Out
NH_3-N, mg/l	18	2

References: 4, 5, 7, 19

FIGURE H-47 BREAKPOINT CHLORINATION

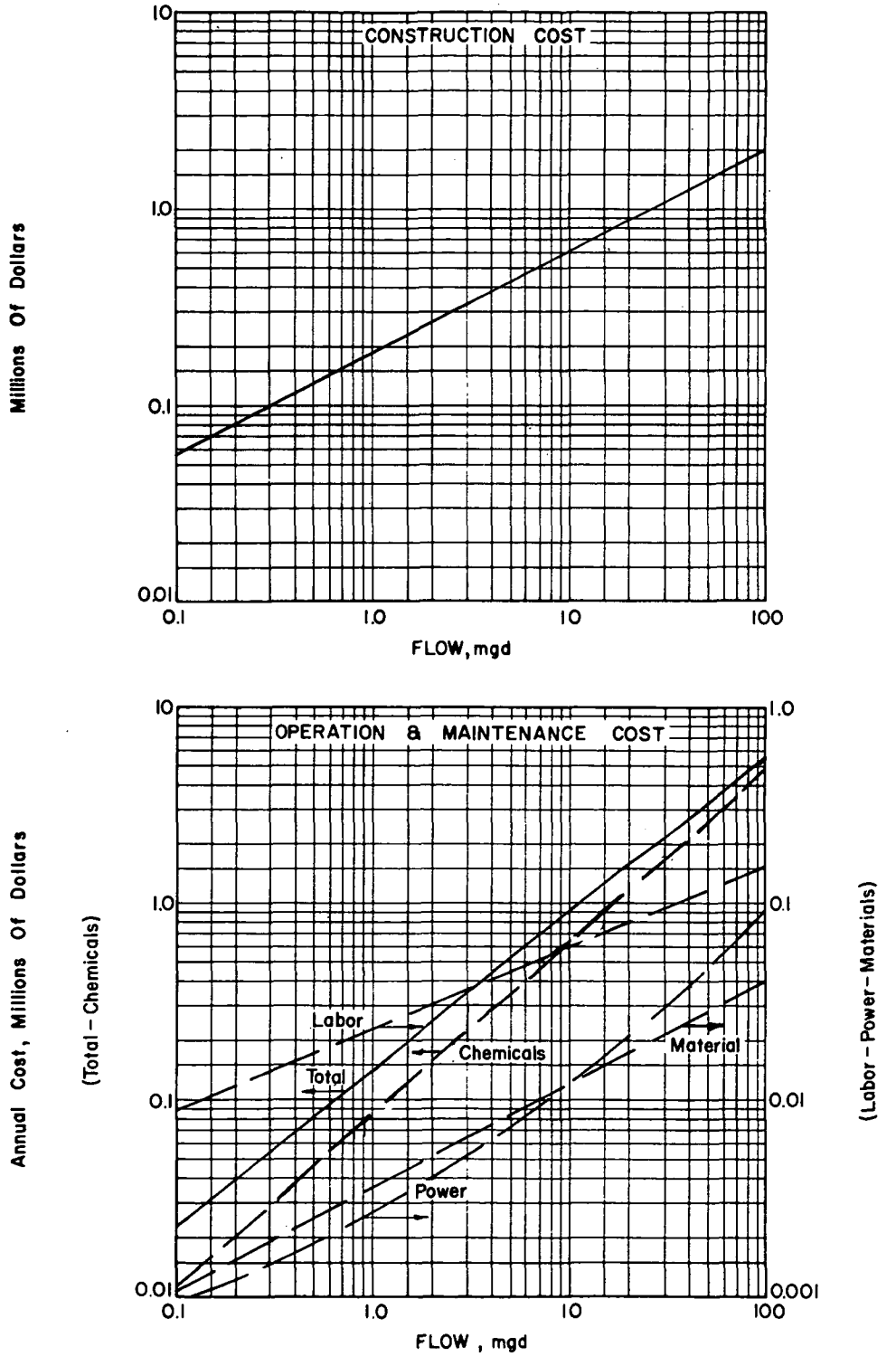


FIGURE H-48

AMMONIA STRIPPING

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:

   a. Ammonia stripping tower - 20' high packed with 1/2 inch diameter schedule 80 PVC pipe at 3 inch centers horizontally with alternate layers placed at right angles at 2 inch centers vertically.

   b. Pumps - 50 ft TDH.

   c. Lime feed facilities to raise pH to 11-11.5 and sulfuric acid facilities to subsequently neutralize the treated effluent.

2. Hydraulic loading: 1.0 gpm/sq ft.

3. Air/water ratio: 400 cu ft/gal.

4. Process performance:

Wastewater Characteristics

	In	Out
NH_3, mg/l	18	3

References: 5, 11, 19, 26

FIGURE H-48 AMMONIA STRIPPING

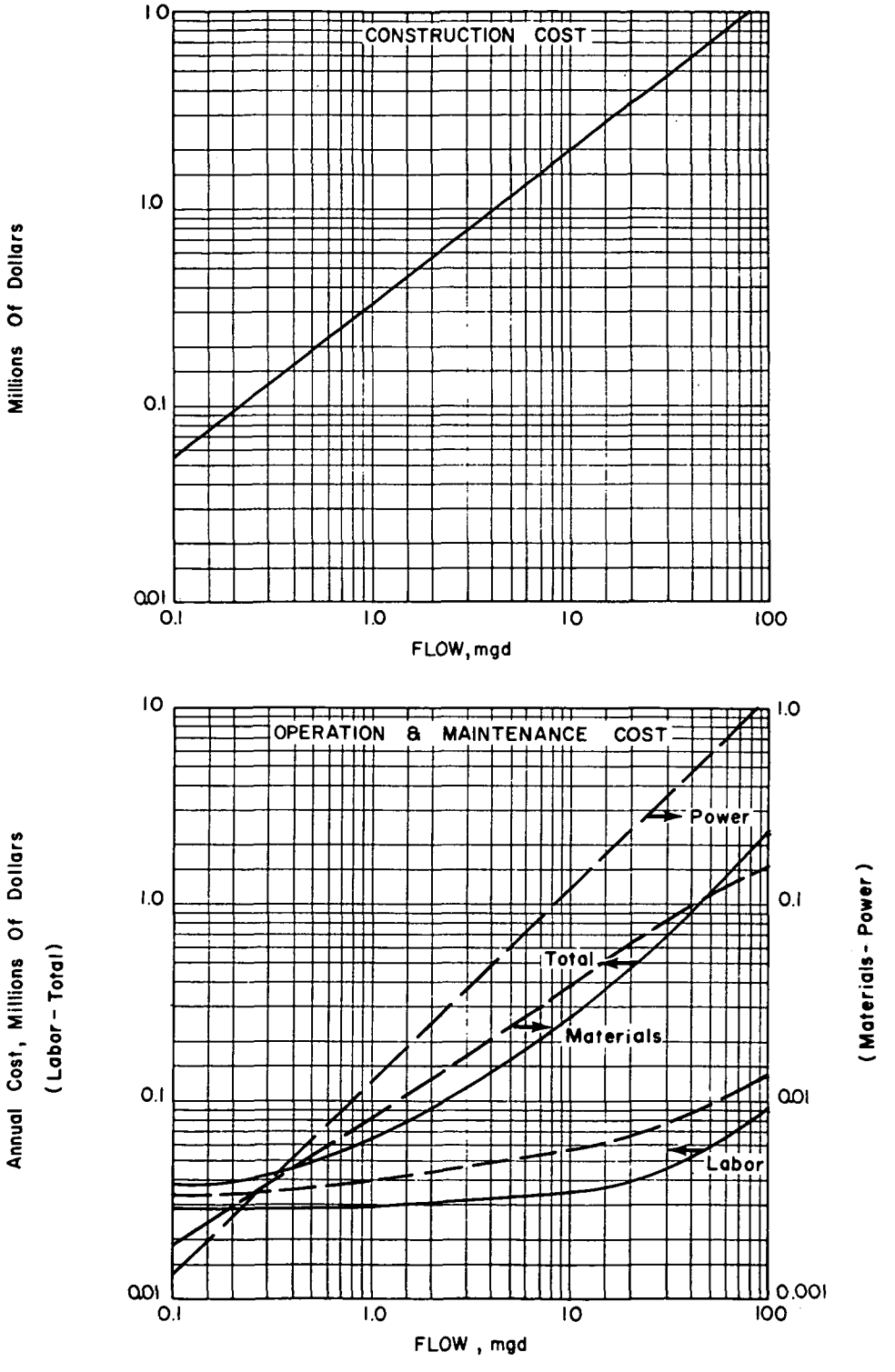


FIGURE H-49

ION EXCHANGE (FOR AMMONIA REMOVAL)

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:

   a. Gravity feed clinoptilolite beds with loading rate of 5.25 gpm/sq ft at 4 ft depth.

   b. Blackwash regeneration facilities at 8 gpm/sq ft.

   c. Influent pumping at 15 ft TDH.

   d. Sodium chloride regeneration facilities using 2 percent NaCl solution, 40 bed volumes/regeneration, and 1 regeneration/ 24 hours.

   e. Closed-loop air stripping tower for regenerant recovery.

   f. Clarification - softening facility for spent regenerant.

2. Chemical costs include: makeup clinoptilolite and makeup regenerant.

3. Ammonia sulfate produced by this system may be sold to defer O & M costs; however, it was not included in this cost estimate.

4. Process performance:

Wastewater Characteristics

	In	Out
NH_3-N, mg/l	18	1

References: 19, 49

FIGURE H-49 ION EXCHANGE (FOR AMMONIA REMOVAL)

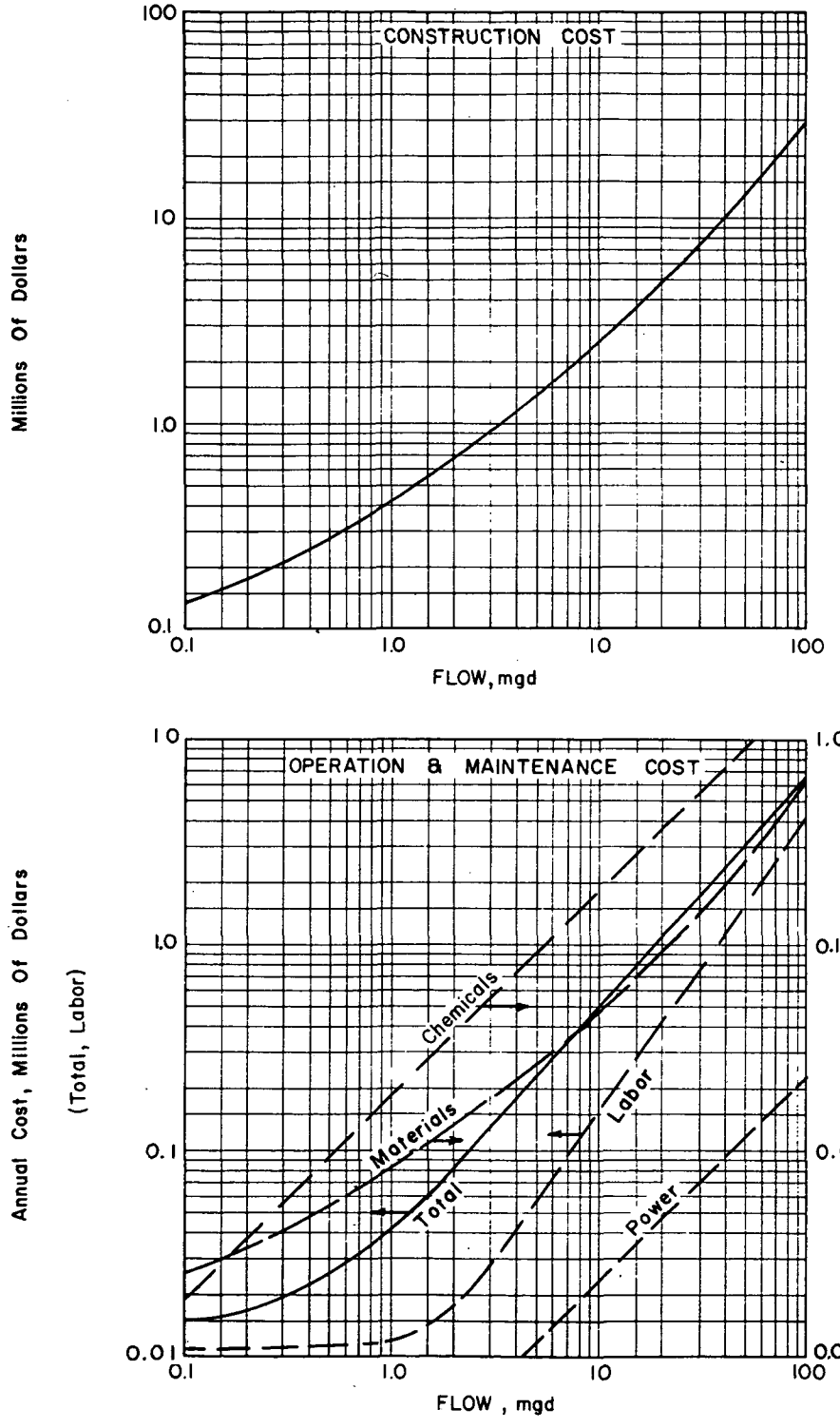


FIGURE H-50

PHOSTRIP

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:
   a. Stripper (10 hours detention at 50 percent of return sludge).
   b. Flash mixer.
   c. Flocculator - clarifier.
   d. Thickeners.
   e. Lime feed and storage facilities.
2. Lime cost based on 225 lbs/mg.
3. Process applies only to activated sludge.
4. Process performance:

Wastewater Characteristics		
	In	Out
Total-P, mg/l	9	2

Reference: 20

FIGURE H-50 PHOSTRIP

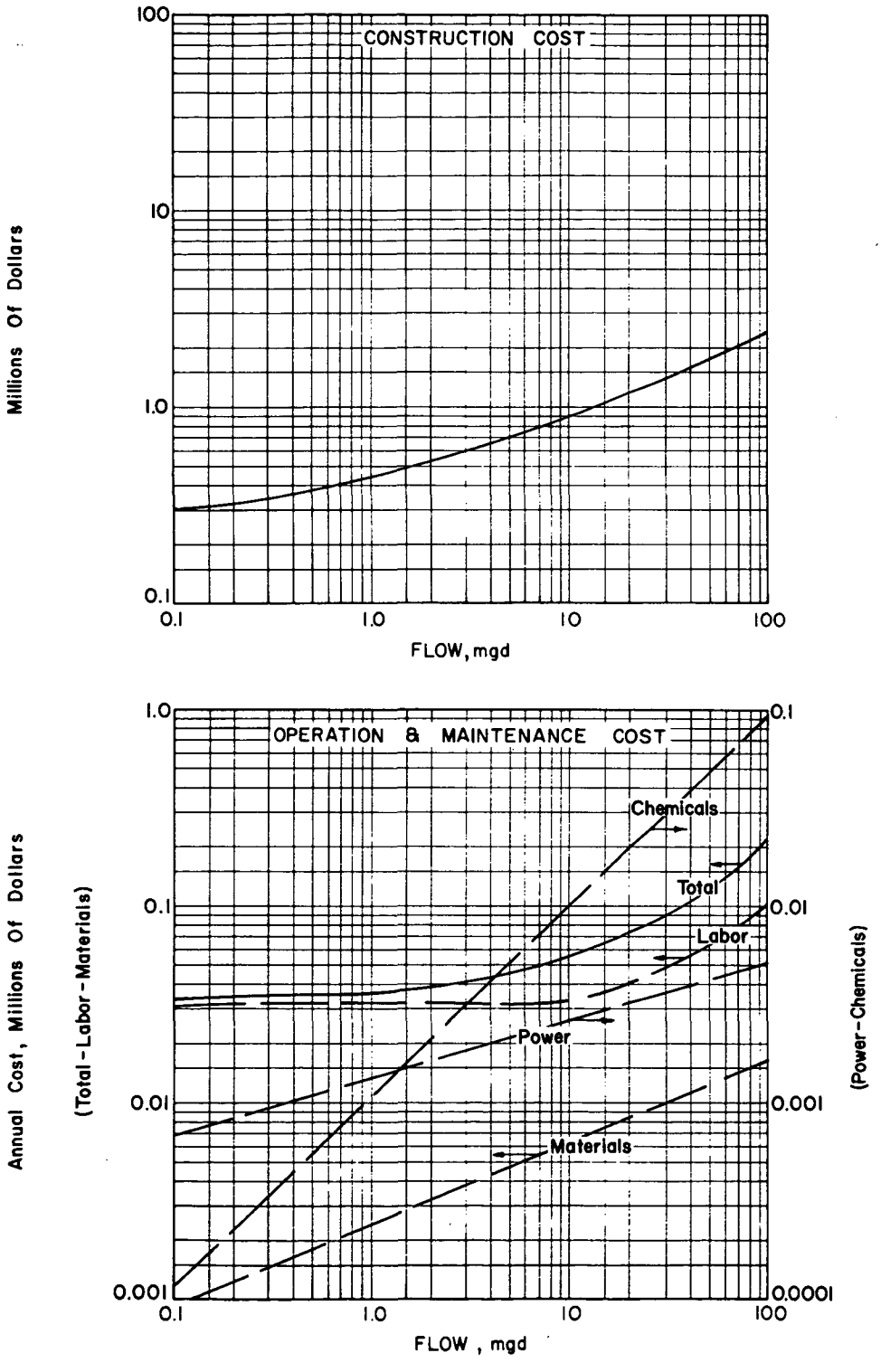


FIGURE H-51

POLYMER FEED SYSTEM

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. System includes chemical storage, chemical feeding, rapid mix; flocculation and settling are included in a separate curve. The system is not for sludge conditioning.

2. Polymer dosage at 1 mg/l (8.34 lbs/mg).

3. Construction costs include:

   a. Use of dry polymer.

   b. Feedstock solution - 0.25 percent.

   c. Piping and building to house the feeding equipment and to store bags are included.

   d. Systems for 1 mgd plant size and smaller, use manual procedures. Two systems of tanks and feeders are included.

   e. For the 10 mgd plant size, the cost includes two feeders and mixing tanks, one day tank, and two solution feeders.

   f. For the 100 mgd plant size, the cost includes four feeders and mixing tanks, two holding tanks, and ten solution feeders.

   g. The rapid-mix tank is concrete, and is equipped with stainless steel mixer and handrails.

   h. For the 0.1 mgd plant size, no separate building is required. Cost includes manual operation with feeder, mix tank, solution feeder, and holding tank.

References: 6, 9, 18, 25

FIGURE H-51 POLYMER FEED SYSTEM

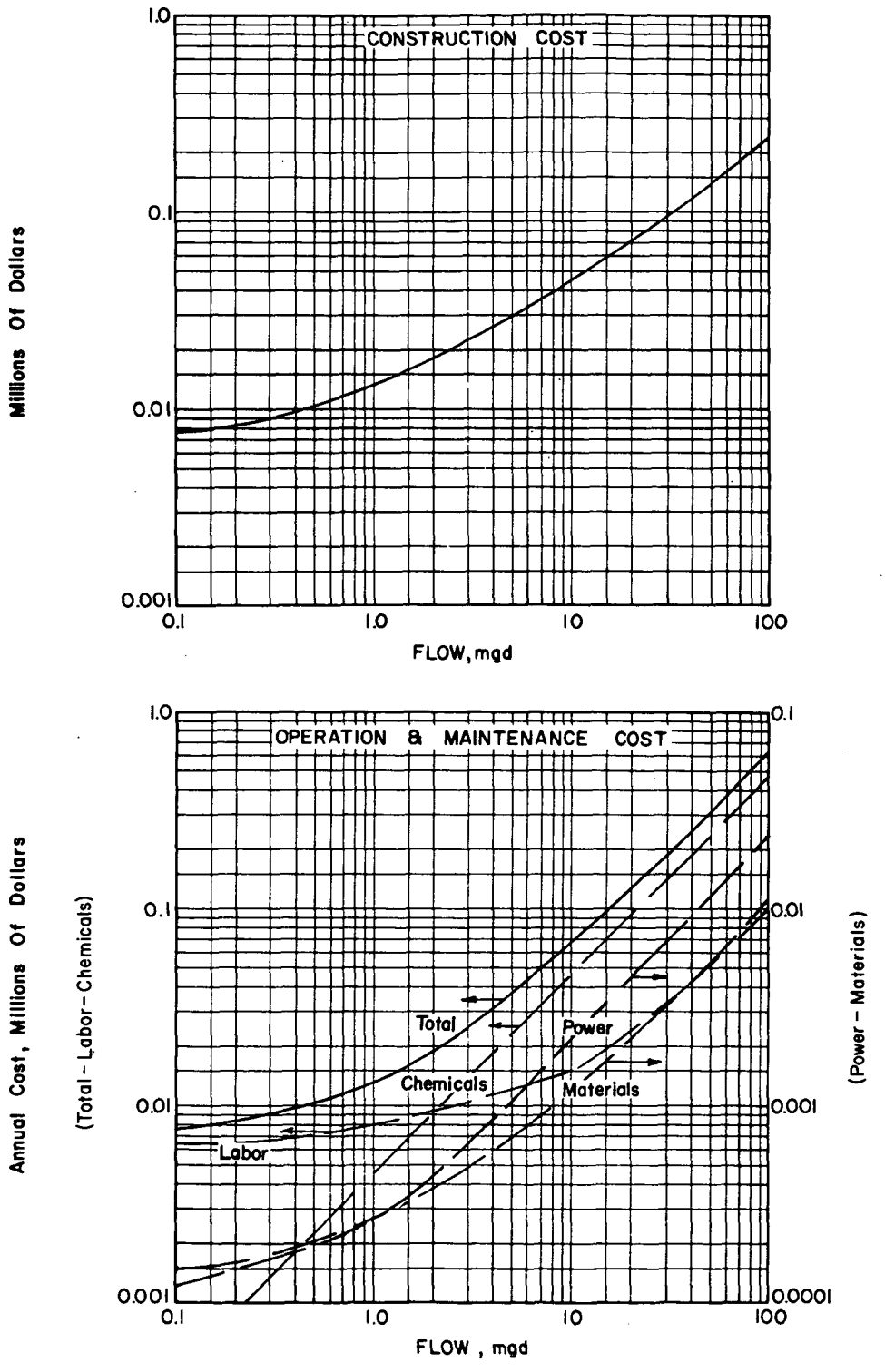


FIGURE H-52

MINERAL ADDITION

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Alum dosage of 200 mg/l and ferric chloride dosage of 100 mg/l. Phosphorus removal - for other dosages see adjustments below.

2. The rapid-mix tank is constructed of concrete, and multiple basins are used for volumes greater than 1,500 cubic feet.

3. Costs include:

   a. Liquid chemicals (8.3 percent Al_2O_3 and 35 percent $FeCl_3$).

   b. Chemical feed equipment sized for twice the average feed rate.

   c. Storage of at least 15 days.

   d. Price of building is included except for plants with a capacity of less than 1 mgd.

   e. Rapid-mix tank includes stainless steel mixer.

Adjustment Factor:

To adjust costs enter curve at effective flow (Q_E)

$$\text{Alum: } Q_E = Q_{DESIGN} \times \frac{\text{Alum Dose}}{200 \text{ mg/l}}$$

$$FeCl_3 = Q_E = Q_{DESIGN} \times \frac{FeCl_3 \text{ Dose}}{100 \text{ mg/l}}$$

FIGURE H-52[1] MINERAL ADDITION (ALUM)

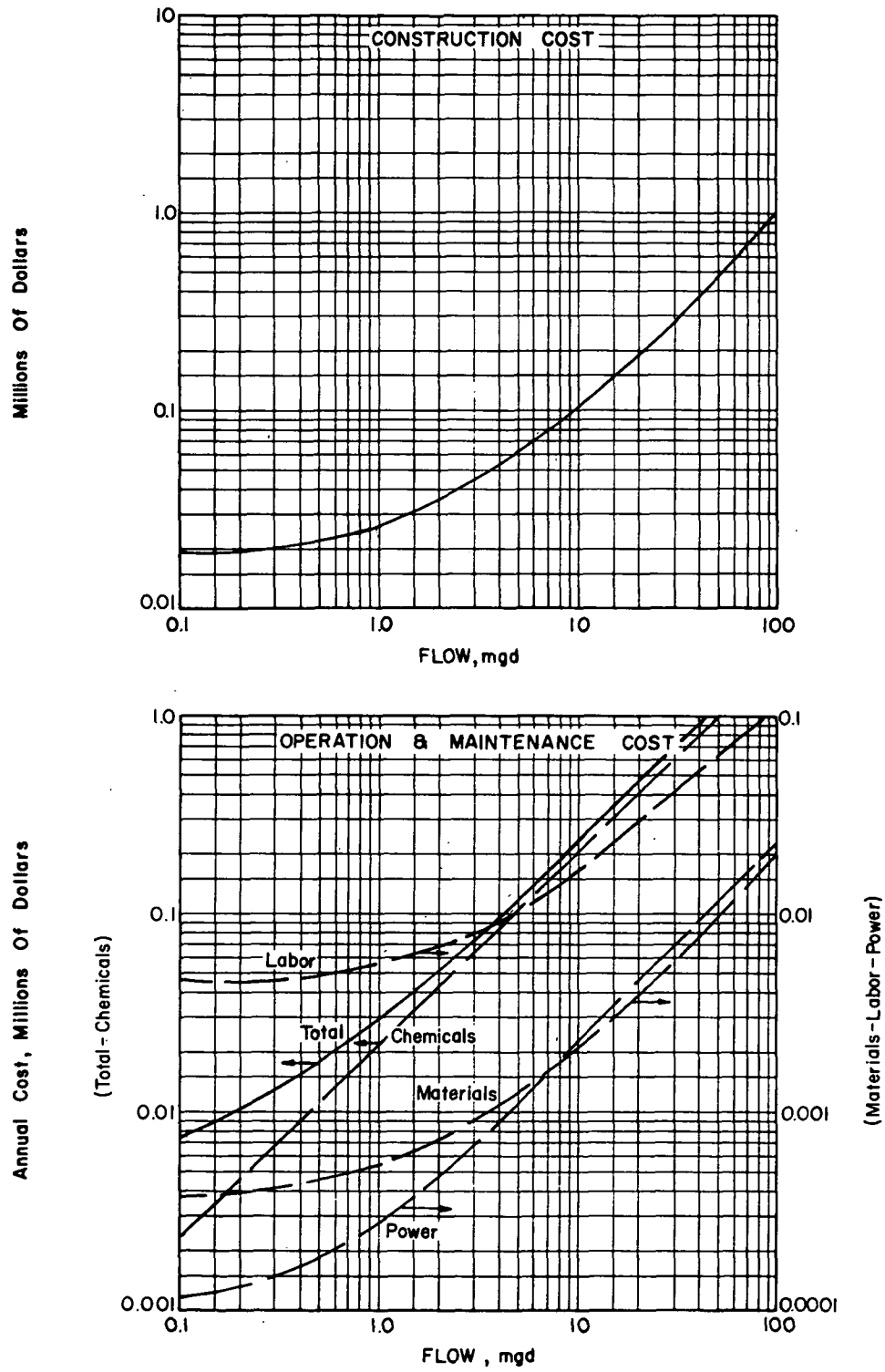


FIGURE H-52

MINERAL ADDITION

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Alum dosage of 200 mg/l and ferric chloride dosage of 100 mg/l. Phosphorus removal - for other dosages see adjustments below.

2. The rapid-mix tank is constructed of concrete, and multiple basins are used for volumes greater than 1,500 cubic feet.

3. Costs include:

   a. Liquid chemicals (8.3 percent Al_2O_3 and 35 percent $FeCl_3$).

   b. Chemical feed equipment sized for twice the average feed rate.

   c. Storage of at least 15 days.

   d. Price of building is included except for plants with a capacity of less than 1 mgd.

   e. Rapid-mix tank includes stainless steel mixer.

Adjustment Factor:

To adjust costs enter curve at effective flow (Q_E)

Alum: $Q_E = Q_{DESIGN} \times \frac{\text{Alum Dose}}{200 \text{ mg/l}}$

$FeCl_3 = Q_E = Q_{DESIGN} \times \frac{FeCl_3 \text{ Dose}}{100 \text{ mg/l}}$

FIGURE H-52[2] MINERAL ADDITION (FERRIC CHLORIDE)

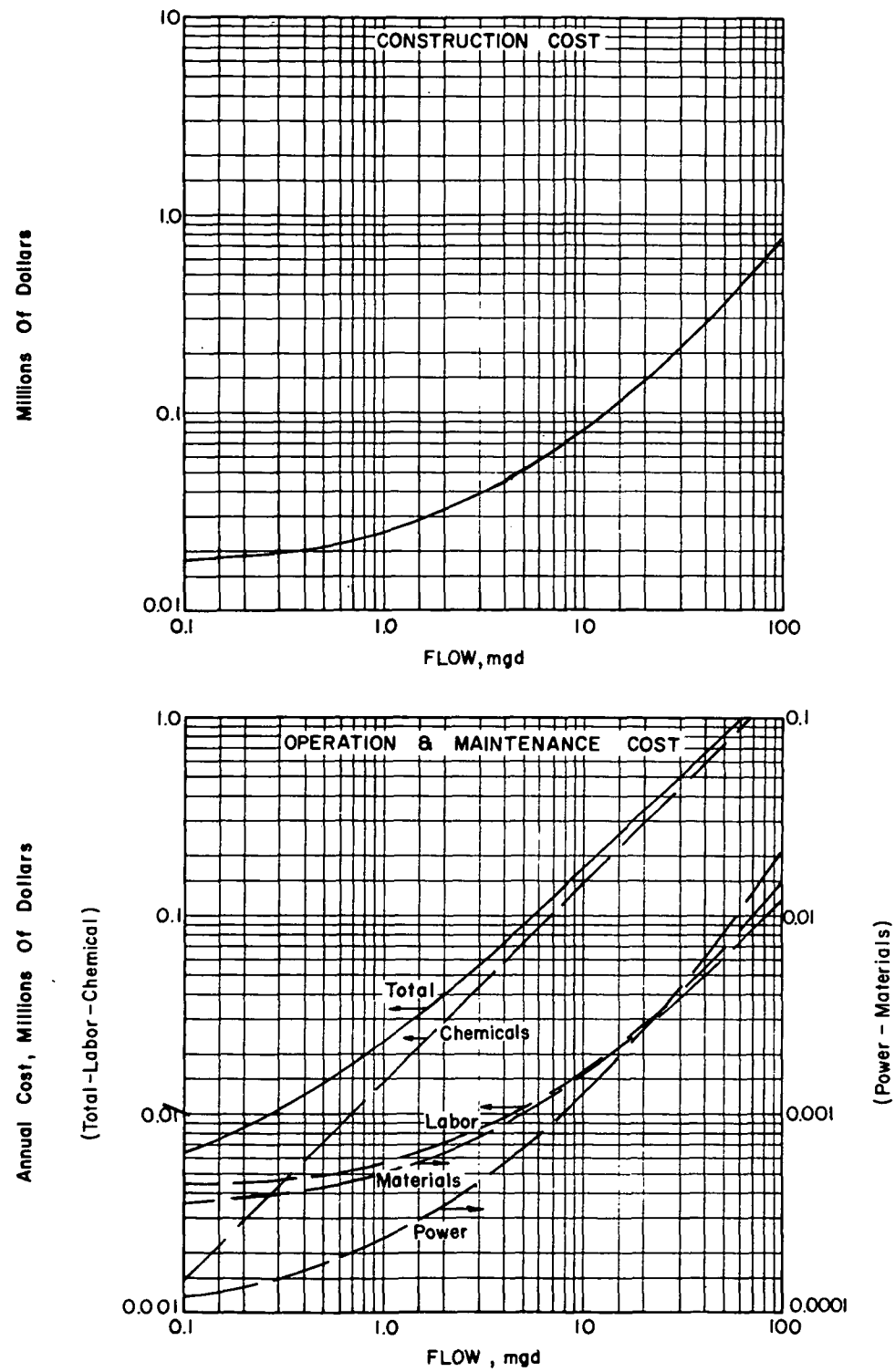


FIGURE H-53

TWO-STAGE TERTIARY LIME TREATMENT
(WITHOUT RECALCINATION)

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Typical secondary effluent as feed to two-stage lime treatment.
2. Lime dosage rate = 400 mg/1 or 3,340 lb/mg as CaO.
3. Clarifier overflow rate = 1,000 gpd/sq ft.
4. Construction cost includes: lime storage and feed facilities, rapid-mix facilities, flocculator/clarifiers, flow and pH controls, and recarbonization facilities.
5. Costs do not include recalcination facilities (see Figure H-54).
6. Process performance:

Wastewater Characteristics

	In	Out
BOD_5, mg/1	20	8
COD, mg/1	45	18
TSS, mg/1	20	20
Total-P, mg/1	8	0.5 (with filtration)
		1.5 (without filtration)

Adjustment Factor:

To adjust costs, enter curves at effective flow (Q_E)

Construction Cost: $Q_E = Q_{DESIGN} \times \frac{1000}{\text{New Design Overflow Rate}}$

Chemical Costs (O&M): $Q_E = Q_{DESIGN} \times \frac{\text{New Lime Dosage}}{400}$

References: 4, 5, 7, 8, 11, 18, 26, 31

FIGURE H-53 TWO STAGE TERTIARY LIME TREATMENT
(WITHOUT RECALCINATION)

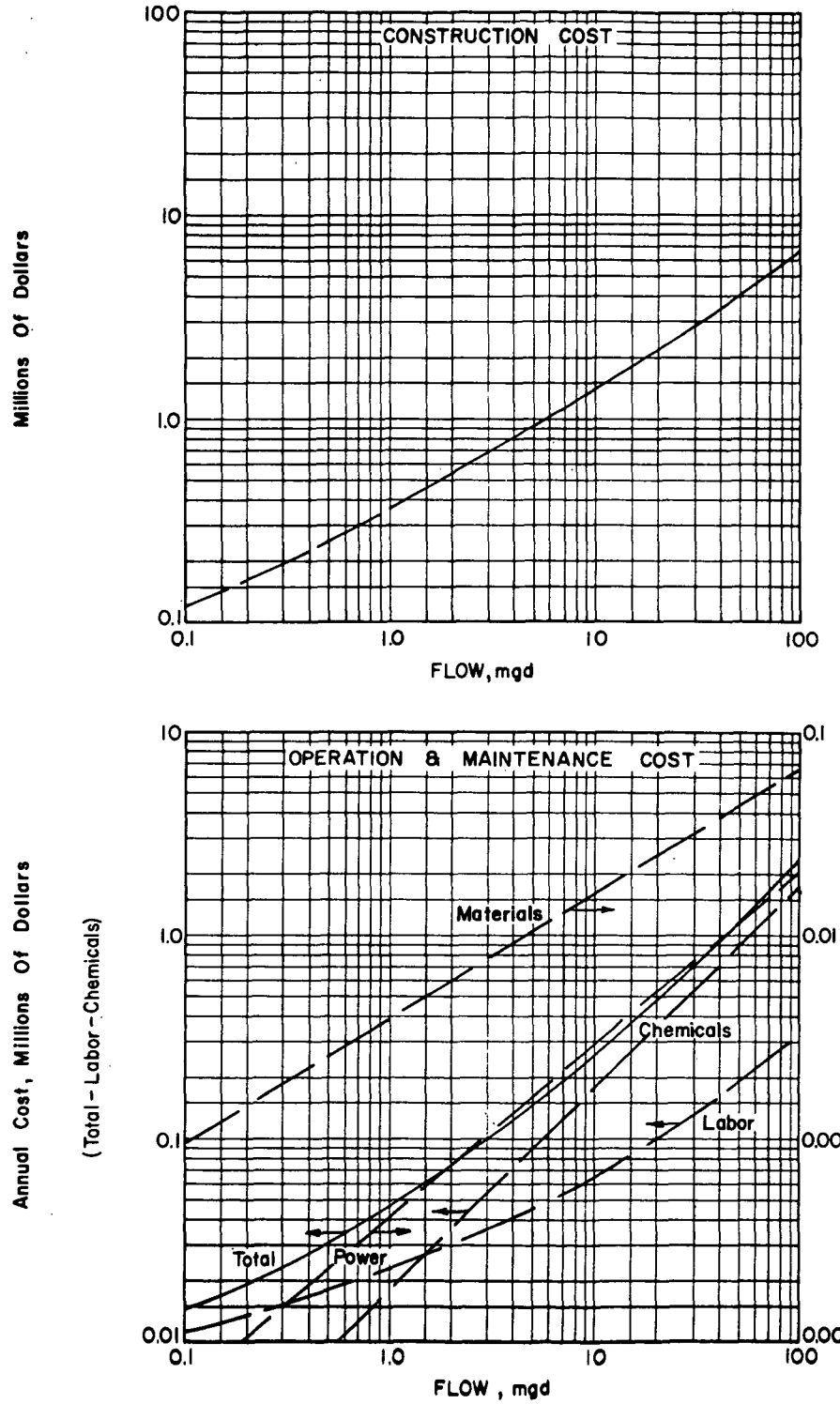


FIGURE H-54

LIME RECALCINATION

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Quantity of lime sludge: 4,500 lb/mg; 30 percent solids (from two-stage tertiary lime treatment).

2. Operations:

Flow	Wet Solids lbs/hr (24 hrs/day)	Days/Week Operating	Hours/Day Operating	Furnace Loading lb/hr	Hearth Area sq ft
0.1	62.5	1	20	525	112
1.0	625	6	16	1,095	256
10.0	6,250	7	20	7,500	2 at 760
100.0	62,500	7	20	75,000	3 at 5,070

3. Fuel requirements (No. 2 fuel oil): 129,000 gal/yr/mgd.

4. Construction costs included: recalcination furnace, sludge conveyors, storage, hoppers, building.

Adjustment Factor:

When lime recalcination is used, reduce the chemical cost of two-stage tertiary lime treatment (Figure H-53) by 70 percent.

References: 4, 7, 8, 11, 18

FIGURE H-54 LIME RECALCINATION

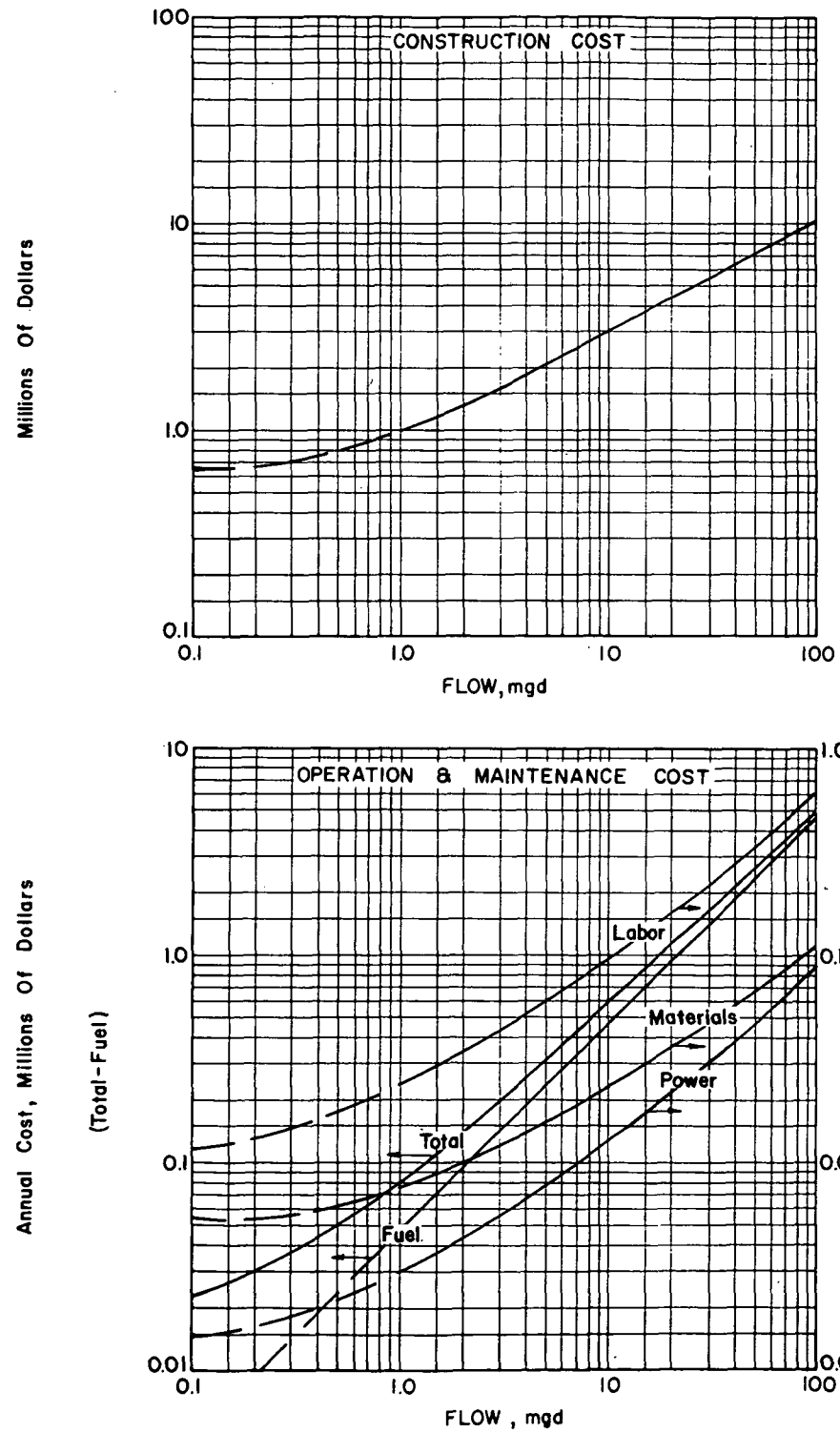


FIGURE H-55

MICROSCREENING

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: tanks, drums, screens, backwash equipment, drive motors, and building. Instrumentation for automatic operation is included.

2. Hydraulic loading = 2.5 gpm/sq ft at average flow.

3. Screen mesh: 25 microns.

4. Peripheral drum speed: 15 fpm at 3-in. head loss.

5. Backwash 3 percent of throughput at 25 psi.

6. Process performance:

Wastewater Characteristics

	In	Out
TSS, mg/l	20	12

Adjustment Factor:

To determine costs for hydraulic loadings different than that above, enter curve at effective flow (Q_E)

$$Q_E = Q_{DESIGN} \times \frac{2.5 \text{ gpm/sq ft}}{\text{New Design Hydraulic Loading}}$$

References: 5, 8, 11, 24, 26

FIGURE H-55 MICROSCREENING

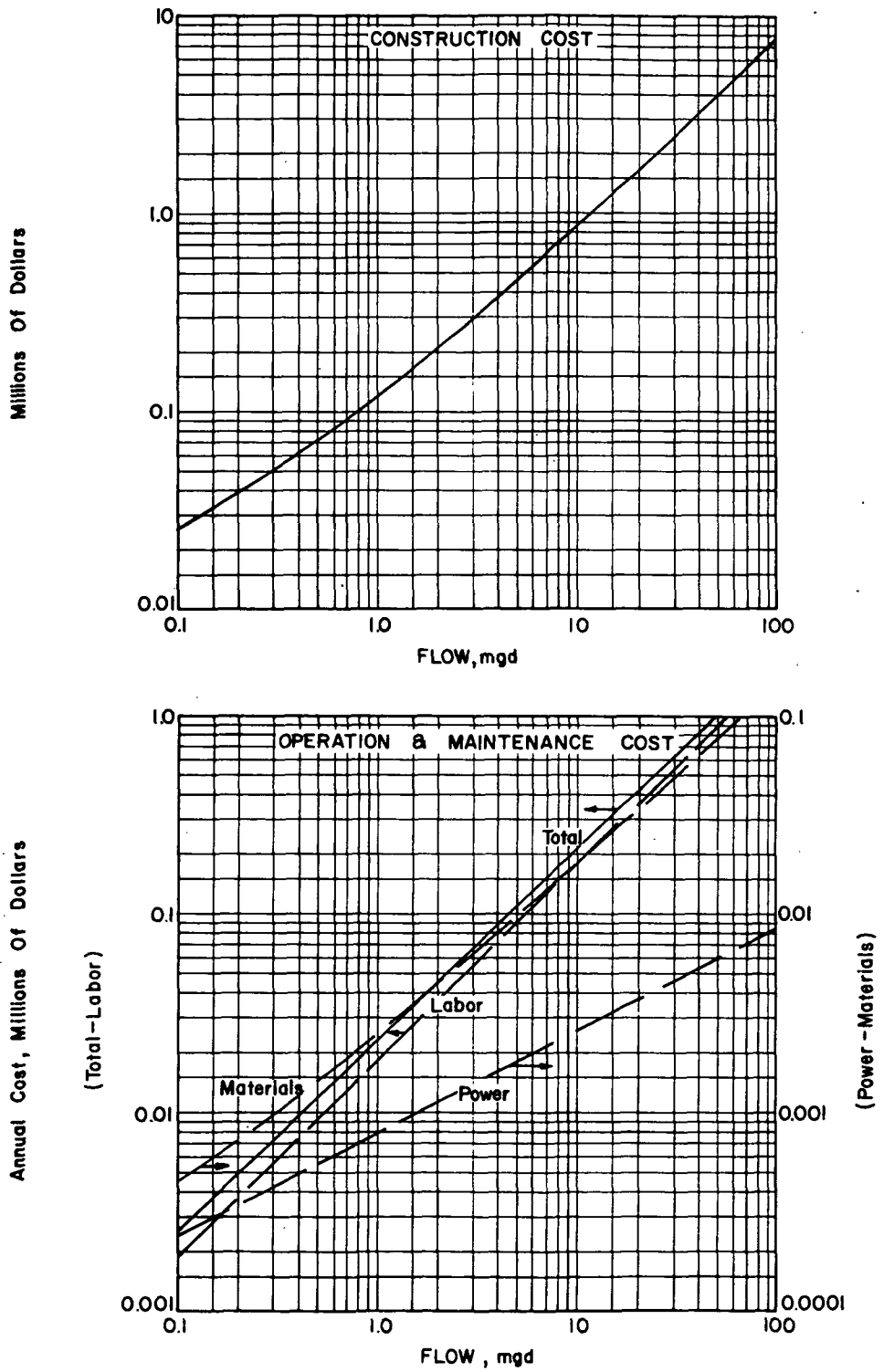


FIGURE H-56

GRAVITY FILTRATION (DUAL-MEDIA)

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Feed rate = 4 gpm/sq ft at average flow.
2. Backwash rate = 15 gpm/sq ft with air scour.
3. Backwash holding tank = capacity of 2 backwash cycles.
4. Construction costs include facilities for backwash storage, all feed and backwash pumps, piping, and building.
5. Process performance:

Wastewater Characteristics

	In	Out
BOD_5, mg/l	20	15
COD, mg/l	40	30
TSS, mg/l	20	5
Total-P, mg/l	2	1

Adjustment Factor:

To adjust for loading rates other than above, enter curve at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{4 \text{ gpm/sq ft}}{\text{New Design Filtration Rate}}$$

References: 4, 7, 24

FIGURE H-56 GRAVITY FILTRATION (DUAL-MEDIA)

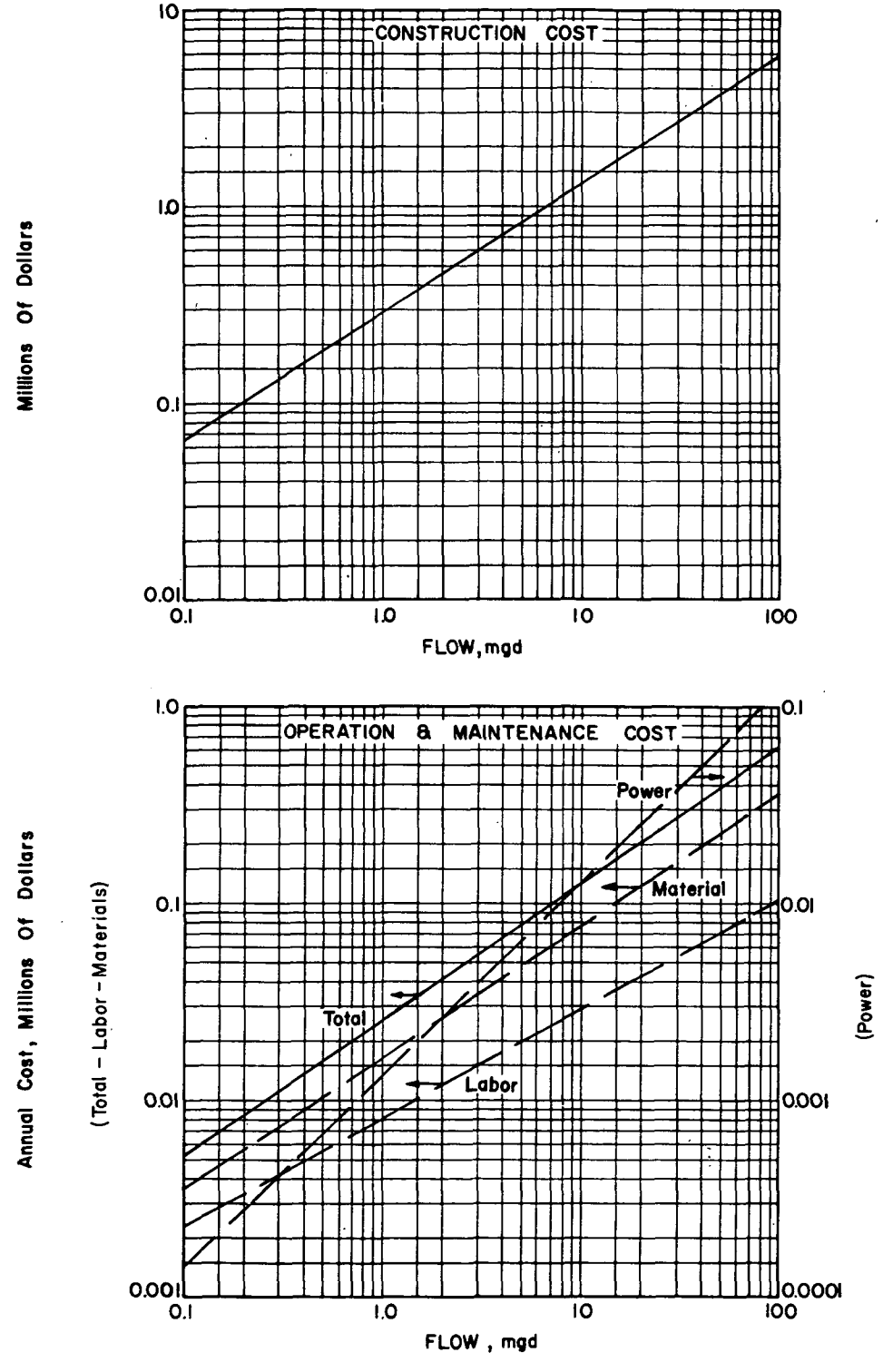


FIGURE H-57

ACTIVATED CARBON ADSORPTION

Service Life: 35 years

Relative Reliability: Level B

Design Basis:

1. Contact time = 30 minutes (empty bed basin).
2. Regeneration furnace loading = 80 lb/sq ft/day.
3. Construction costs include:
   a. For flows above breakpoint:
      1. Carbon columns (minimum of 3).
      2. Virgin & spent carbon storage tanks.
      3. Feed and backwash pumps and all necessary piping.
      4. Operations building.
      5. Regeneration furnace (Multiple Hearth Furnace).
      6. Slurry tank and steam generator.
   b. For flow below breakpoint:
      1. Items a.1-a.4 - no regeneration.
4. Carbon Dosage

   H-57[1] - Tertiary Effluent: for flow less than 3.0 mgd = 30 lb/mg (without regeneration); for flow greater than 3.0 mgd = 3 lb/mg (10 percent make up) (with regeneration)

   H-57[2] - Physical/Chemical: for flow less than 1.5 mgd = 1,650 lb/mg (without regeneration); for flow greater than 1.5 mgd = 165 lb/mg (10 percent make up) (with regeneration).
5. Regeneration heat requirement: 7,000 Btu/lb of carbon regeneration.
6. Effluent from activated sludge, two-stage lime treatment, and gravity dual-media filters as feed to carbon adsorbers.
7. Process performance:

Wastewater Characteristics		
	In	Out
BOD_5, mg/l	15	3
COD, mg/l	30	10
TSS, mg/l	5	5

References: 4, 5, 6, 7, 29

FIGURE H-57[1] ACTIVATED CARBON ADSORPTION

(TERTIARY EFFLUENT)

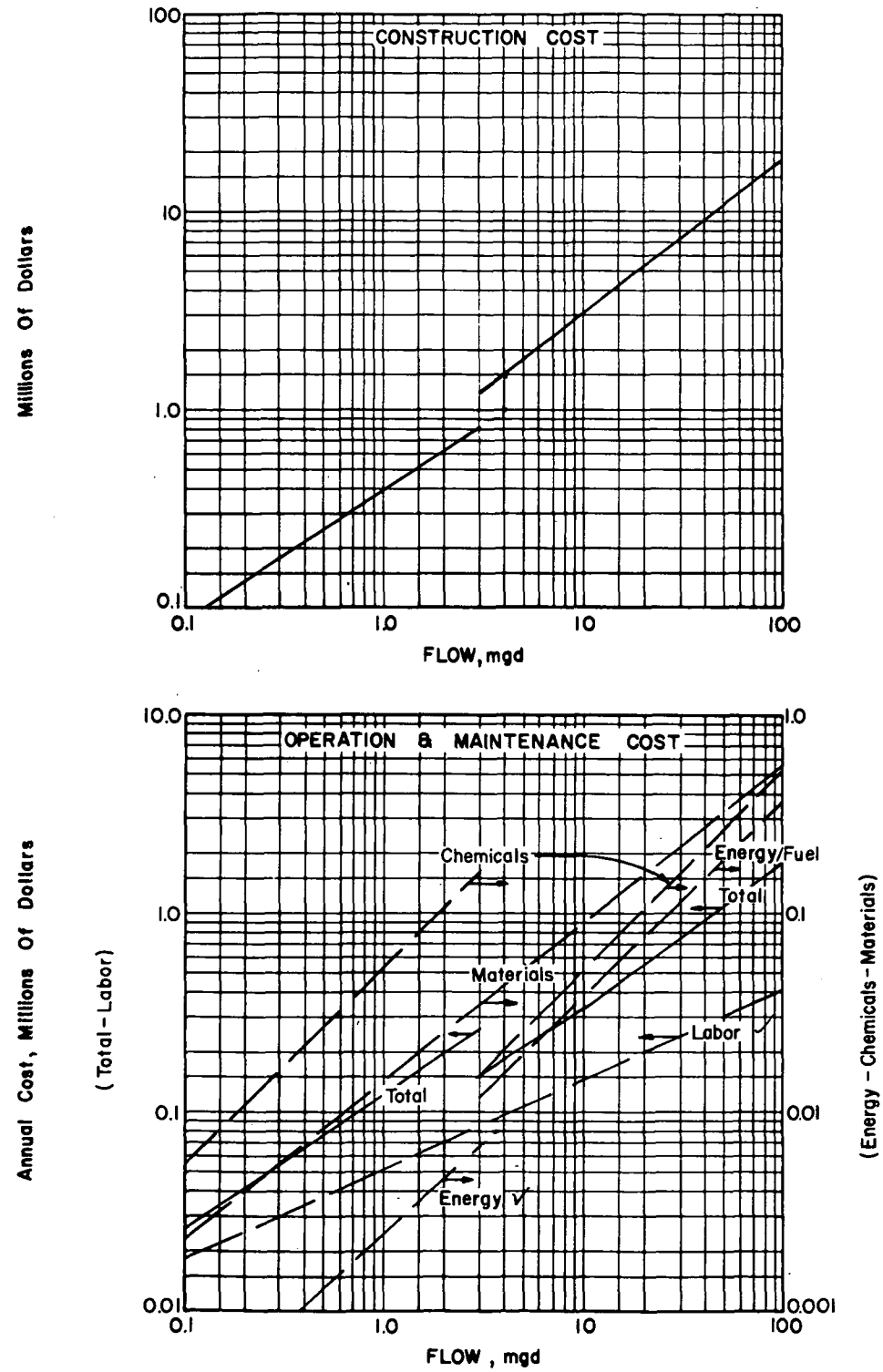


FIGURE H-57

ACTIVATED CARBON ADSORPTION

Service Life: 35 years

Relative Reliability: Level B

Design Basis:

1. Contact time = 30 minutes (empty bed basin).
2. Regeneration furnace loading = 80 lb/sq ft/day.
3. Construction costs include:
   a. For flows above breakpoint:
      1. Carbon columns (minimum of 3).
      2. Virgin & spent carbon storage tanks.
      3. Feed and backwash pumps and all necessary piping.
      4. Operations building.
      5. Regeneration furnace (Multiple Hearth Furnace).
      6. Slurry tank and steam generator.
   b. For flow below breakpoint:
      1. Items a.1-a.4 - no regeneration.
4. Carbon Dosage

   H-57[1] - Tertiary Effluent: for flow less than 3.0 mgd = 30 lb/mg (without regeneration); for flow greater than 3.0 mgd = 3 lb/mg (10 percent make up) (with regeneration)

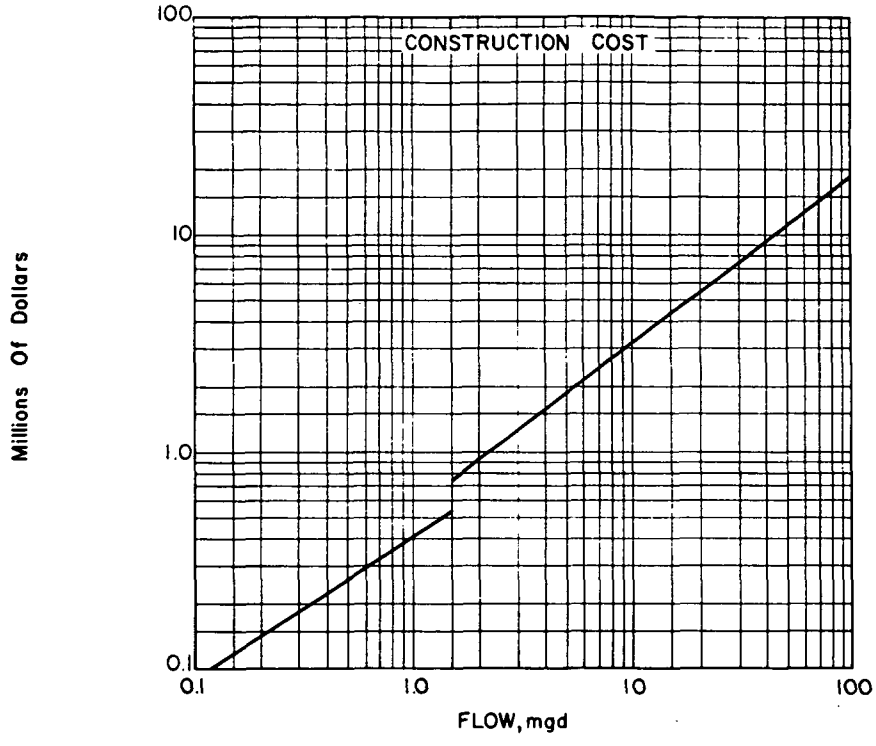
   H-57[2] - Physical/Chemical: for flow less than 1.5 mgd = 1,650 lb/mg (without regeneration); for flow greater than 1.5 mgd = 165 lb/mg (10 percent make up) (with regeneration).
5. Regeneration heat requirement: 7,000 Btu/lb of carbon regeneration.
6. Effluent from activated sludge, two-stage lime treatment, and gravity dual-media filters as feed to carbon adsorbers.
7. Process performance:

Wastewater Characteristics		
	In	Out
BOD_5, mg/l	15	3
COD, mg/l	30	10
TSS, mg/l	5	5

References: 4, 5, 6, 7, 29

FIGURE H-57[2] ACTIVATED CARBON ADSORPTION

(PHYSICAL/CHEMICAL)



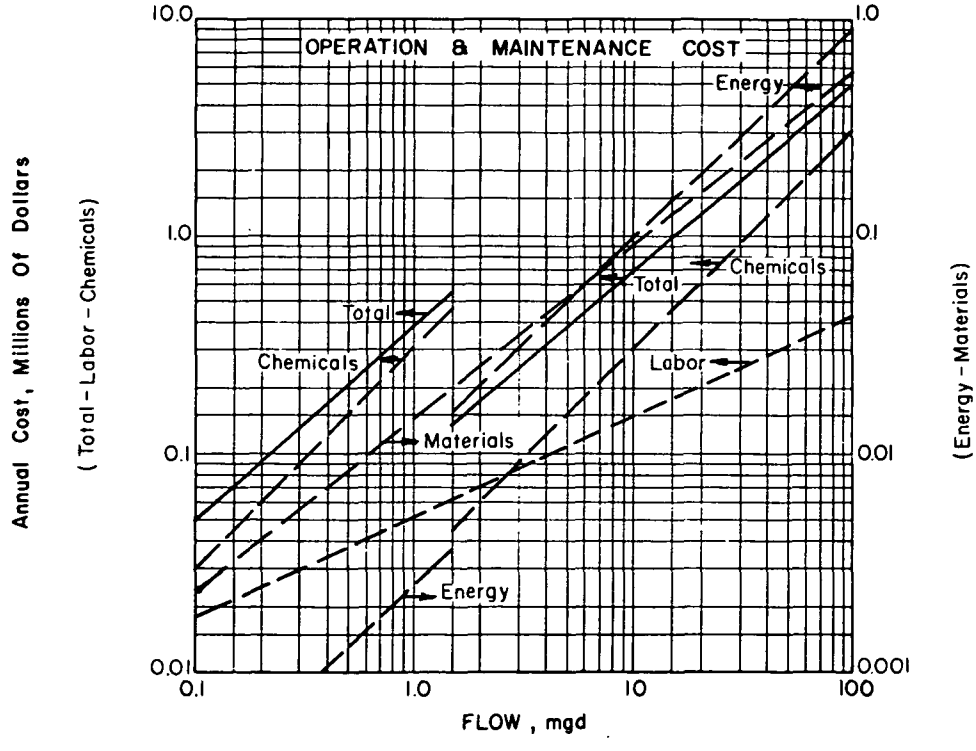


FIGURE H-58

CHLORINATION (DISINFECTION)

Service Life: 15 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:
   a. Chlorination building.
   b. Chlorine storage and handling facilities including hoists, etc.
   c. Chlorinators.
   d. Plug flow contact chamber.
2. Average chlorine dosage, 10 mg/l.
3. Chlorination contact time, 30 min for average flow.
4. Chlorine residual, 1 mg/l.

Adjustment Factor:

To adjust costs, enter curves at effective flow (Q_E)

Chemical Cost (O&M): $Q_E = Q_{DESIGN} \times \frac{\text{New Chlorine Dosage}}{10 \text{ mg/l}}$

References: 3, 7, 9

FIGURE H-58 CHLORINATION (DISINFECTION)

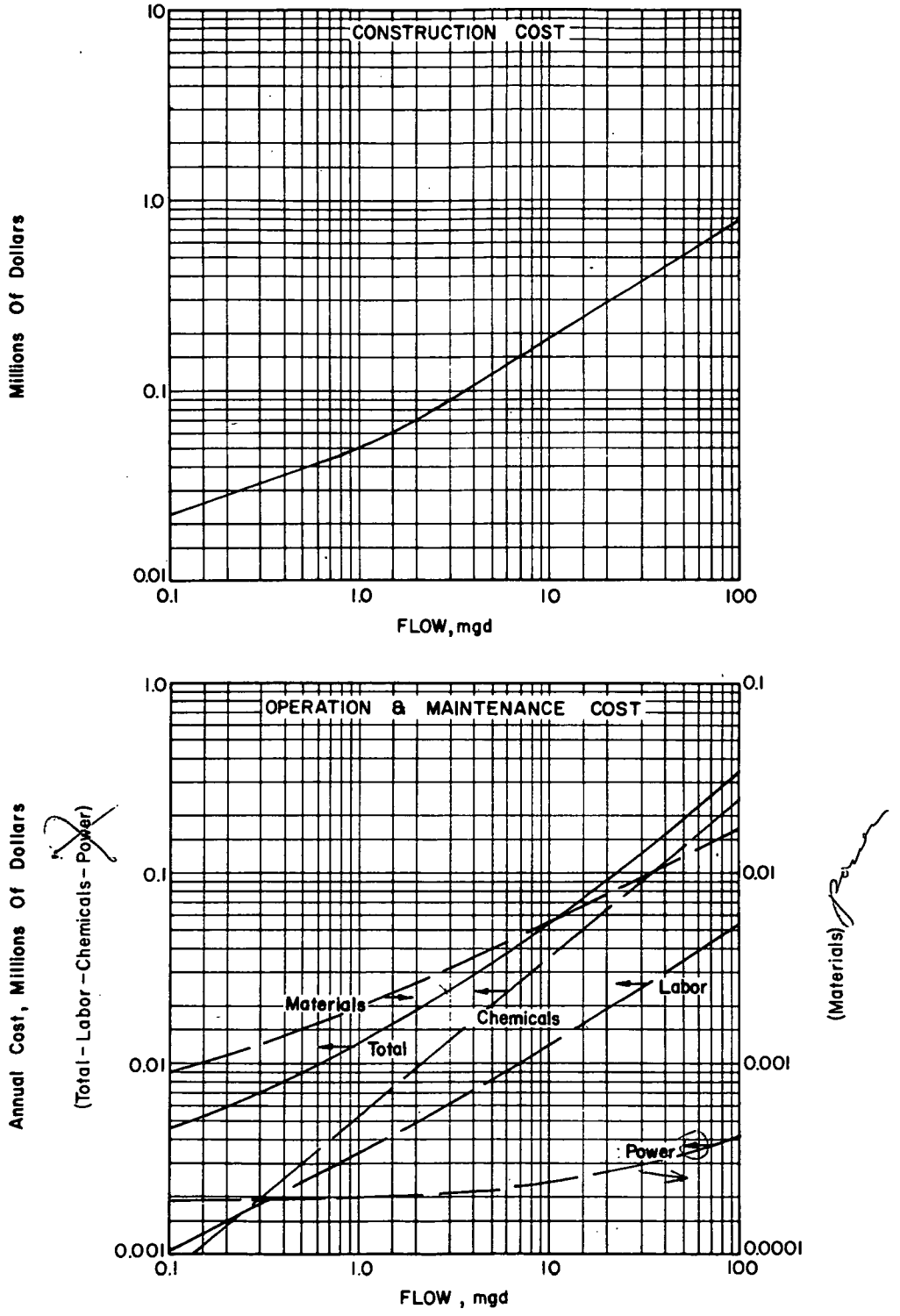


FIGURE H-59

DECHLORINATION USING SULFUR DIOXIDE

Service Life: 15 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include SO_2 feed facilities, reaction tank (1 minute detention), mixer, and storage facilities; building space not included.

2. SO_2 costs based on 20 lb/mg (1 mg/l SO_2 required per mg/l chlorine residual.

Adjustment Factor:

To adjust costs, enter curve at effective flow (Q_E)

Chemical Costs (O&M): $Q_E = Q_{DESIGN} \times \frac{\text{New } SO_2 \text{ Dosage}}{1 \text{ mg/l } SO_2}$

References: 1, 5

FIGURE H-59 DECHLORINATION USING SULFUR DIOXIDE

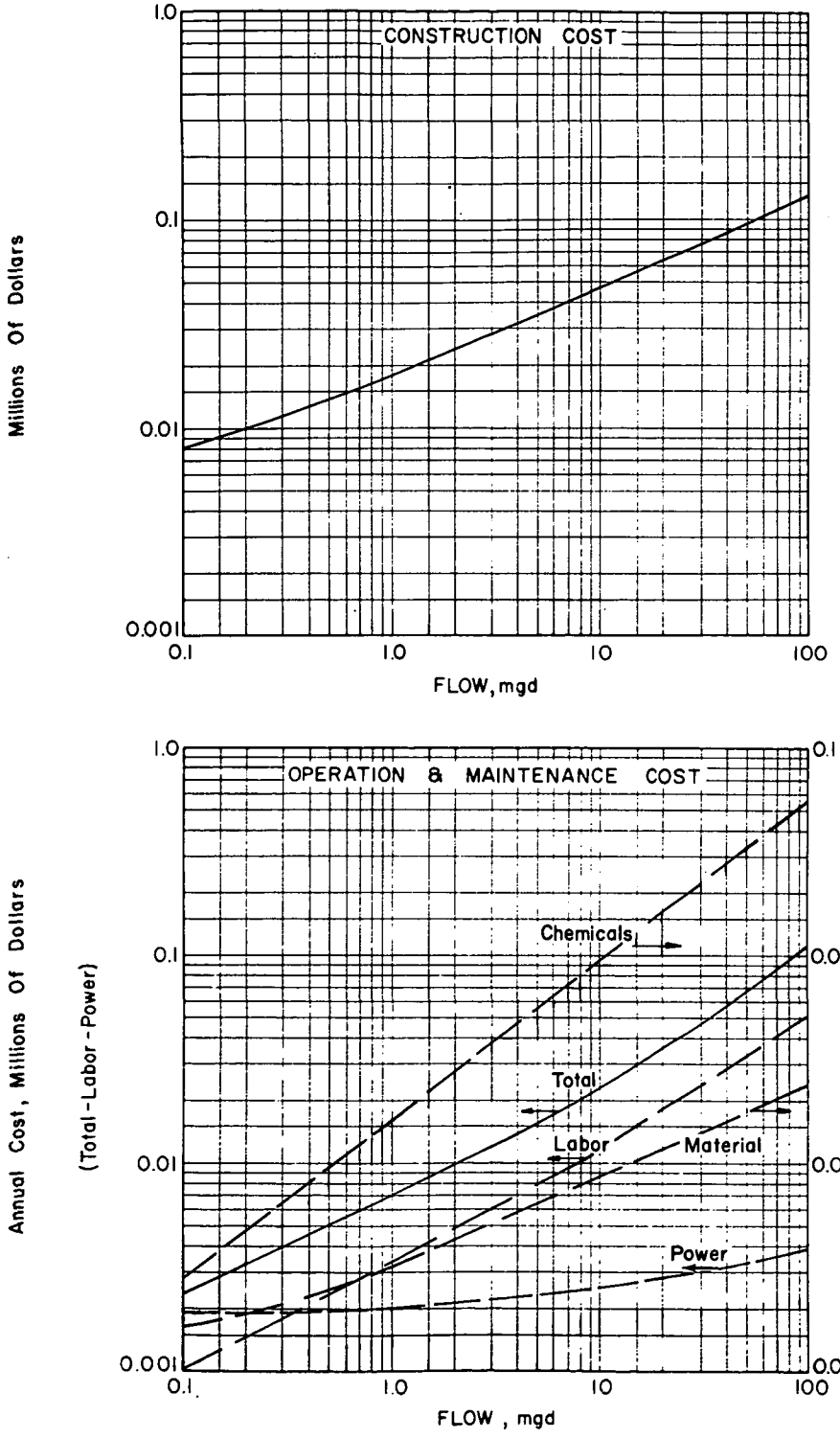


FIGURE H-60

OZONATION (AIR- AND O_2-GENERATED)

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:

   Air Systems:

   a. Air blowers.
   b. Air filters.
   c. Air coolers.
   d. Air dryers.
   e. Ozonator.
   f. Ozone injector.
   g. Ozone-water contact chamber.
   h. Aeration chamber.

   Oxygen Systems:

   a. Oxygen storage.
   b. Ozonator.
   c. Ozone injector.
   d. Ozone-water contact chamber.
   e. Aeration chamber.

2. Oxygen requirements = 3 lb O_2/lb O_3.

3. Ozone dosage = 8 mg/l.

Adjustment Factor:

To adjust costs, enter curve at effective flow (Q_E)

Chemical Costs (O&M): $Q_E = Q_{DESIGN} \times \frac{\text{New Ozone Dosage}}{8 \text{ mg/l}}$

References: 1, 7

FIGURE H-60[1] OZONATION (AIR)

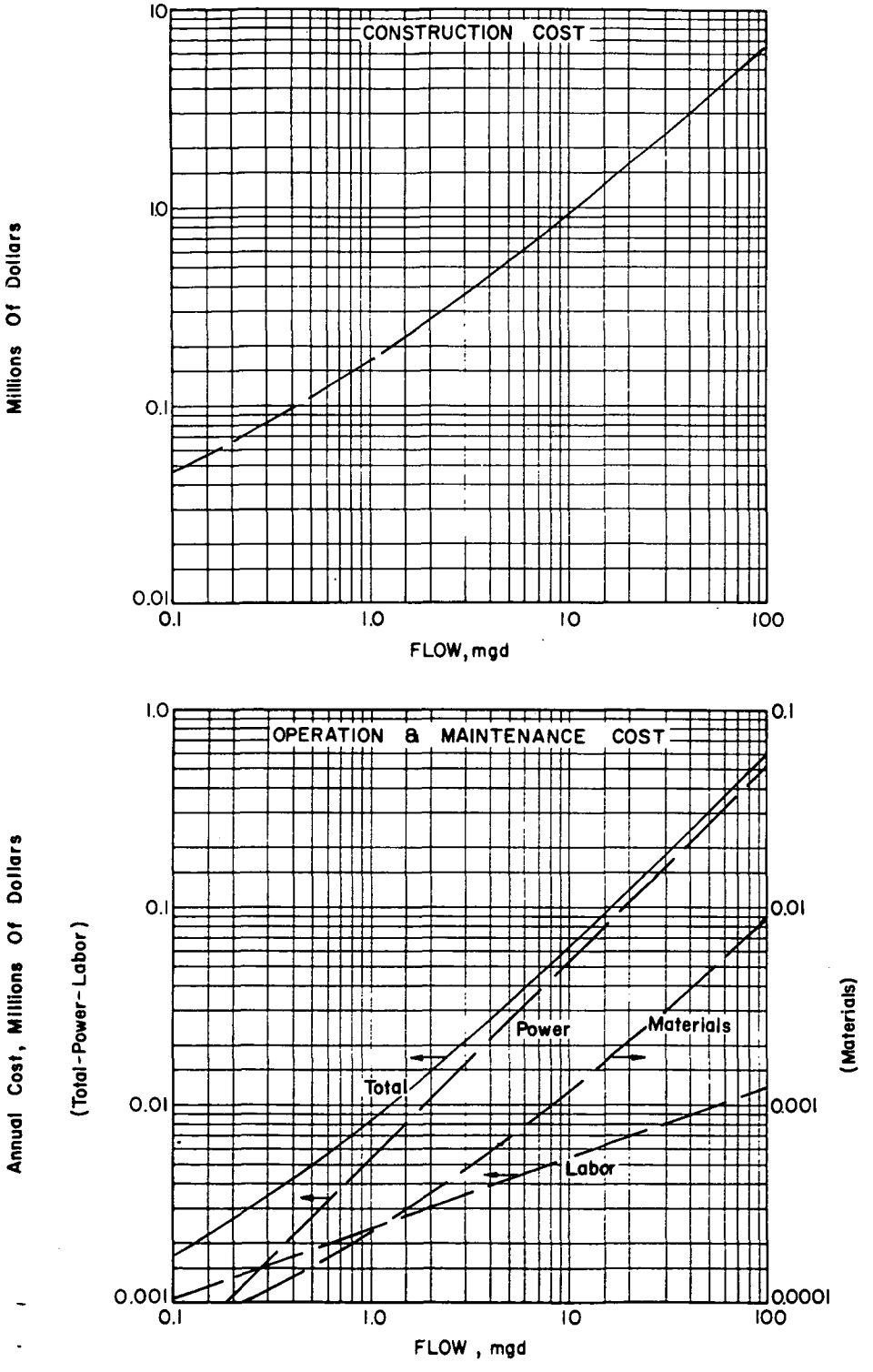


FIGURE H-60

OZONATION (AIR- AND O_2-GENERATED)

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:

   Air Systems:

   a. Air blowers.
   b. Air filters.
   c. Air coolers.
   d. Air dryers.
   e. Ozonator.
   f. Ozone injector.
   g. Ozone-water contact chamber.
   h. Aeration chamber.

   Oxygen Systems:

   a. Oxygen storage.
   b. Ozonator.
   c. Ozone injector.
   d. Ozone-water contact chamber.
   e. Aeration chamber.

2. Oxygen requirements = 3 lb O_2/lb O_3.

3. Ozone dosage = 8 mg/l.

Adjustment Factor:

To adjust costs, enter curve at effective flow (Q_E)

Chemical Costs (O&M): $Q_E = Q_{DESIGN} \times \frac{\text{New Ozone Dosage}}{8 \text{ mg/l}}$

References: 1, 7

FIGURE H-60[2] OZONATION (OXYGEN)

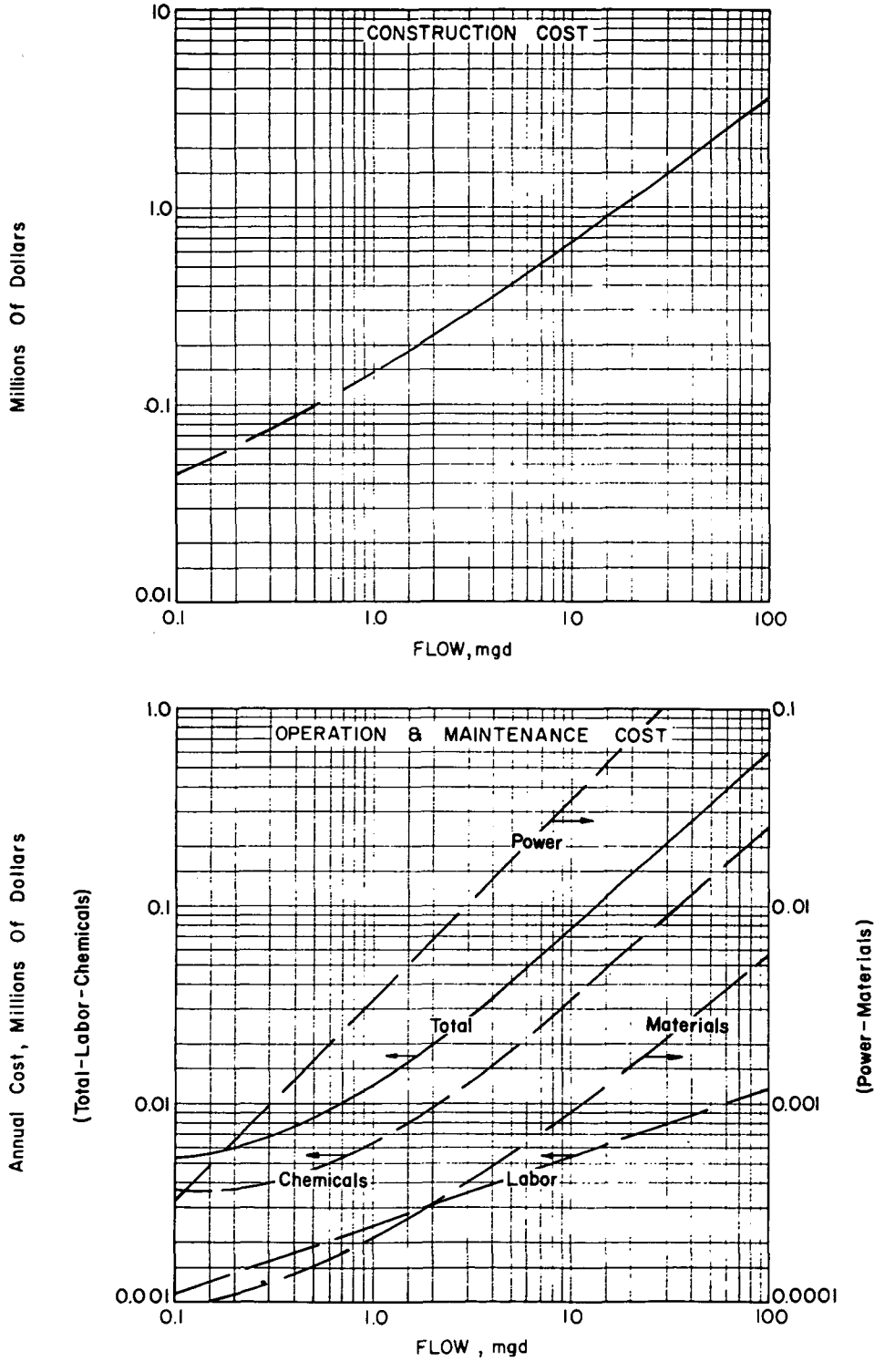


FIGURE H-61

POST AERATION

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include aeration equipment and post aeration basin.

2. Designed to increase dissolved oxygen from 1 mg/l to 5 mg/l.

3. Detention time = 20 minutes.

4. Power information based on transfer of 34 lbs of O_2 per million gallons of wastewater treated.

References: 1, 7

FIGURE H-61 POST AERATION

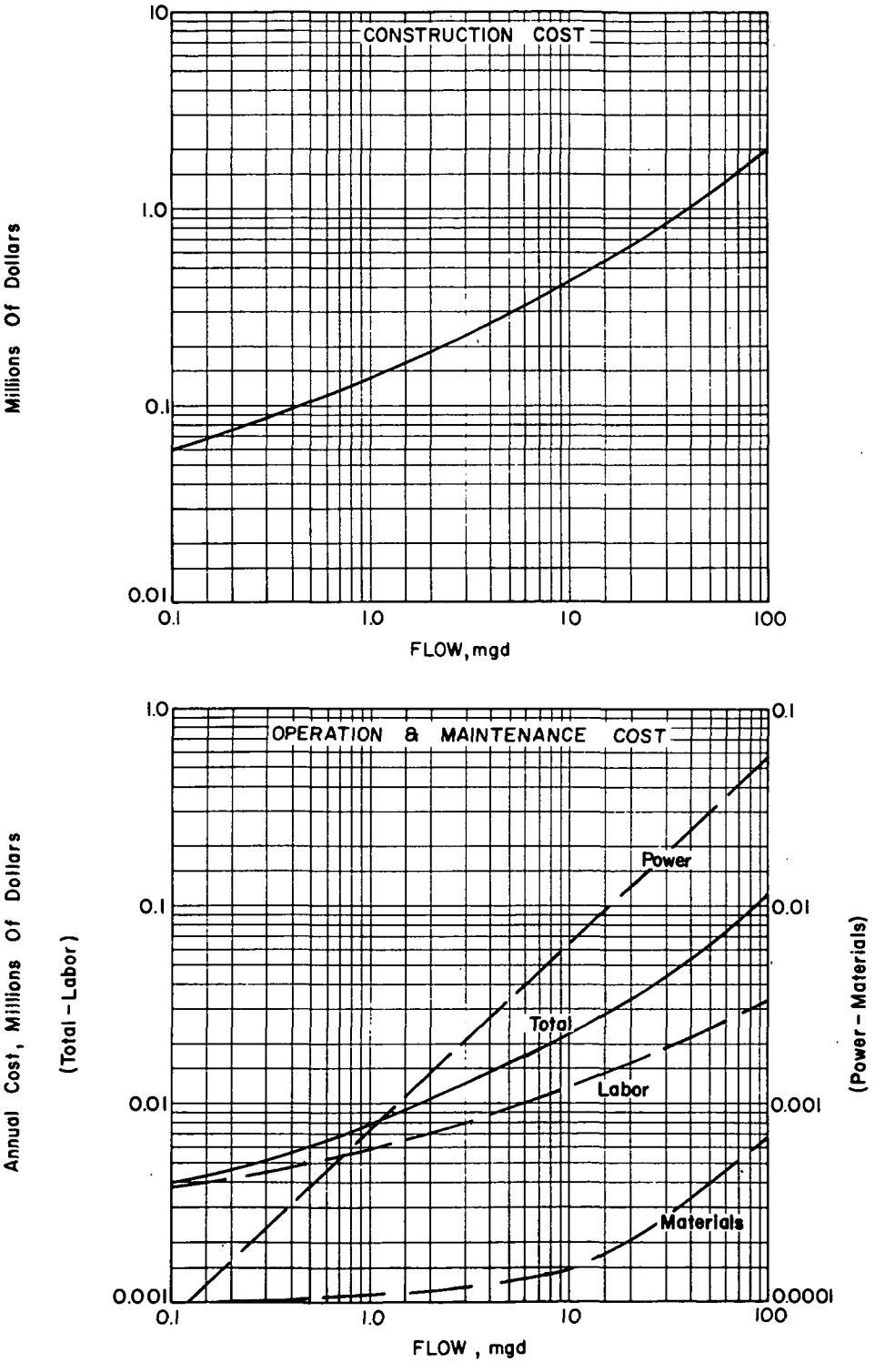


FIGURE H-62

GROUNDWATER RECHARGE (INFILTRATION)

Service Life: 30 years (equipment only); land assumed to have infinite life.

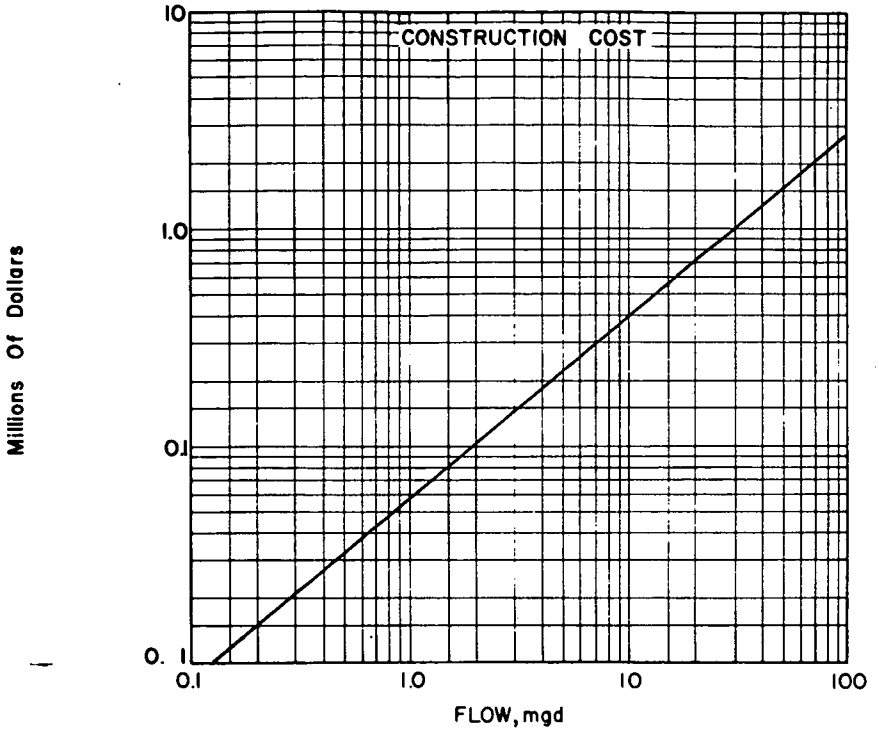
Relative Reliability: Level B

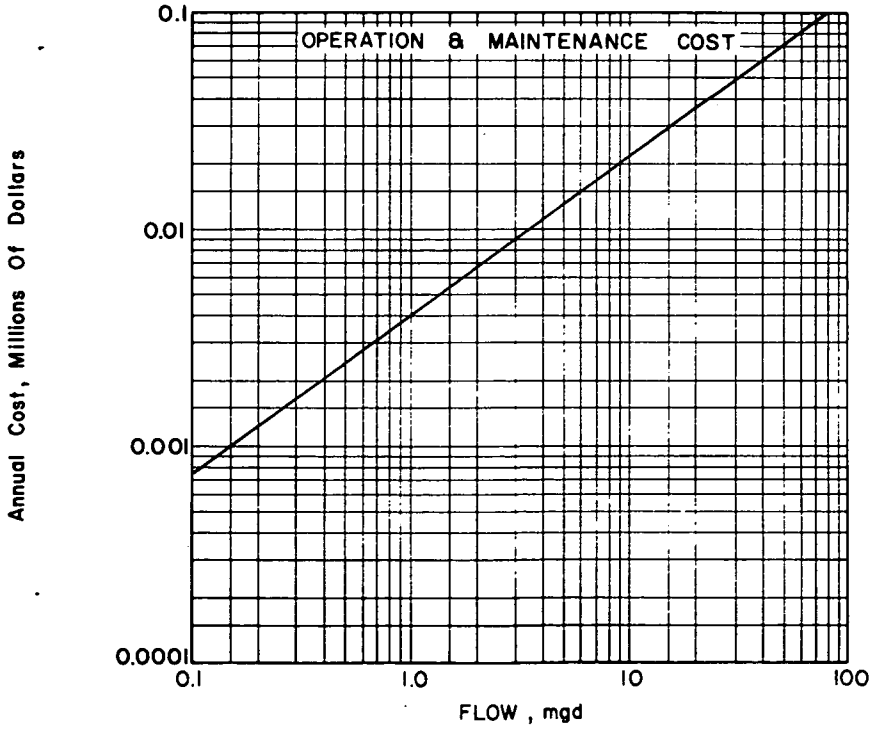
Design Basis:

1. Construction costs include site work, infiltration basin, gravel service road (12 ft), fence (4 ft) and on-site disposal of debris.

Reference: 27

FIGURE H-62 GROUNDWATER RECHARGE (INFILTRATION)





H.8.4 Sludge Treatment and Disposal Processes: Cost Curves, Figures H-63 through H-83

FIGURE H-63

SLUDGE PUMPING

Service Life: 10 years

Relative Reliability: Level B

Design Basis:

1. Costs are based on a sludge loading of 1,900 lbs/mg at 4% solids, i.e., 5,700 gal of sludge/mg for combined primary and secondary sludge after thickening.
2. Non-clog centrifugal pumps.
3. TDH = 30 feet.

Adjustment Factor:

To adjust costs for alternative sludge quantities and characteristics, enter curves at effective flow (Q_E):

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{1{,}900 \text{ lb/mg}} \times \frac{4\%}{\text{New Design Concentration (\%)}}$$

References: 4, 7

FIGURE H-63 SLUDGE PUMPING

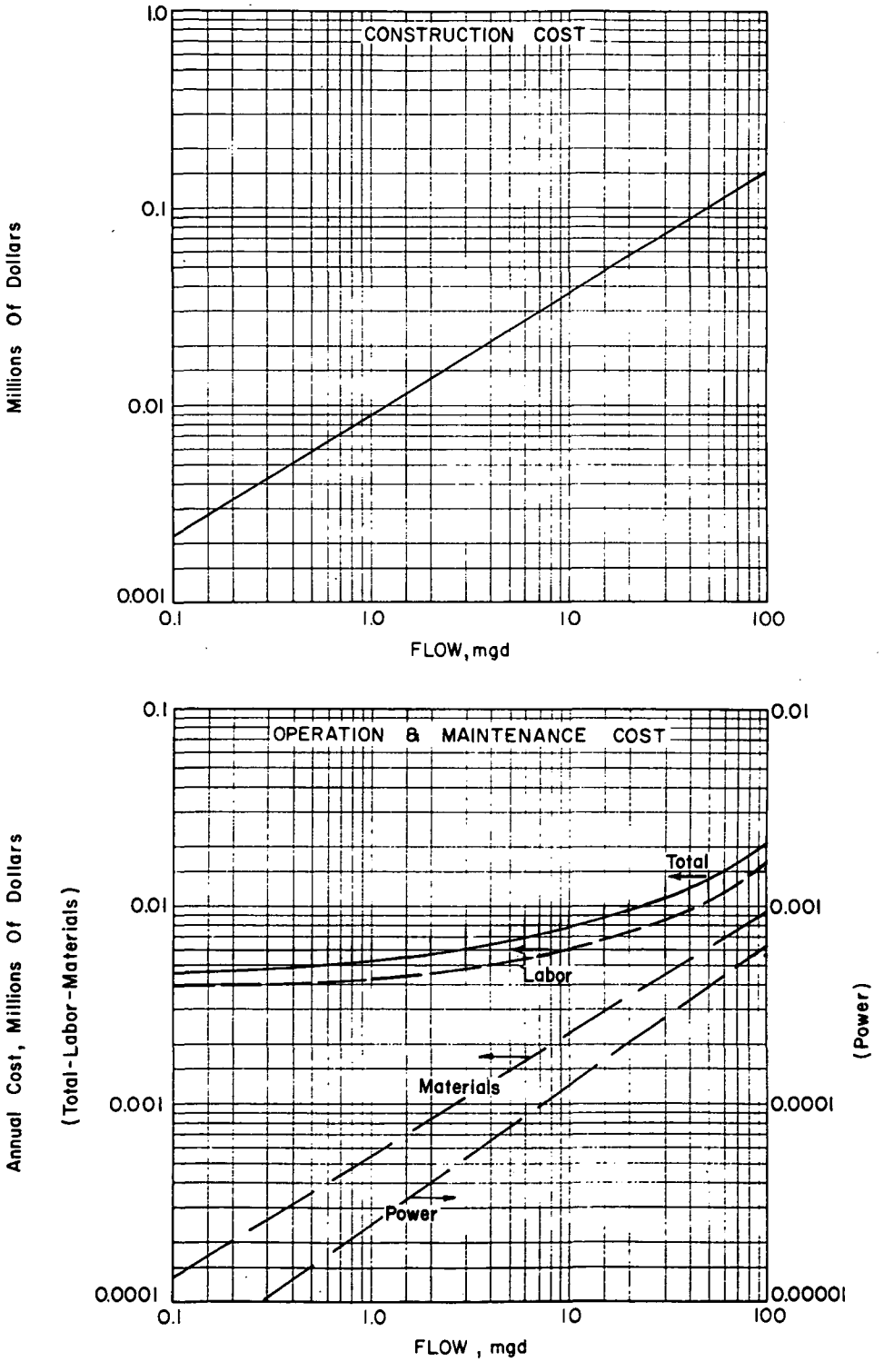


FIGURE H-64

GRAVITY THICKENER

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include thickener and all related mechanical equipment. Pumps are not included. (See Sludge Pumping Figure H-63.)

2. Costs are based on thickening of secondary sludge (820 lb/mg; loading = 6 lb/sq ft/day). See adjustment factors for other sludge loadings.

3. O & M costs do not include polymer or metal addition.

Adjustment Factor:

To adjust costs for alternative sludge quantites, concentrations, and thickening properties, enter curves at effective flow (Q_E)

$$Q_E = Q_{DESIGN} \times \frac{6 \text{ lb/sq ft/day}}{\text{New Design Mass Loading}} \times \frac{\text{New Design Sludge Mass}}{820 \text{ lbs/mg}}$$

References: 5, 6, 7, 13, 18

FIGURE H-64 GRAVITY THICKENER

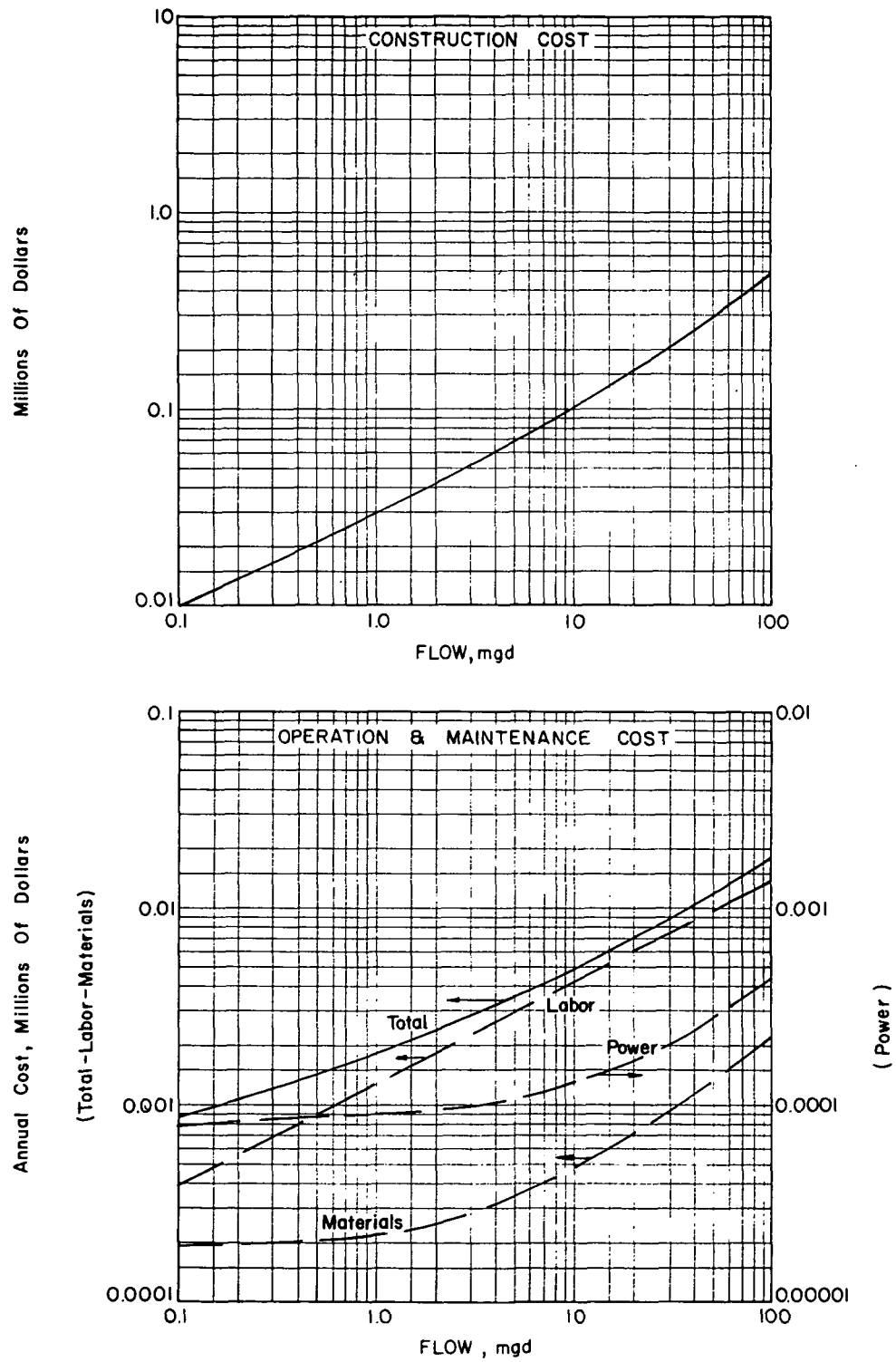


FIGURE H-65

DISSOLVED AIR FLOTATION THICKENER

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:

   a. Flotation chamber (two-hour detention based on sludge flow).
   b. Pressure tanks (60 psig).
   c. Recycle pumps (100% recycle).

2. Costs for thickening of secondary sludge only: 820 lb/mg.

3. Loading rate = 2 lb/sq ft/hr.

4. Operating hours: 0.1 and 1 mgd = 40 hrs/wk; 10 mgd = 100 hrs/wk; 100 mgd = 168 hrs/wk.

Adjustment Factor:

To determine costs at loading rates or sludge quantities other than above, enter curve at effective flow Q_E.

$$Q_E = Q_{DESIGN} \times \frac{2 \text{ lb/sq ft/hr}}{\text{New Design Mass Loading Rate}} \times \frac{\text{New Design Sludge Mass}}{820 \text{ lbs/day/mg}}$$

References: 5, 6, 8, 13, 18

FIGURE H-65 DISSOLVED AIR FLOTATION THICKENER

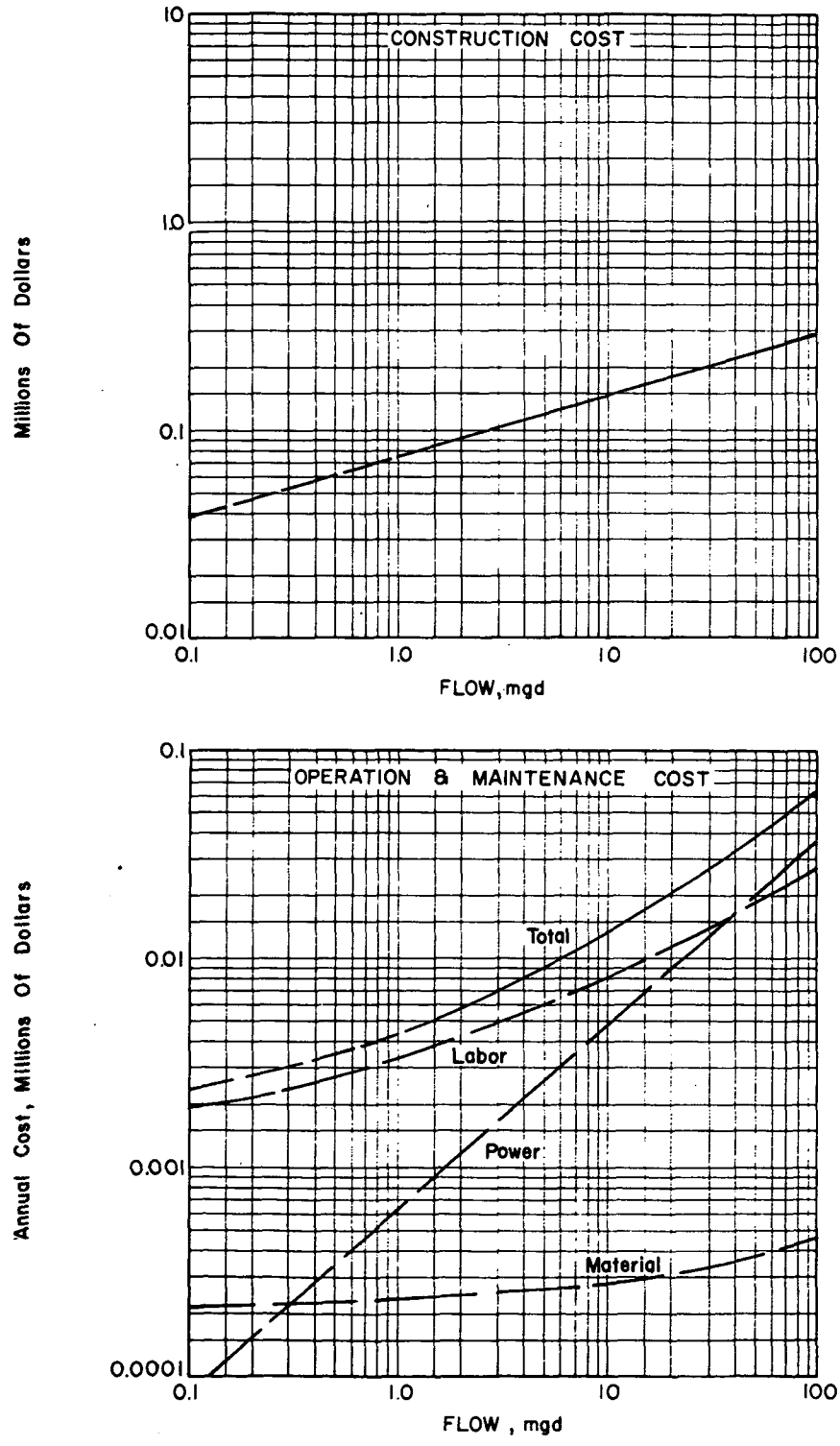


FIGURE H-66

CENTRIFUGATION (LIME SLUDGE DEWATERING)

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: centrifuges (solid bowl), with minimum of one spare; sludge pumps and piping; cake conveyors, internal electrical and building cost.

2. Sludge quantity: 4,500 lb/mg at 10% solids.

3. Operation = 8 hrs/day.

4. Costs do not include centrate handling.

Adjustment Factor:

Costs for sludge quantities and characteristics are different than those for digested primary and secondary sludge. They are determined by entering curves at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{4{,}500 \text{ lbs/mg}} \times \frac{10\%}{\text{New Design Feed Sludge Concentration}}$$

References: 3, 5

FIGURE H-66 CENTRIFUGATION (LIME, SLUDGE DEWATERING)

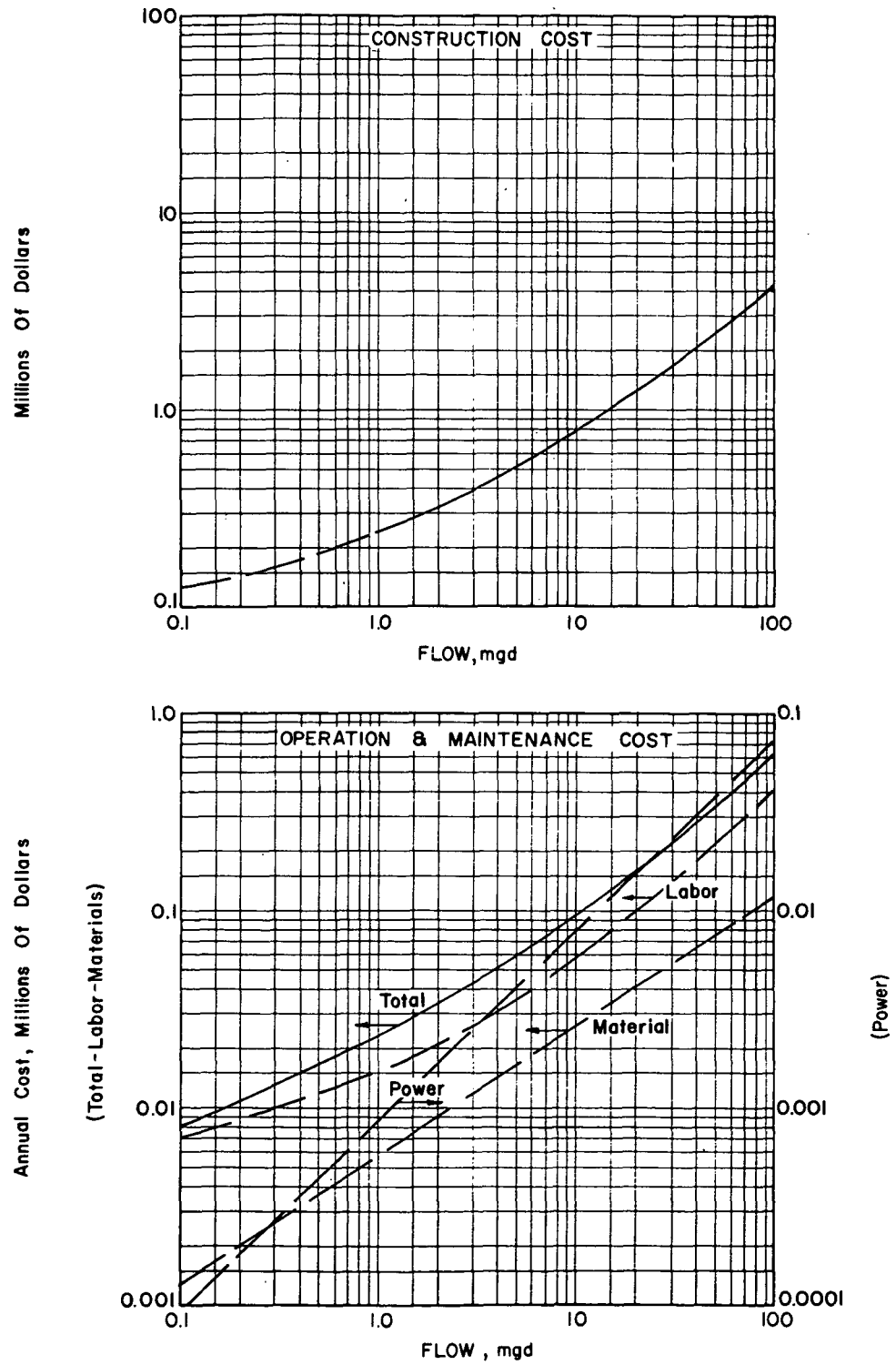


FIGURE H-67

CENTRIFUGATION (BIOLOGICAL SLUDGE DEWATERING)

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: centrifuges with minimum of one spare unit; sludge pumps and piping; cake conveyors; and building costs.

2. Construction costs shown are for digested and undigested primary and secondary sludges.

3. O & M costs are for digested sludge.

4. Sludge quantity: digested primary and secondary sludge - 900 lb/mg at 4%.

5. Operation = 8 hrs/day.

6. Cationic polymer cost based on 10 lb/ton dry solids.

Adjustment Factor:

Costs for sludge quantities and characteristics are different than those for digested primary and secondary sludges. They are determined by entering curves at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{900 \text{ lbs/mg}} \times \frac{4\%}{\text{New Design Sludge Concentration}}$$

References: 3, 4, 5

FIGURE H-67 CENTRIFUGATION (BIOLOGICAL SLUDGE DEWATERING)

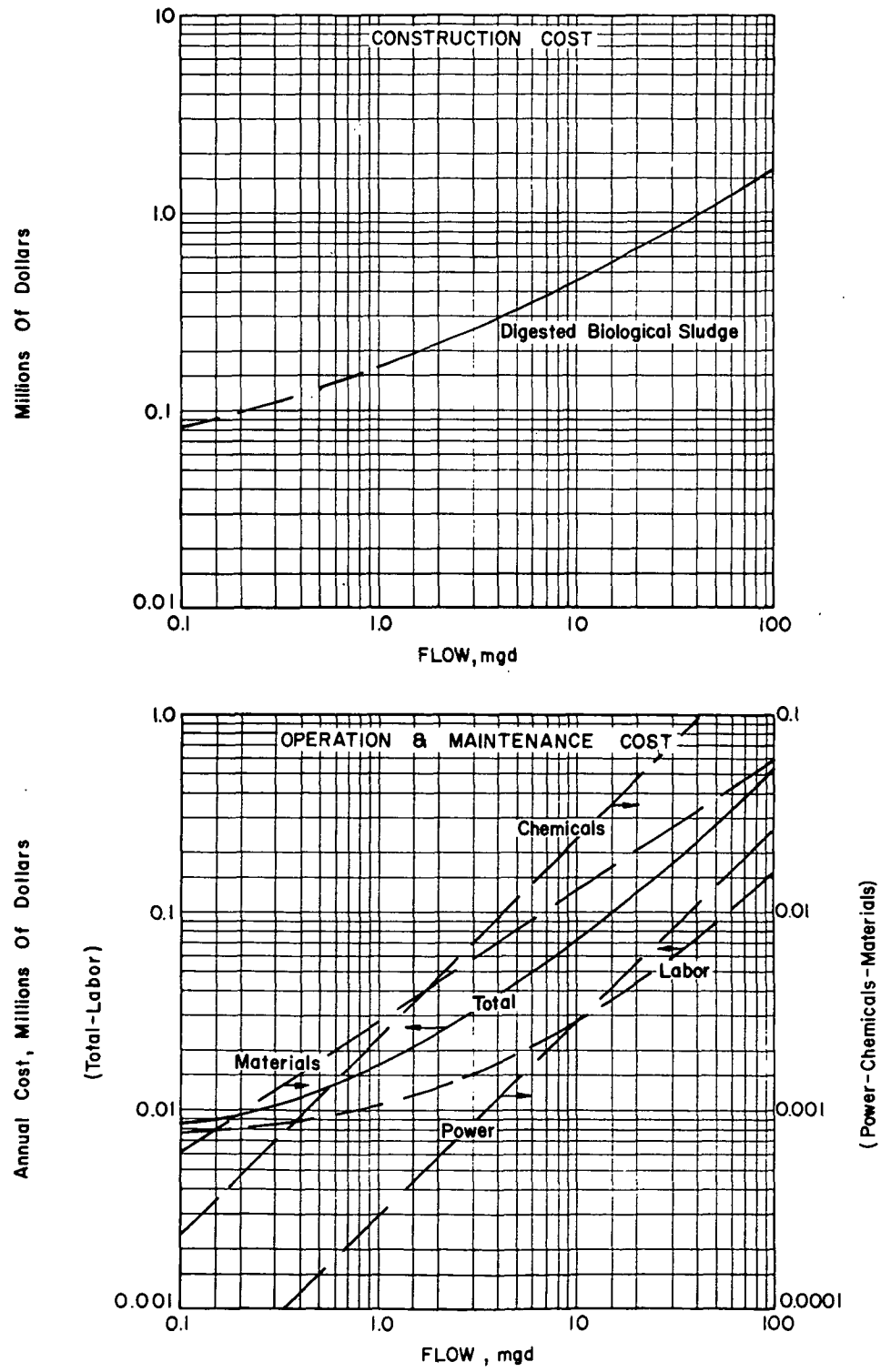


FIGURE H-68

VACUUM FILTRATION (BIOLOGICAL SLUDGE)

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: pumps, internal piping and electrical controls; mechanical equipment, conveyors, and sludge cake storage hopper; building, chemical-handling, and storage facilities.

2. Costs are for dewatering of combined primary and secondary digested sludge - 900 lb/mg. See adjustment factor for dewatering other sludge.

3. Filter yield = 5 lb/sq ft/hr.

4. Operation time (excluding downtime for maintenance):

   6 hrs/day for 1-mgd plant or less.
   12 hrs/day for 10-mgd plant.
   18 hrs/day for 100-mgd plant.
   Interpolate for other plant sizes.

5. Chemical dosage: $FeCl_3$ = 35 lb/mg; CaO = 90 lb/mg.

Adjustment Factor:

Costs for sludge quantities and dewatering characteristics are different than those above. Enter curves at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{900 \text{ lbs/mg}} \times \frac{5 \text{ lbs/sq ft/hr}}{\text{New Design Mass Loading Rate}} \times \frac{\text{Original Design Operation Time}}{\text{New Design Operating Time}}$$

References: 3, 4, 7

FIGURE H-68 VACUUM FILTRATION (BIOLOGICAL SLUDGE)

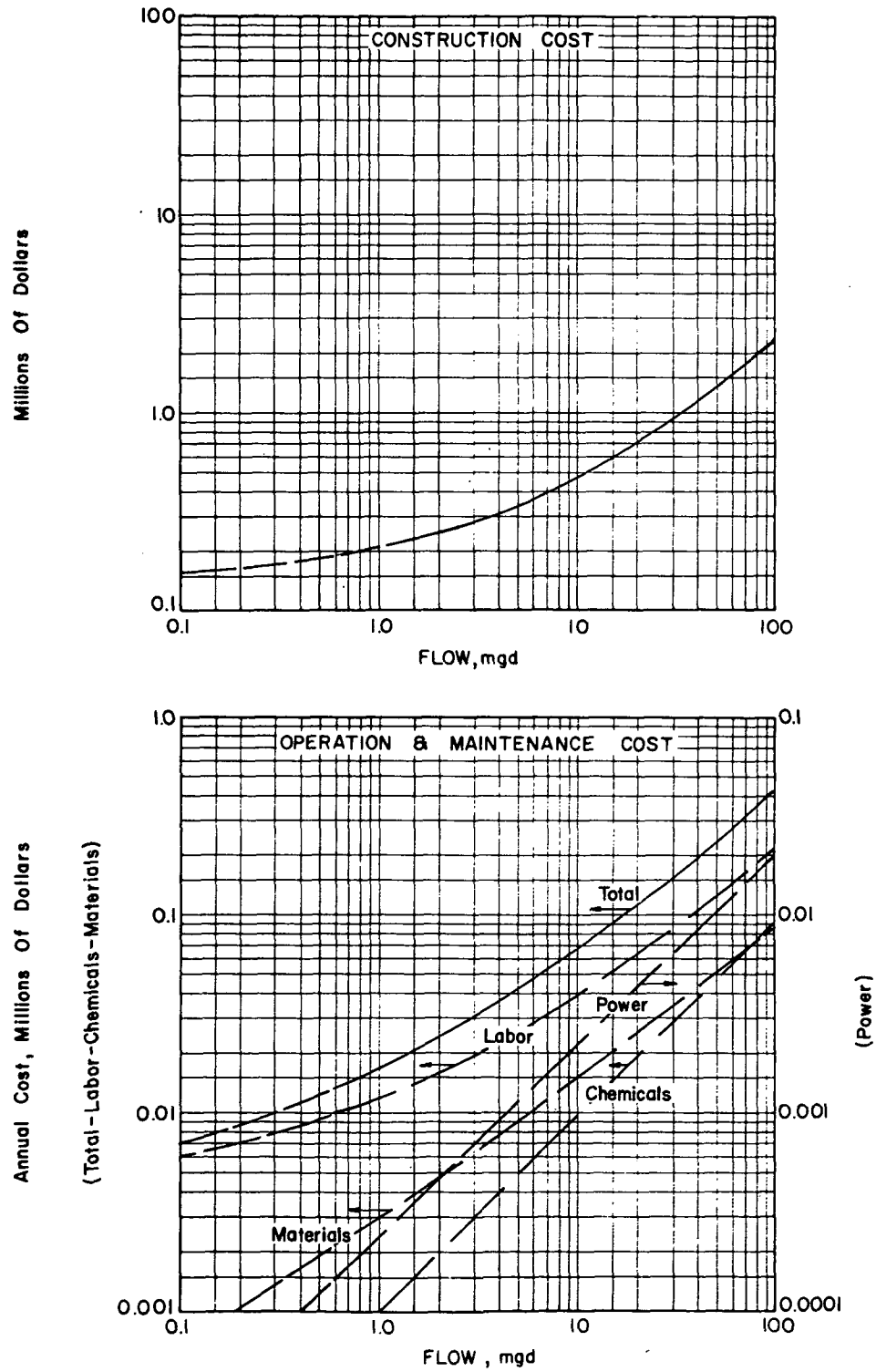


FIGURE H-69

VACUUM FILTRATION (LIME SLUDGE)

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: pumps, internal piping and electrical, controls; mechanical equipment, conveyors, sludge-cake storage hopper; building.

2. Costs are for dewatering of lime sludge (4,500 lb/mg @ 10%).

3. Filter yield = 8 lb/sq ft/hr.

4. Operation time (excluding downtime for maintenance):

   6 hrs/day for 1-mgd plant or less.
   12 hrs/day for 10-mgd plant.
   18 hrs/day for 100-mgd plant.
   Interpolate for other plant sizes.

Adjustment Factor:

Costs for sludge quantities, concentrations, and dewatering characteristics or operating times are different than those above. Enter curves at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{4{,}500 \text{ lbs/mg}} \times \frac{8 \text{ lbs/sq ft/hr}}{\text{New Design Mass Loading Rate}} \times \frac{\text{Original Design Operation Time}}{\text{New Design Operation Time}}$$

References: 3, 4, 7

FIGURE H-69 VACUUM FILTRATION (LIME SLUDGE)

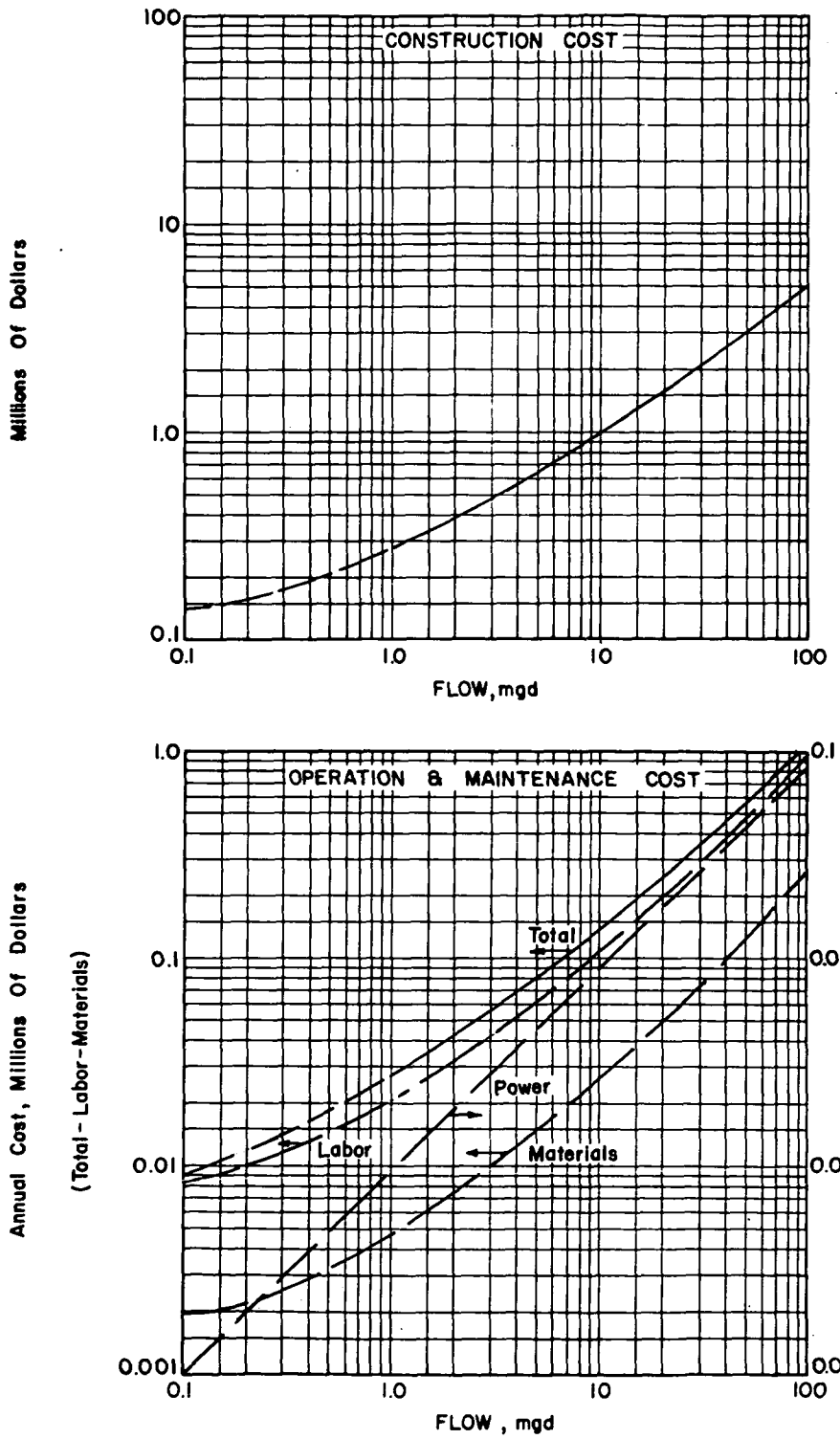


FIGURE H-70

FILTER PRESS (LIME SLUDGE)

Service Life: 15 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: filter presses, pressure pumps, conveyor equipment, sludge storage tanks, and building.

2. Sludge loading: 4,500 lb/mg at 10%.

3. Cake characteristics: density = 75 lb/cu ft; solids content = 45%.

4. Operations: For 0.1 to 1 mgd plants - 20 cycles/week.
   For 1 to 10 mgd plant - 48 cycles/week.
   For 10 to 100 mgd plant - 84 cycles/week.

5. Cycle time: 2 hours/cycle.

Adjustment Factor:

To develop cost for sludge quantities, concentrations, characteristics or cycles per week different than those used to develop these curves, enter curve at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{4{,}500 \text{ lbs/mg}} \times \frac{\text{Original Design Cycles Per Week}}{\text{New Design Cycles Per Week}} \times \frac{\text{New Design Cycle Time}}{2 \text{ hours}}$$

References: 5, 18

FIGURE H-70 FILTER PRESS (LIME SLUDGE)

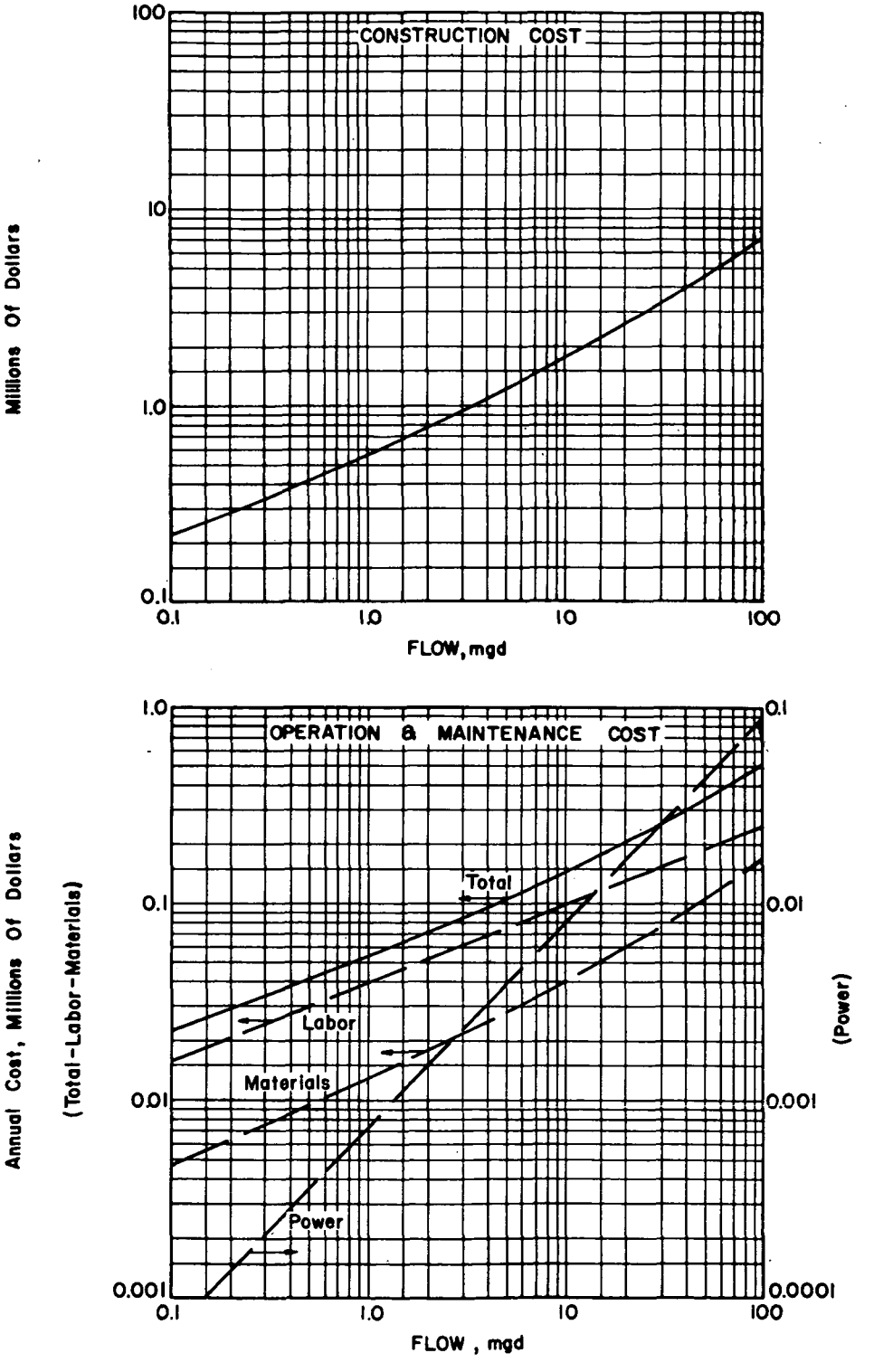


FIGURE H-71

FILTER PRESS (BIOLOGICAL SLUDGE)

Service Life: 15 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: filter presses, pressure pumps, conveyor equipment, chemical feed and storage facilities, conditioning tanks, sludge storage tanks, and building.

2. Sludge loading: digested primary + secondary = 900 lb/mg @ 2.5%

3. Cake characteristics: density = 65 lb/cu ft; solids content = 40%

4. Operations: For 0.1 to 1 mgd plants - 20 cycles/week
   For 1 to 10 mgd plant - 48 cycles/week
   For 10 to 100 mgd plant - 84 cycles/week

5. Conditioning chemicals: $FeCl_3$ = 35 lb/mg; CaO = 90 lb/mg.

Adjustment Factor:

To develop cost for sludge quantities, concentrations, characteristics or cycles per week different than those used to develop these curves, enter curve at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{900 \text{ lb/mg}} \times \frac{\text{Original Design Cycles Per Week}}{\text{New Design Cycles Per Week}} \times \frac{\text{New Design Cycle Time}}{2 \text{ hours}}$$

References: 5, 18

FIGURE H-71 FILTER PRESS (BIOLOGICAL SLUDGE)

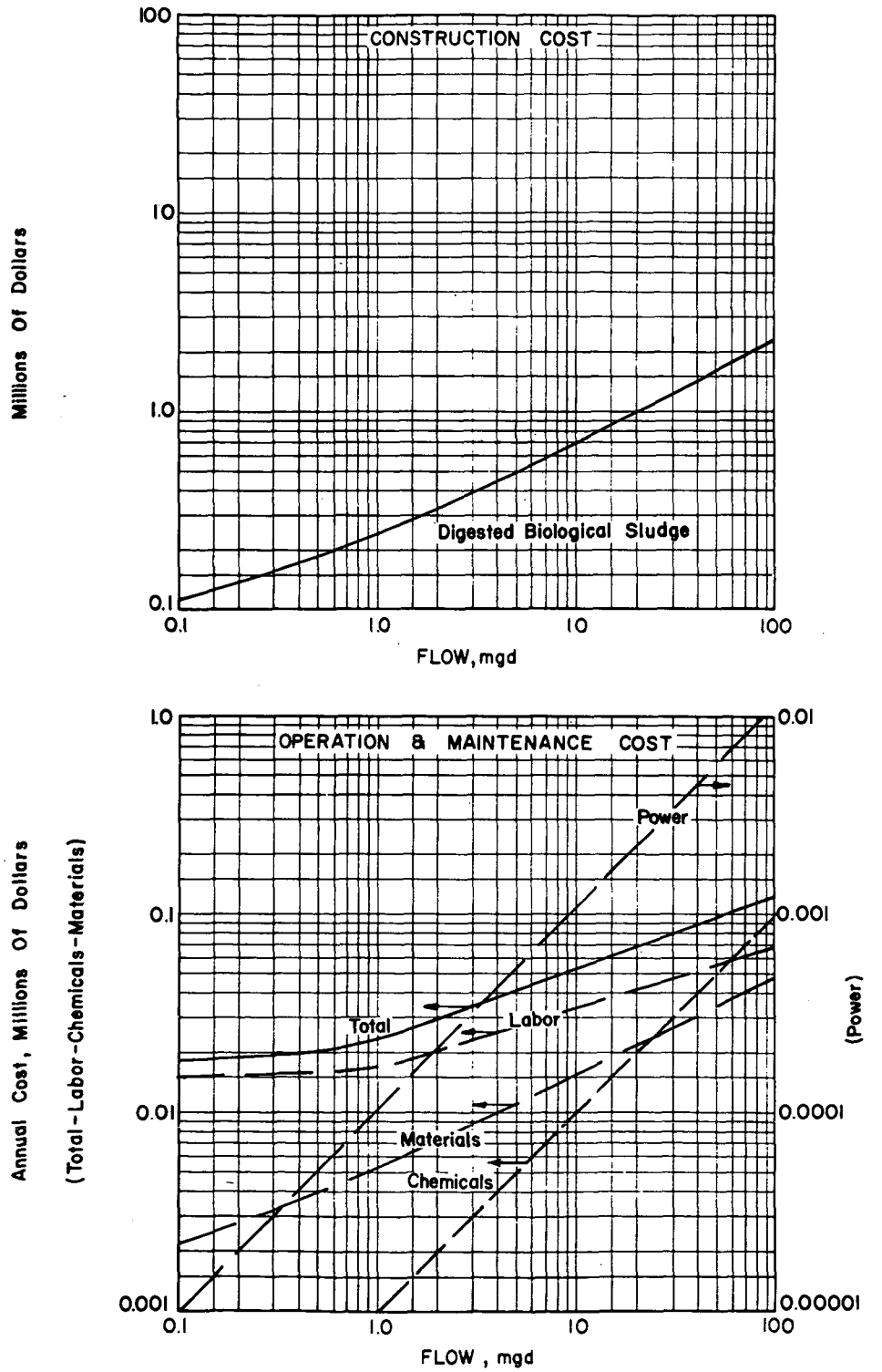


FIGURE H-72

SLUDGE DRYING BEDS

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: sand beds, sludge inlets, underdrains, cell dividers, sludge piping, underdrain return, and other structural elements of the beds.

2. Bed loading: 900 lb of sludge/mg; 20 lb/sq ft/year.

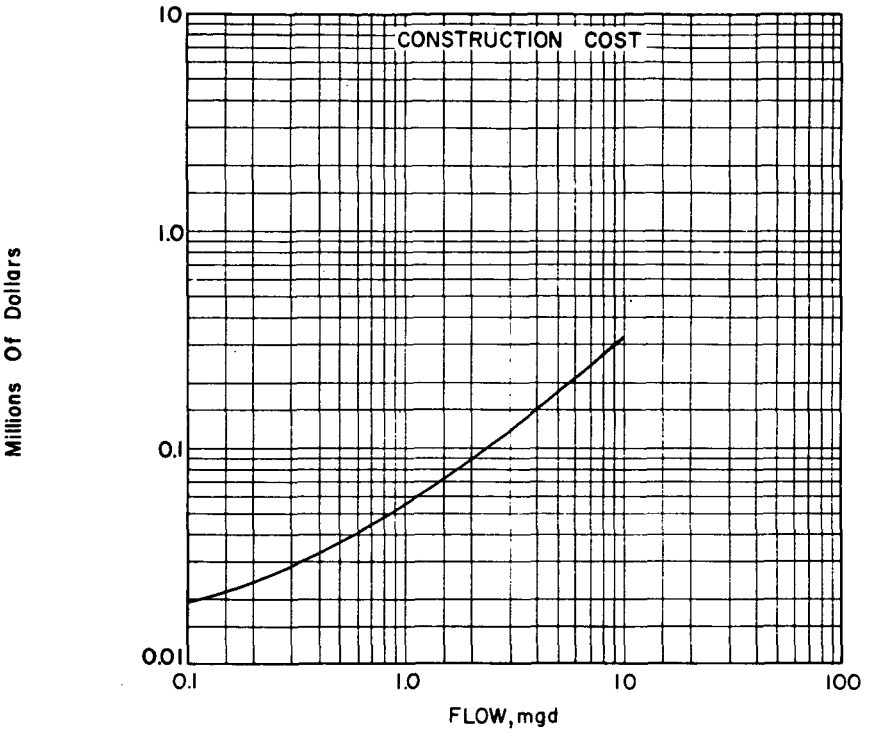
Adjustment Factor:

To adjust costs for bed loading rates, sludge quantities, or characteristics, enter curve at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{900 \text{ lb/mg}} \times \frac{20 \text{ lb/sq ft/yr}}{\text{New Design Bed Loading}}$$

References: 3, 5

FIGURE H-72 SLUDGE DRYING BEDS



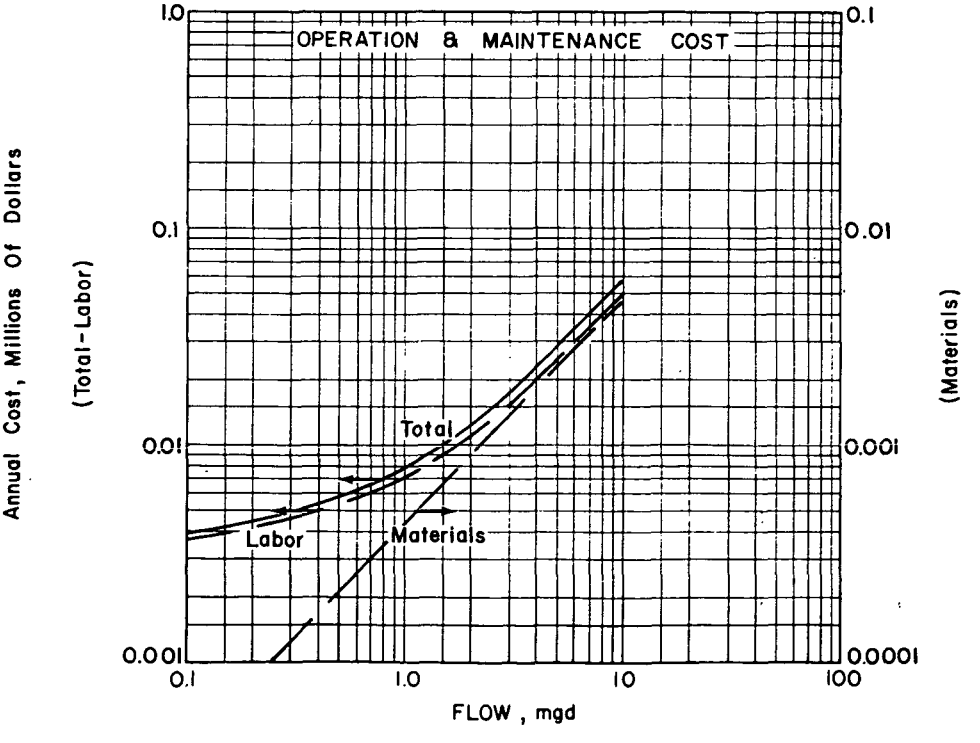


FIGURE H-73

TWO-STAGE ANAEROBIC DIGESTERS

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

1. Capital costs include: digester, heat-exchanger, gas-collection equipment, control building.
2. Feed to digesters is combined primary and is thickened.
3. Feed - 1,900 lb/mg at 4% solids (75% volatile).
4. Effluent from digesters is 900 lb/mg at 2.5% solids.
5. Loading rate - 0.16 lb/cu ft/day.
6. Operating temperature - 85 to 110^{o}F.
7. Digester gas is utilized for heating. Excess gas is not utilized.

Adjustment Factor:

To adjust costs for loading rates different than those presented here, enter curve at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{1{,}900 \text{ lb/mg}}$$

References: 7, 8, 10, 13

FIGURE H-73 TWO STAGE ANAEROBIC DIGESTERS

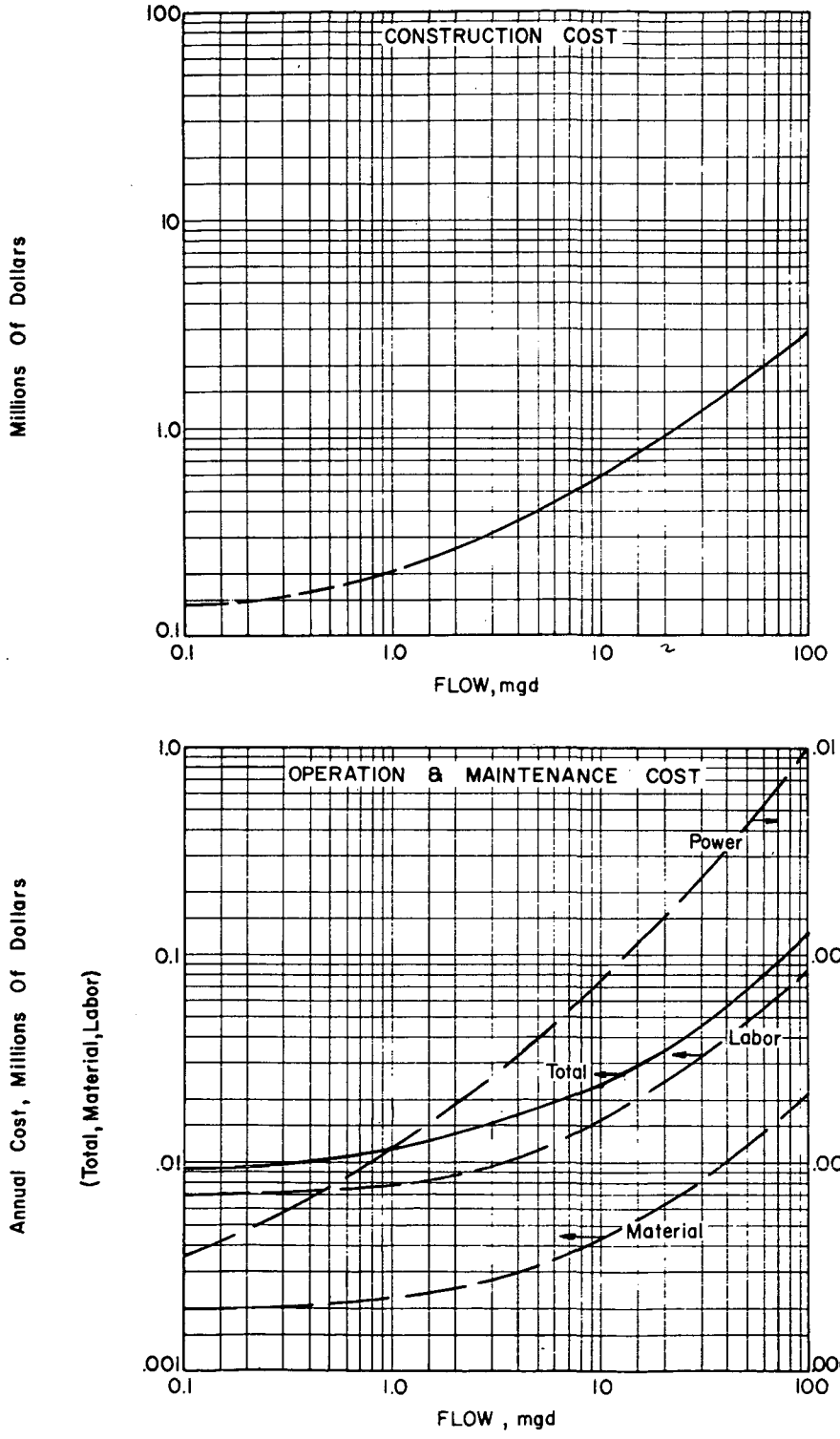


FIGURE H-74

AEROBIC DIGESTERS

Service Life: 40 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include:

   a. Basins (20 day detention time) - sludge flow = 5,700 gal/mg (1900 lb/mg at 4%)

   b. Floating mechanical aerators

2. Mixing requirements: 134 hp/mg.

3. Oxygen requirements: 1.6 lbs O_2/lb VSS destroyed.

Adjustment Factor:

To adjust costs for design factors different from those above, enter curves at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{20 \text{ days}}{\text{New Design Retention Time}} \times \frac{\text{New Design Sludge Mass}}{1{,}900 \text{ lb/mg}} \times \frac{4\%}{\text{New Design Sludge Concentration}}$$

References: 6, 13

FIGURE H-74 AEROBIC DIGESTERS

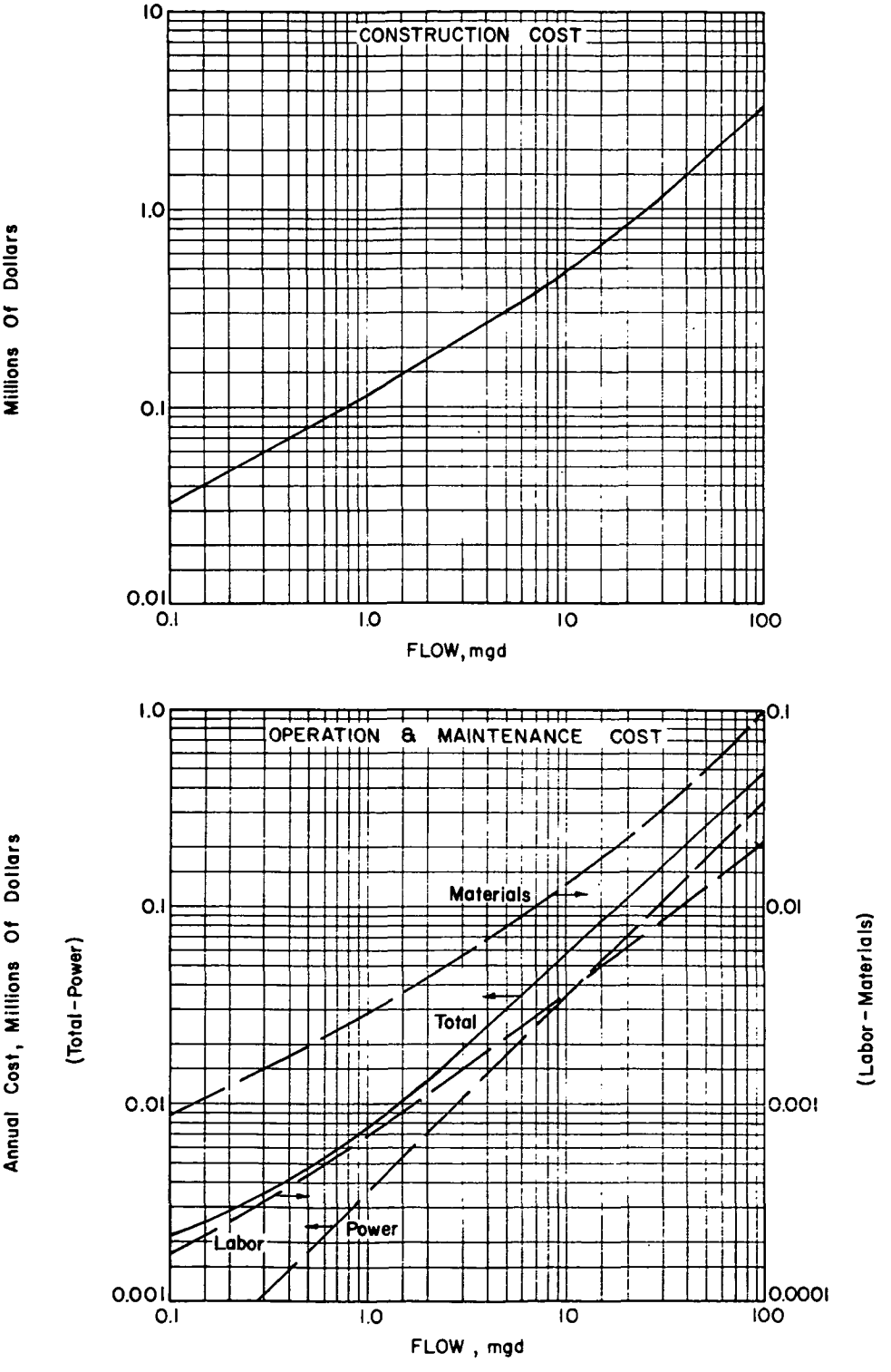


FIGURE H-75

HEAT TREATMENT OF SLUDGE

Service Life: 25 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: sludge feed pumps, grinders, heat exchangers, reactors, boilers, gas separators, and buildings.

2. Costs are related to average wastewater flow by the following:

   a. Sludge quantity = 1900 lbs/mg (undigested, combined thickened primary plus secondary sludge).

   b. Solids concentration = 4.5%

   c. Sludge flow = 3.8 gpm/mgd based on 8,000 operating hrs per year

3. Fuel costs are for steam generation.

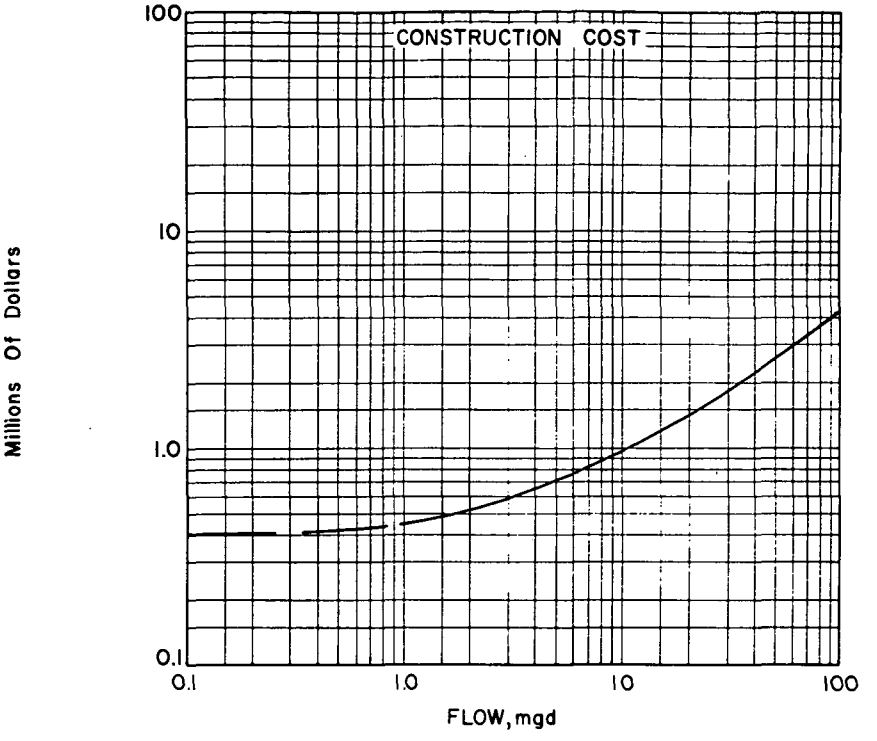
Adjustment Factor:

To adjust costs for design factors different than for those above, enter curve at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{1{,}900 \text{ lb/mg}}$$

References: 4, 23

FIGURE H-75 HEAT TREATMENT OF SLUDGE



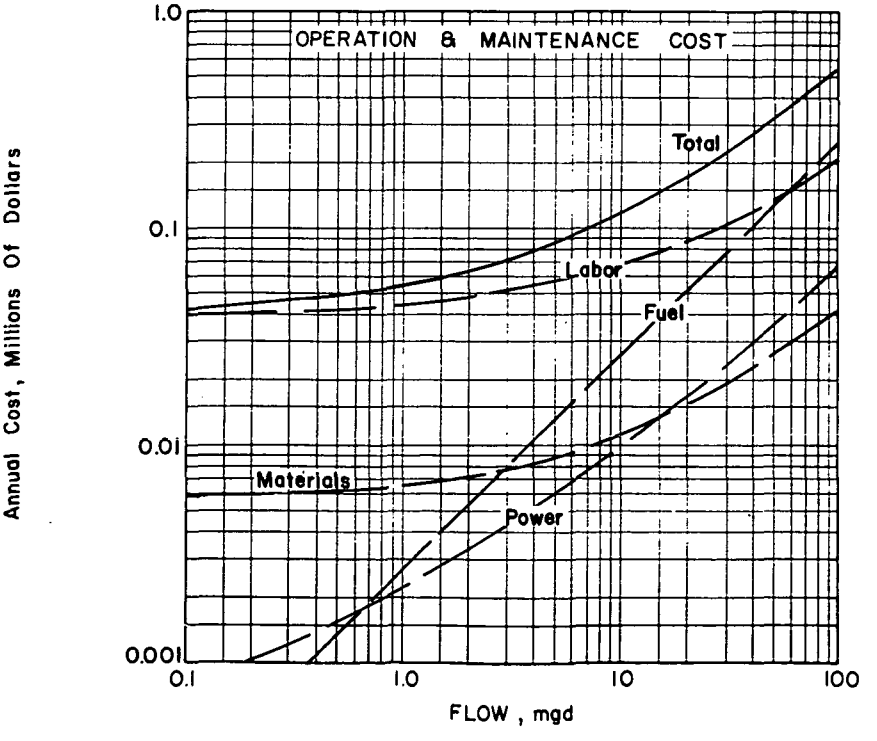


FIGURE H-76

COMPOSTING

Service Life: 17 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: asphalt pad, roads, sewer, drainage pond, electrical work, engineering. Do not apply cost multipliers shown in Table H-2.

2. Sludge production rate: 900 lb/mg.

3. Land Requirement: 0.35 acres/(ton/day).
   Assumed land cost = $10,500/acre.

4. Costs apply to composting of digested or raw biological sludge.

Adjustment Factor:

To adjust cost for sludge composting rates different than 900 lb/mg enter cost curves at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{900 \text{ lb/mg}}$$

References: 53

FIGURE H-76 COMPOSTING

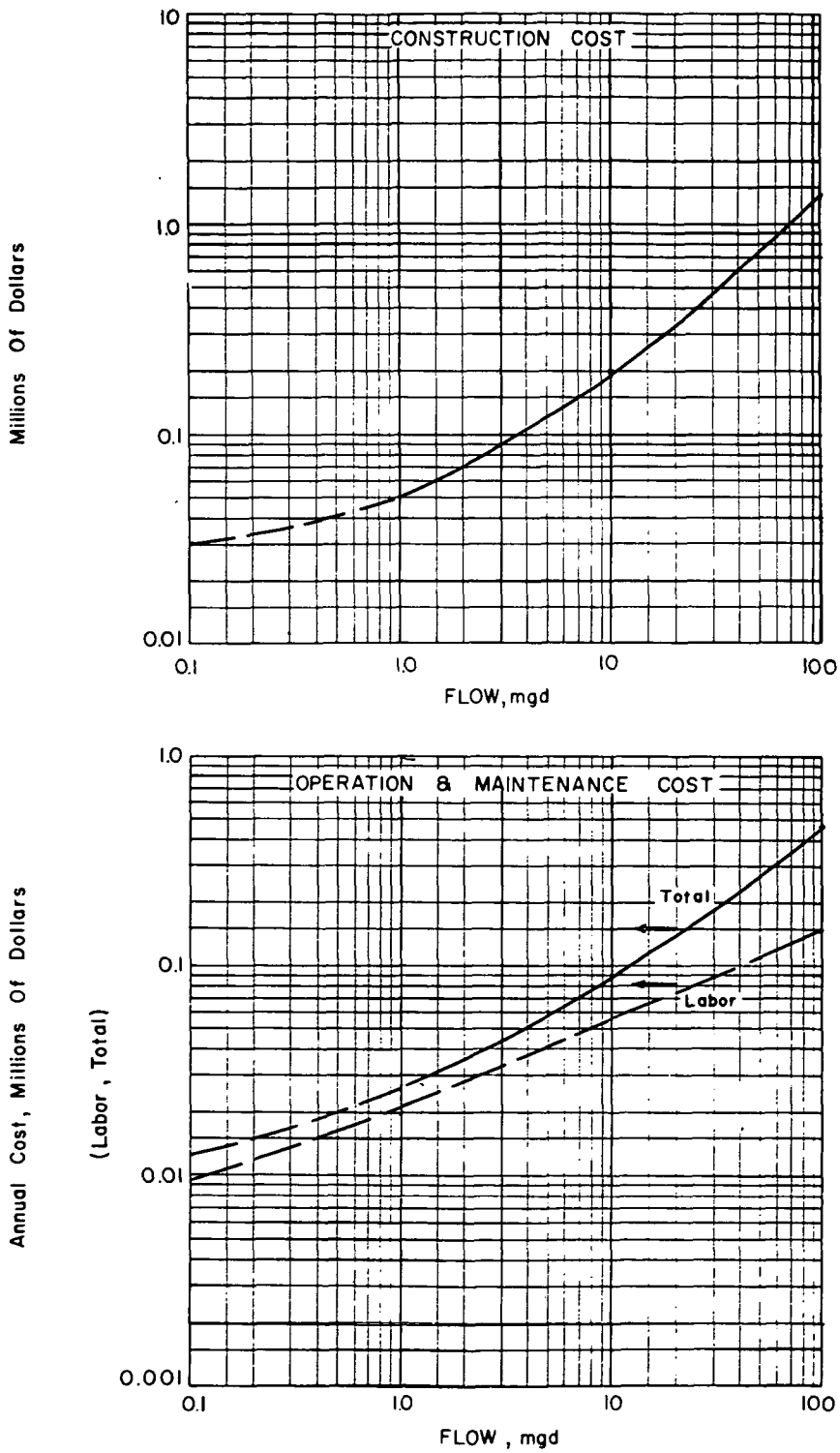


FIGURE H-77

INCINERATION (FLUIDIZED BED)

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: reactor, air blowers and accessories, preheater, scrubbers, fuel pumps, building.

2. Costs are for undigested dewatered primary and secondary sludge (1,900 lb/mg at 20% solids; 75% volatile).

3. Operations:

Flow mgd	Day/Week Operating	Hrs/Day
0.1	1	20
1.0	7	20
10.0	7	20
100.0	7	20

4. Fuel requirements = 100 - 120 gal No. 2 fuel oil/dry ton sludge.

Adjustment Factor:

Fuel costs for conditions different than those given above may be adjusted. However, the relationship of fuel requirements and operating conditions are too complicated to allow for simple adjustment here. Specific detailed study is suggested for each case.

References: 9, 21, 22, 34

FIGURE H-77 INCINERATION (FLUIDIZED BED)

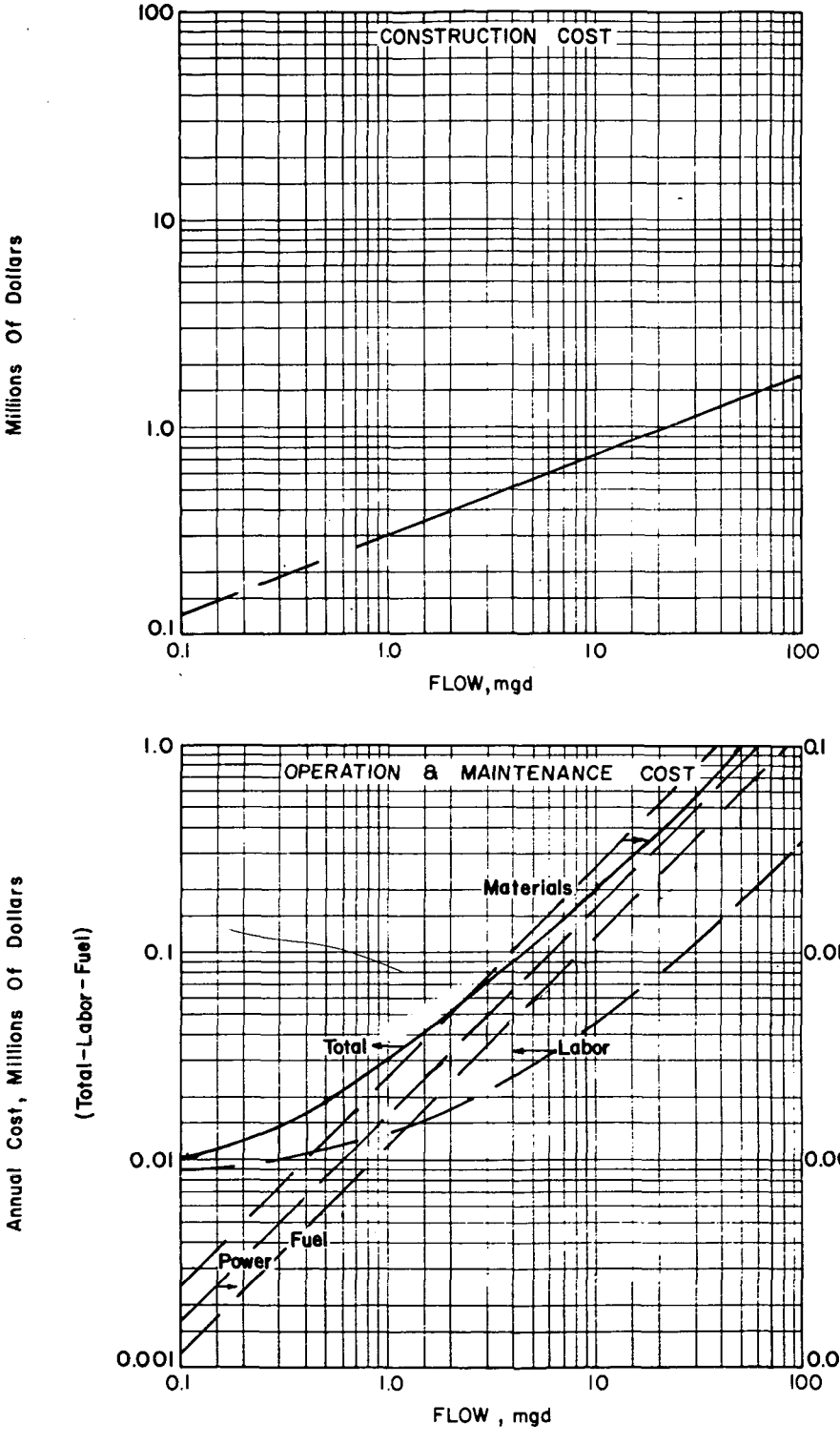


FIGURE H-78

INCINERATION (MULTIPLE HEARTH)

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: incinerator, building, sludge conveyor, ash-handling equipment, gas scrubbers.

2. Costs are for undigested dewatered primary and secondary sludge (1,900 lb/mg at 20% solids; 75% volatile).

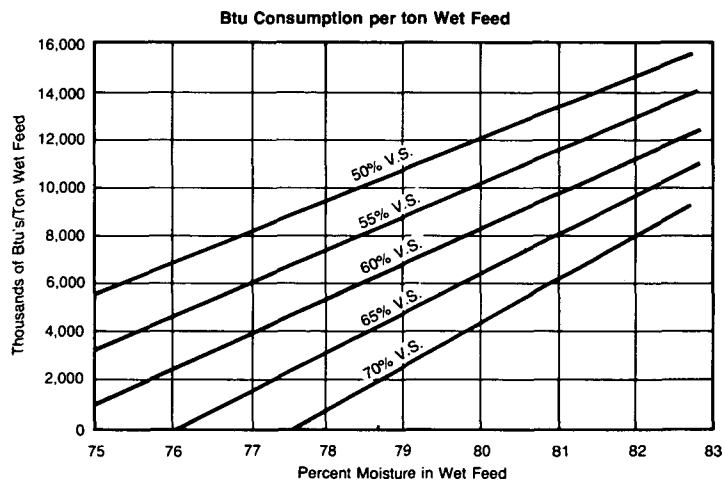
3. Operations:

Flow mgd	Day/Week Operating	Hrs/Day
0.1	1	20
1.0	7	20
10.0	7	20
100.0	7	20

4. Fuel requirements for warm-up and incineration = 32,000 gal of No. 2 fuel oil/yr/mgd.

Adjustment Factor:

To adjust fuel costs of other than those listed, use graph below.



NOTES: 1. Curves are only applicable for continuous operation at feed rates above 4 tons per hour.
2. Sludge heat content is 10,000 btu/lb volatile solids.

References: 3, 4, 5, 8, 9, 13

FIGURE H-78 INCINERATION (MULTIPLE HEARTH)

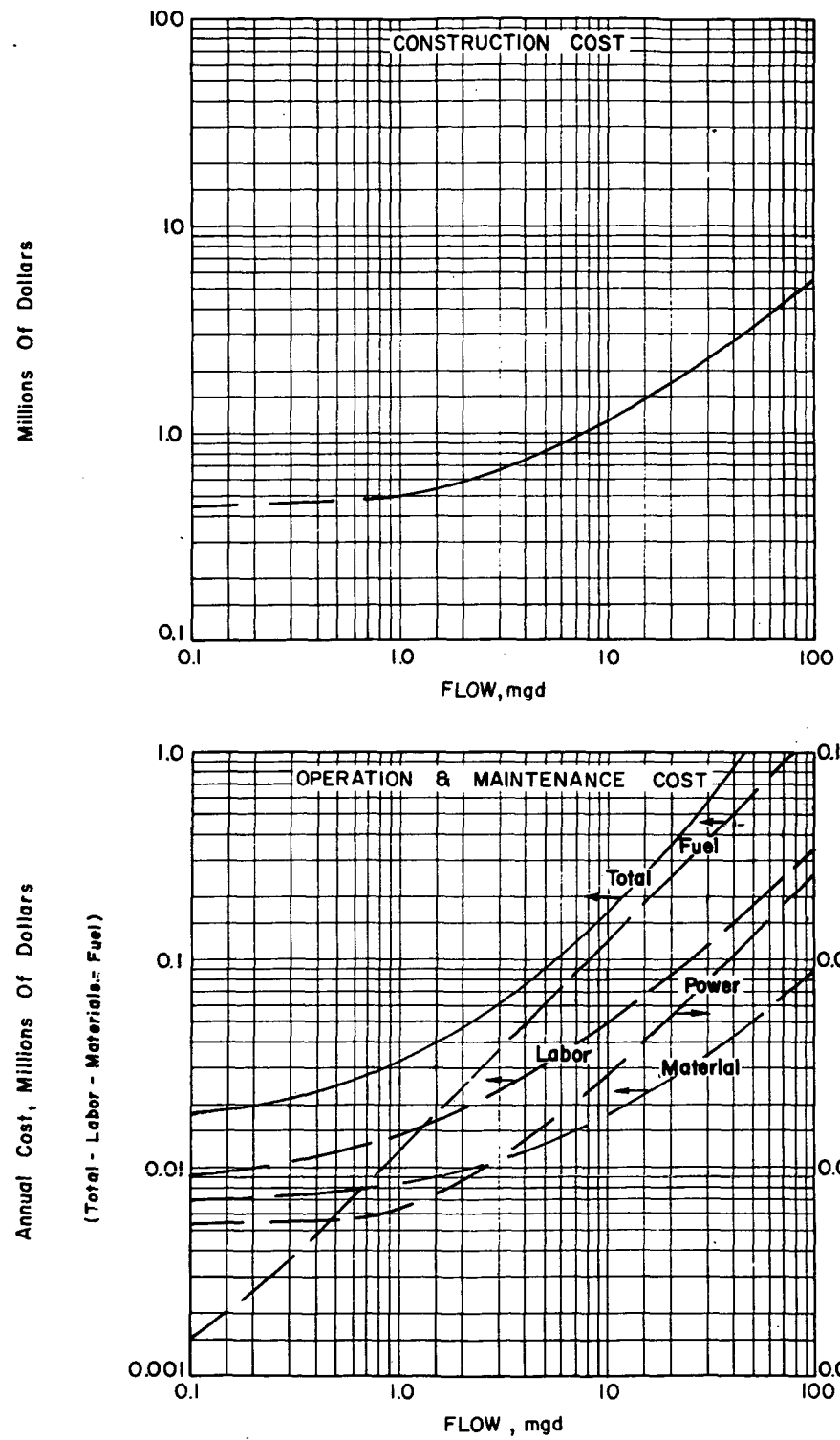


FIGURE H-79

LIME STABILIZATION

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Construction cost includes:

   a. Two volumetric feeder systems with manually loaded bins, dissolving chambers, and accessories for plants less than 1.5 mgd.

   b. For plants larger than 1.5 mgd, two gravimetric feeder-slaker systems and steel bins to hold 1½ truckloads of quick lime each.

   c. For plants larger than 25 mgd, four gravimetric feeder-slaker systems with bin gates and accessories.

   d. Sludge holding tank (3 days detention time).

2. Operations and maintenance costs include:

   a. Slaker - 1 hour of operation/slaker/shift in use.
   b. Feeder - 10 minutes of operation/hour/feeder.
   c. Slurry Pot - feed line (for slake lime) - 4 hours/week.
   d. Power requirements of 7.0 kwh/1,000 lbs lime.

3. Use hydrated lime ($Ca(OH)_2$) for plants less than 1.5 mgd.

4. Use quicklime (CaO) for plants larger than 1.5 mgd.

5. Sludge mass is 1900 lbs dry solids/mg for thickened primary and secondary sludge at 4% solids.

6. Lime dosage is 212 lb (as CaO)/ton of dry solids.

References: 13, 18, 50

FIGURE H-79 LIME STABILIZATION

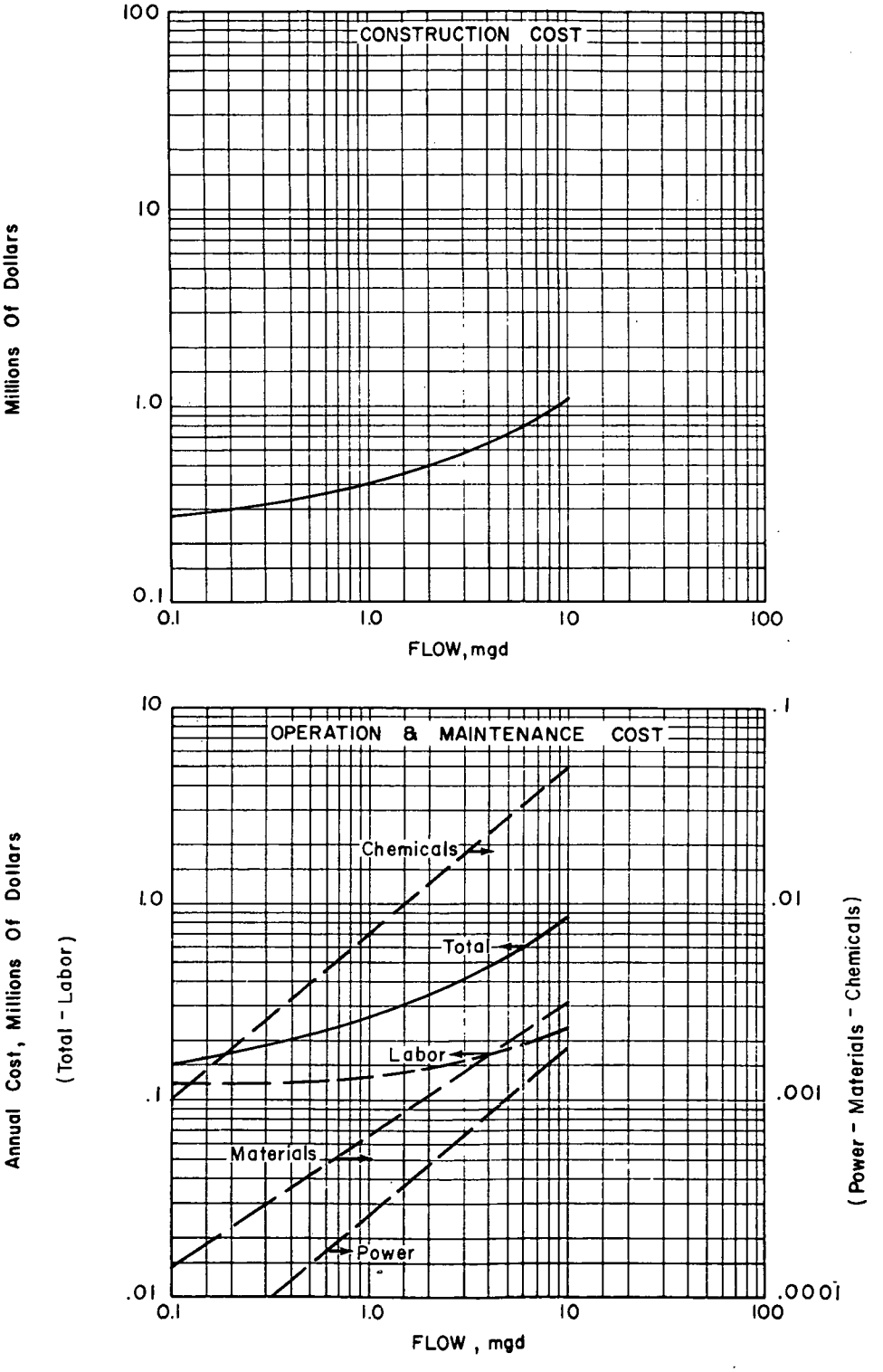


FIGURE H-80

SLUDGE STORAGE

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Costs are for storage of thickened primary and secondary sludge (1,900 lb/mg; at 4% solids).

2. Mixing by diffused air (25 cfm/1,000 cu ft, or approximately 130 hp/mg).

3. Construction costs include: storage tank and air-supply system for mixing and aeration.

Adjustment Factor:

To adjust costs for other sludge quantities and concentrations, enter the curves at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{1{,}900 \text{ lbs/mg}} \times \frac{4\%}{\text{New Design Sludge Concentrations}}$$

References: 3, 7, 9, 13

FIGURE H-80 SLUDGE STORAGE

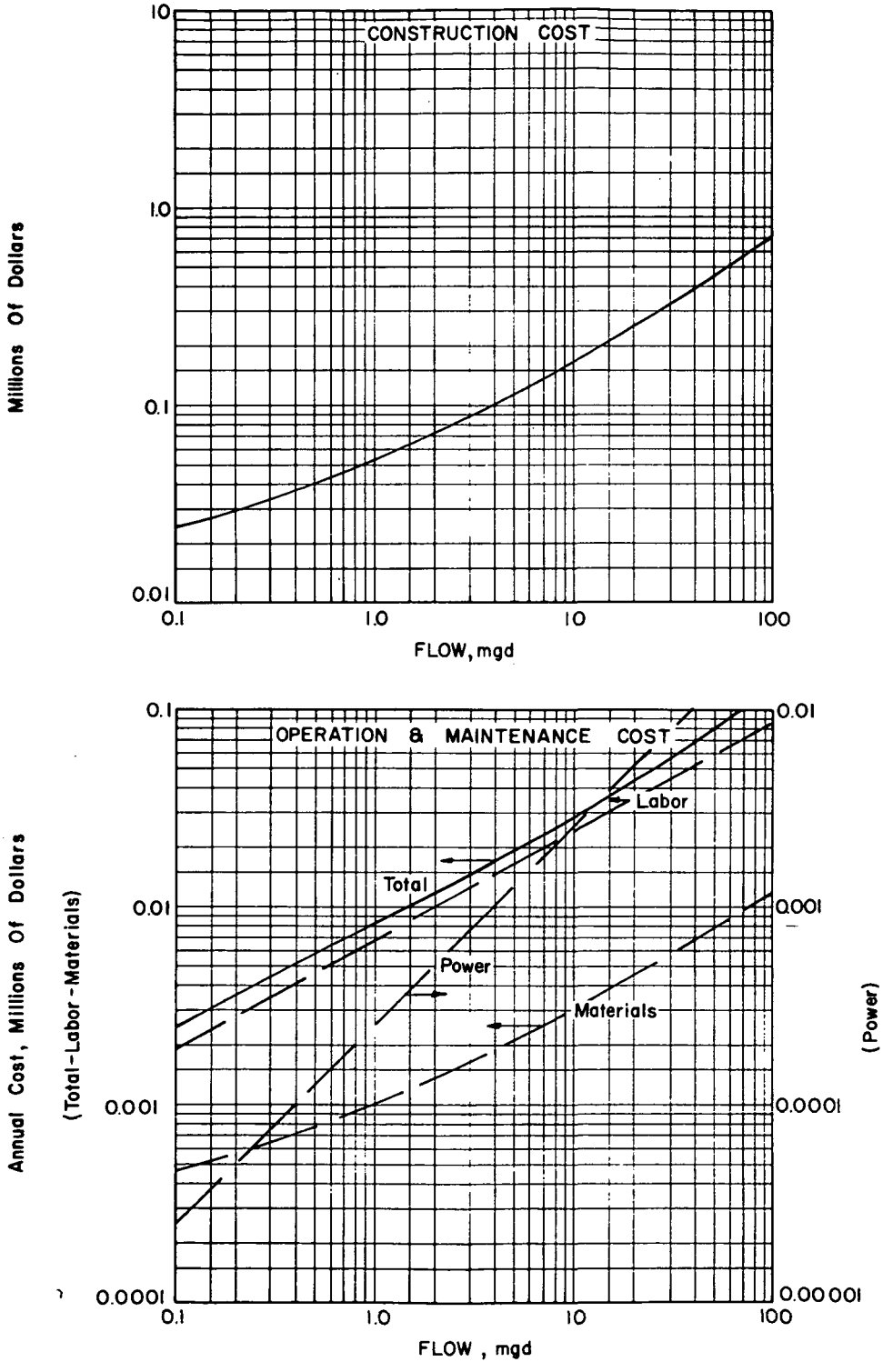


FIGURE H-81

LANDFILLING (BIOLOGICAL SLUDGE EXCLUDING TRANSPORTATION)

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Construction costs do not include cost for land (1.1 acres/mgd including allowance for 25% of volume for cover and 10% loss of surface area for roads and buffer zone).

2. Construction costs include: site preparation, front end loaders, monitoring wells, fencing, leachate collection and treatment.

3. Operation and maintenance costs include:

   a. Labor costs for operation, preventive maintenance, and minor repairs.

   b. Material costs to include replacement parts, soil for mixing with sludge (1 part soil to 3 parts sludge), fuel for bulldozer operation, and repair work performed by outside contractors.

4. Costs are for landfilling of dewatered and digested biological sludge. Costs for landfilling of other predominantly biological sludges can be obtained by using the adjustment factor.

5. Sludge quantity = 900 lb/mg at 20% solids; 2.7 cu yd/mg.
   Note: at 20% concentration, may require blending with soil.

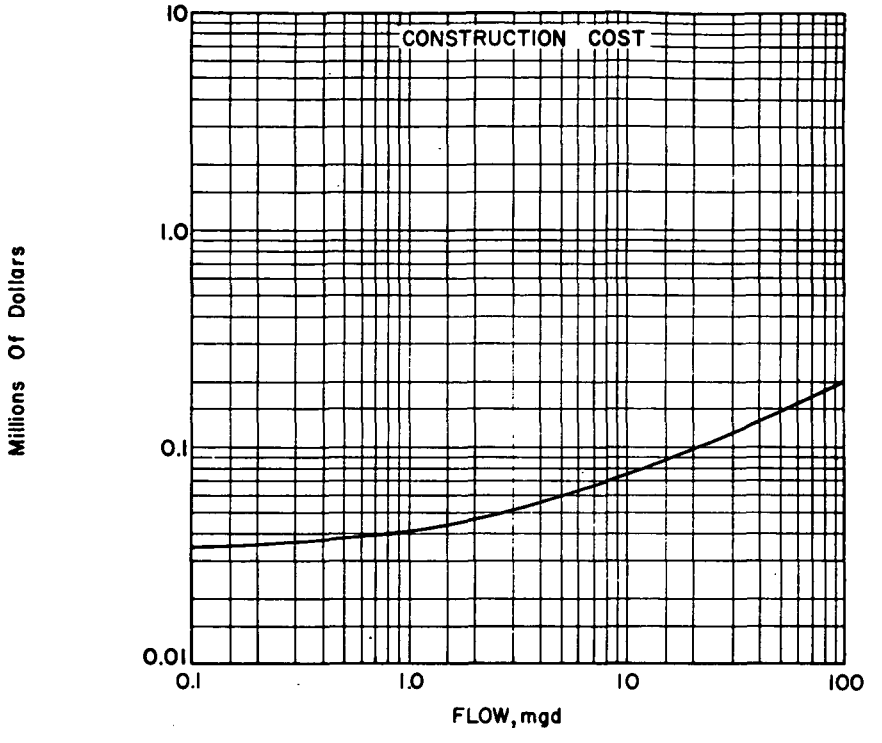
Adjustment Factor:

For sludge quantities and concentrations other than those listed above, enter curve at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{900 \text{ lb/mg}} \times \frac{20\%}{\text{New Design Sludge Concentration}}$$

References: 9, 32

FIGURE H-81 LANDFILLING (BIOLOGICAL SLUDGE, EXCLUDING TRANSPORTATION)



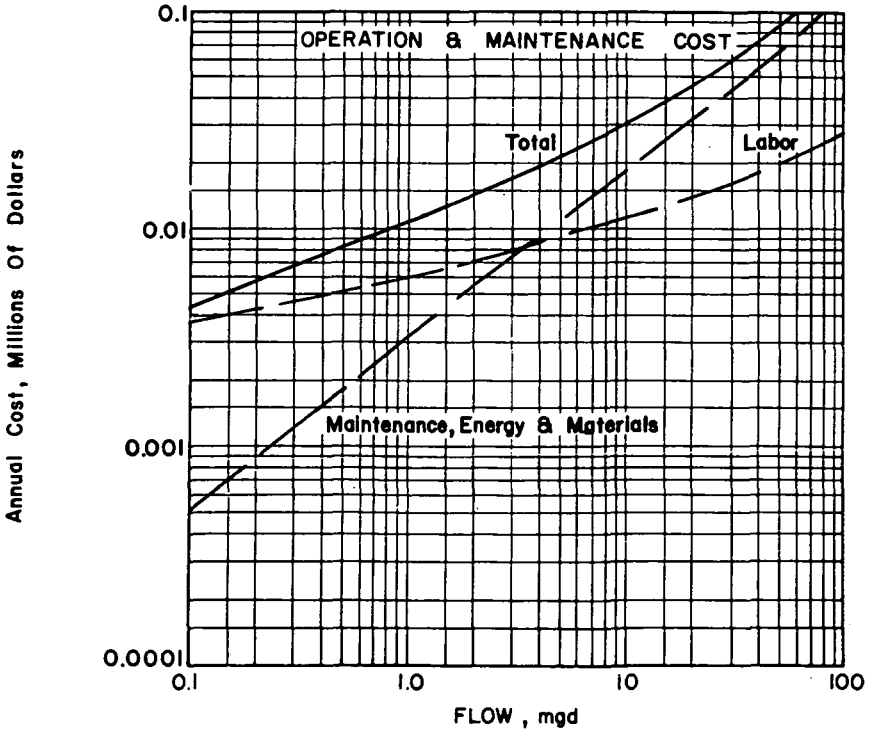


FIGURE H-82

LANDFILLING (LIME SLUDGE EXCLUDING TRANSPORTATION)

Service Life: 20 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: site preparation, bulldozers, monitoring wells, fencing, and leachate collection and treatment.

2. Construction costs do not include cost for land (3.1 acres/mgd; including allowance for 25% of volume for cover and 10% loss of surface area for roads and buffer zone).

3. Operation and maintenance costs include:

   a. Labor costs for operation, preventive maintenance, and minor repairs.

   b. Material costs to include replacement parts, soil for mixing with sludge (1 part soil to 3 parts sludge), fuel for bulldozer operation, and repair work performed by outside contractors.

4. Lime sludge quantity = 4,500 lb/mg at 30%; 7.4 cu yd/mg.

Adjustment Factor:

To adjust costs for other sludge quantities and concentrations, enter the curves at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{\text{New Design Sludge Mass}}{4{,}500 \text{ lb/mg}} \times \frac{30\%}{\text{New Design Concentration}}$$

References: 9, 32

FIGURE H-82 LANDFILLING (LIME SLUDGE, EXCLUDING TRANSPORTATION)

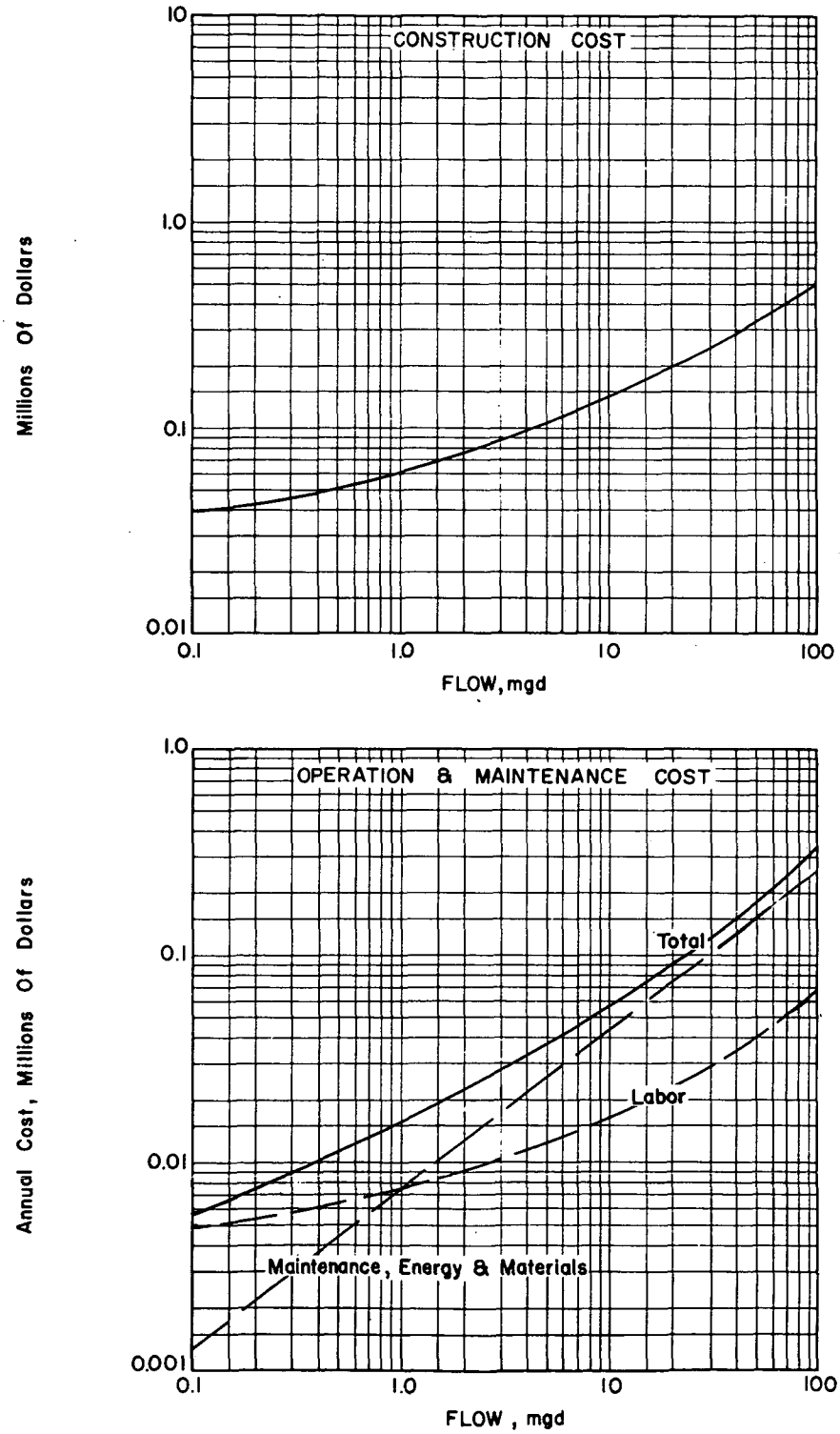


FIGURE H-83

LAND APPLICATION OF SLUDGE

Service Life: 30 years

Relative Reliability: Level B

Design Basis:

1. Construction costs include: storage lagoon (6 weeks); land preparation, monitoring wells (3 at 0.1 mgd, 5 at 1 mgd, 8 at 10 mgd, 25 at 100 mgd, and services roads.

2. Costs are for application of digested biological sludge - 900 lb/mg at 4% solids.

3. Transport of sludge to site is included in the appropriate transportation curve. (See Figures H-86 through H-90.)

4. Sludge application rate - 10 ton (dry)/acre/year.

5. Land costs are not included.

6. Sludge application is by subsurface injection - unit attached to haul truck.

7. Operation and maintenance costs include:

   a. Labor costs for sludge operation, preventive maintenance, and minor repairs.

   b. Material costs to include replacement of parts and repair performance by outside contractors.

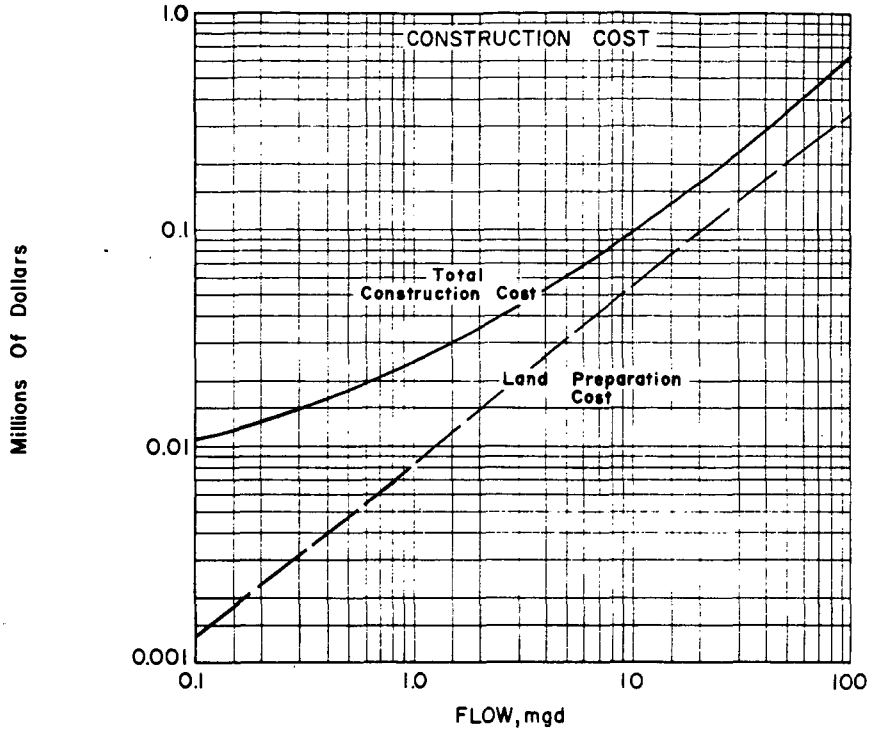
Adjustment Factor:

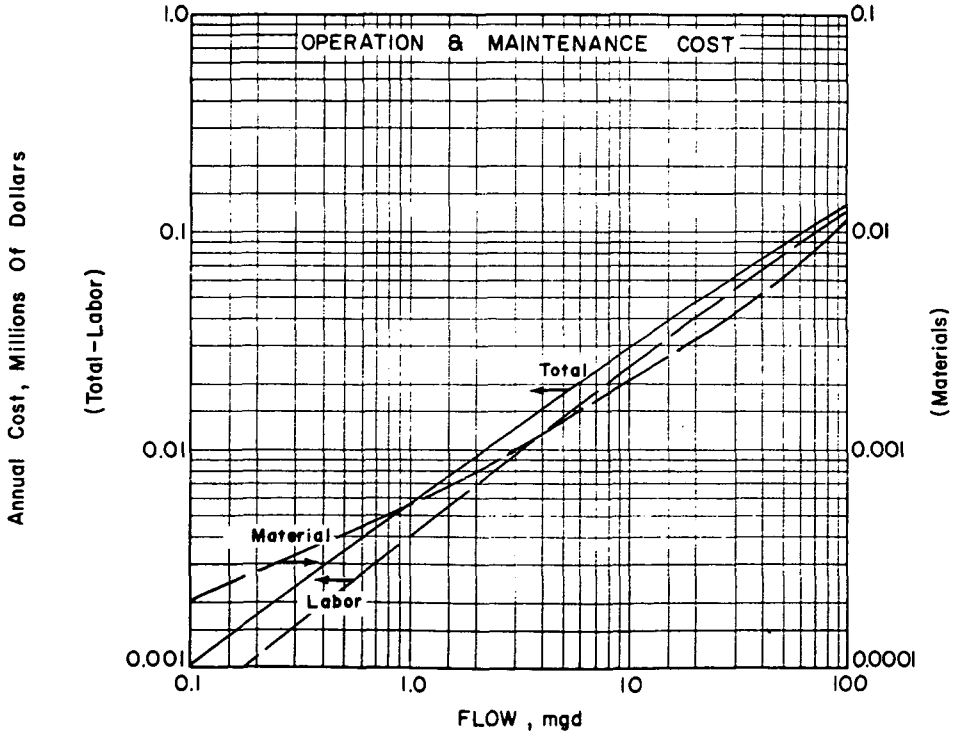
If costs are desired for different application rates enter curve at effective flow (Q_E).

$$Q_E = Q_{DESIGN} \times \frac{10 \text{ ton (dry)/acre/yr}}{\text{New Design Application Rate}}$$

Reference: 9

FIGURE H-83 LAND APPLICATION OF SLUDGE





H.8.5 Wastewater Transportation Methods: Cost Curves, Figures H-84 and H-85

FIGURE H-84

GRAVITY SEWERS

Service Life: 50 years

Relative Reliability: Level B

Design Basis:

1. Peaking factor allowance ranges from 3 for flows of 1 mgd and less average flow, to 2 for flows with greater than 10 mgd average flow.
2. Average slope of 0.002 to 0.005.
3. Velocity of not less than 2 fps when flowing full at peak flow; minimum sewer size = 8 inches.
4. Repaving of road surface required for 10% of distance.
5. Does not include right-of-way or aerial crossing, etc.

References: 38, 52

FIGURE H-84 GRAVITY SEWERS

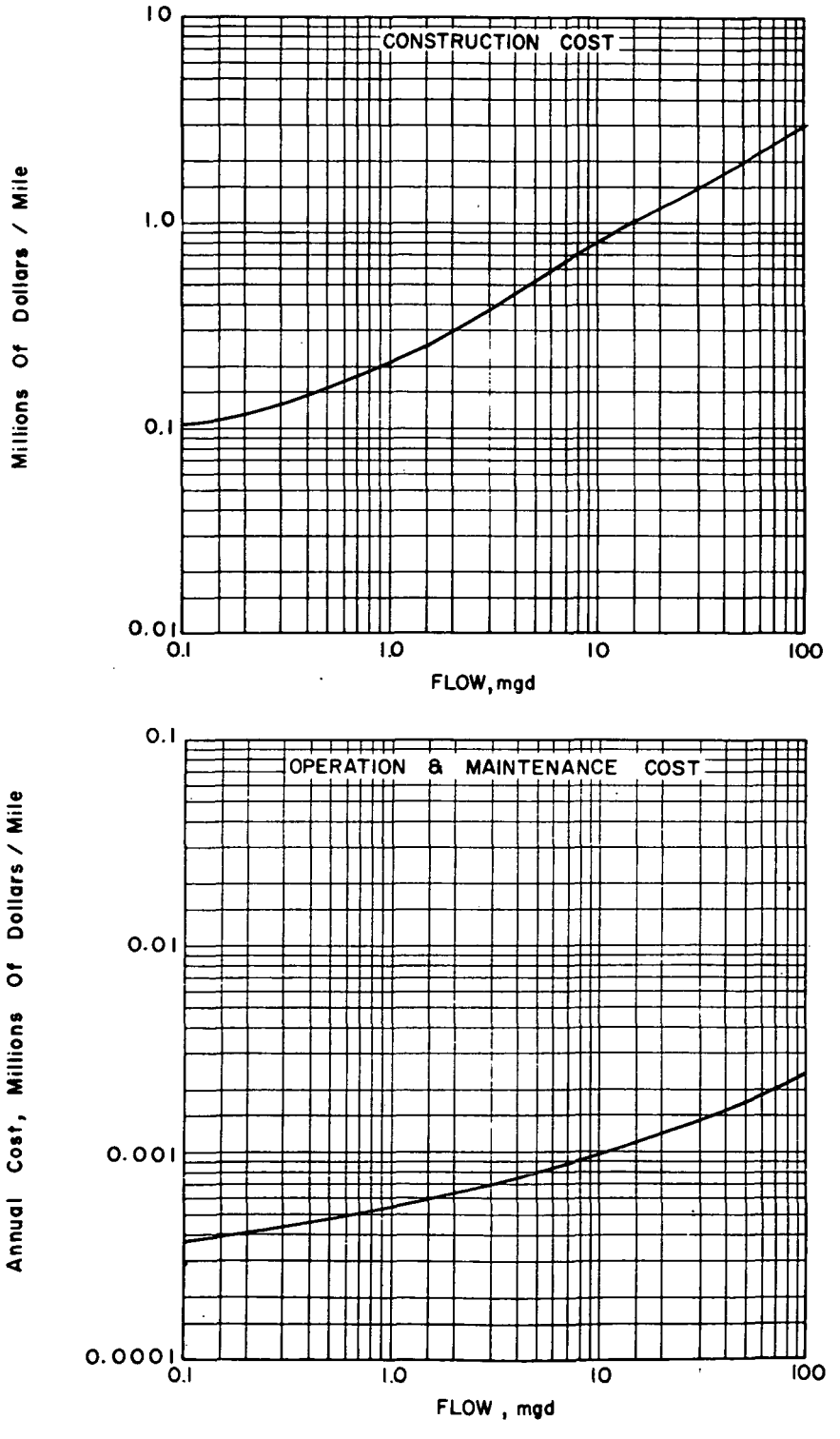


FIGURE H-85

TRANSMISSION FORCE MAIN

Service Life: 50 years

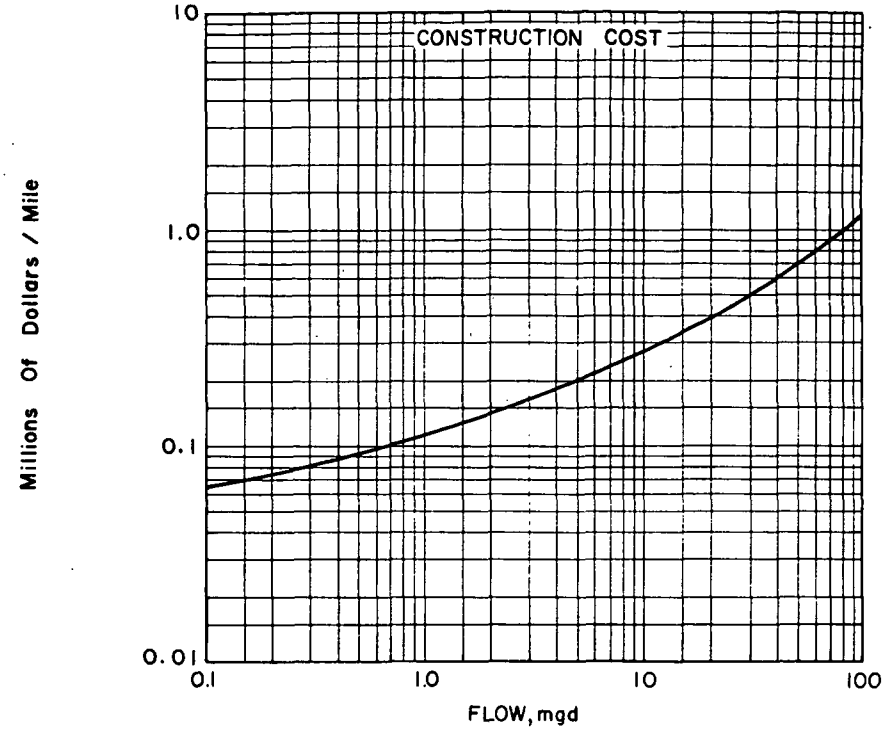
Relative Reliability: Level B

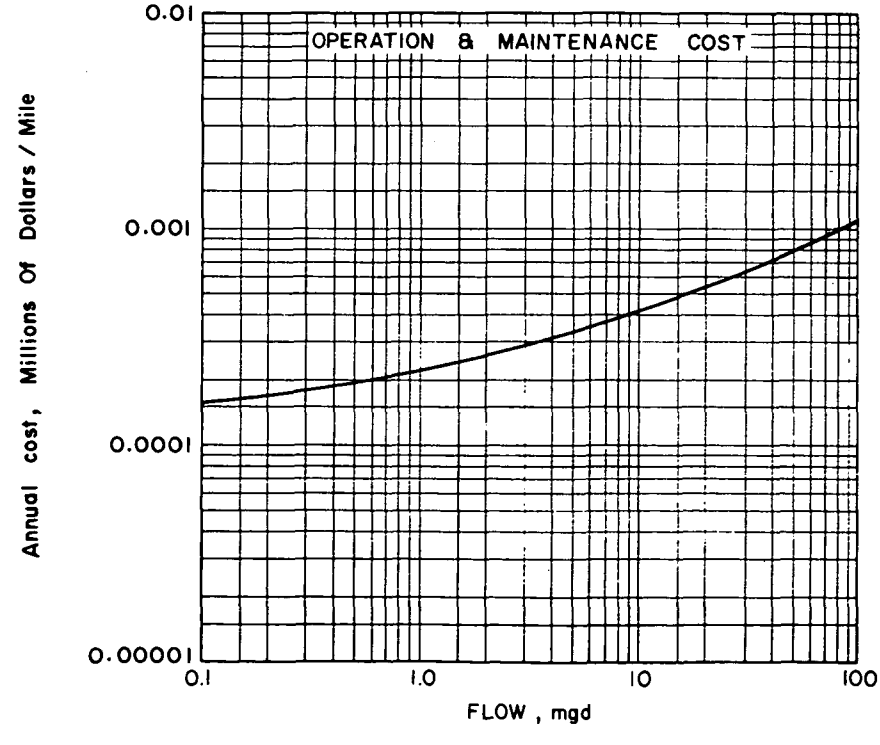
Design Basis:

1. Peaking factor allowance ranges from 3 for flows of 1 mgd and less average flow, to 2 for flows with greater than 10 mgd average flow.
2. 5-ft. depth of cover over crown of pipe.
3. Average velocity of 5 fps.
4. Repaving of road surface required for 10% of distance.
5. Does not include pump stations, right-of-way, aerial crossings, etc.

Reference: 38

FIGURE H-85 TRANSMISSION FORCE MAIN





4.8.6 Sludge Transportation Methods: Cost Curves, Figures H-86 through H-90

FIGURE H-86

LIQUID SLUDGE TRANSPORT (RAIL)

Service Life: 15 years

Level of Reliability: Level B

Design Basis:

1. Construction cost includes loading facilities.
2. Storage at plant equals one day's production.
3. Pumping and piping are sized to fill 1, 10, 20, and 100 unit car trains in 1.5, 2, 3, and 15 hours, respectively.
4. Rail cars discharge by gravity into unloading storage.
5. Storage at unloading area not included in cost.
6. Travel distances of 40, 80, and 160 miles one way.
7. Costs based on 8 hours operation per day.
8. Operation and maintenance costs include car lease, labor, electrical energy, supplies, and rail maintenance.
9. Assume a Full Car Maintenance Lease rate of $525/car.
10. Rail haul charges are based on the following:

Area	Approximate Railroad Rate Variation, Adjust Accordingly
North Central and Central	Average rate as presented here
Northeast	25% higher than average
Southeast	25% lower than average
Southwest	10% lower than average
West Coast	10% higher than average

The railroads generally allow a rebate of $0.06 to $0.20 per mile per car if the shipper provides the car.

11. Solids content of sludge = 4%.
12. Rail cars have 20,000 gallon capacity.

Reference: 46

FIGURE H-86 LIQUID SLUDGE TRANSPORT (RAIL)

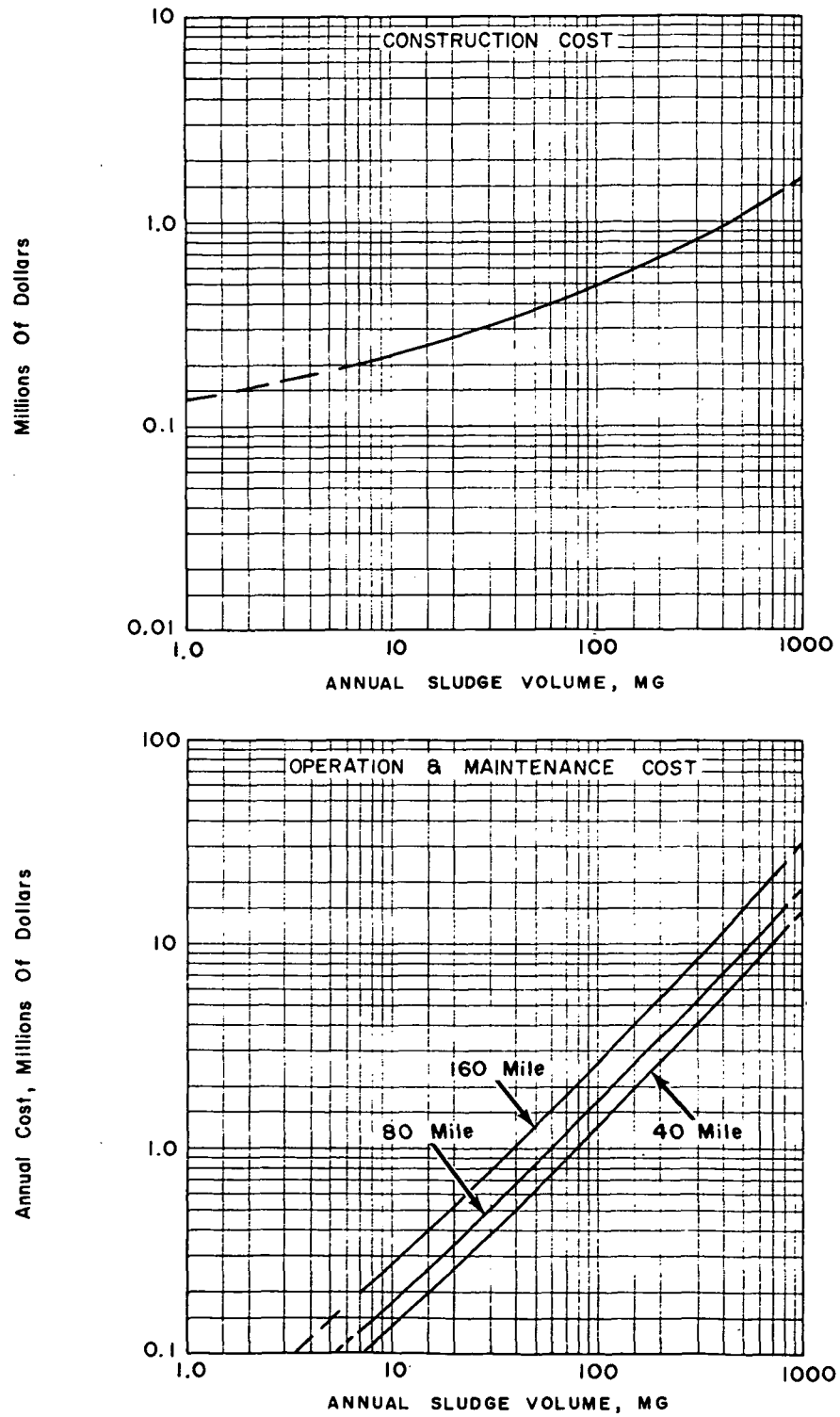


FIGURE H-87

DEWATERED SLUDGE TRANSPORT (RAIL)

Service Life: 15 years

Relative Reliability: Level B

Design Basis:

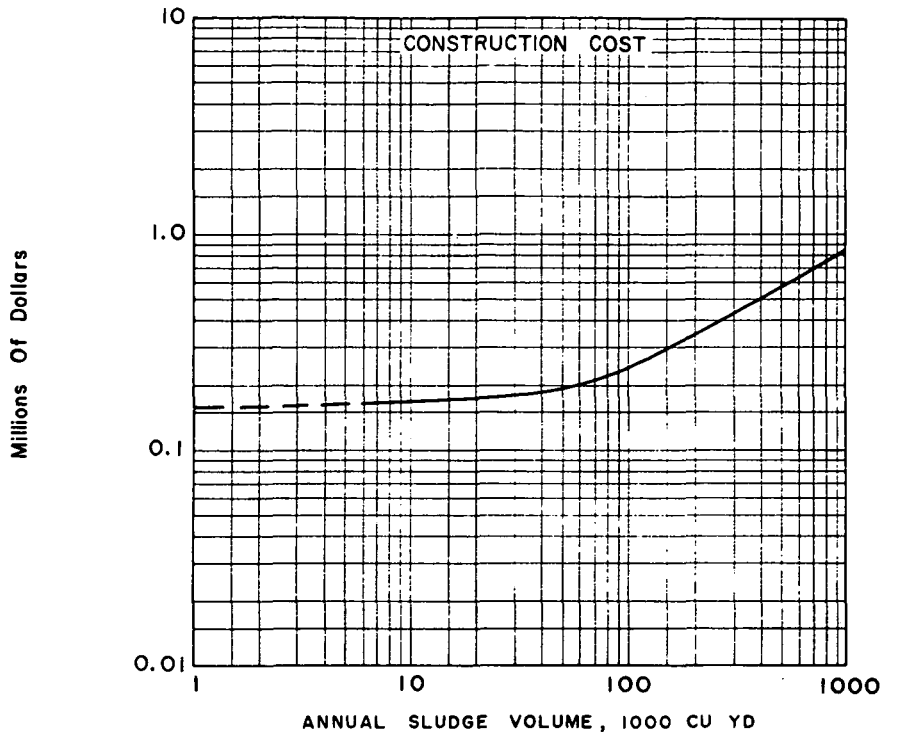
1. Construction cost includes construction of loading facilities.
2. Loading storage hopper is sized for one car load.
3. Cars are gravity loaded.
4. Rail cars dump by gravity into unloading storage.
5. Storage at unloading area is not included in the cost.
6. Operation and maintenance costs include rail haul charges, labor, electrical power, and supplies for the loading facilities.
7. Assume the railroad provides the hopper cars.
8. Use 50 yd^3 cars for 0-74,000 yd^3 of sludge. Use 100 yd^3 cars for greater than 75,000 yd^3 of sludge.
9. Rail haul charges are based on the following:

Area	Approximate Railroad Rate Variation, Adjust Accordingly
North Central and Central	Average rate as presented here
Northeast	25% higher than average
Southeast	25% lower than average
Southwest	10% lower than average
West Coast	10% higher than average

10. Travel distances presented are 40, 80, or 160 miles one way.
11. Costs based on 8 hours operation per day.

Reference: 46

FIGURE H-87 DEWATERED SLUDGE TRANSPORT (RAIL)



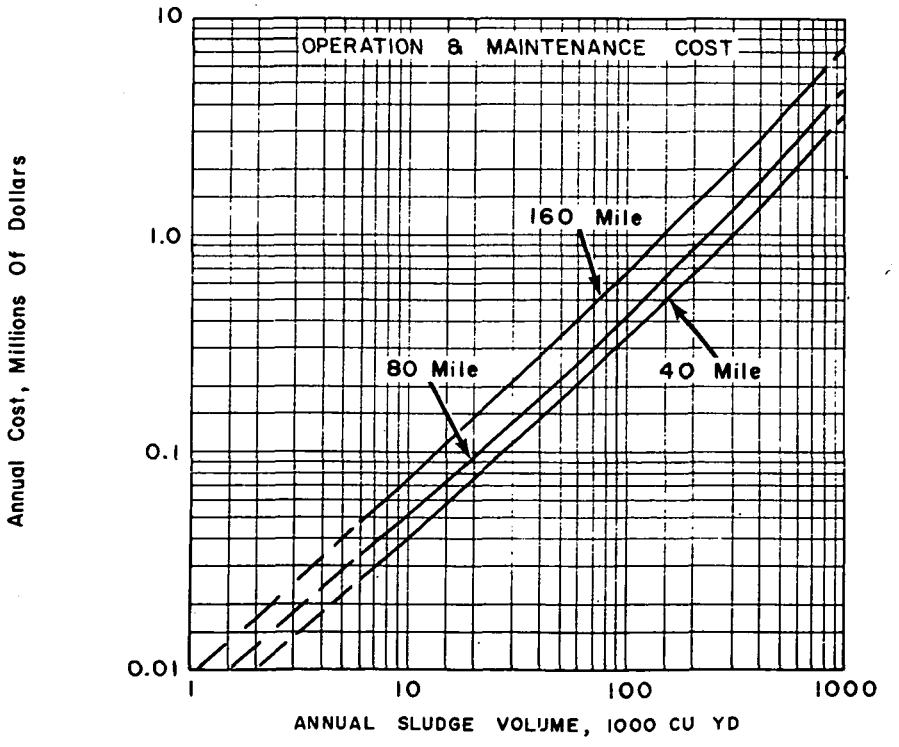


FIGURE H-88

DEWATERED SLUDGE TRANSPORT (TRUCK)

Service Life: 15 years

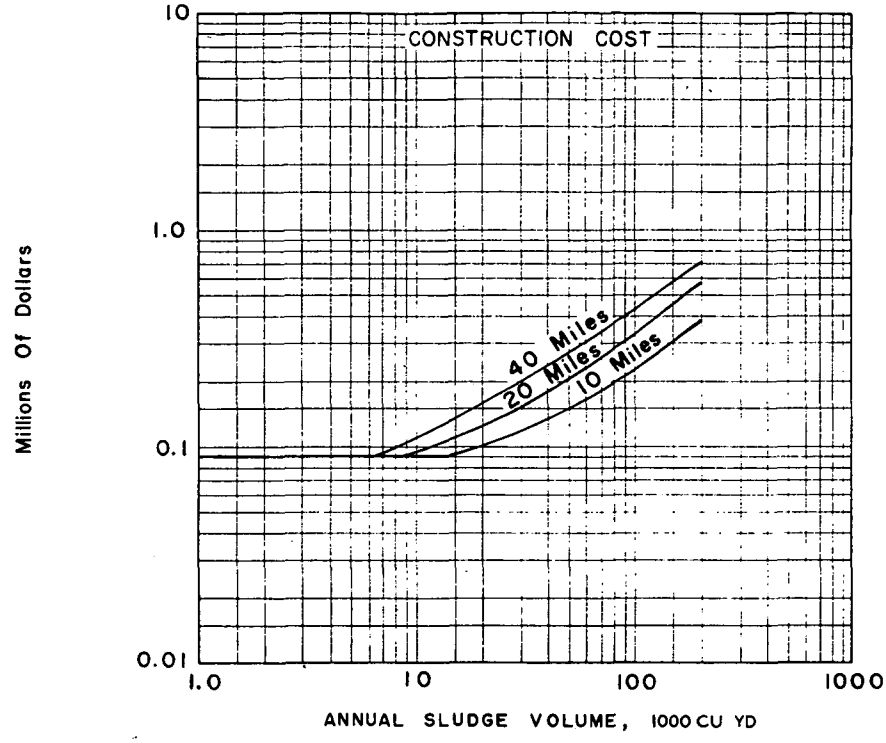
Level of Reliability: Level B

Design Basis:

1. Construction cost includes truck purchase and load/unload facilities.
2. Operation and maintenance costs include truck maintenance, truck operation and fuel, labor for truck and loading facility operation, electrical power, and supplies for the loading facility.
3. Costs are based on use of most cost effective size trucks per volume of sludge transported. Truck sizes are 10 yd^3 and 30 yd^3.
4. Loading hopper is sized for one truck.
5. Equipment is sized to fill truck in 20 minutes maximum.
6. Loading into truck is by gravity. Truck unloading at disposal sites is also by gravity.
7. Storage at unloading site is not included in cost.
8. Costs are based on 8 hours operation per day for 360 days per year.
9. Travel distances of 10, 20, or 40 miles one way to disposal site.
10. Dewatered sludge cake solid content: 25%

Reference: 46

FIGURE H-88 DEWATERED SLUDGE TRANSPORT (TRUCK)



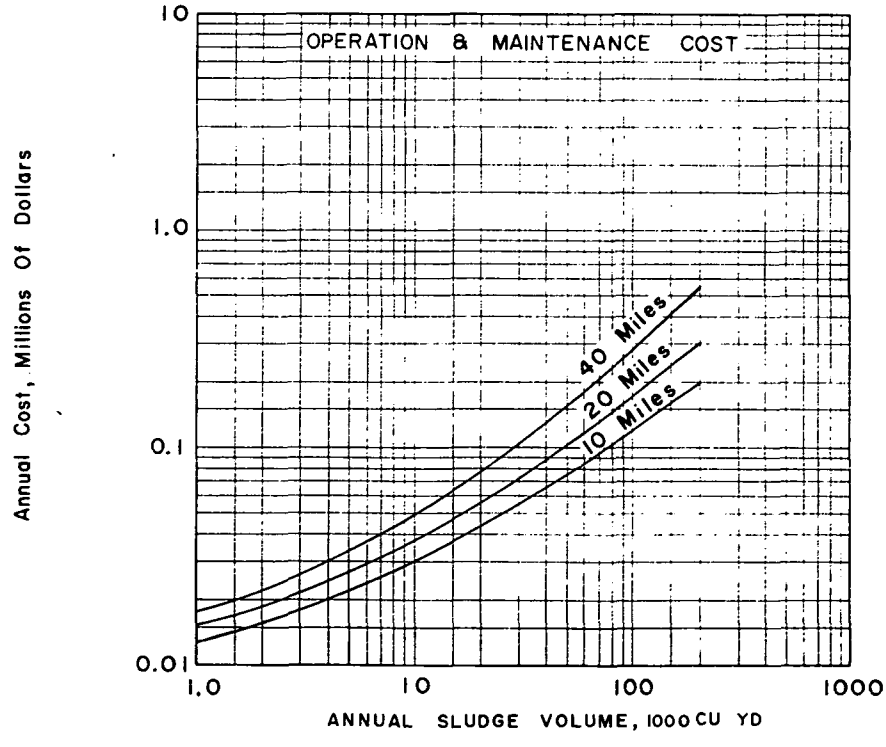


FIGURE H-89

LIQUID SLUDGE TRANSPORT (TRUCK)

Service Life: 15 years

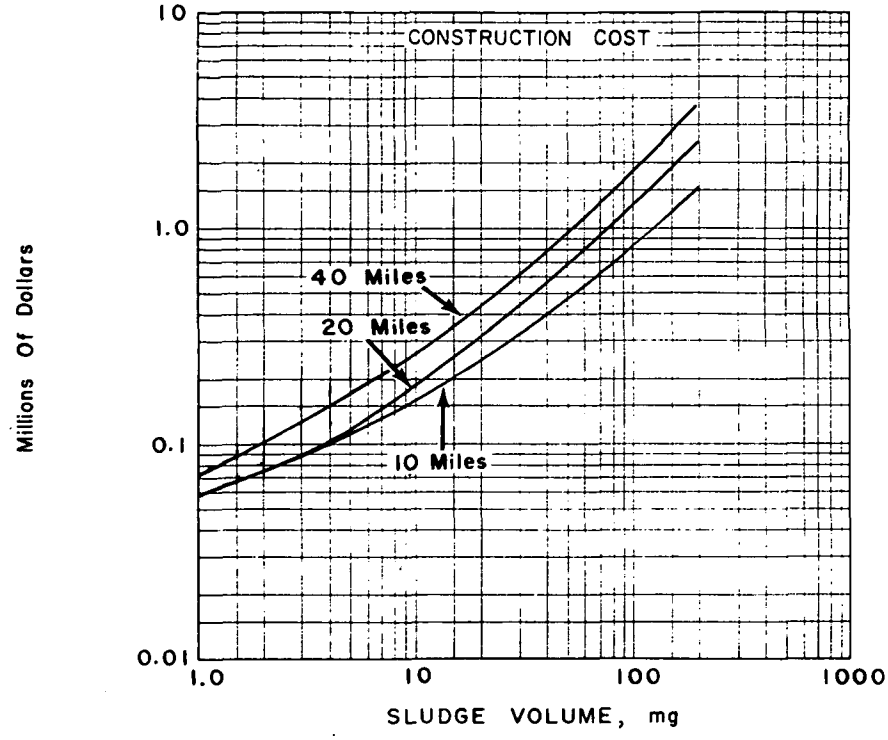
Level of Reliability: Level B

Design Basis:

1. Construction costs include truck purchase and load/unload facilities.
2. Operation and maintenance costs include truck maintenance, truck operation and fuel, labor for truck and loading facility operation, electrical power, and supplies for the loading facility.
3. Costs are based on use of most cost effective size trucks per volume of sludge transported. Truck sizes are 1200 gallon, 2500 gallon, and 5500 gallon.
4. Equipment is sized to fill truck in 20 minutes maximum.
5. Loading into truck is by gravity. Truck unloading at disposal site is also by gravity.
6. Storage at unloading site is not included in cost.
7. Costs are based on 8 hours operation per day for 360 days per year.
8. Travel distances of 10, 20, or 40 miles one way to disposal site.
9. Fuel cost (gasoline) = $0.60/gallon.
10. Liquid sludge = 4% solids.

Reference: 46

FIGURE H-89 LIQUID SLUDGE TRANSPORT (TRUCK)



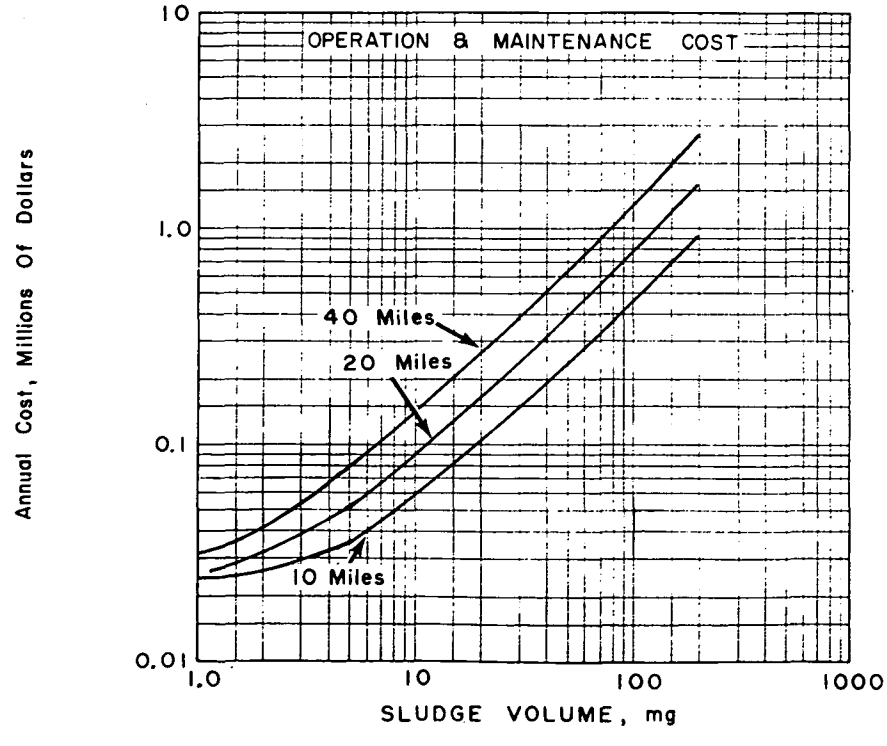


FIGURE H-90

SLUDGE TRANSPORT (PIPELINE)

Service Life: 15 years

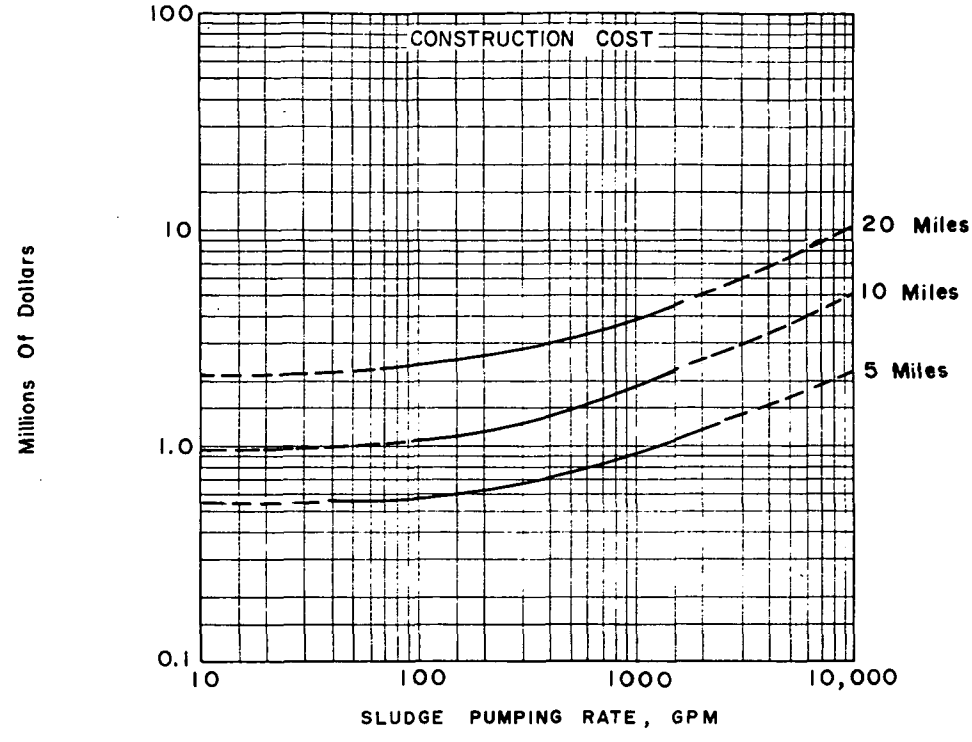
Relative Reliability: Level B

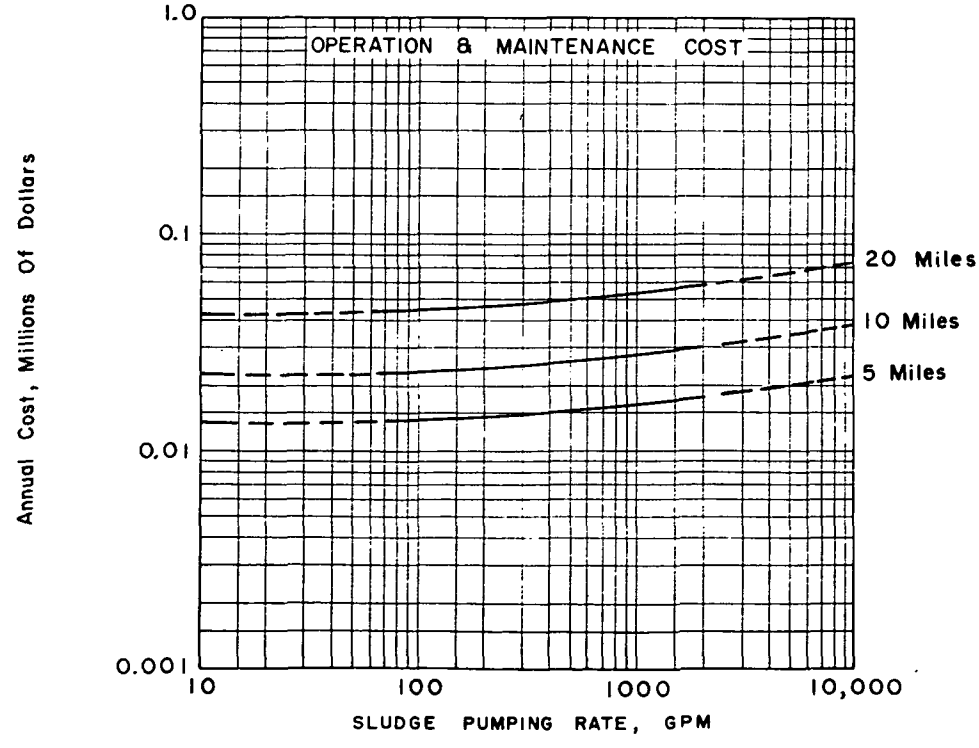
Design Basis:

1. Construction cost includes: pipeline and pumping stations. Cost includes one major highway crossing per mile, one single rail crossing per 5 miles, and a nominal number of driveways and minor road crossings.
2. Flow velocity = 4.0 fps.
3. Pumping distances of 5, 10, and 20 miles.
4. No rock excavation or major unusual problems (for hard rock add 70% to cost).
5. Pipeline is buried 3 to 6 feet. For 6-10 feet, add 15% to cost.
6. No elevation change in pipeline.
7. Minimum pipe size is 4 inches.
8. Pipeline is cement-lined cast iron or ductile iron.
9. Costs based on 12 hours pumping per day.
10. Operation and maintenance cost includes labor, supplies and electrical power for pump stations.
11. Pumps are dry-pit, horizontal or vertical, non-clog centrifugal pumps operating at 1780 rpm.

Reference: 46

FIGURE H-90 SLUDGE TRANSPORT (PIPELINE)





H.9 References

1. Smith, R., et al., "Cost of Alternative Processes for Wastewater Disinfection". U.S. EPA, Cincinnati, Ohio (October, 1974).

2. Smith, R., "The Cost of Dispersed Floc Nitrification and Dentrification for Removal of Nitrogen from Wastewater". U.S. EPA, Cincinnati, Ohio (November, 1970).

3. Patterson, W.L. and Banker, R.F., "Estimating Costs and Manpower Requirements for Conventional Wastewater Treatment Facilities". U.S. EPA Report No. 17090 DAN 10/71 (October, 1971).

4. Van Note, R.H., et al., "A Guide to the Selection of Cost-Effective Wastewater Treatment Systems". U.S. EPA Report No. 430/9-75-002 (July, 1975).

5. Metcalf and Eddy, Inc., "Assessment of Technologies and Costs for Publicly Owned Treatment Works". Report to the National Commission on Water Quality (September, 1975).

6. Roy F. Weston, Inc., Cost Curves Developed for U.S. EPA Miscellaneous Chemical Industry Effluent Guidelines Project (January, 1975).

7. Roy F. Weston, Inc., Internal Cost-Estimating Computer Program - DYNAMO 1.

8. Smith, R., "Electrical Power Consumption for Municipal Wastewater Treatment". U.S. EPA, Cincinnati, Ohio (August, 1972).

9. Roy F. Weston, Inc., Cost Projections Based on National Average Unit Costs Obtained From: a) Literature References; b) Weston In-house Cost Data; c) Manufacturers' Information.

10. Wyatt, J.M. and White, P.E. Jr., "Sludge Processing, Transportation and Disposal/Resource Recovery: A Planning Perspective". U.S. EPA Report No. WA-75-R024 (December, 1975).

11. Smith, R. and McMichael, W.F., "Cost and Performance Estimates for Tertiary Wastewater Treating Processes". U.S. Department of the Interior Report No. TWRC-9 (June, 1969).

12. Toftner, R.O., "Planner's Handbook for Residual Waste Management". U.S. EPA Draft Report (March, 1976).

13. U.S. EPA Technology Transfer, "Process Design Manual for Sludge Treatment and Disposal". Report No. 625/1-74-006 (October, 1974).

14. Hagerty, D.J., Pavoni, J.L., and Heer, J.E. Jr., "Solid Waste Management". New York: Van Nostrand-Reinhold Company (1973).

15. Pavoni, J.L., Heer, J.E. Jr., and Hagerty, D.J., "Handbook of Solid Waste Disposal: Materials and Energy Recovery". New York: Van Nostrand-Reinhold Company (1975).

16. Cruver, J.E., Beckman, J.E., and Bevege, E.E., "Water Renovation of Municipal Effluents by Reverse Osmosis". U.S. EPA Report No. 17040 EOR (August, 1972).

17. Universal Oil Products Company, ROGA Division, "Reverse Osmosis Principles and Applications". Training Manual (October, 1970).

18. Culp, R.L., Wesner, W.J., and Culp, G.L., "Costs of Chemical Clarification of Wastewater". U.S. EPA Draft Report for Contract No. 68-03-2186 (January, 1976).

19. U.S. EPA Technology Transfer, "Process Design Manual for Nitrogen Control" (October, 1975).

20. "Relative Cost Study of Alternate Phosphorous Removal Processes for Wastewater Application". Boyle Engineering (June, 1972).

21. Ducan, G.J. and Levin, P., "Mathematical Model of Sewage Sludge Fluidized Bed Incinerator Capacities and Costs". FWPCA Report No. TWRC-10 (September, 1969).

22. Mayrose, D.T., "Fluidized Bed Reactors Ease Problems", Water and Wastes Engineering, 13, No. 10, pg. 56 (1976).

23. Weing, Lewis J., Almgran, H.H., and Culp, R.L., "Effects of Thermal Treatment of Sludge on Municipal Wastewater Treatment Costs". U.S. EPA Contract No. 68-03-2186 (1976).

24. U.S. EPA Technology Transfer, "Process Design Manual for Suspended Solids Removal". No. 625/1-75-003a (January, 1975).

25. U.S. EPA Technology Transfer, "Process Design Manual for Phosphorus Removal" (October, 1974).

26. Eilers, R.G., "Condensed One-Page Cost Estimates for Wastewater Treatment". U.S. EPA, Cincinnati, Ohio (November, 1970).

27. Pound, C.E., Crites, R.W., Griffes, D.A., "Costs of Wastewater Treatment by Land Application". U.S. EPA Contract No. 68-01-0966 (June, 1975).

28. Smith, R., "Cost-Effective Analysis for Water Pollution Control". U.S. EPA (1974).

29. U.S. EPA Technology Transfer, "Process Design Manual for Carbon Adsorption" (October, 1973).

30. Roesler, J.F., Smith, R., "A Mathematical Model for a Trickling Filter". U.S. Department of the Interior, FWPCA, Taft Water Research Center, 1969.

31. Environmental Quality Systems, Inc., "Technical and Economic Review of Advanced Waste Treatment Process". Office of the Chief of Engineers, U.S. Army Corp of Engineers (March, 1973).

32. Council on Environmental Quality and U.S. EPA "Evaluation of Municipal Sewage Treatment Alternatives" (February, 1974).

33. Smith, R., "New Technology for Carbon and Nitrogen Removal - Single Stage Activated Sludge, Bio-Disc, or Fluidized Bed". U.S. EPA, Cincinnati, Ohio.

34. Burd, R.G., "A Study of Sludge Handling and Disposal". FWPCA (May, 1968).

35. Ehlich, W.F., "What's Best for Sludge Transport". Water and Wastes Engineering (October, 1976).

36. Benjes, H.H., Jr., "Estimating Construction Costs and Operating and Maintenance Requirements for Combined Sewer Overflow Storage and Treatment Facilities". Prepared for Municipal Environmental Research Laboratory, Cincinnati, Ohio (May, 1976).

37. Hydroscience, Inc., "Texas Stormwater Manual on Treatment", Draft Report.

38. "Costs of Wastewater Treatment by Land Application". U.S. EPA Report No. 430/9-75 (1975).

39. Kalinske, A.A., "Comparison of Air and Oxygen Activated Sludge Systems". Journal Water Pollution Control Federation, pg. 2472 (November, 1976).

40. Chapman, T.D., Matsch, L.C., and Zander, H.H., "Effect of High Dissolved Oxygen Concentration in Activated Sludge Systems". Journal Water Pollution Control Federation, pg. 2486 (November, 1976).

41. Parker, D.S. and Merrill, S.L., "Oxygen and Air Activated Sludge: Another View". Journal Water Pollution Control Federation, pg. 2511 (November, 1976).

42. "Urban Stormwater Management and Technology - An Assessment". U.S. EPA Report No. 670/2-74-040 (1974).

43. Benjes, H.H., Jr., "Small Community Wastewater Treatment Facilities". Presented at EPA Technology Transfer, National Conference on Small Wastewater Treatment Systems, Portland, Oregon, March 8-10, 1977.

44. "Energy Conservation in Municipal Wastewater Treatment". Draft Report, U.S. EPA Contract No. 68-03-2186 (November, 1976).

45. "Estimating Staffing for Municipal Wastewater Treatment Facilities". U.S. EPA Contract No. 68-01-0328.

46. Ettlich,W.F., "Transport of Sewage Sludge". EPA Contract No. 68-03-2186 (August, 1976).

47. Culp/Wesner/Culp, "A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastewater". U.S. EPA Contract No. 68-03-2186.

48. Benjes, H.H., Jr., "Evaluation of Biological Wastewater Treatment Processes".

49. "Energy Conservation in Municipal Wastewater Treatment". U.S. EPA Draft Report, Contract No. 68-03-2186, Task No. 9 (November, 1976).

50. Smith, R. and Eilers, R.G., "Computer Evaluation of Sludge Handling and Disposal Costs". U.S. EPA, Cincinnati, Ohio.

51. "Cost Estimates for Construction of Publicly-Owned Wastewater Treatment Facilities - Summaries of Technical Data for Combined Sewer Overflows and Stormwater Discharge, 1976 Needs Survey". U.S. EPA Report No. 430/9-76-012, MCD Report No. 48C (February 10, 1977).

52. "Cost Estimates for Construction of Publicly-Owned Wastewater Treatment Facilities - Summaries of Technical Data, Categories I-IV, 1976 Needs Survey". U.S. EPA Report No. 430/9-76-011, MCD Report No. 48C (February 10, 1977).

53. Colacicco, D. et al. "Costs of Sludge Composting." Agricultural Research Service, U.S. Dept. of Agriculture ARS-NE-79, February 1977.

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

1. REPORT NO.	2.	3. RECIPIENT'S ACCESSION NO.
EPA-600/9-76-014		

4. TITLE AND SUBTITLE	5. REPORT DATE
Areawide Assessment Procedures Manual	July 1976 (Issuing Date)
	6. PERFORMING ORGANIZATION CODE

7. AUTHOR(S)	8. PERFORMING ORGANIZATION REPORT NO.
Municipal Environmental Research Laboratory	

9. PERFORMING ORGANIZATION NAME AND ADDRESS	10. PROGRAM ELEMENT NO.
U.S.Environmental Protection Agency, MERL Wastewater Research Division 26 West St. Clair Street Cincinnati, Ohio 45268	1HC619 11. CONTRACT/GRANT NO. 68-03-2428 68-03-2445 68-03-2437 68-01-4158

12. SPONSORING AGENCY NAME AND ADDRESS	13. TYPE OF REPORT AND PERIOD COVERED
Municipal Environmental Research Laboratory Office of Research and Development U.S. Environmental Protection Agency Cincinnati, Ohi 45268	Planning Manual 14. SPONSORING AGENCY CODE EPA/600/14

15. SUPPLEMENTARY NOTES The manual is prepared as an information document and users planning manual to support the Agency's 208 areawide waste treatment management and planning effort. It is a joint effort of the Office of Research and Development and the Office of Water Planning and Standards.

16. ABSTRACT Pertinent to the implementation of State and areawide planning under Section 208 of P.L. 92-500, this manual provides an environmental management statement of procedures available for water quality management with particular emphasis on urban stormwater. The manual summarizes and presents in condensed form a range of available procedures and methodologies that are available for identifying and estimating pollutant load generation and transport from major sources within water quality management planning areas. Although an annotated chapter is provided for the assessment of non-urban pollutant loads, the major emphasis of the manual is directed toward the assessment of problems and selection of alternatives in urban areas, with particular concern for stormwater related problems. Also included in the manual are methodologies for assessing the present and future water quality impacts from major sources as well as summaries of available information and techniques for analysis and selection of structural and nonstructual control alternatives.

This manual is structured to present problem assessment and impact analysis approaches for several levels of planning sophistication. Simple procedures are recommended for initial analysis to develop the insight and problem understanding to guide the application of more complex techniques where required. Presented in three volumes the specific sections are as follows: procedures for assessment of urban and non-urban pollution sources and loadings, pollutional load stream impact analysis, methodology for evaluation and selection of control alternatives, rainfall runoff and water quality model applicability, land use and rainfall data analysis, runoff and water quality monitoring and parameter handbook, and cost and performance of structural and nonstructural pollution control alternatives.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Water quality, Assessments, Mathematical models, Systems analysis, Urban areas, Rural areas, Waste water--water pollution, Waste treatment, Rainfall, Runoff, Regional planning, Cost effectiveness--management methods	Areawide planning, Statistical analysis, Stream impact analysis, Treatment alternative evaluation, AAPM, Section 208, P.L. 92-500, Monitoring, Point source pollution, Nonpoint source pollution	13B

18. DISTRIBUTION STATEMENT	19. SECURITY CLASS *(This Report)*	21. NO. OF PAGES
Release Unlimited	Unclassified	2129
	20. SECURITY CLASS *(This page)*	22. PRICE
	Unclassified	

EPA Form 2220-1 (9-73)

☆ U. S. GOVERNMENT PRINTING OFFICE: 1977-757-056/6443 Region No. 5-11